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A DISPLACEMENT-BASED ADAPTIVE PUSHOVER FOR ASSESSMENT OF BUILDINGS AND BRIDGES

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Abstract: Estimating seismic demands on structures requires explicit consideration of the structural inelastic behaviour: to this end, the use of nonlinear static procedures, or pushover analyses, is inevitably going to be favoured over complex, impractical for widespread professional use, nonlinear time history methods. Currently employed pushover methods are performed subjecting the structure to monotonically increasing lateral forces with invariant distribution until a target displacement is reached, basing both the force distribution and target displacement on the assumptions that the response is controlled by the fundamental mode, unchanged after the structure yields. However, these invariant force distributions cannot account for the contributions of higher modes to response, nor for the redistribution of inertia forces because of structural yielding and the associated changes in the vibration properties: in order to overcome drawbacks arising from conventional methods, an innovative displacement-based adaptive pushover technique for estimation of the seismic capacity of RC structures is illustrated. Analytical parametric studies on a suite of continuous multi-span bridges and framed buildings show that, with respect to conventional pushover methods, the novel approach can lead to the attainment of significantly improved predictions, which match very closely results from dynamic nonlinear analysis.

Key words: displacement-based; adaptive pushover; DAP; buildings; bridges; storey drift

1. INTRODUCTION

A major challenge in performance-based engineering is to develop simple, yet accurate methods for estimating seismic demand on structures considering their inelastic behaviour: the use of nonlinear static procedures, or pushover analyses, is inevitably going to be favoured over complex, impractical for widespread professional use, nonlinear time-history methods.

It is observed that traditional pushover methods, prescribed in a number of seismic design codes for buildings, feature a number of drawbacks, mainly related to the impossibility of a fixed force pattern to accurately model the varying response characteristics of reinforced concrete structures subjected to strong transverse motion. On the contrary, the alternative, and most innovative, displacement-based adaptive pushover algorithm proposed is shown to lead to the attainment of significantly improved predictions, which match very closely results from dynamic nonlinear analysis.

The objectives of the present research are to verify the applicability of different pushover procedures, either adaptive or conventional, to RC structures. Analytical parametric studies have been conducted on a number of regular and irregular bridges and buildings: the effectiveness of each methodology in modelling both global behaviour and local phenomena is assessed by comparing static analysis results with the outcome of nonlinear time-history runs.

2. PUSHOVER METHODOLOGIES IN EARTHQUAKE ENGINEERING

The term ‘pushover analysis’ describes a modern variation of the classical ‘collapse analysis’ method, as fittingly described by Kunnath (2004). The procedure consists of an incremental-iterative solution of the static equilibrium equations corresponding to a nonlinear structural model subjected to a monotonically increasing lateral load pattern. The structural resistance is evaluated and the stiffness matrix is updated at each increment of the forcing function, up to convergence. The solution proceeds until (i) a predefined performance limit state is reached, (ii) structural collapse is incipient or (iii) the program fails to converge. Within the framework of earthquake engineering, pushover analysis is employed with the objective of deriving an envelope of the response parameters that would otherwise be obtained through many possible dynamic analyses, corresponding to different intensity levels.

2.1 Nonlinear static pushover in current practice

According to recently introduced code provisions, such as FEMA-356 (BCCS, 2000) and Eurocode 8 (CEN 2002), pushover analysis should consist of subjecting the structure to an increasing vector of horizontal forces with invariant pattern. Both the force distribution and target displacement are based on the assumptions that the response is controlled by the fundamental mode and the mode shape remains unchanged until collapse occurs. Two lateral load patterns, namely the first mode proportional and the uniform, are recommended to approximately bound the likely distribution of the inertia forces in the elastic and inelastic range, respectively.

However, a number of recent studies, summarised in the FEMA-440 (ATC, 2005) report, raise doubts on the effectiveness of these conventional force-based pushover methods in estimating the seismic demand throughout the full deformation range: (i) inaccurate prediction of deformations when higher modes are important and/or the structure is highly pushed into its nonlinear post-yield range, (ii) inaccurate prediction of local damage concentrations, responsible for changing the modal response, (iii) inability of reproducing peculiar dynamic effects, neglecting sources of energy dissipation such as kinetic energy, viscous damping, and duration effects, (iv) difficulty in incorporating three-dimensional and cyclic earthquake loading effects. Krawinkler and Seneviratna (1998) summarised the above with a single statement; fixed load patterns in pushover analysis are limiting, be they first modal or multimodal derived, because no fixed distribution is able of representing the dynamic response throughout the full deformation range.

2.2 The new generation of pushover procedures

In an attempt to include higher modes effects, a number of Multi-Modal Inelastic Procedures (MMP) has been recently developed. These may be referred to as “pushover-based procedures”, as opposed to “pure pushover” analysis methods, since they estimate the seismic demand at one or more specific seismic levels (i.e. “individual point” on the pushover curve) rather than providing a structural capacity curve throughout the whole deformation range. Such methods essentially consist in performing conventional pushover analyses per each mode separately and then estimating the structural response by combining the action effects derived from each of the modal responses (alternatively, the “most critical mode” may be considered in isolation). Paret *et al.* (1996) first suggested the Multi-Modal Pushover procedure, which was then refined by Moghadam and Tso (2002). Chopra and Goel (2002), on the other hand, have developed and proposed a Modal Pushover Analysis (MPA) technique, which Hernández-Montes *et al.* (2004) have then adapted into an Energy-based Pushover formulation.

Although the aforementioned methods constitute a significant improvement over traditional techniques, they still do not account for the damage accumulation, and resulting modification of the modal parameters, that characterise structural response at increasing loading levels. The latter motivated the recent development and introduction of the so-called *Adaptive Pushover* methods whereby the loading vector is updated at each analysis step, reflecting the progressive stiffness degradation of the structure induced by the penetration in the inelastic range. These methods, also termed as *incremental response spectrum analysis* by some researchers (e.g. Aydinoglu, 2003), can evidently consider the effects of the higher modes and of the input frequency content.

Adaptive procedures have been proposed by Bracci *et al.* (1997), Sasaki *et al.* (1998), Satyarno *et al.* (1998), Matsumori *et al.* (1999), Gupta and Kunnath (2000), Requena and Ayala (2000), Elnashai (2001), Antoniou *et al.* (2002), Aydinoglu (2003). The methodologies elaborated by latter four are conceptually identical, with the difference that Elnashai (2001) and Antoniou *et al.* (2002) implemented the procedure within a fibre analysis framework, allowing for a continuous, rather than discrete, force distribution update to be carried out.

These adaptive procedures have led to an improvement in the agreement between static and dynamic analysis results, thanks to the consideration of: (i) spectrum scaling, (ii) higher modes contributions, (iii) alteration of local resistance and modal characteristics induced by the accumulated damage, (iv) load updating according to the eigen-solutions from instantaneous nonlinear stiffness and mass matrix. However, despite such apparent conceptual superiority, or at least despite its conspicuously more elaborated formulation, the improvement introduced by current force-based adaptive pushover procedures is not-necessarily impressive, with respect to its traditional non-adaptive counterparts, particularly in what concerns the estimation of deformation patterns of buildings, which are poorly predicted by both types of analysis (e.g. Kunnath and John, 2000; Antoniou and Pinho, 2004a; ATC, 2005).

As shown by Kunnath (2004) and López-Menjivar (2004), the main reason for such underperformance seems to be the quadratic modal combination rules (SRSS, CQC) used in computing the adaptive updating of the load vector; these rules will inevitably lead to monotonically increasing load vectors, since the possibility of sign change in applied loads at any location is precluded, whilst it may be needed to represent the uneven redistribution of forces after an inelastic mechanism is triggered at some location. It is thus perhaps equally evident that in order to overcome such limitations, alternative modal combination schemes should be derived and proposed. In their recent work, Kunnath (2004) and López-Menjivar (2004) make tentative proposals in such direction, which, however, cannot yet be considered as valid for general application.

2.3 Displacement-based Adaptive Pushover

Antoniou and Pinho (2004b) have proposed, as an alternative solution to the problems highlighted in the previous paragraphs, a paradigm shift in pushover analysis, by introducing the innovative concept of displacement-based pushover. Contrarily to what happens in non-adaptive pushover, where the application of a constant displacement profile would force a predetermined and possibly inappropriate response mode, thus concealing 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. And, indeed, this displacement-based pushover algorithm caters for the reproduction of reversal of storey shear distributions (Figure 1) even if a quadratic rule is employed to combine the contribution of the different modes, since the latter are here represented by their displacement vectors, with forces/shear coming as a result of the structural equilibrium to the applied displacement pattern.

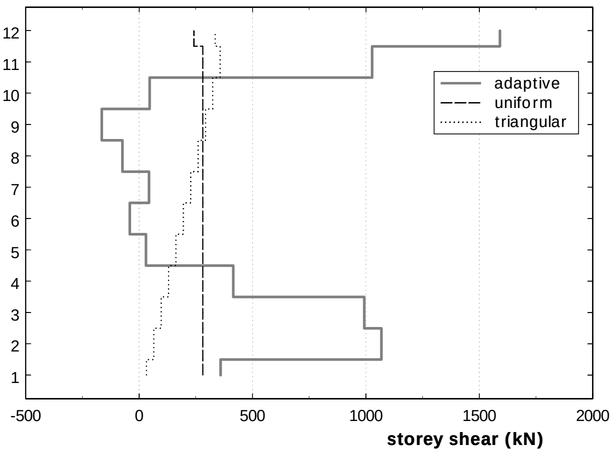


Figure 1. Storey shear distribution for a 12-storey building subjected to pushover analyses using (i) constant uniform force, (ii) constant triangular force and (iii) adaptive displacement loading vectors (Antoniou and Pinho, 2004b)

Hence, by adopting a displacement-based adaptive pushover, not only the attainment of more accurate results (deformation profiles and capacity curves) are warranted, but the entire structural assessment exercise becomes coherent with recent seismic design/assessment trends where the direct use of displacements, as opposed to forces, is preferred as a recognition of the conspicuous evidence that seismic structural damage is in fact induced by response deformations. In addition, and as far as the effort of the modeler/engineer is concerned, the additional modelling and computational effort requested to run such type of analysis is, with respect to conventional pushover procedures, negligible.

3. PARAMETRIC STUDY

In the current work, the innovative displacement-based adaptive pushover procedure (DAP) proposed by Antoniou and Pinho (2004b) is assessed through an analytical comparative study involving different pushover methods, either single or multi mode, adaptive or conventional, and dynamic nonlinear analysis of reinforced concrete buildings and bridges. The "true" dynamic response is deemed to be represented by the results of the Incremental Dynamic Analysis procedure (IDA) (e.g. Vamvatsikos and Cornell, 2002), which is a parametric analysis method by which a structure is subjected to a series of nonlinear time-history analyses of increasing intensity.

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. Due to the marked difference of the two structural typologies, observations and conclusions withdrawn from studies on the latter cannot really be extrapolated to the case of the former, and two different approaches are therefore employed in the subsequent sections. In the case of buildings, the study aimed to explore the potentials of alternative and more effective rules in the proposed adaptive algorithm; for this reason, attention focused on output obtained from single accelerograms, as opposed to the statistical average of all cases, in order to spot structural response peculiarities introduced by individual motions without being smoothed out through results averaging. The dearth of research data, within the framework of bridge applications, implied instead the need of a first comprehensive parametric study, whereby a suite of bridge configurations subjected to a large ensemble of seismic records is analysed in a more statistical perspective.

The Finite Elements Analysis package used in the present work, SeismoStruct (SeismoSoft, 2005), 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 structural configurations under static or dynamic loading, accounting for geometric nonlinearities and material inelasticity. Its accuracy in predicting the seismic response of building and bridge structures has been demonstrated through comparisons with experimental results derived from pseudo-dynamic tests carried out on full or large-scale models (e.g. Pinho and Elnashai, 2000; Casarotti, 2004). Further, the package features also the readily availability of the displacement-based adaptive pushover algorithm employed in this study.

3.1 Building Parametric Study

Three different configurations of common RC structures were employed: a 12 storey regular frame, an eight storey irregular frame and a dual wall-frame system. The latter are based on buildings previously designed for different ductility classes and design ground acceleration, on medium soil type ‘B’ of EC8 (Fardis, 1994), resulting in a total of 12 models, as described in Table 1. The overall plan dimensions of the three configurations are 15m by 20m. The storey height is 3m except the first storey of the irregular set, which is 4.5m high. A detailed description of models and load conditions, as well as of their FE modelling, can be found in López-Menjivar (2004).

Table 1. Considered building systems

Structural System	Storeys (Height)	Structure Reference	Ductility Level	Design PGA (g)	Behavior Factor (q)	$T_{uncracked}$ (s)
Regular Frame	12 (36 m)	RH30	High	0.30	5.00	0.697
		RM30	Medium		3.75	0.719
		RM15	Medium	0.15	3.75	0.745
		RL15	Low		2.50	0.740
Irregular Frame	8 (25.5 m)	IH30	High	0.30	4.00	0.565
		IM30	Medium		3.00	0.536
		IM15	Medium	0.15	3.00	0.613
		IL15	Low		2.00	0.614
Regular Wall-Frame	8 (24 m)	WH30	High	0.30	3.50	0.569
		WM30	Medium		2.625	0.557
		WM15	Medium	0.15	2.625	0.601
		WL15	Low		1.75	0.588

Four input time-histories, consisting of one-artificially generated accelerogram (A975) and three natural records (Loma Prieta earthquake, USA, 1989), were employed: the selection of these four records aimed at guaranteeing a wide-ranging type of earthquake action, in terms of frequency content, peak ground acceleration, duration and number of high amplitude cycles (Antoniou *et al.*, 2002). Upper and lower bounds of the main characteristics of the records are summarised in Table 2, where the significant duration is defined as the interval between the build up of 5% and 95% of the total Arias Intensity (Bommer and Martinez-Pereira, 1999).

Table 2. Bounding characteristics of the employed set of records for buildings

	Peak Ground Acceleration	Peak Response Acceleration	5% Arias Intensity threshold	Significant Duration t_{eff}	Total Duration t_{tot}	t_{eff} / t_{tot}
Min	0.12 g	0.50 g	1.02 s	7.24 s	10.0 s	22.3%
Max	0.93 g	4.25 g	11.23 s	10.43 s	40.0 s	72.4%

3.1.1 Analyses and result post-processing

The two non-adaptive pushover schemes, proposed in the NEHRP Guidelines (ATC, 1997), were applied to each set of buildings: the uniform distribution, whereby lateral forces are proportional to the total mass at each floor level, and the triangular distribution, in which seismic forces are proportional to the product of floor mass and storey height. The adaptive pushover algorithm was used in both its force and displacement-based variants, with spectrum scaling, employing SRSS or CQC modal combination rules.

It is noteworthy that the DAP procedure employed in this building parametric study made use of the interstorey drift-based scaling algorithm, whereby maximum interstorey drift values obtained directly from modal analysis, rather than from the difference between not-necessarily simultaneous maximum floor displacement values, are used to compute the scaling displacement vector. This comes as a reflection of the fact that the maximum displacement of a particular floor level, being essentially the relative displacement between that floor and the ground, provides insufficient insight into the actual level of damage incurred by buildings subject to earthquake loading. On the contrary, interstorey drifts, obtained as the difference between floor displacements at two consecutive levels, feature a much clearer and direct relationship to horizontal deformation demand on buildings. Readers are referred to the work of Antoniou and Pinho (2004b) for further details on this formulation.

The inter-storey drift profiles obtained from each pushover analysis are compared to the drift profiles from the nonlinear dynamic analysis and the standard error of the pushover results, with respect to the dynamic, is calculated as:

$$\mathrm{Error}(\%) = 100\sqrt{\frac{1}{n}\sum_{i=1}^{n}\left(\frac{\Delta_{iD} - \Delta_{iP}}{\Delta_{iD}}\right)} \tag{5}$$

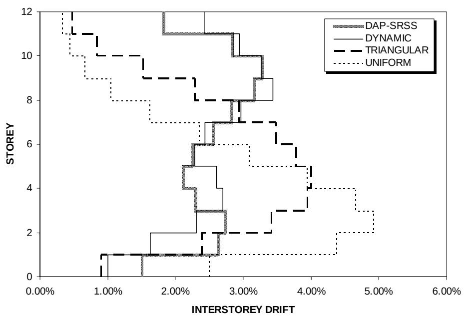
The interstorey drift profiles are monitored at four different deformation levels: the pre-yield state (0.5% total drift), the point of global yielding

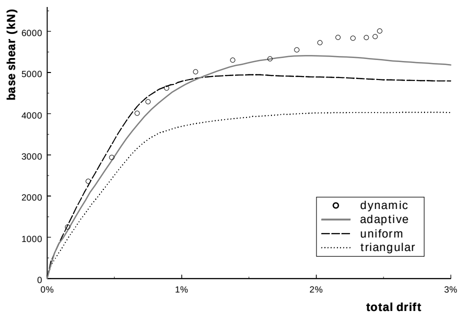
(1.0% and 1.5%), where the stiffness changes significantly and the local distributions are rapidly updated, and the deeply inelastic range (2.5%).

The Standard Error of the non-adaptive and adaptive pushover schemes was computed for all the structures and earthquakes that the authors used in their research. In order to spot the presence of possible response peculiarities introduced by individual input motions but smoothed out through results averaging, the standard error is given separately for each time history analysis, as a unique value, averaging the standard error of all the storeys, in the building, and deformation levels.

3.1.2 Obtained results

The Mean Standard Error of the DAP, FAP, Triangular and Uniform pushovers, considering all structures and ground motions, are 19.11%, 30.90%, 21.11% and 38.76%, respectively. These overall results seem to indicate only a marginal advantage of DAP with respect to non-adaptive triangular distribution. However, a closer inspection of interstorey drift profiles (Figure 2a) and capacity curves (Figure 2b) for some particularly difficult cases, renders much more conspicuous the gains provided by the employment of displacement-based adaptive pushover in the prediction of the seismic demand/capacity of framed buildings subjected to seismic action.





(a) (b)

Figure 2. Representative results obtained with model RM15 subjected to one of the natural accelerograms employed in this study (Hollister)

3.2 Bridges Parametric Study

The parametric study has considered two bridge lengths (50 m spans), with regular, irregular and semi-regular layout of the pier heights and with two types of abutments; (i) continuous deck-abutment connections supported on piles, exhibiting a bilinear behaviour, and (ii) deck extremities supported on pot bearings featuring a linear elastic response. The total number of bridges is therefore twelve, as shown in Figure 3, where the label numbers 1, 2, 3 characterise the pier heights of 7 m, 14 m and 21 m, respectively.

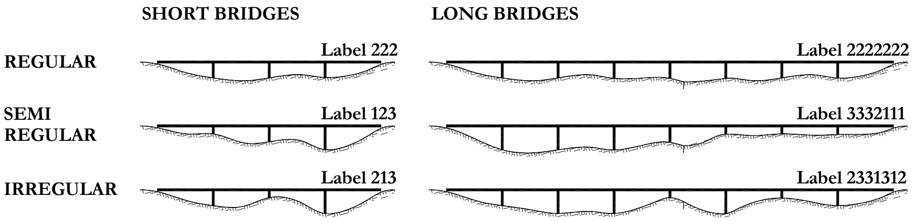


Figure 3. Analysed Bridge Configurations

A sufficiently large number of records has been employed so as to bound all possible structural responses. The employed set of seismic excitation is defined by an ensemble of 14 large magnitude (6-7.3) small distance (13-30 km) records selected from a suite of historical earthquakes scaled to match the 10% probability of exceedance in 50 years uniform hazard spectrum for Los Angeles (SAC Joint Venture, 1997). The bounding characteristics of the records are summarized in Table 3. Further details on modelling and input can be found in Casarotti (2004).

Table 3. Bounding characteristics of the employed set of records for bridges

	Peak Ground Acceleration	Peak Response Acceleration	5% Arias Intensity threshold	Significant Duration t_{eff}	Total Duration t_{tot}	t_{eff} / t_{tot}
Min	0.30 g	0.84 g	1.25 s	5.3 s	14.95 s	9%
Max	1.02 g	3.73 g	12.5 s	19.66 s	80.00 s	52%

3.2.1 Analyses and result post-processing

The response of the bridge models is estimated through the employment of Incremental Dynamic Analysis (IDA), Force-based Conventional Pushover with uniform load distribution (FCPu), Force-based Conventional Pushover with first mode proportional load pattern (FCPm), Force-based Adaptive Pushover with Spectrum Scaling (FAP) and Displacement-based Adaptive Pushover with Spectrum Scaling (DAP). Results are presented in terms of the bridge capacity curve, i.e. a plot of the reference point displacement versus total base shear, and of the deck drift profile.

Each level of inelasticity is represented by the deck centre drift, selected as independent damage parameter, and per each level of inelasticity the total base shear V_{base} and the displacements Δ_i at the other deck locations are monitored. Results of pushover analyses are compared to the IDA median value out of the responses to the 14 records, of each response quantity R, be it total base shear or deck drift:

$$\hat{R}_{i,IDA} = \text{median}_{j=1:14}\left[R_{i,j-IDA}\right] \tag{1}$$

Pushover analyses with spectrum scaling (i.e. adaptive pushovers) 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. Finally, 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 4, and translated in Eq. (2). Representing results in terms of ratios between the "approximate" and the "exact" procedures, immediately indicates the bias in the approximate procedure, as the ideal target value of the different pushovers is always one.

$$\overline{R}_{i,PUSHOVERtype} = \frac{R_{i,PUSHOVERtype}}{\hat{R}_{i,IDA}} \cdots \xrightarrow[\text{ideally}]{} 1 \tag{2}$$

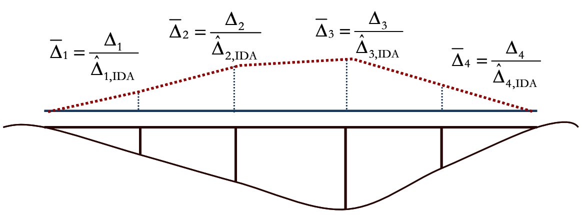


Figure 4. Normalised Transverse Deformed Pattern

Given that fact that a "realistic" capacity curve does not imply reliable estimations of the inelastic displacement pattern at increasing levels of inelasticity, the control of the deformed pattern is of the same relevance of the capacity curve prediction.

Having the same unitary target value, all normalized deck displacements become comparable, and a bridge index BI can measure the precision of the obtained deformed shape. Per each level of inelasticity, such bridge index is defined as the median of results over the m deck locations (Eq. 3a), with the standard deviation measuring the dispersion with respect to the median (Eq. 3b). The latter indicates the stability of the estimate of displacements along the deck: a small scatter means that predicted normalised displacements along the deck are averagely close to their median value BI.

$$BI_{PUSHOVERtype} = median_{i=1:m}(\overline{\hat{\Delta}}_{i,PUSHOVERtype}) \tag{3a}$$

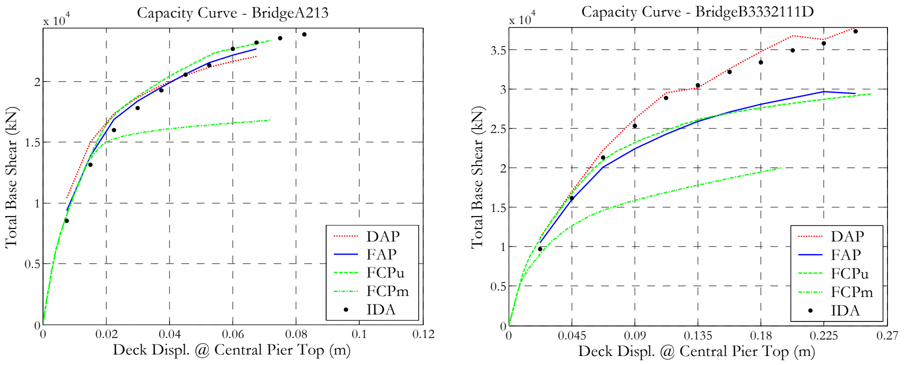
$$\delta_{PUSHOVERtype} = \sqrt{\frac{\sum_{i=1}^{m}\left(\overline{\hat{\Delta}}_{i,PUSHOVERtype} - BI_{PUSHOVERtype}\right)^2}{m-1}} \tag{3b}$$

3.2.2 Obtained results

Current code recommendations require performing pushover analysis by pushing the entire structure with distributed load. In case of bridges, the additional option of pushing only the deck has been investigated, observing that the superstructure is the physical location where the most of the structural mass, i.e. the source of the inertia forces on the bridge, is usually concentrated and where it is relatively free to be excited. A preliminary investigation indicated a significant improvement in terms of stability and velocity of analysis in case of DAP and a very poor influence on results with the application of the latter option, which is thus recommended and employed in the parametric study.

Two main pertinent observations can be withdrawn from a scrutiny of the capacity curves obtained by the different pushover analyses in Figure 5: first, FCPm tends to significantly underestimate the structural stiffness, mainly due to the fact that, for the same base shear, central deck forces are generally higher compared to the other load patterns, thus results in larger displacement at that location. Then, on occasions, a “hardening effect” in the pushover curve occurs, which is sometimes reproduced only by employing DAP: once piers saturate their capacity, abutments absorb the additional seismic demand, proportionally increasing shear response and hardening the capacity curve.

In Figure 6, the Bridge Index, as computed at each level of deck centre drift, is plotted as black filled marks so as to cater for an easier comparison with the IDA-normalised deck displacements, represented as empty marks in the background. In this manner, it results immediately apparent the level with which each pushover analysis is able to capture the deformed pattern of the whole bridge, at increasing deformation levels. For the sake of succinctness, only two analysis types are considered, FCPm and DAP, which are those leading to the worst and best predictions, respectively.



(a) (b)

Figure 5. Capacity curve results

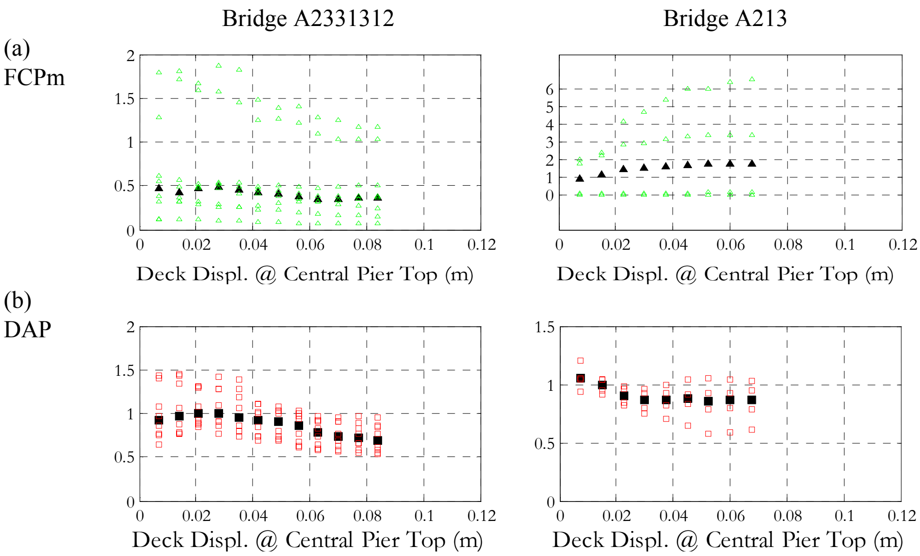


Figure 6. Prediction of the deformed pattern: BI and relative scatter

Table 4 provides global averages of means, maximum and minimum values of BI and respective dispersion as well as of the normalised total base shear, over the whole bridge ensemble. It is noted that FCPm heavily underestimates predictions, featuring also a very high BI dispersion value, (ii) FCPu performs very well for regular bridges and underestimating otherwise, (iii) DAP features the best overall behaviour, despite the slight underprediction of deformed shape values, with the lowest values of scatter.

Table 4. Global averages of the summaries of results

Means	Bridge Index			Dispersion			Normalised Base Shear		
	mean	min	max	mean	min	max	mean	min	max
FCPm	0.74	0.57	0.92	0.79	0.58	1.00	0.80	0.69	0.95
FCPu	0.87	0.75	1.03	0.24	0.17	0.34	1.03	0.92	1.18
FAP	0.88	0.78	1.01	0.22	0.13	0.34	0.99	0.89	1.10
DAP	0.87	0.78	0.99	0.19	0.14	0.27	1.03	0.95	1.13

4. CONCLUDING REMARKS

Given that current performance-based design trends require simple, yet accurate methods for estimating seismic demand on structures considering their full inelastic behaviour, in the current work the effectiveness of pushover analysis applied to buildings and bridges has been investigated. In particular, the effectiveness of applying a displacement-based adaptive pushover to estimate the seismic response of buildings and bridges subjected to earthquake action was investigated.

It was observed that the employment of such an innovative adaptive pushover technique lead to the attainment improved response predictions, throughout the entire deformation range, in comparison to those obtained by force-based methods, either adaptive or conventional. Indeed, prediction of the global behaviour (capacity curves), as well as of the deformed shapes and shear/moment distributions, proved to be very effective.

In other words, within the scope of buildings and bridge applications, 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 observe that a static procedure will never be able to completely replace a dynamic analysis; nevertheless, a methodology has been searched to obtain response information reasonably close to that predicted by the nonlinear dynamic analyses. The innovative displacement-based adaptive pushover method is therefore shown to constitute an extremely appealing displacement-based tool for structural assessment, fully in line with the recently introduced deformation- and performance-oriented trends in the field of earthquake engineering.

Of equally noteworthy status is perhaps the fact that the proposed adaptive pushover schemes are as simple to use as standard pushover methods and have been implemented in an Internet-downloadable Finite Element program, adequate for general usage, and thus rendering the presented analytical methodologies readily available to the practicing and research communities.

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