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DISPLACEMENT-BASED ADAPTIVE PUSHOVER

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Abstract. *A number of recent studies raised doubts on the effectiveness of conventional pushover methods, whereby a constant incremental force vector is applied to the structure, in estimating the seismic demand/capacity of framed buildings subjected to earthquake action. The latter motivated the recent development of the so-called adaptive pushover methods whereby the loading vector is updated at each analysis step, considering one or more response modes, reflecting in this way the effects that damage progression have on the response characteristics of structures subjected to increasing loading levels.*

Within such adaptive framework, the application of a displacement incremental loading vector becomes not only feasible but also advantageous since it leads to superior response predictions, with little or no additional modelling/analysis effort, in comparisons with conventional pushover procedures. In addition, such single-run Displacement-based Adaptive Pushover (DAP) method features also the advantage, with respect to recently proposed multiple-run alternatives (e.g. MPA, AMCP), of leading to structural response estimates that correspond to equilibrated structural stress-states.

The improvements in accuracy gained by the employment of DAP have been scrutinised and confirmed through extensive comparisons with the results of nonlinear dynamic analysis, commonly considered as the most accurate tool to estimate seismic demand/capacity of structures. A multitude of case-studies were considered, including non-seismically and seismically designed buildings and bridges, reinforced concrete and steel constructions, 2D and 3D modelling scenarios, all of which subjected to tens of earthquake records.

In addition, and in order to enable the incorporation of this innovative pushover algorithm within design/assessment codes and guidelines, a complete Nonlinear Static Procedure (NSP), involving thus also the calculation of target displacement, that capitalises fully on the potential of the DAP algorithm has been developed and proposed; Adaptive Capacity Spectrum Method (ACSM). The results obtained with the latter, and its comparison with other NSPs (CSM, N2, MPA, AMCP), are also referred to in this work.

1 INTRODUCTION

Nonlinear dynamic analysis is arguably considered as the most accurate method for assessing the response of structures subjected to earthquake action. Indeed, any type of static analysis will always be inherently flawed, given the conspicuous absence of time-dependent effects. However, as noted by Goel and Chopra [1], amongst others, such type of analysis is not without its difficulties or drawbacks, particularly for what concerns application within a design office environment.

Firstly, in order to employ dynamic analysis for seismic design/assessment of structures, an ensemble of site-specific ground motions compatible with the seismic hazard spectrum for the site must be simulated. As described by Bommer and Acevedo [2], amongst others, this is, however, a far from simple task, since seismic design codes feature insufficient or inadequate guidance on procedures to either (i) generate artificial spectrum-compatible records, (ii) produce synthetic accelerograms from seismological models or (iii) select appropriate suites of real acceleration time-series, eventually modified to better fit a given code response spectrum. It is believed that until better guidance on record selection/generation will be made available to earthquake engineer designers, this first step will remain as a very difficult-to-overcome hurdle to the use of dynamic time-history analysis in design office applications.

Secondly, notwithstanding the significant increase in computing power witnessed in recent years, nonlinear time-history analysis remains computationally demanding, especially when fibre-based (distributed inelasticity) structural analysis programs, which are simpler to calibrate than their plastic-hinge (concentrated plasticity) counterparts, are employed to model the seismic response of large multi-storey irregular buildings, requiring 3D models with thousands of elements. This problem becomes even the more significant if one considers that the analyses will need to be repeated a significant amount of times, not only because design codes or guidance documents request for a relatively large number of earthquake records to be employed in order to warrant minimum probabilistic validity of the results, but also, and perhaps mainly, because the process of analysing any given structure is invariably an iterative one, given that modelling errors are commonly encountered as the design/assessment process evolves.

Thirdly, even in those situations where the expertise and resources for running time-history analyses are available, it is often the case that preliminary simpler analysis (i.e. modal and static analyses) are run to enable a first check of the model; errors in the definition/assemblage of a finite elements model are difficult to detect from dynamic analysis results, whilst they tend to be relatively evident from the output of eigenvalue or pushover runs. As an example, inspection of the first modes of vibration of a given building model may be used to check if member stiffness has been correctly allocated or if the mass has been appropriately distributed, whilst examination of a force-displacement monotonic capacity curve may serve to quickly assess if member strength and ductility has been properly assigned. Static analyses, even if representing simplified methods, also provide important structural response information such as (i) identification of critical regions, where large inelastic deformations may occur, (ii) individuation of strength irregularities in plan and elevation that might cause important changes in the inelastic dynamic response characteristics, (iii) evaluation of the force demand in potentially brittle elements, and (iv) prediction of the sequence of yielding and/or failure of structural members. In addition, the explicit insight that pushover-derived base shear vs. top displacement capacity curves provide into the stiffness, strength and ductility of a given structure, constitutes the type of qualitative data that is always most informative and useful within a design application, even when time-history analysis is then employed for the definitive verifications.

The above constitute, in the opinion of the authors, strong reasons for nonlinear static analysis methods to continue to be developed and improved, so that these tools can become even more reliable and useful when employed either as a replacement to time-history analysis in the seismic design/assessment of relatively simple non-critical structures, or as a complement to dynamic analysis of more complex/critical facilities.

1.1 Definition and Scope

The term pushover analysis describes a modern variation of the classical collapse analysis method, as fittingly described by Kunnath [3]. It refers to an analysis procedure whereby an incremental-iterative solution of the static equilibrium equations is carried out to obtain the response of a structure subjected to 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. In this manner, each point in the resulting displacement vs. base shear capacity curve represents an effective and equilibrated stress-state of the structure, i.e. a state of deformation that bears a direct correspondence to the applied external force vector.

Within the framework of earthquake engineering, pushover analysis is employed with the objective of deriving, with relative ease, an envelope of the response parameters that would otherwise be obtained through a much more complex and time-consuming Incremental Dynamic Analysis (IDA) procedure, as can be construed by Figure 1. IDA is a parametric analysis method by which a structure is subjected to a series of nonlinear time-history analyses of increasing intensity [4], with the objective of attaining an accurate indication of the "true" dynamic response of a structure subjected to earthquake action.

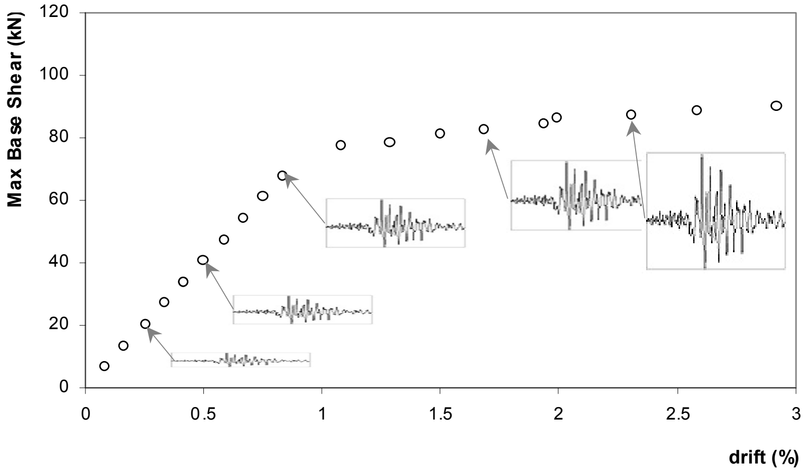


Figure 1: Maximum base-shear and top displacement values obtained with incremental dynamic analysis.

Recent years have also witnessed the development and introduction of an alternative type of nonlinear static analysis, which involves 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). Paret et al. [5] first suggested the Multi-Modal Pushover procedure, which was then refined by Moghadam and Tso [6]. Chopra and Goel [7], on the other hand, have developed and proposed a Modal Pushover Analysis (MPA) technique, which Hernández-Montes et al. [8] have then adapted into an Energy-based Pushover formulation. A further refinement of such multiple-pushover procedures, with the aim to account for the alteration of local resistance and modal characteristics of the structure induced by the accumulation of damage, consists in the employment of adaptive updating of the loading pattern [9,10], effectively meaning that the methods may now be categorized as piecewise linear response spectrum analysis. As highlighted by 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. The associated drawback, however, is that the methods are inevitably more complex than running a single pushover analysis, as noted by Maison [11]. Furthermore, some of the proposed "multiple-run" procedures, either they have an adaptive or non-adaptive nature, lead to difficulties when applied within capacity-spectrum type of procedure due to the need to handle capacity curves associated with higher mode force patterns that display a reversal of the roof displacement as inelasticity develops in the structure [8,12,13]. For all of the above, these multiple-pushover based approaches do not constitute the scope of the current presentation, where focus is placed instead on "single-run" pushover analysis procedures.

In tandem with the present drive for performance-based seismic engineering, there is also currently a thrust for the development and code implementation of displacement or, more generally, deformation-based design and assessment methods. Therefore, it would seem that applying displacement loading, rather than force actions, in pushover procedures would be an appropriate option for nonlinear static analysis of structures subjected to earthquake action. However, due to the unvarying nature of the applied displacement loading vector, conventional (non-adaptive) displacement-based pushover analysis can conceal important structural characteristics, such as strength irregularities and soft storeys, should the displacement pattern adopted at the start of the analysis not correspond to the structure's post-yield failure mechanism. Consequently, when only non-adaptive static nonlinear analysis tools are available, as has been the case throughout the past, force-based pushover does constitute a preferable choice over its displacement-based counterpart.

On the other hand, however, if one is able to apply displacements, rather than forces, in an adaptive fashion, that is, with the possibility of updating the displacement loading pattern according to the structural properties of the model at each step of the analysis, then a conceptually appealing deformation-based nonlinear static analysis tool is obtained.

1.2 Recent developments in single-run pushover analysis

According to recently introduced code provisions, such as FEMA-356 [14] and Eurocode 8 [15], 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 report [12], 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.

In Figure 2 and Figure 3, examples of inadequate prediction of both the capacity curve as well as the deformation response characteristics of a 12-storey reinforced concrete frame subjected to a natural earthquake recording (case-study RM15-NR2 in Antoniou and Pinho [16]) and of a 4-storey irregular frame subjected to an artificial accelerogram (ICONS full-scale test specimen, described in Pinho and Elnashai [17]) are given. It is noted that although the 12-storey building is regular in height, its response is heavily influenced by higher mode effects, effectively rendering its seismic behaviour highly irregular in height, as conspicuously shown by Figure 3a. The standard pushover results have been carried out using both triangular and uniform loading distributions, and are compared with the envelope of results obtained with incremental dynamic analysis.

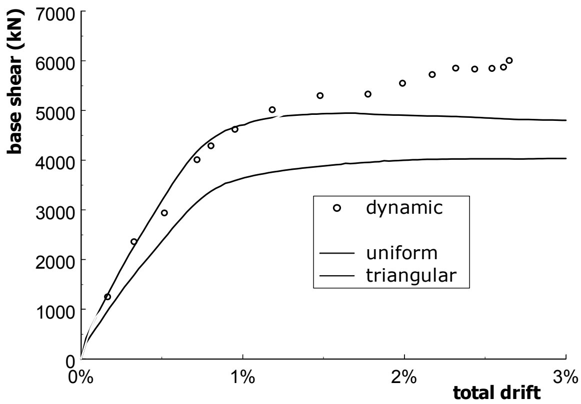
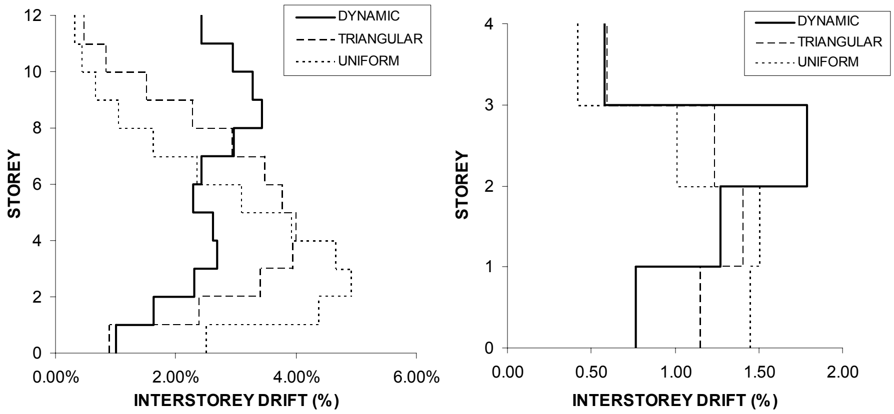


Figure 2: Capacity curves of a 12-storey building, obtained with standard pushover.



(a) (b)

Figure 3: Interstorey drift profiles of (a) 12-storey building and (b) 4-storey irregular frame, obtained with standard pushover.

The main reason behind the underperformance of these conventional pushover methods is the fact that they do not account for the effect that damage accumulation, induced by the increasing deformation levels imposed on the structure, has on the response of the latter. Cumulative material straining introduces a reduction in stiffness, which, in turn, causes an elongation of the periods of vibration (Figure 4), which then, depending on the shape of the response spectrum being considered (or on the frequency content of an input record), may trigger significant changes in the response characteristics of the buildings (Figure 5). Krawinkler and Seneviratna [18] 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.

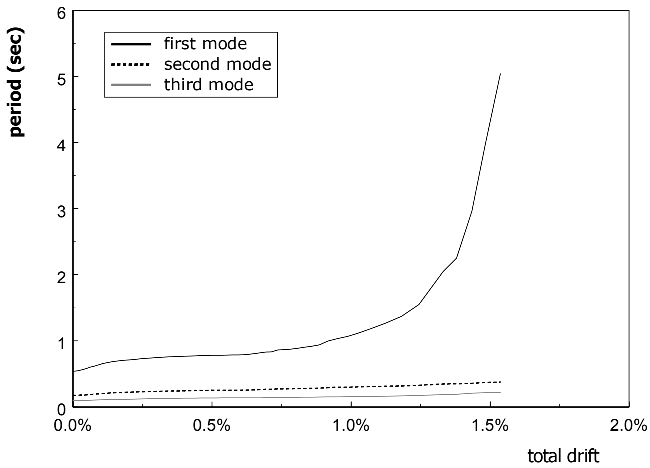


Figure 4: Periods of vibration of 4-storey building under increasing levels of deformation.

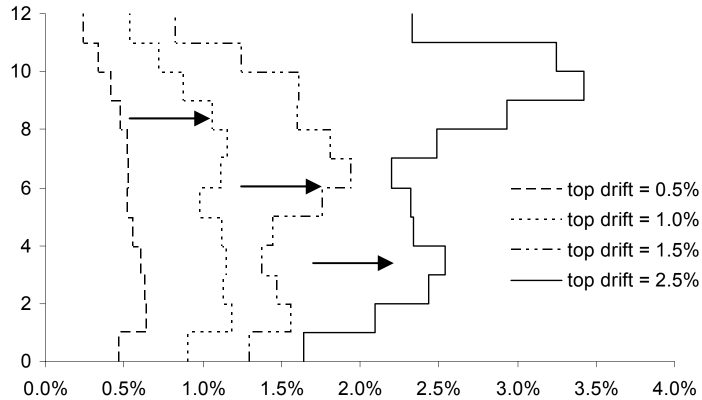


Figure 5: Interstorey drift profiles of a 12-storey building subjected to increasing levels of deformation.

As a result of the aforementioned limitations, recent years have witnessed the development and introduction of 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 (Figure 6). The response of the structure is thus computed in incremental fashion, through piecewise linearization (Figure 7), hence rendering it possible to use the tangent stiffness at the start of each increment, together with the mass of the system, to compute modal response characteristics of each incremental pseudo-system through elastic eigenvalue analysis, and use such modal quantities to congruently update (i.e. increment) the pushover loading vector.

Force-based adaptive pushover procedures have been proposed by Reinhorn [19], Bracci et al. [20], Satyarno et al. [21], Requena and Ayala [22], Elnashai [23] and Antoniou et al. [24]. With the exception of the work of Satyarno et al. [21], where a single mode adaptive pushover pattern was employed, all other adaptive methodologies considered the effects of the higher modes and of the input frequency content. Furthermore, Elnashai [23] and Antoniou et al.[24] implemented their adaptive algorithm within a fibre analysis framework, allowing for a continuous, rather than discrete, force distribution update to be carried out.

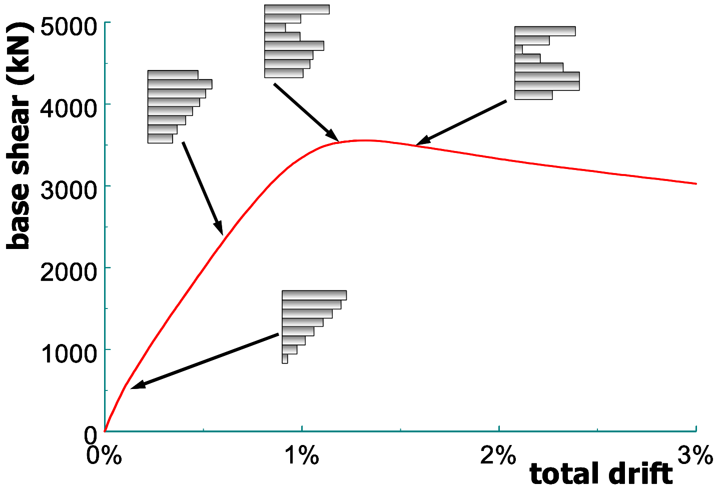


Figure 6: Adaptive pushover: shape of loading vector is updated at each analysis step.

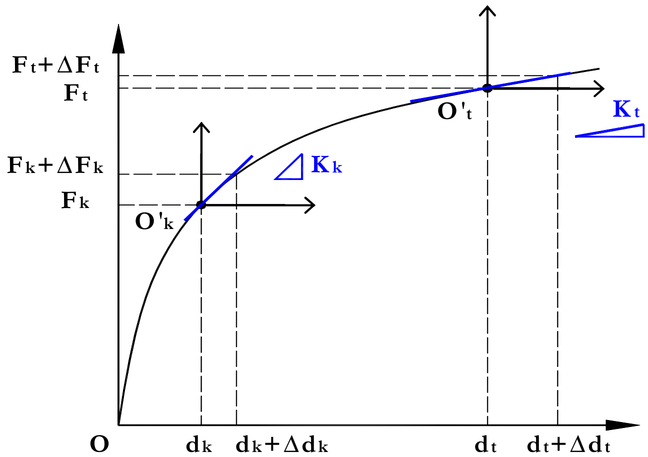


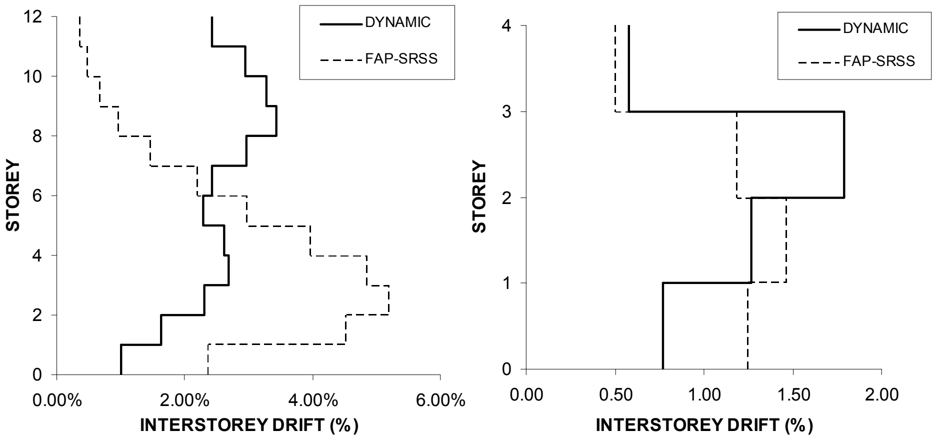
Figure 7: The use of tangent stiffness in updating (i.e. incrementing) the loading vector.

Despite their apparent conceptual superiority, or at least despite their conspicuously more elaborated formulation, the improvement introduced by such Force-based Adaptive Pushover (FAP) procedures was not-necessarily impressive, with respect to its traditional non-adaptive counterparts, particularly in what concerns the estimation of deformation patterns of buildings, which seemed to be poorly predicted by both types of analysis, as shown in Figure 8. As described by Kunnath [3] and Antoniou and Pinho [16], the main reason for such underperformance seems to be the quadratic modal combination rules (e.g. SRSS, CQC) used in computing the adaptive updating of the load vector; such 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.

With the above in mind, Kunnath [3] and López-Menjivar [25] have proposed an alternative modal combination scheme, consisting of a weighted Direct Vectorial Addition (DVA) of the different modal shapes that can be mathematically expressed as:

$$F_i = \sum_{j=1}^{n} \alpha_j \Gamma_j \phi_{j,i} M_j Sa_j \qquad (1)$$

where i is the storey number, j is the mode number, n is the highest mode of interest, Γj is the

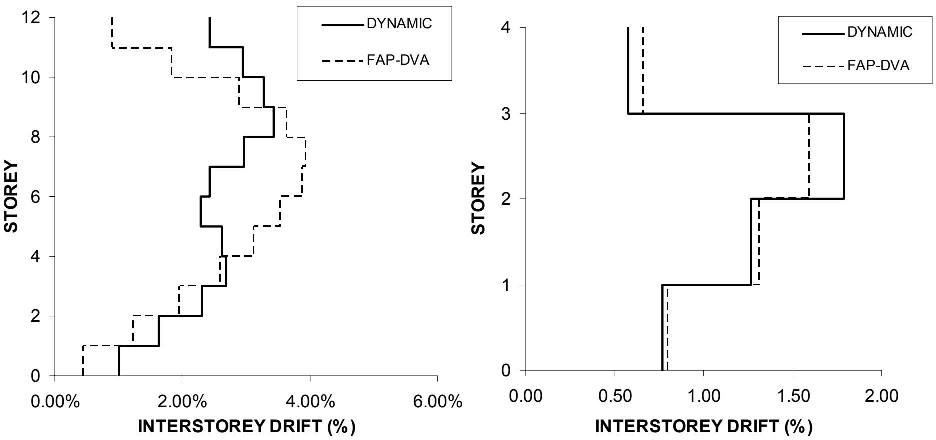


(a) (b)

Figure 8: Interstorey drift profiles of (a) 12-storey building and (b) 4-storey irregular frame, obtained with Force-based Adaptive Pushover using SRSS modal combination.

modal participation factor for the jth mode, ϕi,j is the mass normalised mode shape value for the ith storey and the jth mode, Mi is the mass of the ith storey and Saj represents the acceleration response spectrum ordinate corresponding to the period of vibration of the jth mode. Finally, αj is a weighting factor that aims at accounting for the varying relative importance that each mode j has on the maximum response of the structure.

The employment of such alternative modal combination procedure, may indeed lead to the attainment of improved results, as demonstrated by the interstorey drift profiles given in Figure 9, obtained through consideration of the first three modes of vibration of the buildings, and using $\alpha_1 = 1.0$, $\alpha_2 = -1.0$ and $\alpha_3 = 1.0$ in Eq. (1). However, the arbitrary nature of these weighting factors αj renders the method unfeasible for practical application, as explicitly acknowledged in Kunnath [3] and demonstrated in López-Menjivar [25]. Indeed, in the latter work it is demonstrated how values of αj that lead to optimum results for some building configurations, lead then to poor predictions in buildings with diverse characteristics. Therefore, and until a general procedure to correctly determine the values of the weighting factors is found, the DVA adaptive pushover modality cannot really be deemed as a valid solution for practical application.



(a) (b)

Figure 9: Interstorey drift profiles of (a) 12-storey building and (b) 4-storey irregular frame, obtained with Force-based Adaptive Pushover using DVA modal combination.

2 DISPLACEMENT-BASED ADAPTIVE PUSHOVER (DAP)

With a view to overcome the limitations described above, Antoniou and Pinho [26] have proposed a paradigm shift in pushover analysis, by introducing the innovative concept of Displacement-based Adaptive Pushover (DAP). 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 concentrating 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.

2.1 Methodology

The implementation of DAP can be structured in four main stages; (i) definition of the nominal load vector and inertia mass, (ii) computation of the load factor, (iii) calculation of the normalised scaling vector and (iv) updating of the loading displacement vector. Whilst the first step is carried out only once, at the start of the analysis, its three remaining counterparts are repeated at every equilibrium stage of the nonlinear static analysis procedure, as described in the following subsections.

The loading vector shape is automatically defined and updated by the solution algorithm at each analysis step, for which reason the nominal vector of displacements, U0, must always feature a uniform (rectangular) distribution shape in height, so as not to distort the load vector configuration determined in correspondence to the dynamic response characteristics of the structure at each analysis step. In addition, it is noteworthy that the adaptive pushover requires the inertia mass M of the structure to be modelled, so that the eigenvalue analysis, employed in updating the load vector shape, may be carried out.

The magnitude of the load vector U at any given analysis step is given by the product of its nominal counterpart U_0, defined above, and the load factor λ at that step (see Eq. 2). The latter is automatically increased, by means of a load control strategy (Antoniou and Pinho, [16]), until a predefined analysis target, or numerical failure, is reached.

$$U = \lambda \cdot U_0 \tag{2}$$

The normalized modal scaling vector, $\bar{D}$, used to determine the shape of the load vector (or load increment vector) at each step, is computed at the start of each load increment. In order for such scaling vector to reflect the actual stiffness state of the structure, as obtained at the end of the previous load increment, an eigenvalue analysis is carried out. To this end, the Lanczos algorithm [27] is employed to determine the modal shape and participation factors of any given predefined number of modes. Modal loads can be combined by using either the Square Root of the Sum of the Squares (SRSS) or the Complete Quadratic Combination (CQC) methods.

Since application to the analysis of buildings is the scope of the present work, use is made 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 Antoniou [28] for further details on this formulation.

In such an interstorey drift-based scaling technique, the eigenvalue vectors are thus employed to determine the interstorey drifts for each mode Δ_{ij}, as shown in Eq. 3, while the displacement pattern D_i at the ith storey is obtained through the summation of the modal-combined inter-storey drifts of the storeys below that level, i.e. drifts Δ_1 to Δ_i:

$$D_i = \sum_{k=1}^{i} \Delta_k \text{ with } \Delta_i = \sqrt{\sum_{j=1}^{n} \Delta_{ij}^2} = \sqrt{\sum_{j=1}^{n} \left[\Gamma_j \left(\phi_{i,j} - \phi_{i-1,j}\right)\right]^2} \tag{3}$$

Since only the relative values of storey displacements (D_i) are of interest in the determination of the normalised modal scaling vector $\overline{D}$, which defines the shape, not the magnitude, of the load or load increment vector, the displacements obtained by Eq. 3 are normalized so that the maximum displacement remains proportional to the load factor, as required within a load control framework:

$$\overline{D_i} = \frac{D_i}{\max D_i} \tag{4}$$

Once the normalized scaling vector and load factor have been determined, and knowing also the value of the initial nominal load vector, the loading vector U_t at a given analysis step t is obtained by adding to the load vector of the previous step, U_{t-1} (existing balanced loads), a newly derived load vector increment, computed as the product between the current load factor increment $\Delta\lambda_t$, the current modal scaling vector $\overline{D_t}$ and the nominal vector U_0, as mathematically translated into Eq. 5 and graphically depicted in Figure 10.

$$U_t = U_{t-1} + \Delta\lambda_t \cdot \overline{D_t} \cdot U_0 \tag{5}$$

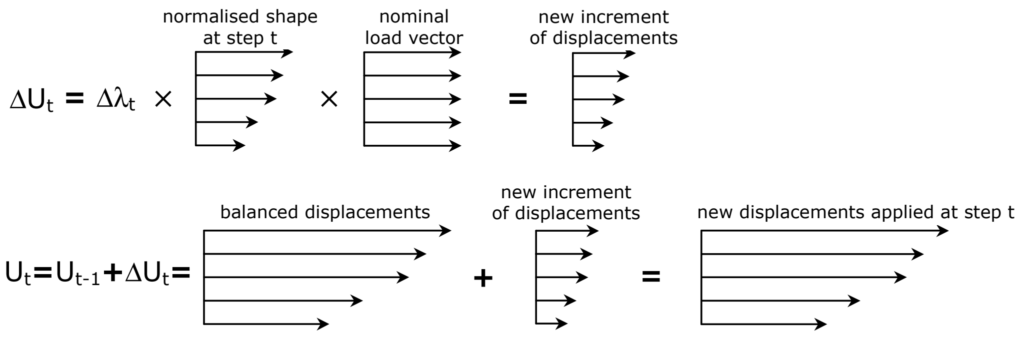


Figure 10: Updating of the loading displacement vector.

The DAP algorithm has been implemented in the computer code SeismoStruct [29], a fibre element-based program for seismic analysis of framed structures, downloadable from the internet. The program incorporates both local (beam-column effects) and global (large displacements/rotations effects) sources of geometric nonlinearity as well as the interaction between axial force and transverse deformation of the element. The spread of material inelasticity along the member length is explicitly represented through the employment of a fibre modelling approach, implicit in the formulation of the inelastic beam-column frame elements adopted in the analyses. Various verification studies have been carried out with the aforementioned program on a four-storey reinforced concrete frame (Figure 11), a reinforced concrete bridge (Figure 12a) and a two-storey steel frame (Figure 12b) all of which show the ability of the analytical models to replicate the seismic response of full-scale structures.

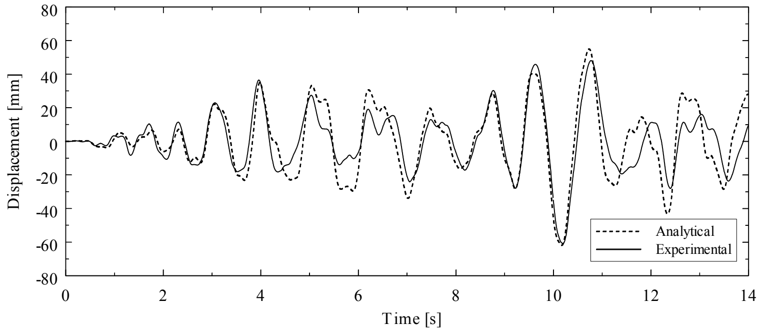
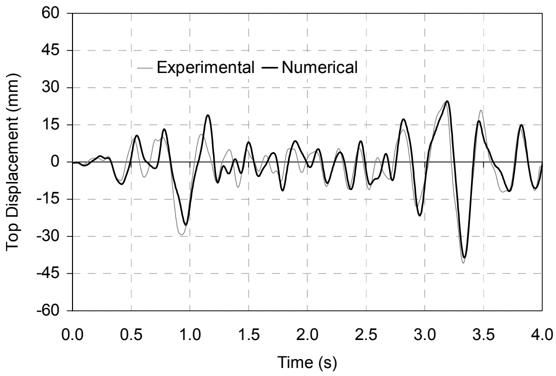
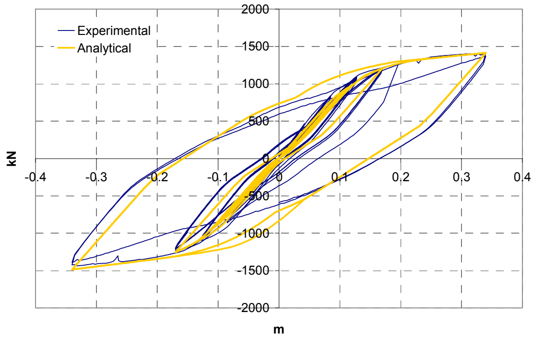


Figure 11: Verification of a fibre-element analytical model of a 4-storey RC frame (López-Menjivar [25]).



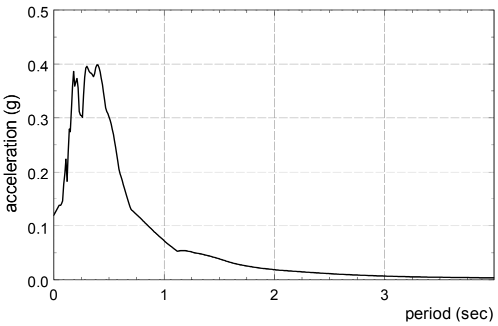


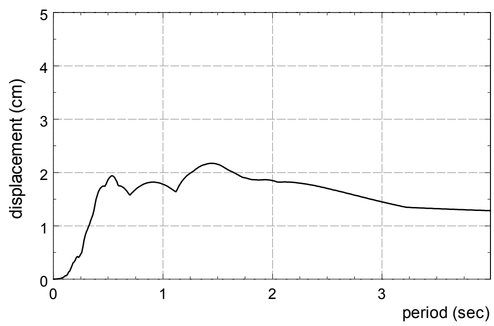
(a) (b)

Figure 12: Verification of fibre element analytical models of (a) a RC bridge (Casarotti and Pinho [30]) and (b) a 2-storey steel frame (Pietra et al [31]).

2.2 Illustrative results

Two clearly distinct building frames, both of which featuring an irregular type of dynamic response, are considered herein. The first of these is a 12-storey five-bay reinforced concrete structure designed according to Eurocode 8 [15]. It displayed a highly irregular dynamic behaviour (Figure 3) when subjected to an accelerogram (Hollister station, Loma Prieta earthquake, USA, 1989) that presented a very high amplification in the short-period and thus lead to a response very much dominated by the 2nd and 3rd modes of vibration. Indeed, and as can be observed in Figure 13, these two higher modes ($0.15 < T_2, T_3 < 0.30$ secs) feature a spectral amplification, in acceleration, that is ten times higher than that corresponding to first mode of vibration ($T_1 > 1.4$ secs). Further details on this case-study can be found in Antoniou et al. [24].



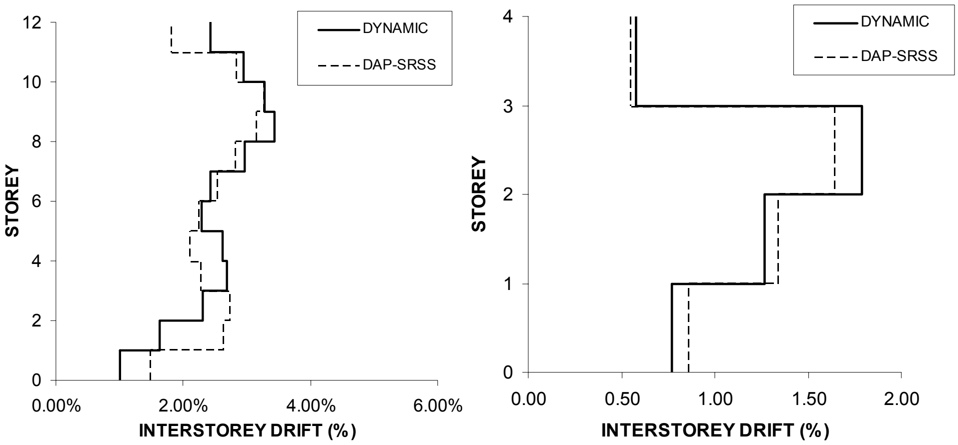


(a) (b)

Figure 13: (a) Acceleration and (b) displacement response spectra of accelerogram employed in the analysis of 12-storey building.

The second structure is a 4-storey three-bay building refers to a full-scale test specimen, built to represent typical design and construction practice in most South-European countries in the 1950's, and tested under pseudo-dynamic conditions (Pinho and Elnashai,[17]) at the JRC in Ispra (Italy). The frame was designed for gravity loads only, without any consideration of ductility provisions or capacity design principles. Consequently, it exhibited a soft-storey type of deformation mechanism at the third storey level (e.g. Figure 3) caused mainly by the drastic stiffness/strength variation present at such location, as well as by inadequate lap-splicing and defective column shear capacity. The input motion consisted of artificial accelerograms aiming at being representative of European seismicity (Campos-Costa and Pinto, [32]).

In Figure 14, the interstorey drift profiles of these two case-studies, as obtained with the employment DAP analyses, are given. It is observed that the predictions now match much closer the dynamic response of these two structures, which effectively means that the response irregularities caused by the flexibility of the 12-storey structure, and subsequent amplification of higher modes, as well as the strength irregularity of the 4-storey prototype, have been fully and correctly captured by the proposed static analysis algorithm.



(a) (b)

Figure 14: Interstorey drift profiles of (a) 12-storey building and (b) 4-storey irregular frame, obtained with Displacement-based Adaptive Pushover using SRSS combination.

In Figure 15, on the other hand, the capacity curves of the 12-storey building, as derived by both DAP and standard pushover curves are compared with the Incremental Dynamic Analysis envelope. The advantages of using an adaptive displacement-based pushover can be inferred also from this type of results.

The reason behind the most-improved predictions obtained with the displacement-based adaptive pushover procedure is the fact that storey forces or shears are no longer applied directly to the structure but rather come as a result of structural equilibrium to the applied displacement pattern, thus allowing for the reproduction of reversal of storey shear distributions, observed in dynamic analysis, even if a quadratic rule is employed to combine the contribution of the different modes. In effect, DAP drift profiles, despite carrying a permanently positive sign, do, in any case, feature changes of their respective gradient (i.e. the trend with which drift values change from one storey to the next), introduced by the contribution of higher modes. When such gradient variations imply a reduction of the drift of a given storey with respect to its adjacent floor levels, then the corresponding applied storey horizontal force must also be reduced, in some cases to the extent of sign inversion, as observed in Figure 16.

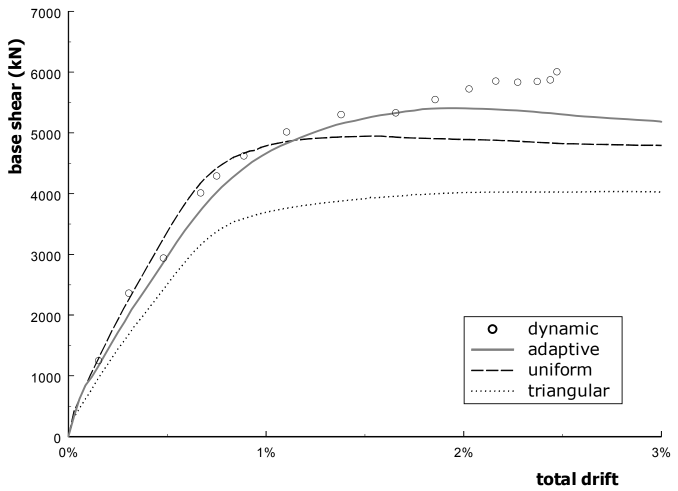


Figure 15: Capacity curves of a 12-storey building, obtained with DAP and standard pushovers, and compared against IDA envelopes.

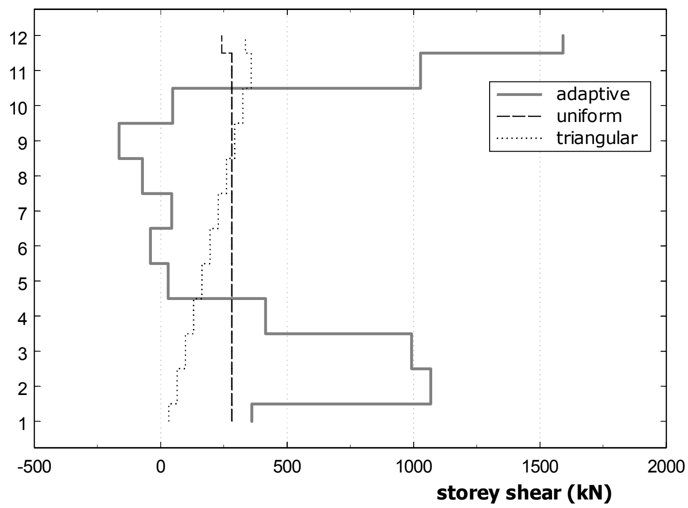


Figure 16: Storey shear distributions of a 12-storey building obtained with Displacement-based Adaptive Pushover as well as with standard non-adaptive pushovers.

In other words, given that in DAP, shear distributions are automatically derived to attain structural equilibrium with the imposed storey drifts, rather than being a result of the loads directly applied to the structure, the previously described limitations evidenced by force-based adaptive schemes that use quadratic modal combination rules can be overcome and, consequently, results as whole (i.e. deformation profiles and capacity curves) become more accurate.

2.3 Ease-of-use, Computational Effort and Numerical Stability

When compared with nonlinear time-history analysis, pushover methods are advantaged by their (i) higher user-friendliness, (ii) reduced running time and (iii) increased numerical stability. Therefore, it is important that the proposed displacement-based algorithm, capable of producing improved structural response predictions in comparison with existing non-adaptive pushover techniques, does also feature these three advantages over dynamic analysis.

From a usability point-of-view, the proposed displacement-based adaptive pushover algorithm effectively presents no additional effort and/or requirements with respect to its conventional non-adaptive counterparts. In effect, the only element of novelty, in terms of analysis input, is the introduction of the building's inertia mass, which, however, can readily be obtained directly from the vertical gravity loads, already included in any type of pushover analysis.

With regards to computational effort, in general, the amount of time necessary to complete an adaptive pushover analysis is typically double the time necessary for a conventional procedure, approximately. Obviously, the duration of such finite element runs will vary according to the computing capacity of the workstation being used, as well as with the characteristics of the model (mainly the number of elements and level of fibre discretisation of the sections). In any case, adaptive pushover proved to be up to ten times quicker than nonlinear dynamic analysis of a same model (keeping in mind that fibre-based finite element modelling has been adopted for the current work), hence the time-advantage of static methods versus their dynamic counterparts is not lost with the addition of the adaptive features.

As far as numerical stability is concerned, no particular problems have been observed in the studies described above, and those given in subsequent sections, noting that structures were pushed well into their post-peak inelastic response range.

3 VERIFICATION PARAMETRIC STUDIES

In this Section, an analytical comparative study involving different pushover methods, either single or multi mode, adaptive or conventional, and dynamic nonlinear analysis of buildings and bridges is presented. The "true" dynamic response is deemed to be represented by the results of the Incremental Dynamic Analysis procedure (IDA) (e.g. [4]), which is a parametric analysis method by which a structure is subjected to a series of nonlinear time-history analyses of increasing intensity.

3.1 Reinforced Concrete Frames

(a) Characteristics of Input Motion and Structural Models. 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 (Figure 17). The latter are based on buildings previously designed for different ductility classes and design ground acceleration, on medium soil type 'B' of EC8 [33], resulting in a total of 12 models, as described in Table 1. The overall plan dimensions of the three configurations are 15 m by 20 m. The storey height is 3 m except the first storey of the irregular set, which is 4.5 m high. A detailed description of models and load conditions, as well as of their FE modelling, can be found in López-Menjivar [25].

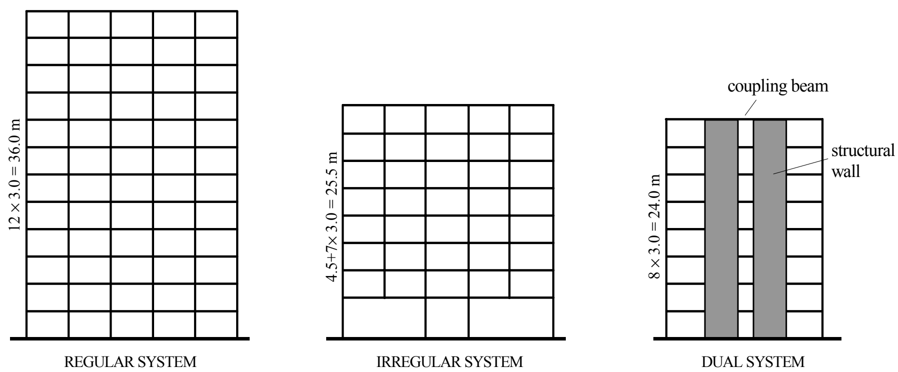


Figure 17: Geometric characteristics of the regular, irregular and dual systems.

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. Their acceleration response spectra are shown in Figure 18. 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 ([34]).

(b) Analyses and Results Post-Processing. The two non-adaptive pushover schemes, proposed in the NEHRP Guidelines [35], 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.

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
		RH30	High		5.00	0.697

Table 1: Considered building systems.

	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%

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

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:

$$\mathrm{Error}(\%) = 100\sqrt{\frac{1}{n}\sum_{i=1}^{n}\left(\frac{\Delta_{iD}-\Delta_{iP}}{\Delta_{iD}}\right)} \tag{6}$$

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 %).

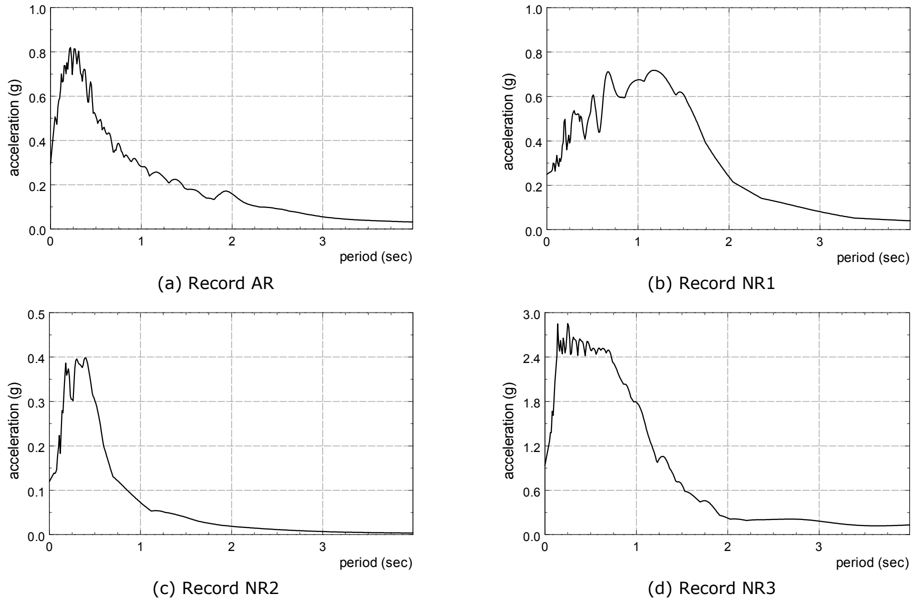


Figure 18: Elastic response spectra of the four records (5% equivalent viscous damping).

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

(c) 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 19) 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.

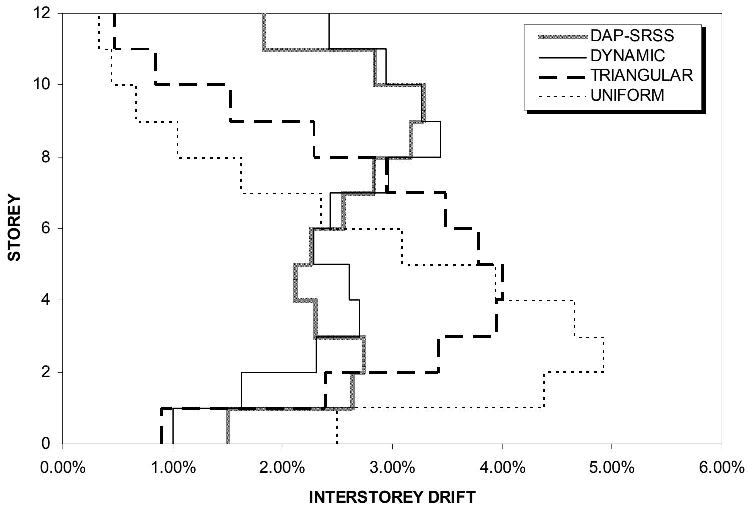


Figure 19: Representative results obtained with model RM15 subjected to one of the natural accelerograms employed in this study (NR2).

3.2 Steel Frames

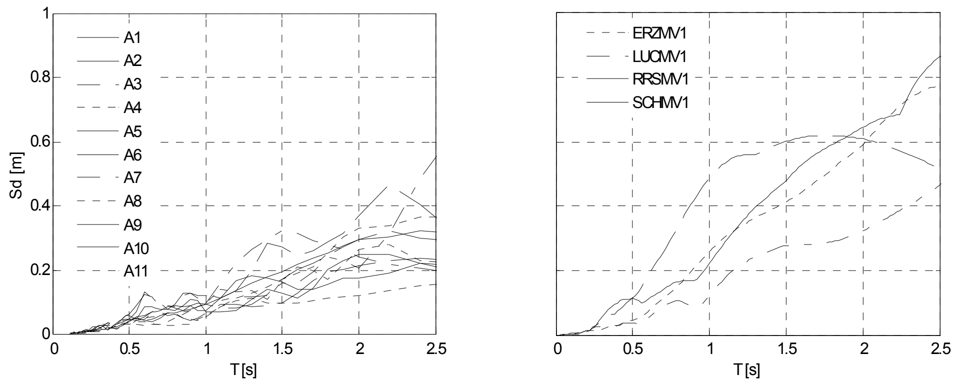
(a) Characteristics of Input Motion and Structural Models. In order to enable a direct comparison with the extensive and most thorough parametric study described in FEMA-440 [12], the same structural models and earthquake records that have been used in such work have been adopted here. Hence, considered ground motions include Near-Field type records (NF records) as well as ordinary records (Ordinary Ground Motions, OGMs). Their main properties are summarized in Table 3, whilst displacement response spectra are represented in Figure 20. OGMs (11 records) consist of site class C accelerograms selected from strong-motion records that do not present near-fault or near-field characteristics (strong velocity pulses, short duration, high frequency content, etc.). NF accelerograms, on the other hand, consist of motions recorded close to the epicentre and which contain very strong velocity pulses, originally included in the FEMA-440 work with the objective of verifying, at least in preliminary fashion, the validity of employing pushover methods in areas where pulse-like near-fault ground motions are likely to occur. For further details the reader is referred to [12] and Somerville et al. [36].

Identifier	EQ	M_s	Station	Component	PGA [g]	PGV [cm/s]	Effective Length[s]	Source
			Ordinary Ground Motions					
A1	Superstitn 11-24-87	6.6	El Centro Imp Co. Cent(01335)	000	0.358	46.4	23	CDMG
A2	Northridge 1-17-94	6.7	Canyon Country-W Lost Cany (90057)	000	0.41	43	11	USC
A3	Loma Prieta 10-18-89	7.1	Gilroy Array #2 (47380)	090	0.322	39.1	14	CDMG
A4	Chi-Chi, Taiwan 8-20-99	7.6	(TCU122)	N	0.261	34	35	CWB
A5	Loma Prieta 10-18-89	7.1	Gilroy Array #3 (47381)	090	0.367	44.7	17	CDMG
A6	Northridge 1-17-94	6.7	Canoga Park-Topanga Can (90053)	196	0.42	60.8	14	USC
A7	Chi-Chi, Taiwan 8-20-99	7.6	(CHY101)	W	0.353	70.6	32	CWB
A8	Superstitn 11-24-87	6.6	El Centro Imp Co. Cent(01335)	090	0.258	40.9	27	CDMG
A9	Northridge 1-17-94	6.7	Canoga Park-Topanga Can (90053)	106	0.356	32.1	16	USC
A10	Imperial Valley 10-15-79	6.9	El Centro Array #2 (5115)	140	0.315	31.5	17	USGS
A11	Imperial Valley 10-15-79	6.9	El Centro Array #11 (5058)	230	0.38	42.1	18	USGS
			Near-Field Ground Motions					
ERZ	Erzican 3-13-92	6.9	Erzican Station	NA	0.442	126	7	EERL Caltech
LUC	Landers 6-28-92	7.3	Lucerne Valley Station	280	0.732	147	14	EERL Caltech
RRS	Northridge 1-17-94	6.7	Rinaldi Receiving Station	213	0.891	186	6.5	EERL Caltech
SCH	Northridge 1-17-94	6.7	Sylmar County Hospital Parking Lot	190	0.865	138	5.5	EERL Caltech

Table 3: Ground Motion characteristics (ATC, 2005).

The prototype buildings analyzed in the current endeavour consist therefore of two steel moment-resisting frames (nine and three storeys, Figure 21 and Figure 22) designed as a part of the FEMA-funded SAC joint venture project [37]. These frames were considered in both regular and irregular (weak-storey at the ground floor) configurations, thus leading to a total of four frames.

(b) Analyses and Results Post-Processing. The pushover schemes considered include invariant static load patterns, such as (i) uniform (Uniform), (ii) inverted triangular (Triangular), (iii) first mode shape (1st Mode), (iv) a code-specified period-dependent distribution (where lateral forces changes from a linear distribution for low period systems to a parabolic shape



(a) (b)

Figure 20: Ground Motions: unscaled displacement response spectrum for (a) Ordinary- and (b) Near-Field-type records [12].

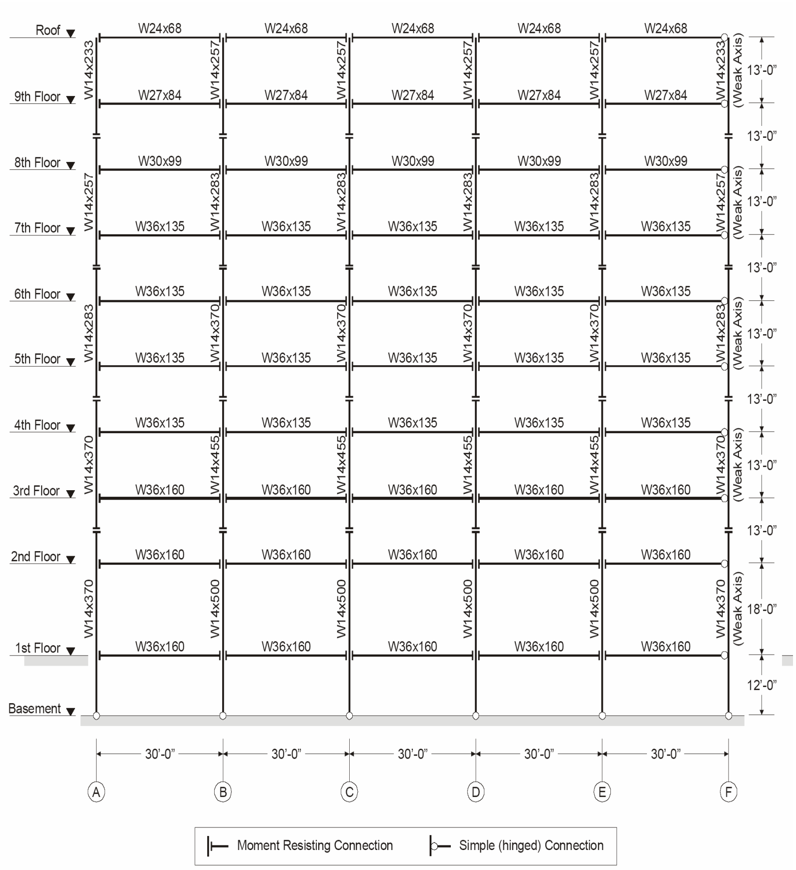


Figure 21: 9-Storey frame considered in this study [12].

for more flexible models) (Code), and the Displacement-based Adaptive Pushover (DAP). In addition, an alternative adaptive approach has been tested in the case of OGMs, consisting in the employment of the average response spectrum of all records to compute the modal spectral amplification that is considered in the calculation of the incremental adaptive loading vector (Displacement-based Adaptive Pushover with Average Spectrum scaling (DAP-AS)). Whilst such procedure might be adopted in the case of OGMs, where records have been scaled to meet the same target displacement, it cannot be used for NF records, which were employed without scaling and thus involve different drift responses for the same prototype building.

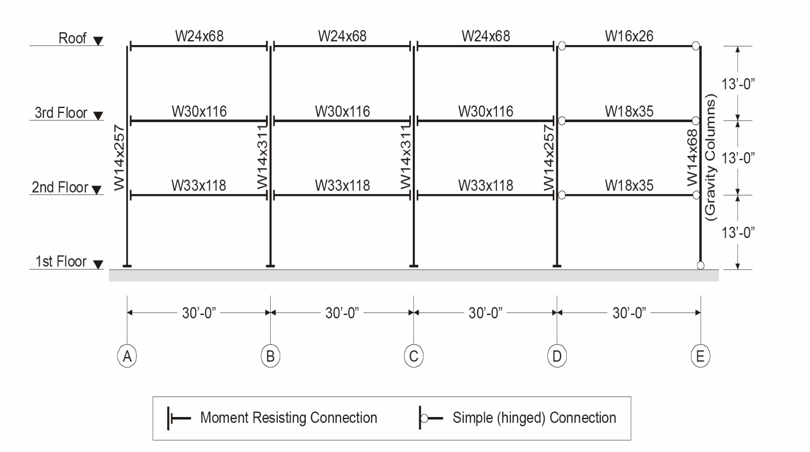


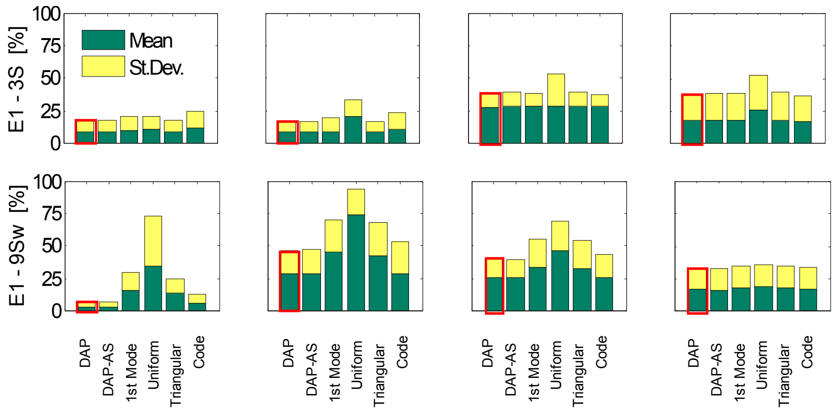
Figure 22: 3-Storey frame considered in this study [12].

Verification of the pushover algorithm at the "global level" was carried out through comparisons between the base shear vs. top floor IDA envelopes (assumed as representative of the true behaviour of the frame) and the pushover curves. With the objective of assessing also the accuracy of DAP in estimating local response parameters, the dynamic response at different ductility demand levels has been considered, where each Ductility Level (DL) is identified by means of the total drift value (i.e. top floor displacement/building height). Three different drift levels are assumed (0.5, 2 and 4%) and each ordinary record has been scaled in order to get the predefined drift for each prototype building. Response parameters of interest (displacement, drift, shear and moment) recorded in time-history analyses are then compared, at each level location, with those predicted by the pushover procedures at the same roof displacement magnitude. For the 9-Storey weak frame system an ultimate drift level of 2.7% has been selected (corresponding to a top floor displacement of 1m), instead of the 4% defined in a preliminary stage, since higher values were leading to the development of a global failure mechanism of the structure (corresponding to an ultimate steel strain in frame elements conservatively fixed in 15% [38]) under several records, and would thus prevent a complete statistical evaluation of the results.

The effectiveness of the different static procedures in predicting the local dynamic response is quantified and compared by means of error measure E1, which provides a direct insight on how inaccurate is the static method (evidently, the mean of a pushover response estimate is computed only in the case of record-dependent DAP analysis, for all other pushover schemes the single response value is used):

$$E1 = \left| \frac{Mean_{PUSHOVER} - Mean_{DYNAMIC}}{Mean_{DYNAMIC}} \right| \tag{7}$$

(c) Obtained Results. Figure 23 summarises the results obtained through this parametric study, putting in evidence the fact that, when compared with other pushover procedures, DAP leads to higher accuracy in the prediction of global and local response parameters of steel buildings, particularly in those cases where the influence of higher modes of vibration is important (e.g. high-rise buildings). It is also shown that the employment of an average response spectral shape leads to satisfactory results, thus rendering the procedure very much applicable within a design application framework, where standard code spectral shapes are prescribed.



(a) Floor Displacement (b) Interstorey Drift (c) Interstorey Shear (d) Interstorey Moment

Figure 23: Mean values and standard deviation of error measure E1, averaged along the height of buildings models 3S and 9Sw at the intermediate drift level (top floor drift level of 2%).

3.3 RC Continuous-Span Bridges

(a) Characteristics of Input Motion and Structural Models. 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 24, where the label numbers 1, 2, 3 characterise the pier heights of 7 m, 14 m and 21 m, respectively.

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 [39]. The bounding characteristics of the records are summarized in Table 4. Further details on modelling and input can be found in Casarotti et al. [40].

(b) Analyses and Results 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

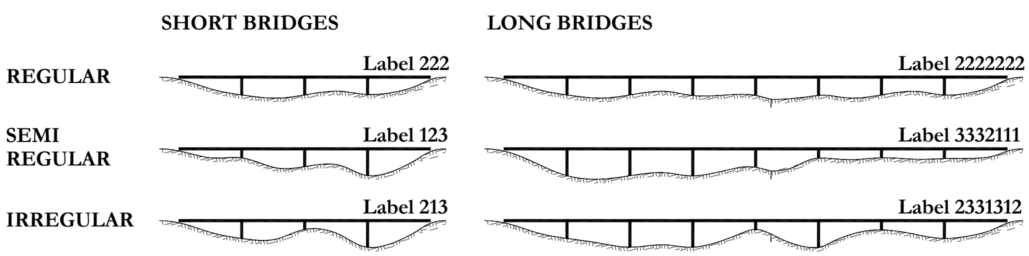


Figure 24: Analysed bridge configurations.

	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%

Table 4: Bounding characteristics of the employed set of records for bridges.

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:

$$\hat{R}_{i,IDA} = \text{median}_{j=1:14}\left[R_{i,j-IDA}\right] \tag{8}$$

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, and translated in Eq. 9 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.

$$\overline{R}_{i,PUSHOVERtype} = \frac{R_{i,PUSHOVERtype}}{\hat{R}_{i,IDA}} \cdots \xrightarrow[\text{ideally}]{} 1 \tag{9}$$

Given the 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.

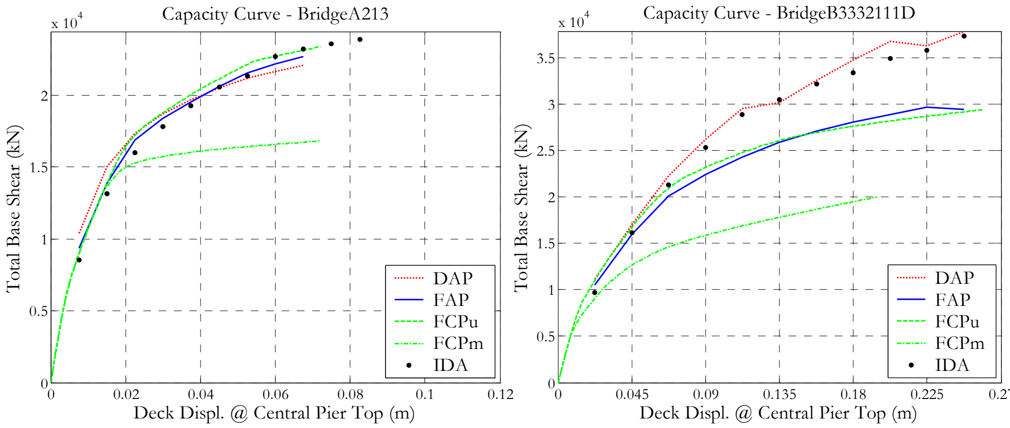
Having the same unitary target value, all normalized deck displacements become comparable, and a bridge index BI can measure the precision of the obtained deformed shape. Per each level of inelasticity, such bridge index is defined as the median of results over the m deck locations Eq. 10 with the standard deviation measuring the dispersion with respect to the median Eq. 11. 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.

$$BI_{PUSHOVERtype} = \text{median}_{i=1:m}\left(\overline{\hat{\Delta}}_{i,PUSHOVERtype}\right) \tag{10}$$

$$\delta_{PUSHOVERtype} = \sqrt{\frac{\sum_{i=1}^{m}\left(\overline{\hat{\Delta}}_{i,PUSHOVERtype} - BI_{PUSHOVERtype}\right)^2}{m-1}} \tag{11}$$

(c) Obtained Results. Two main pertinent observations can be withdrawn from a scrutiny of the capacity curves obtained by the different pushover analyses in Figure 25: 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 26, 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



(a) (b)

Figure 25: Capacity curve results.

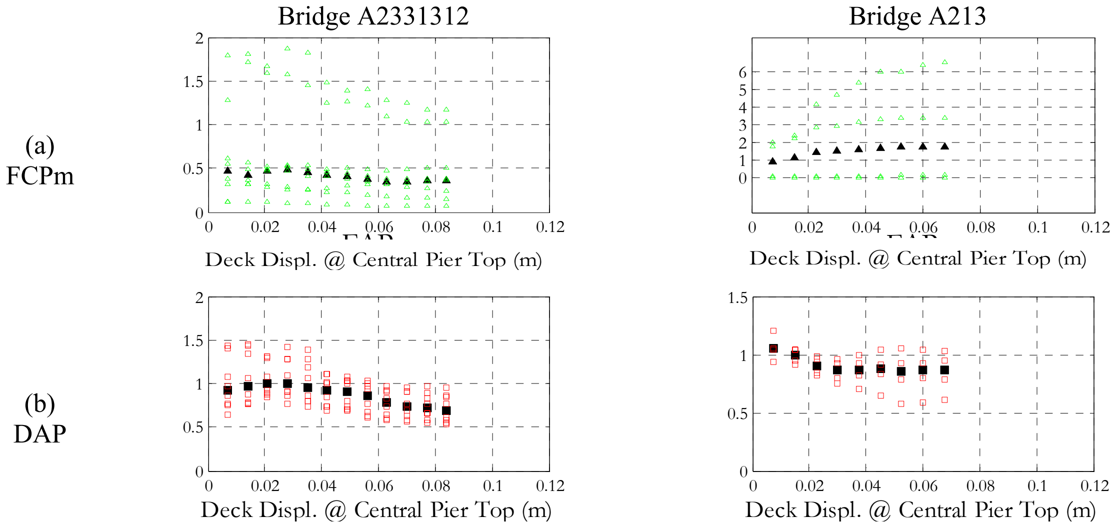


Figure 26: Prediction of the deformed pattern: BI and relative scatter.

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.

4 CONCLUSIONS

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 is 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.Finally, it is worth noting that in order for it to be employed in design office applications, DAP needs obviously to be introduced within a complete Nonlinear Static Procedure (NSP), such as the Capacity Spectrum Method (CSM) [41,42] and the N2 Method [43,44]. With this in mind, the Adaptive Capacity Spectrum Method has been developed [45], leading to promising and somewhat reassuring results [46-48].

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