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VERIFICATION OF DISPLACEMENT-BASED ADAPTIVE PUSHOVER FOR SEISMIC ASSESSMENT OF HIGH-RISE STEEL BUILDINGS

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SUMMARY

A number of recent studies raised doubts on the effectiveness of conventional pushover methods, whereby a constant single-mode incremental force vector is applied to the structure, in estimating the seismic demand/capacity of framed buildings subjected to earthquake action, in particular when higher modes are involved in the structural response. 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 damage accumulation and resulting modification of the modal parameters that characterise the structural response at increasing loading levels.

Within such adaptive framework, the application of a displacement, as opposed to force, incremental loading vector becomes not only feasible, since the latter is updated at each step of the analysis according to the current dynamic characteristics of the structure, but also very appealing, since inline with the present drive for development and code implementation of displacement or, more generally, deformation-based design and assessment methods. Further, such innovative displacement-based pushover algorithm seems to lead to superior response predictions, with little or no additional modelling and computational effort, with respect to conventional pushover procedures.

The verification of the accuracy of these innovative adaptive pushover techniques, however, has so far been restrained to the cases of reinforced concrete buildings and continuous-span bridges, with steel construction having been essentially overlooked. Therefore, the current paper aims at addressing such knowledge gap, by describing the results of a parametric study, whereby the accuracy of the Displacement-based Adaptive Pushover algorithm (DAP) in predicting the seismic response of 9- and 20-storey high steel buildings responding in the inelastic range is investigated. A large set of natural records (from the SAC project) is used in the dynamic analyses that are carried out for comparison.

1. INTRODUCTION

In the last few years, with the development of performance-based design strategies and their diffusion in the current engineering design practice, the call for easy, yet accurate methods for estimating seismic demand on structures considering their inelastic behaviour has increased. Within this framework, Nonlinear Static Procedures (NSPs) do represent an easier and thus appealing alternative with respect to nonlinear dynamic analyses. In fact, they overcome the major drawbacks of nonlinear time-history analyses, (a) which require the assemblage of an ensemble of site-specific ground motions compatible with the seismic hazard spectrum for the

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site, (b) and are quite computationally demanding, particularly when fibre-based structural analysis programs are employed to model the seismic performance of large buildings, requiring 3D models with thousands of elements, considering also that a large number of analysis runs are required in order to get the average response of the structure. A detailed discussion on pros and cons of nonlinear static, with respect to dynamic, analysis can be found in Pinho et al. [2006].

NSPs are commonly based on 'pushover analysis', which represents a modern variation of the classical 'collapse analysis' method as fittingly described in Kunnath [2004], where the static equilibrium equations corresponding to a nonlinear structural model subjected to a monotonically increasing lateral load pattern are solved in an incremental-iterative fashion. 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 (a) a predefined limit state is reached, (b) structural collapse is incipient or (c) the program fails to converge. NSPs assume that all the structural response quantities of interest (displacements, internal forces, plastic deformations, etc.) can be estimated by means of those recorded in a pushover analysis at the design displacement level.

Many authors [e.g. Lawson et al., 1994; Krawinkler and Seneviratna, 1998; Kim and D'Amore, 1999; Naeim and Lobo, 1999; Antoniou and Pinho, 2004a; Chopra and Goel, 2006] have however noted that nonlinear static methods suffer from a series of limitations, which stem essentially from their inherently static nature. Such limitations become particularly evident when high-rise flexible frames, whose response may be heavily influenced by higher modes, are being assessed. In this work, therefore, high-rise steel frames that have been used in past publications [Goel and Chopra, 2004] to demonstrate the inability of current nonlinear static procedures in predicting their response, will be analysed using the recently proposed Displacement-based Adaptive Pushover algorithm [Antoniou and Pinho, 2004b] with a view to assess the capability of the latter in overcoming some of the shortcomings discussed above.

2. CASE STUDY

2.1. Previous work on high-rise steel frames

Goel and Chopra [2004] assessed the performance of a number of nonlinear static analysis procedures, including conventional pushover analysis with varied force distributions as well as the Modal Pushover Analysis (MPA) procedure that has been proposed and developed by the same authors in recent years [e.g. Chopra and Goel, 2002, 2006]. Prototype buildings were 9- and 20-storey frames, each of which considered in three variants; the buildings had been designed according to the Seattle (ICBO, 1994), Boston (BOCA, 1993) and Los Angeles (ICBO, 1994) local codes.

Results, analysed under the perspective of the interstorey drifts and plastic rotations estimates, show that all the static approaches, and MPA in particular, perform better in the case of Seattle and Boston frames, where the response under the selected input records is nearly elastic, whilst the accuracy decreases as the structure response starts to enter its inelastic regime, as in the case of LA frames. Indeed, as it might be observed in Figure 1 and Figure 2 where the interstorey drift profile and the interstorey drift ratio (i.e. bias, the ratio between the pushover estimate and the dynamic value) are represented, MPA performs well in the case of Seattle and Boston frames, whilst results became worst in the case of LA frames. In particular, MPA improved results with respect to conventional procedures for buildings that respond in the elastic range but overestimate the drift demand at lower storeys in the 9-Storey LA model and at upper storeys in the 20-Storey LA frame. Moreover, as noted by Goel and Chopra [2004], the bias in all static methods (MPA included) is unacceptably large for the 20-Storey LA building that experience roof drift in the region of rapid decay in lateral capacity of the building; their recommendation is for nonlinear dynamic analyses to be carried out in such cases.

The difficulties encountered with the LA model are further emphasised by the fact that, when using MPA, in order to avoid the reversal of direction of the roof displacement in the second mode pushover analysis of the 20-Storey LA frame soon after initiation of yielding, ad-hoc modifications of the building model, consisting in the somewhat arbitrary reduction of applied gravity loads, had to be introduced.

Due to all of the above, the structural models designed according to the Los Angeles building code have been taken as reference for the parametric study presented in this paper, in order to test the efficiency of the innovative Displacement-based Adaptive Pushover procedure within a worst-case type of scenario.

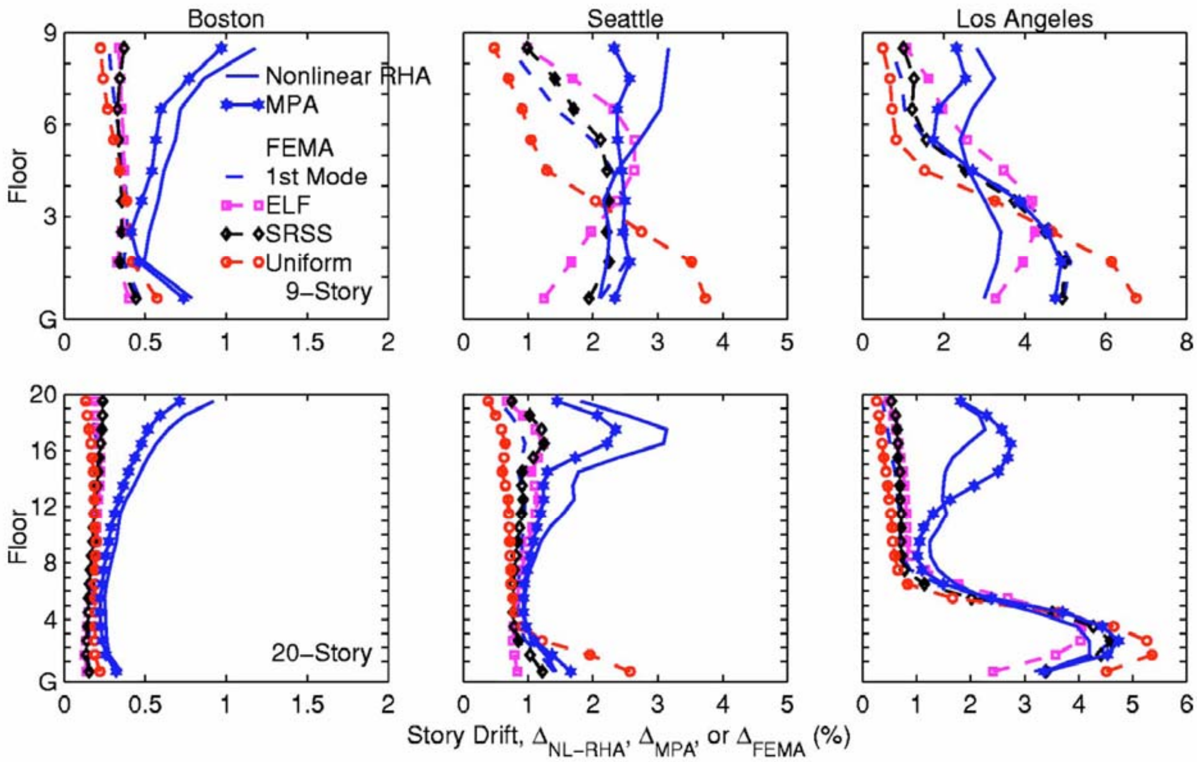

Figure 1: Interstorey drift profile [from Goel and Chopra, 2004]

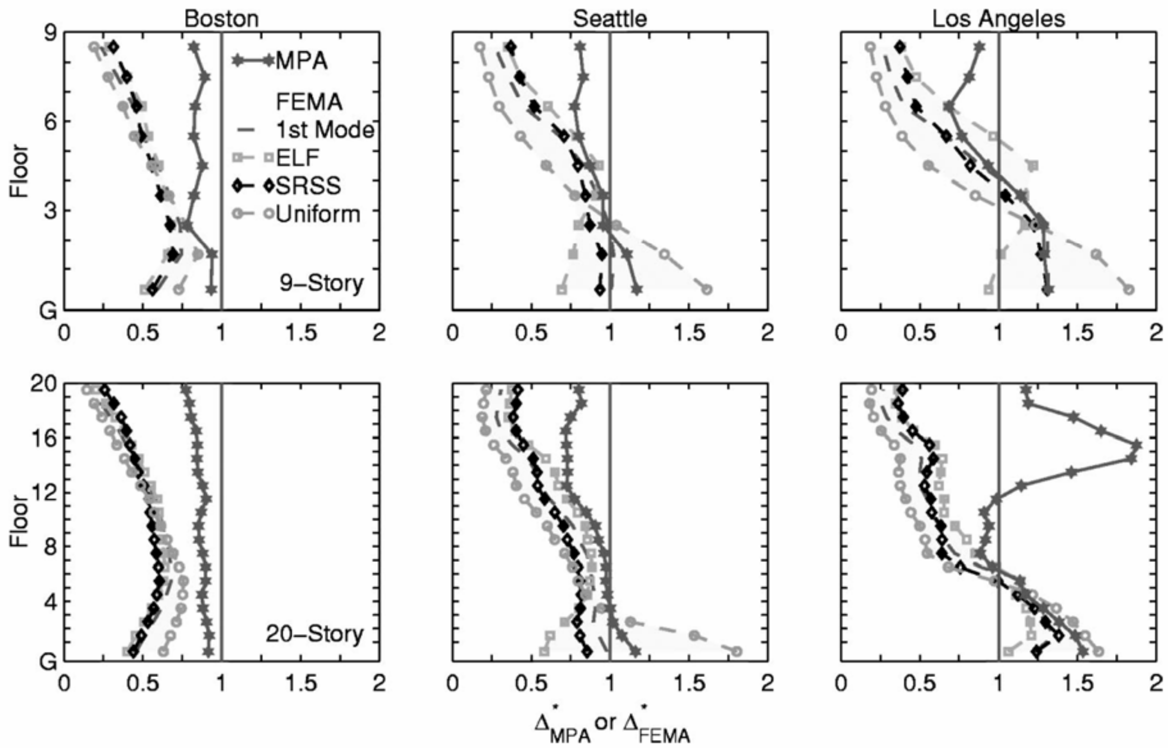

Figure 2: Interstorey drift ratio [from Goel and Chopra, 2004]

2.2. Building models and ground motions

As stated above, the two steel frame models under scrutiny and the ensemble of 20 ground motions adopted for dynamic (and DAP) analyses thus refer to the 9- and 20-Storey steel frames and the 2/50 set of records developed for the SAC joint venture [Gupta and Krawinkler, 1999]. Ground motions consist in 20 records representing a probability of exceedance of 2% in 50 years (acceleration and displacement response spectrum are represented in Figure 3 and Figure 4), whilst the steel frames (schematically represented in Figure 5 and Figure 6) have been designed according to the Los Angeles local code requirements (ICBO 1994), following pre-Northridge standards. The reader is referred to the work of Goel and Chopra [2004] for further details.

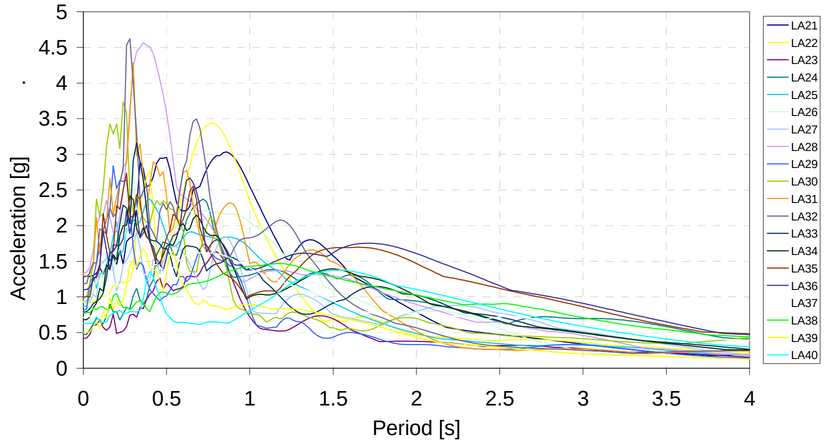

Figure 3: 2/50 set SAC records: Acceleration Response Spectrum.

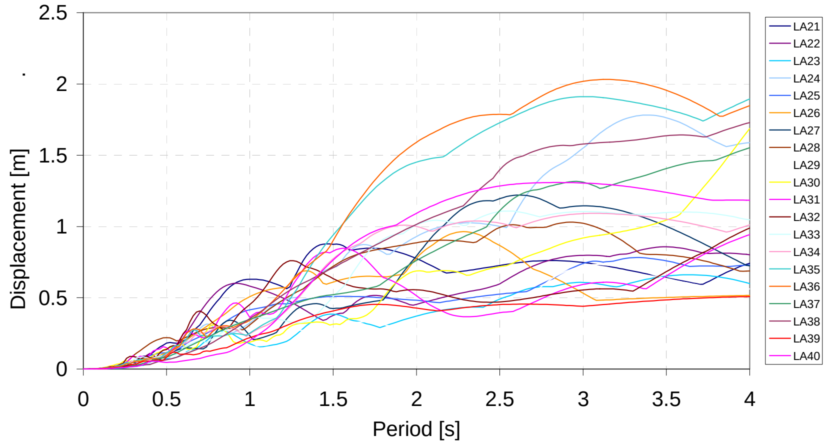

Figure 4: 2/50 set SAC records: Displacement Response Spectrum.

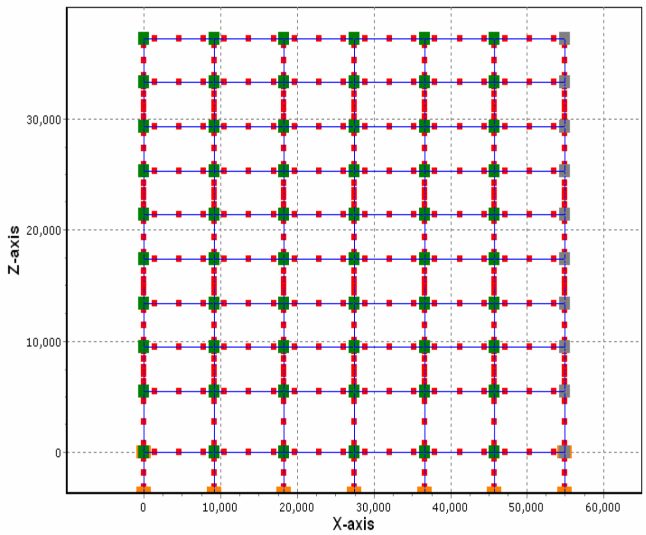

Figure 5: 9-Storey frame: finite-element model.

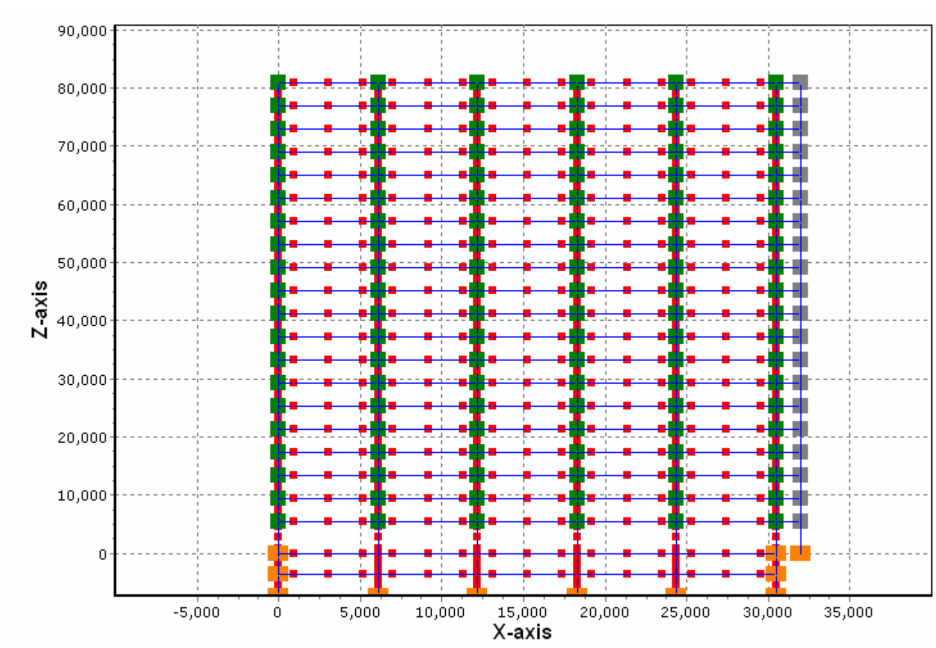

Figure 6: 20-Storey frame: finite-element model.

3. STRUCTURAL MODELLING AND ANALYSIS

3.1 Verification of structural analysis software

All analyses have been carried out using finite element analysis program SeismoStruct [SeismoSoft, 2005], a fibre element-based program for seismic analysis of framed structures, freely 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 analysis.

Whilst the accuracy of such FE program in predicting the nonlinear dynamic seismic response of reinforced concrete frames and bridges has been extensively verified [e.g. López-Menjivar, 2004; Casarotti et al., 2005], the same has not occurred for the case of steel frames. Therefore, in what follows, the program is validated through a comparison with experimental results obtained from quasi-static tests on a full-scale steel structural model [Matsumiya et al., 2004; Nakashima et al., 2006].

The prototype building is a three-storey, two-span by one-span steel moment frame tested at the University of Kyoto. The test structure plan dimensions are 12 m in the longitudinal direction and 8.5 m in the transversal direction (Figure 7) and it has been designed following the most common design considerations exercised in Japan for post-Kobe steel moment frames. Columns were extended to the approximate mid-height at the third storey (Figure 8), at which level steel braces were connected horizontally to the columns by high strength bolts through gusset plates to allow for the rotation at the column top. The quasi-static cyclic loading with increasing displacement amplitude has been applied at this level by means of two hydraulic jacks (one per frame). Since the two lateral resisting frames work essentially in independent fashion, only one of these has been actually modelled.

Results obtained are reported in Figure 9, in terms of global response (i.e. total base shear vs. total drift) and in terms of local response (i.e. first storey shear vs. storey drift). Good agreement is observed between the numerical predictions and the experimental results, with both strength and stiffness being estimated with a satisfactory level of accuracy. It is also noted that the pinching that occurs after large displacement cycles cannot be reproduced, because the slip-type of response that was experimentally observed at the column base (due to the accumulation of plastic deformation in the anchor bolts) was not modelled in the current application, given its reduced importance within the scope of the current application.

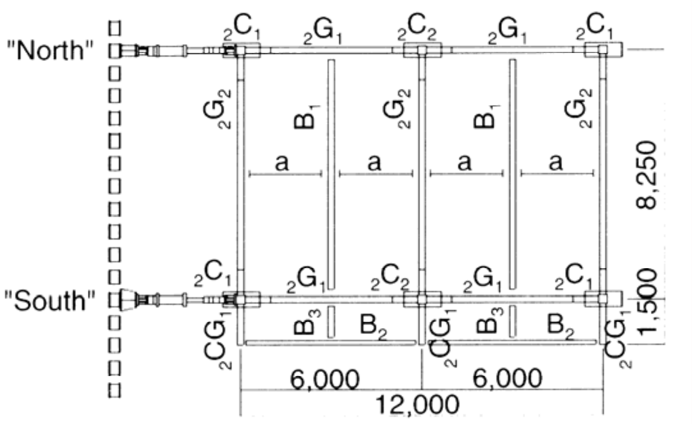

Figure 7: Plan view of the test structure - units: mm [Nakashima et al., 2006].

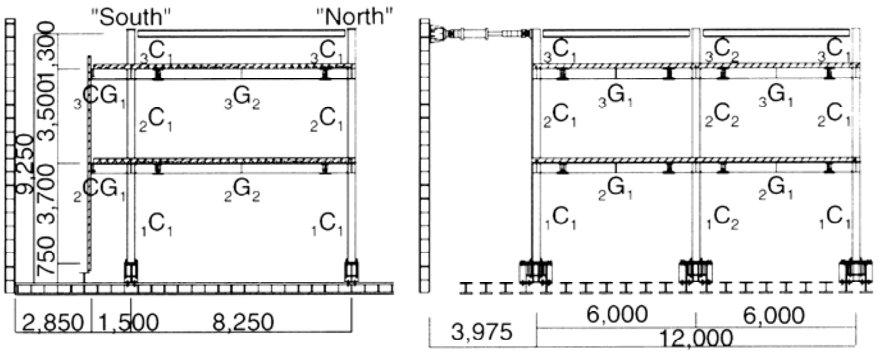

Figure 8: Elevation of the test structure - units: mm [Nakashima et al., 2006].

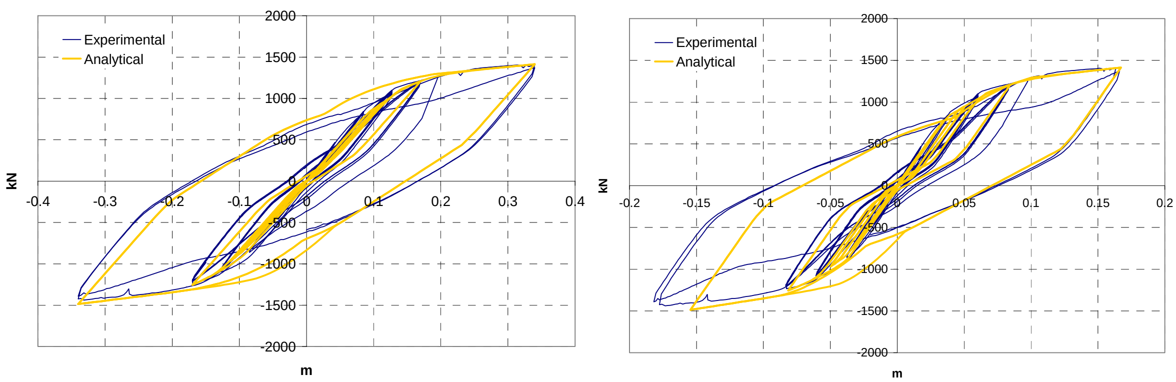

Figure 9: Total base shear vs. total drift (left) and 1st storey shear vs. storey drift

3.2 Analysis and results post-processing

In order to provide an extensive evaluation of the performance provided by alternative pushover procedures in predicting dynamic results of high-rise steel frames responding in the inelastic range, a wide set of pushover schemes have been considered. The latter consist in conventional (i.e. constant-shape load vector) schemes where the lateral load profile follows alternatively the first mode, uniform, triangular shape or a code-specified period-dependent distribution [FEMA 368, 2000], and the displacement-based adaptive scheme described above.

As far as the dynamic time-history analyses are concerned, it is perhaps worth noting that, notwithstanding the fact that recent work by Hall [2005] and Priestley and Grant [2005] seem to clearly indicate the inadequacy of introducing Rayleigh damping, the latter was still employed with a view to render the results more directly comparable to the work of Goel and Chopra [2004], where Rayleigh damping was indeed employed. Hence, for the 9-storey building a 2% equivalent viscous damping ratio was assumed for the 1st mode and at a period of 0.2 sec, whilst for the 20-storey frame a 2% equivalent viscous damping was considered for the 1st and 5th modes.

It might also be noted that, being spectrum-dependent, DAP analyses are performed for every ground motion adopted in dynamic analyses, selecting as target displacement the maximum top floor displacement obtained in each dynamic run. A 5%-damped spectrum has been arbitrarily, and pragmatically, to be used in the spectrum scaling of the different modes (which are employed in the updating of the shape of the adaptive load vector increment), with the CQC rule being then employed in the combination of the results, again assuming a damping ratio of 5% in all modes. The reader is referred to Pinho and Antoniou [2005] for further details on DAP.

Results are presented in terms of median values of the main structural response parameters of interest (interstorey drift, interstorey shear and overturning moment), and their bias with respect to dynamic analyses results. Structural responses predicted by the different pushover procedures have been evaluated at the top floor displacement level obtained in each dynamic analysis and then median values are compared. Moreover, the ratio between the estimate provided by each static scheme and the result of time-history run has been considered, in order to readily evaluate if the static procedure is overestimating or underestimating the dynamic results. The median value of this ratio has been considered, together with its dispersion, the latter being defined as: δ=ln(84th percentile)–ln(median value).

4. ANALYSES RESULTS

In Figure 10 and Figure 11, representative results in terms of storey response parameters obtained in this study are represented. It can be observed that although the actual values of drift are slightly underestimated by the displacement-based adaptive pushover scheme (DAP), the latter predicts the shape of the drift profiles with superior accuracy, in comparison to conventional pushover (and also with respect to what is observed in Figure 1 and Figure 2). This characteristic of DAP is particularly positive and encouraging, since interstorey drift profiles provide valuable information on the yield/failure mechanism that the structure will develop and can also be directly related to structural and non-structural damage. Hence, whilst the under-prediction of the actual values can be overcome with relative ease by the introduction of a scaling parameter (empirically derived from a series of parametric studies), the correct prediction of the interstorey drift profile requires an algorithm that is able to take due account of both higher mode contributions and distribution of inelastic deformations among structural members.

The superiority of DAP with respect to non-adaptive schemes is confirmed also in terms of prediction of internal forces, interstorey shear and overturning moment. Compared with the other pushover schemes, DAP leads to results closer to the dynamic values, as clearly represented in Figure 12 where results regarding interstorey overturning moments are represented. From the observation of the dispersion of the bias along the height of the

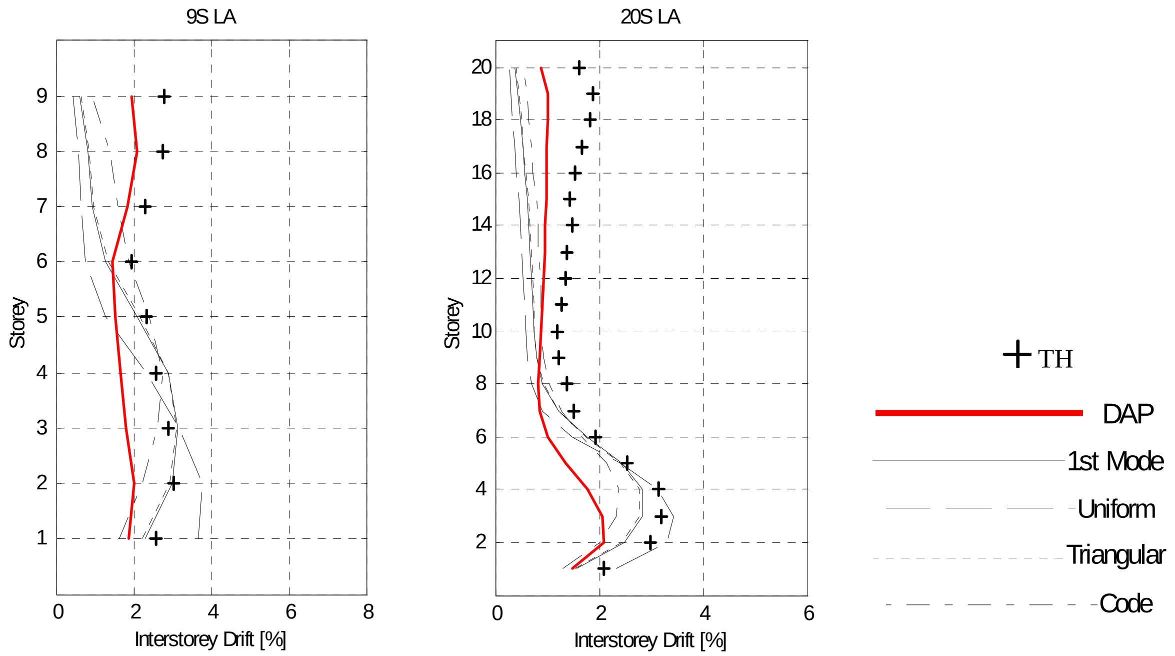

Figure 10: Interstorey drift profile: 9-Storey frame (left) and 20-Storey frame (right).

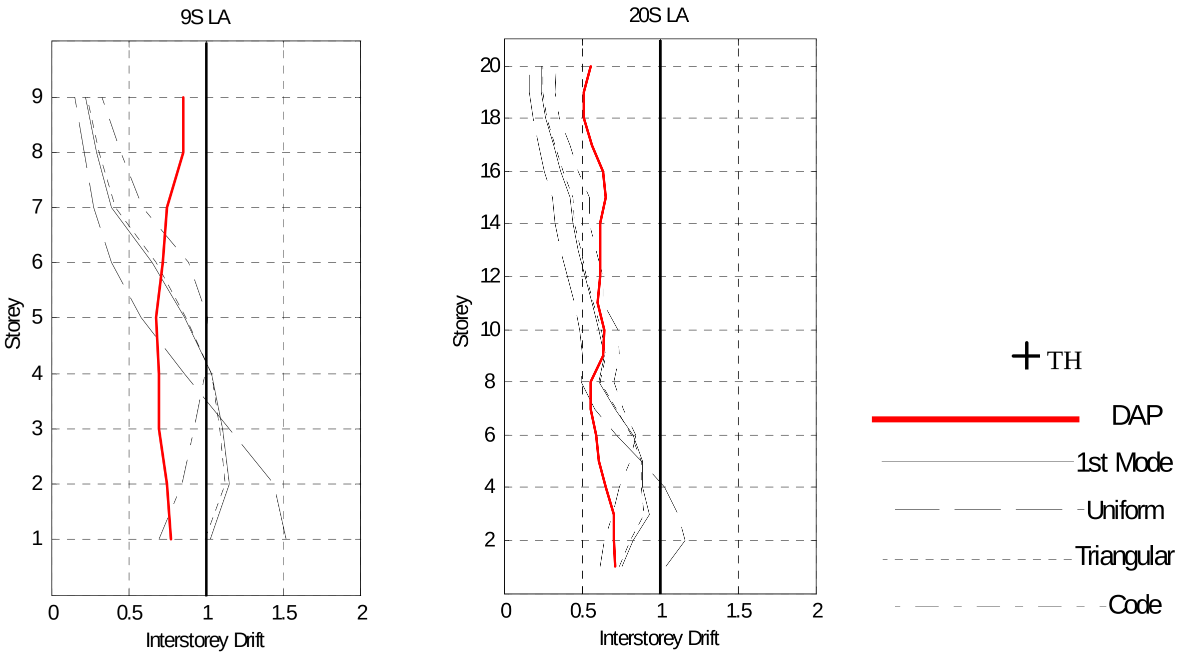

Figure 11: Interstorey drift ratio (bias): 9-Storey frame (left) and 20-Storey frame (right).

prototype structures [Pietra, 2006], it was also possible to conclude that the scatter of DAP values is of the same order of magnitude of the other procedures, and it is satisfactorily low, implying that results fall within a narrow range of values, thus confirming the reliability of the adaptive pushover estimates.

As briefly mentioned before, in order to overcome the drift under-prediction of DAP, which, by the way, has been observed also in other studies for different structural typologies [e.g. Casarotti et al., 2005; Meireles et al., 2006], one could perhaps think of deriving, though a series of parametric studies, empirical scaling parameters. Simply as an exemplificative case, in Figure 13 interstorey DAP results scaled up by a constant factor (1.3 for the 9-storeys building and 1.6 for the 20-storeys frame) are shown, highlighting how such a simple measure could lead to tremendous benefits drift results, if parametric would lead to the conclusion that such a scaling parameter could be derived. This possibility will be studied in future parametric/statistical studies.

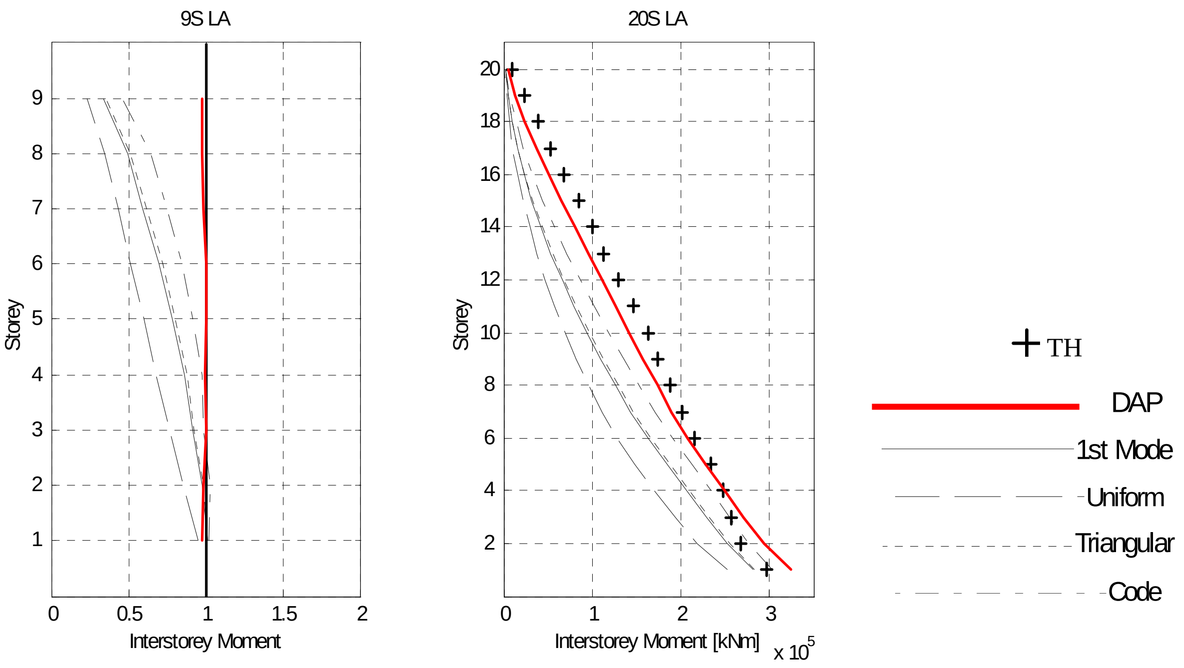

Figure 12: Interstorey moment: bias for the 9-Storey frame (left) and moment profile for the 20-Storey frame (right).

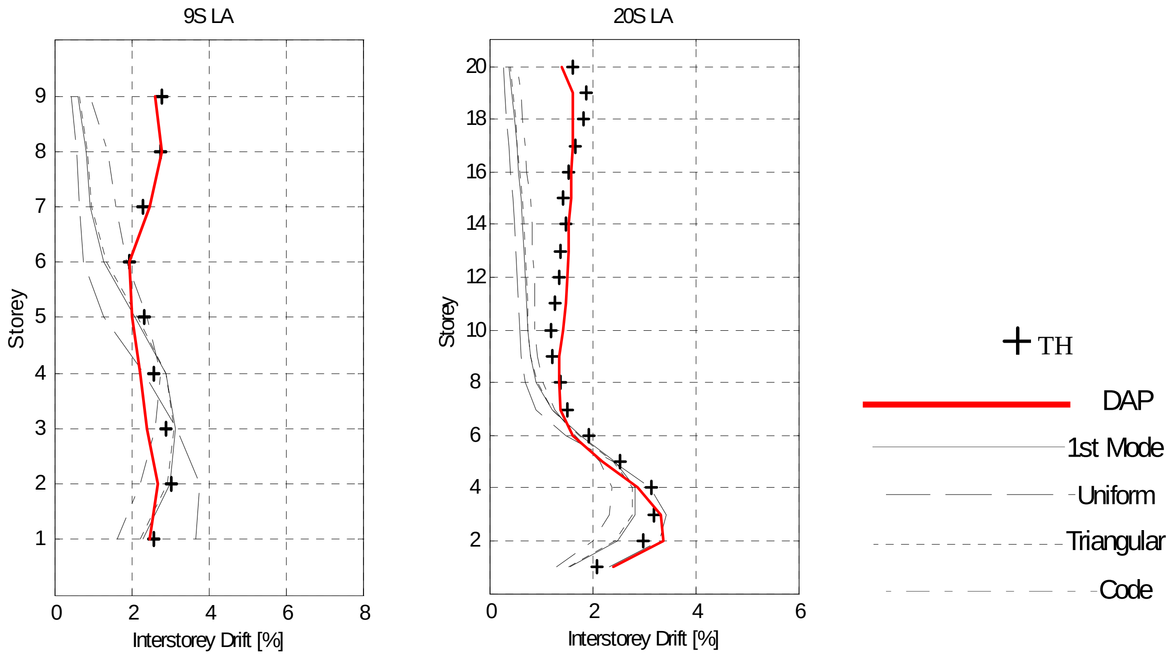

Figure 13: Scaled-up interstorey drift profile: 9-Storey frame (left) and 20-Storey frame (right).

5. CONCLUSIONS

In the current work, the capability of conventional pushover schemes as well as the Displacement-based Adaptive Procedure (DAP) to estimate the dynamic response of two high-rise steel frames responding in the inelastic range has been evaluated. Results clearly show the superiority of the DAP methodology with respect to constant-shape lateral load patterns, which can more accurately estimate the structural response both in terms of structural deformations, i.e. storey drifts, and internal forces, i.e. interstorey shears and moments. In particular, such innovative procedure performs well against the higher modes contributions, associated for example with large drift demands in the upper storeys, thus overcoming the major limitation of conventional schemes, that largely underestimate the structural responses at the upper levels.

In addition, even if estimates are not completely satisfactory, in terms of magnitude, in particular for the 20-Storey frame, DAP, compared with non-adaptive procedures, represents an alternative simpler procedure (involving a single pushover analysis) that allows predicting the response shape of tall steel buildings with an accuracy that is at least as good as that obtained with more complex multiple-pushover procedures. DAP seems also to be essentially insensitive to peculiar structural responses, hence no special measures or restrictions need to be introduced for structures that exceed a given number of storeys.

ACKNOWLEDGEMENTS

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