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ASSESSMENT OF STATIC NONLINEAR APPROACHES FOR EARTHQUAKE-RESISTANT DESIGN OF TALL REINFORCED CONCRETE BUILDINGS

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ABSTRACT

The simplified nonlinear static procedures (conventional pushover) have become a useful tool to assess the seismic performance of buildings. Beyond their inherent static character, such methods include two major inaccuracies: poor consideration of the higher-mode effects and lack of consideration of the changes in the modal characteristics of the structure (mainly, the modal shapes) as the damage progresses; both issues are particularly relevant for tall buildings. To cope with these two subjects, the modal and adaptive pushover strategies have been proposed, respectively. The objective of this paper is to evaluate the suitability of these procedures for high-rise buildings. The research approach consists in selecting two representative prototype buildings with 30 and 45 stories and carrying out a number of nonlinear static and dynamic analyses and comparing their results. Preliminary results show that current pushover methods cannot predict with enough accuracy the response of high-rise buildings under dynamic seismic action.

INTRODUCTION

Nowadays, in earthquake engineering, nonlinear dynamic analysis has become a common practice because of the growing severity of the code requirements and the capacity of the computation software. Nonlinear response history analysis (NLRHA) utilizing a set of carefully selected ground motion records, is one of the most accurate procedures to estimate the effects of the seismic demand. However, our limited present knowledge about the site seismicity makes highly difficult to identify future seismic demands. Moreover, the precision of NLRHA is strongly affected by modelling parameters such as damping, mass distribution and hysteretic behaviour, among others. Conversely, nonlinear static analysis (pushover) can constitute a practical earthquake-resistant design tool for day use, given its simplicity, thus requiring less computational effort. However, in the conventional pushover methods, the structure is subjected to monotonically increasing lateral forces with an invariant load pattern (FEMA-273 1997, FEMA-356 200, ATC-40 1996). Such pattern defines the variation of the pushing forces along the height of the building; three major patterns have been proposed: uniform, triangular and modal. In the uniform and triangular patterns, the forces are proportional to the floor mass and to a factor which is either constant (uniform pattern) or is proportional to the floor height (triangular pattern). In the modal pattern, the forces are consistent with the lateral force distribution determined in elastic analysis. When a modal pattern is considered, their invariant character is a significant limitation because the vibratory properties of the structure change as damage progresses, leading to different modal shapes of the lateral forces. Furthermore, the contribution of the higher modes is not accounted for. Since the higher mode effects and the variation of modal shapes are particularly relevant for tall

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buildings, conventional pushover methods might be not sufficiently adequate for them. Thus, in recent years attempts have been made to develop some improved procedures to consider higher-mode effects and the variation of the modal shapes.

One of the first pushover methods able to take into account the higher-mode effects was the “multi-mode pushover” (MMP) (Sasaki et al. 1998). The procedure uses load patterns based on the modes of interest. The capacity curve is obtained for each mode load pattern and then the Capacity Spectrum Method (CSM) (ATC-40 1996) is used to compare the structure capacity of each mode with the earthquake demand. It was concluded that MMP can be useful in identify failure mechanism due to the higher modes. Another static approach is the Modal Pushover Analysis (MPA) proposed in (Chopra, Goel 2002). In this method, a series of independent pushover analysis are carried out considering invariant modal lateral force distribution in accordance with linear elastic theory. Then, the capacity curves for each mode are converted in capacity curves for equivalent SDOF. The seismic demands are separately evaluated for each SDOF by nonlinear response history analysis or from the inelastic design spectrum (Chopra, 2001). Finally the demands are combined by the SRSS method. The drawback of the previously mentioned methods is that they do not take into account the interaction between the modes in the nonlinear range.

All of the methods described above are based on invariant load pattern during the analysis and, therefore, cannot take into account the changes in the vibration properties of the structure. For this reason adaptive pushover methods have been developed. Their main feature is that the load pattern is updated in each step of the pushover analysis. One of the recently developed methods was proposed in (Antoniou et al. 2004). In this method, eigenvalue analysis is carried out considering the stiffness state at the end of the previous load step and, based on the eigen-solution, the load pattern are calculated for each mode and combined by SRSS or CQC methods. Then, the calculated forces are applied to the structure to obtain the structural response and calculate the updated tangent stiffness matrix and return to the previous step of the algorithm. This method can take into account the spectral amplification by using elastic or inelastic response spectrum.

In this context, the objective of this work is to carry out a series of analysis to assess the adequacy of conventional and advanced static nonlinear approaches for practical earthquake-resistant design of tall reinforced concrete buildings. The study focuses on modern buildings, towering above 30 stories, with plan symmetry and regularity in elevation, and located in high seismicity regions (i.e. seismic design acceleration higher or equal than 0.4 g).

REPRESENTATIVE PROTOTYPE BUILDINGS

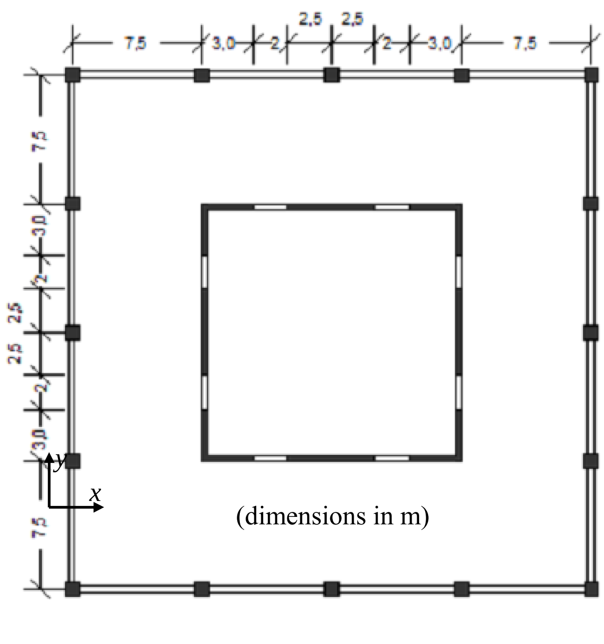
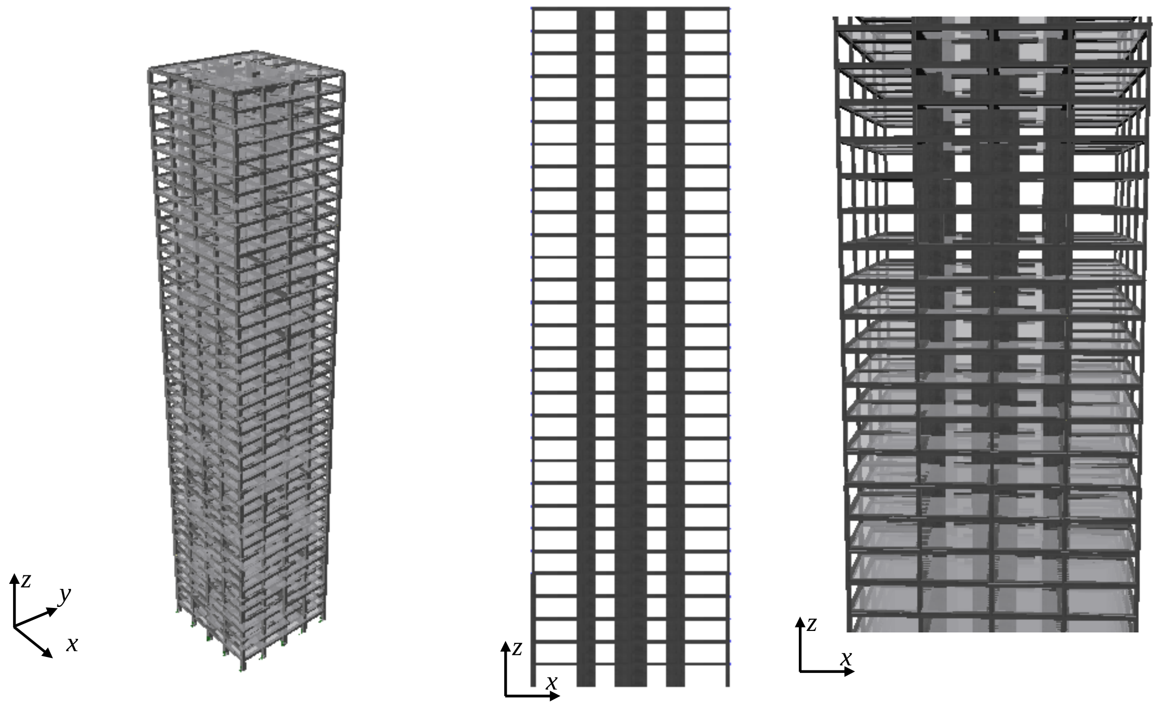


Figure 1. Plan view of the prototype buildings

Two prototype buildings are considered. These buildings have 30 and 45 floors respectively, and the lateral-load resisting system consists of structural core walls and special moment resisting frames (dual system). The buildings are 105 and 157.5 m high, respectively; the story height is 3.5 m. The dimensions of the floor plan are shown in Figure 1 and Figure 2 displays several views of the buildings.

The two considered buildings are designed according to the Eurocode 8 (EN-1998 2004) for equivalent static forces determined from type 1 acceleration response spectra (e.g. for earthquakes with $M_w \geq 5.5$). The structural members are pre-designed by simple empirical criteria. From this initial design, the fundamental period has been determined by classical eigenvalue analysis; the bending stiffness of the members has been reduced with the empirical factors suggested by the Eurocode 8. The behaviour factor is determined assuming "ductility class high"; the ratio α_u / α_1 is initially taken as the suggested default value and, once the structural members are designed, more refined values are determined by pushover analyses. The value of the fundamental period is updated, following a trial-and-error strategy, accounting for the actual sizes of the members and for the right reduction factors of the bending stiffness.

Table 1 and Table 2 describe the main characteristics of the prototype buildings. In the notation B# , "#" refers to the number of floors. The periods were determined from the numerical models of the buildings described in the next section, to be considered for the pushover and dynamic analyses. The last column contains the total weight of the buildings corresponding to the loading combination G + 0.3 Q (G and Q represent the dead and live loads, respectively). The buildings were designed for ground acceleration 0.4 g and behaviour factor q = 3.5. Figures from Table 2 confirm the relevance of the higher-mode effects.



(a) 3D representation (B45) (b) Elevation (B30) (c) View

Figure 2. Prototype buildings

Table 1. Main characteristics of the prototype buildings

Building	Floors / height (m)	Plan / core size (m)	First/ top floor columns size (m)	First / top floor core walls width (m)	Floor slab depth (m)	Perimeter beams (l × b × h) (m)	First/ top floor coupling beams (b × h) (m)	Weight (G + 0.3 Q) (MN)	First three natural periods (s)
B30	30 / 105	30 × 30 / 15 × 15	0.65 / 0.40	0.45 / 0.30	0.25	7.50 × 0.50 × 06	0.45 × 0.80 / 0.30 × 0.80	325.97	2.26 / 0 .69 / 0.33
B45	45 / 157.5	30 × 30 / 15 × 15	1.0 / 0.70	0.55 / 0.35	0.25	7.50 × 0.70 × 0.80	0.55 × 0.80 / 0.35 × 0.80	616.19	3.81 / 1.12 / 0.57

Table 2. Equivalent modal masses of the prototype buildings (% of the total mass)				
Building	**Mode No.**			
	1	**2**	**3**	**4**
B30	68.92	17.12	4.84	2.40
B45	65.26	18.8	5.39	2.64

NUMERICAL MODELLING OF THE STRUCTURAL BEHAVIOUR

The nonlinear behaviour of the prototype buildings was modelled as is shown in Figure 3 and is explained in this section. Nonlinear static and dynamic analyses have been performed using the software SeismoStruct (SeismoSoft 2011).

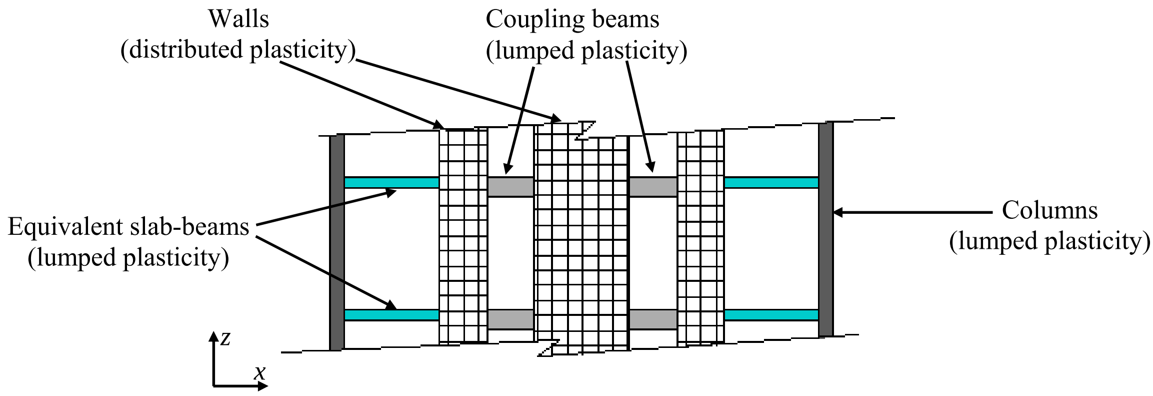


Figure 3. Nonlinear modelling of the structure of the buildings

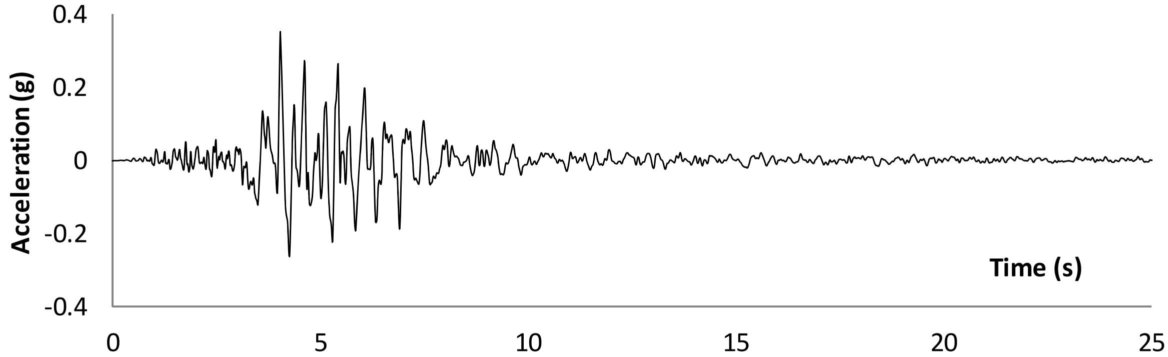
Columns and beams are modelled as linear elements with plastic hinges located at their ends (lumped plasticity) while the structural walls are modelled with fibber models (distributed plasticity). The floor slab needs to be taken into account since it contributes considerably to the lateral stiffness of the structure (Zekioglu et al. 2008). In this study, the influence of the floor slab is modelled by an equivalent slab-beam model based on the concept proposed in (Hwang and Moehle 2000). The equivalent slab-beam is an elastic flexural member with rectangular section; plastic hinges are defined at the ends of the element. The nonlinear behaviour of the concrete is represented by a five-parameter constant-confinement concrete model (Mander, Priestley, Park, 1988; Martínez-Rueda, Elnashai, 1997; Madas, 1993); the confinement effect is described by an effective confinement stress which depends on the longitudinal and transverse reinforcement. Since this model can experience numerical instabilities under large displacements, the modifications suggested by (Martínez-Rueda, Elnashai, 1997) are considered; the therefrom-arising model can predict the strength and stiffness degradation under cyclic motion. The behaviour of the reinforcement steel is described by uniaxial bilinear constitutive laws with 5‰ kinematic strain hardening; the hardening rule for the yield surface is a linear function of the increment of plastic strain. The damping is represented by a Rayleigh model (linear combination of the mass and initial stiffness matrices) The geometric nonlinear effects are also considered (second-order analysis).

CONSIDERED SEISMIC INPUTS

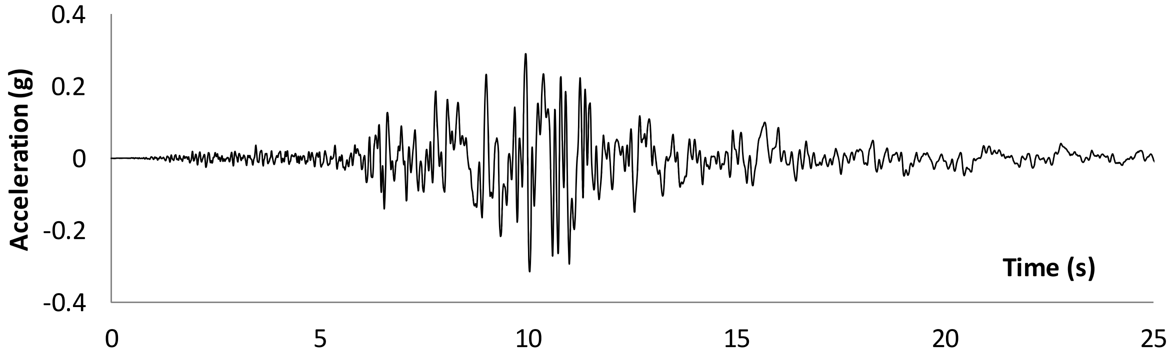
Three strong seismic inputs have been selected; given the big damaging potential of the impulsive registers, all of them are impulsive. Table 3 depicts the most relevant features of the considered inputs (PEER). I_A is the Arias Intensity (Arias 1970) given by $I_A = \frac{\pi}{2\,g}\int \ddot{x}_g^2\, dt$ where $\ddot{x}_g$ is the input ground acceleration; the Arias intensity is an estimator of the input severity. I_D is the dimensionless seismic index (Manfredi 2001) given by $I_D = \frac{\int \ddot{x}_g^2\, dt}{PGA\, PGV}$. The dimensionless index accounts for the velocity pulses content; it is generally assumed that $I_D < 10$ corresponds to impulsive registers and $I_D > 10$ corresponds to vibratory ones. The Triffunac duration is defined as the time between the 5% and the 95% of the Arias Intensity I_A (Triffunac, Brady 1975). The hypocentral distance corresponds to the straight separation between the hypocentre and the recording station. The closest distance corresponds to the shortest way to the rupture surface. v_{s30} is the average shear wave velocity in the top 30 m of the foundation soil. Figure 4 displays the time histories of the inputs and Figure 5 represents response spectra of relative displacement and of absolute acceleration.

Table 3. Considered input registers

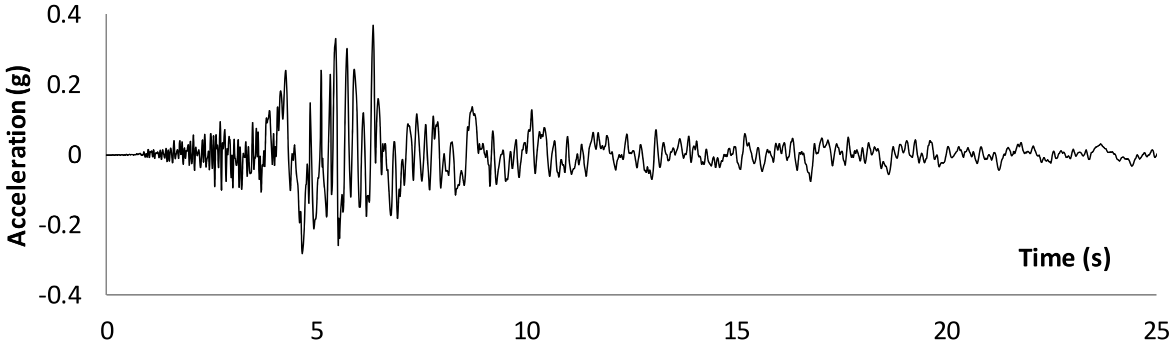
Earthquake	Date	M_w	Hypocentre depth [km]	Station	Component	PGA [g]	PGV [m/s]	PGD [cm]	I_A [m/s]	I_D	Triffunac duration [s]	Hypocentral distance [km]	Closest distance [km]	v_{s30} [m/s]
Imperial Valley	15/10/1979	5.2	9.5	5115 El Centro Array	H-E02140	0.315	0.315	14.12	1.265	8.11	8.93	20.39	15.33	188.8
Loma Prieta	18/10/1989	6.9	17.5	47381 Gilroy Array	G03090	0.367	0.447	19.25	1.348	5.23	11.25	35.93	12.82	349.9
Friuli	06/05/1976	6.5	5.1	8012 Tolmezzo	TMZ000	0.351	0.22	4.1	0.78	6.43	4.21	20.87	15.82	424.8



(a) Friuli

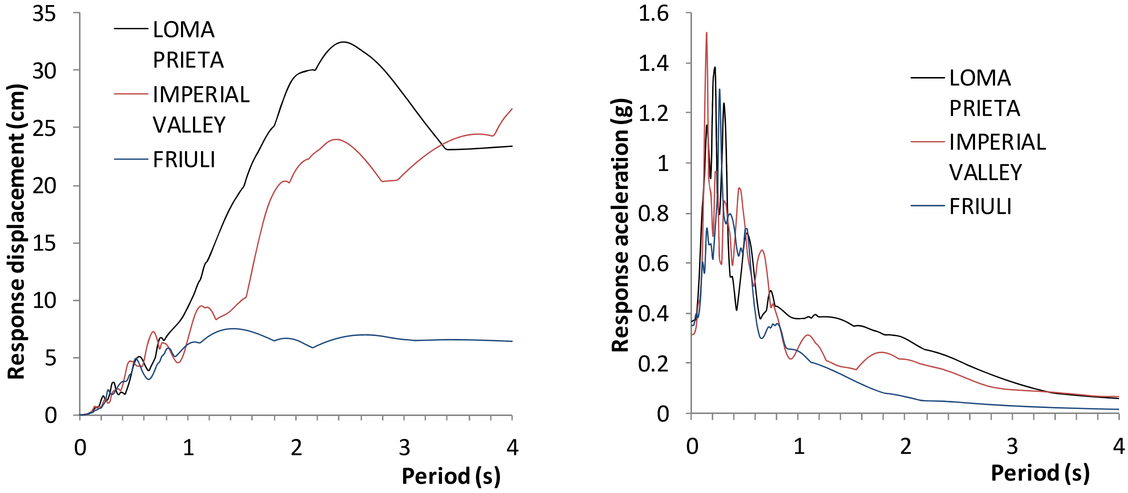


(b) Imperial Valley



(c) Loma Prieta

Figure 4. Selected ground motions



(a) Relative displacement spectra (b) Absolute acceleration spectra

Figure 5. Response spectra of the selected ground motions

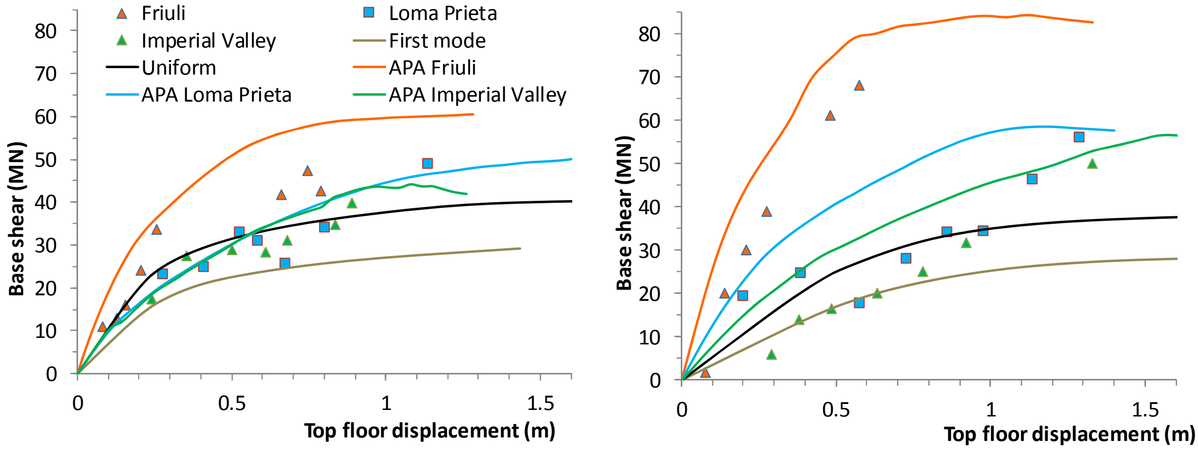
NUMERICAL RESULTS

The following types of analyses have been carried out:

- **Ordinary pushover**. Two different lateral load patterns are considered, uniform and proportional to the first mode (according to the Eurocode 8). The target displacements (performance points) for the selected inputs are obtained by the N2 method (Fajfar et al. 1996).
- **Modal pushover**. The first three modes are considered.
- **Adaptive pushover**. The previous calculations are based on an invariant load pattern during the analysis; to take into account the variability of vibration properties due the stiffness degradation, adaptive pushover analysis is performed. In this work, a displacement-based approach is used. The influence of the frequency content of a specific input can be considered in this method, by adopting a given response spectrum. Thus, in this work, adaptive pushover is carried out considering the three different displacement response spectra shown in Figure 5.
- **IDA**. To compare the shape of the capacity curves obtained by the different load patterns with those obtained through adaptive pushover, Incremental Dynamic Analyses (IDA) (Vamvatsikos et al. 2002) have been carried out for each selected ground motion. Incremental dynamic analysis is a parametrical method where a structure is subjected to a series of ground motion records, which are scaled to multiple levels of intensity. Then, joining all the points that correspond to different levels of intensity, we can build a dynamic capacity curve. From IDA results, different criteria may be used to obtain the corresponding point to draw the dynamic capacity curve. In the present work, each point corresponding to different levels of intensity was selected considering the maximum top displacement vs. the corresponding maximum base shear for each NLRHA found in a time window of ±0.25 s. In a wide sense, the IDA results are taken as an "exact" (or reference) solution.

Figure 6 displays the capacity curves of the two prototype buildings obtained by ordinary pushover methods, by modal pushover and by incremental dynamic analyses for the three selected inputs. For both prototype buildings, it can be observed that each lateral load pattern leads to different capacity curves. The load distribution which is proportional to the first mode is the lower bound of all the static capacity curves. The dynamic capacity curve of each ground motion is below its corresponding adaptive curve. In both buildings the adaptive curves corresponding to Friuli displacement spectra are the upper bound.

The story displacement is obtained using target displacement of each method to determinate the displacement of each story. It can be observed that displacement shape obtained by adaptive pushover follows better the obtained from reference solution (IDA)



(a) Building B30 (b) Building B45

Figure 6. Capacity curves of the prototype buildings

The accurate estimation of interstory drift and its distribution along the height of the structure is very important in seismic engineering since the structural damage is directly related to the interstory drift. Therefore, Figures 7 and 8 present the results in form of interstory drift and story displacement for each building, method and ground motion. In the case of adaptive pushover, the structure was pushed until the displacement obtained by nonlinear dynamic analysis for each seismic input.

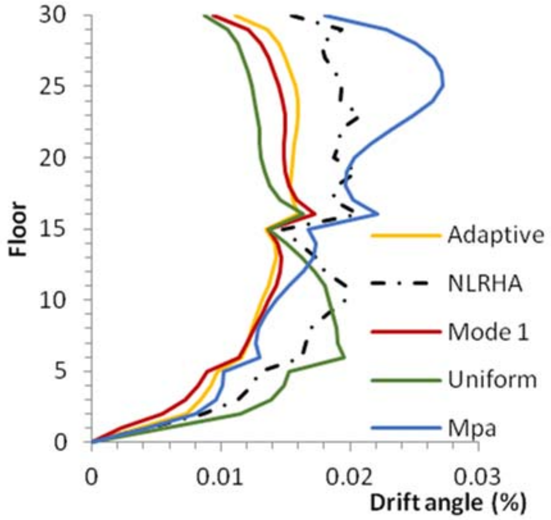
To quantifying the accuracy of the considered methods in determining the interstory drifts, the relative error ε_j is defined as:

$$\varepsilon_j = \frac{\Delta_{j,NLRHA} - \Delta_{j,Method}}{\Delta_{j,NLRHA}} \qquad j\text{: floor number}$$

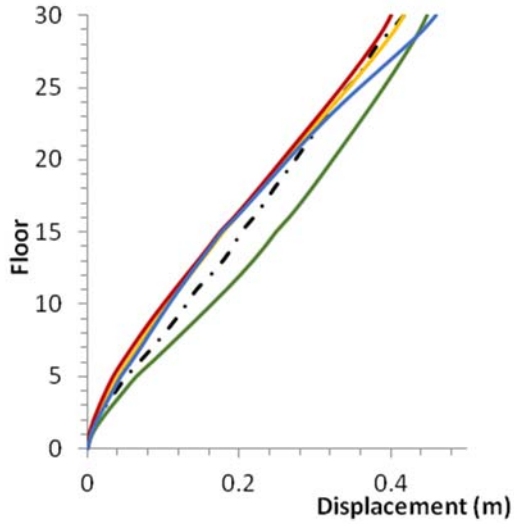
A positive value of ε_j indicates a non-conservative estimation of the response in the *j*-th floor.

Figure 9 displays, similarly to Figures 7 and 8, the drift errors for each building, method and ground motion.

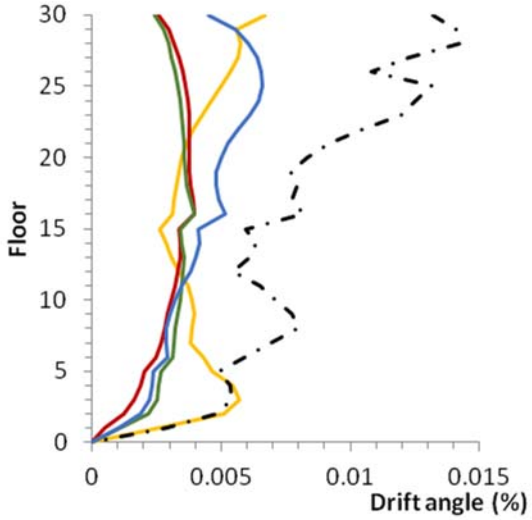
Figure 9 shows that the minimum average error corresponds to the MPA procedure with 28.5%. The average error for the adaptive pushover method is 37% for both case studies and the maximum error corresponds to uniform load pattern with 61%. It can be observed that the error due to uniform and first mode proportional lateral load is bigger in the building B45 since those analysis cannot take into account the higher-mode effects.



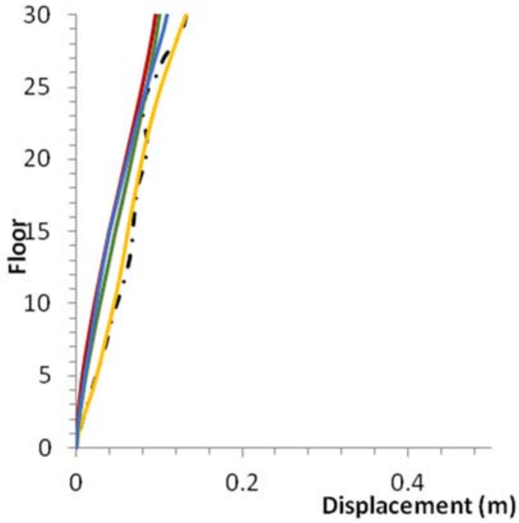
(a) Drift displacement. Loma Prieta



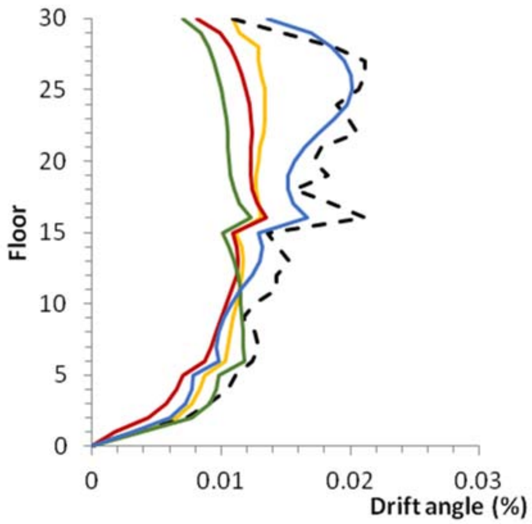
(b) Relative displacement. Loma Prieta



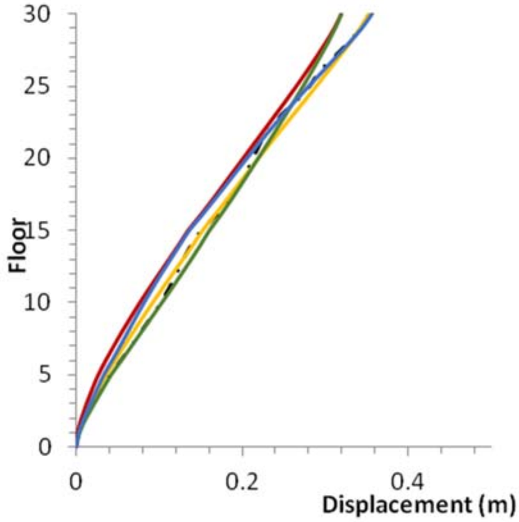
(c) Drift displacement. Friuli



(d) Relative displacement. Friuli

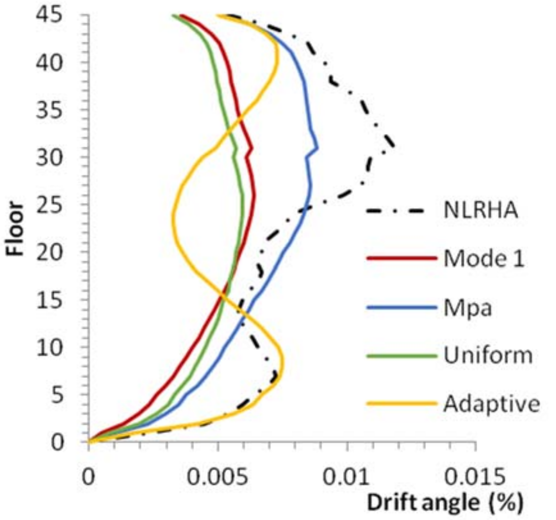


(e) Drift displacement. Imperial Valley

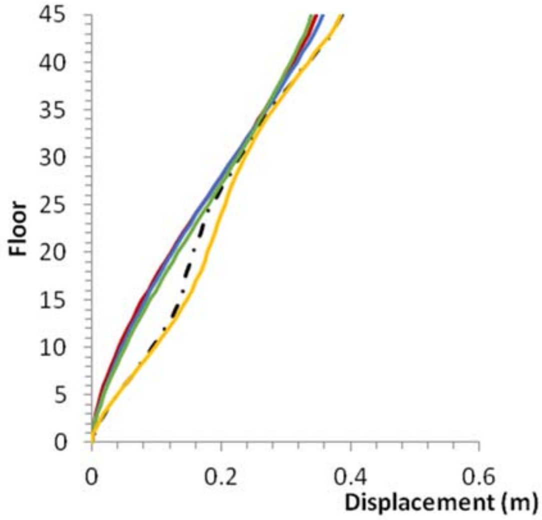


(f) Relative displacement. Imperial Valley

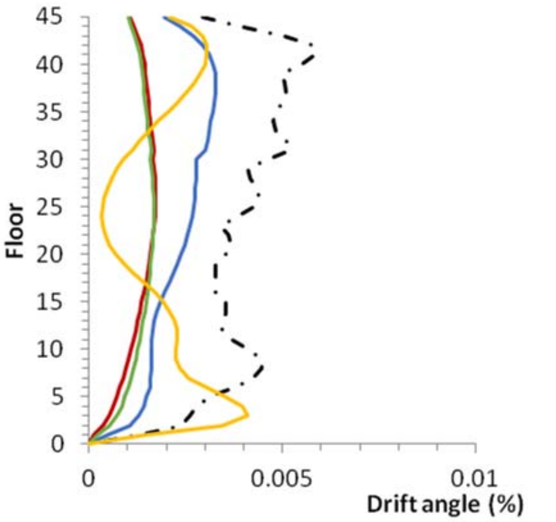
Figure 7. Interstory drift and horizontal relative displacement of the prototype building B30



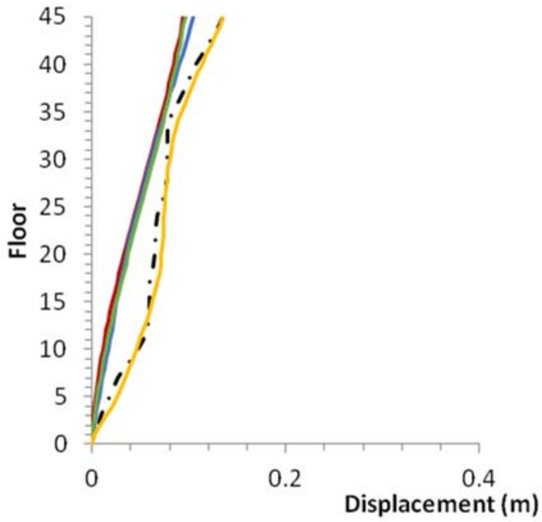
(a) Drift displacement. Loma Prieta



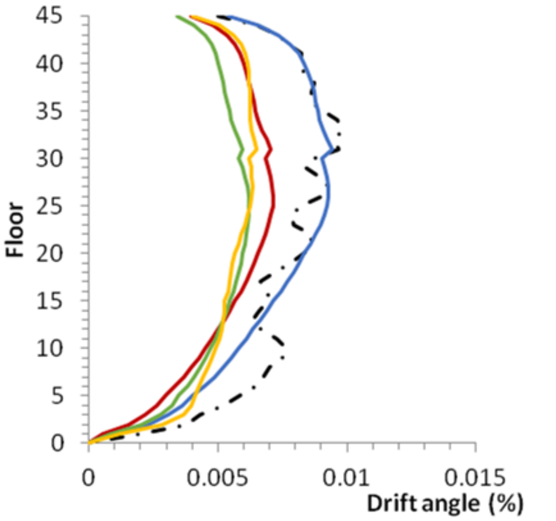
(b) Relative displacement. Loma Prieta



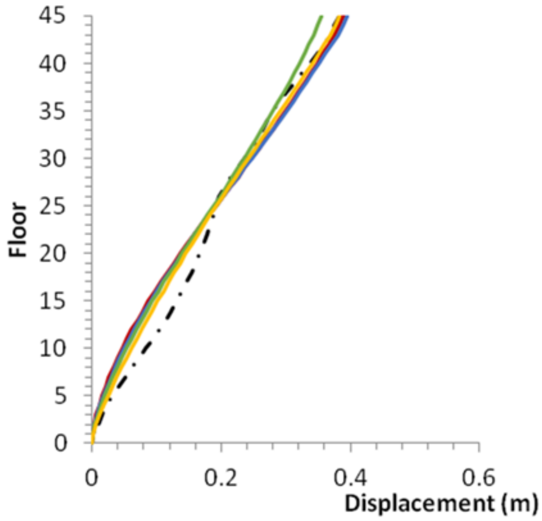
(c) Drift displacement. Friuli



(d) Relative displacement. Friuli

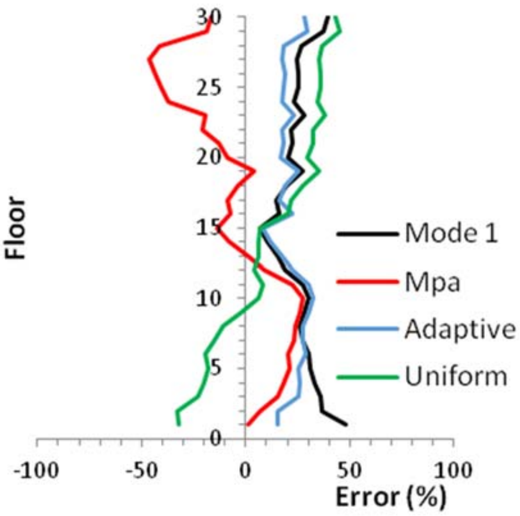


(e) Drift displacement. Imperial Valley

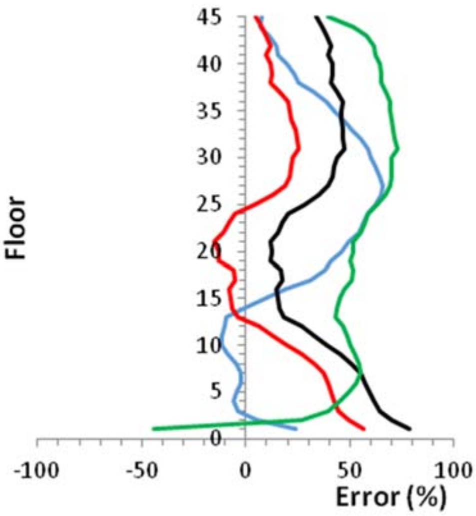


(f) Relative displacement. Imperial Valley

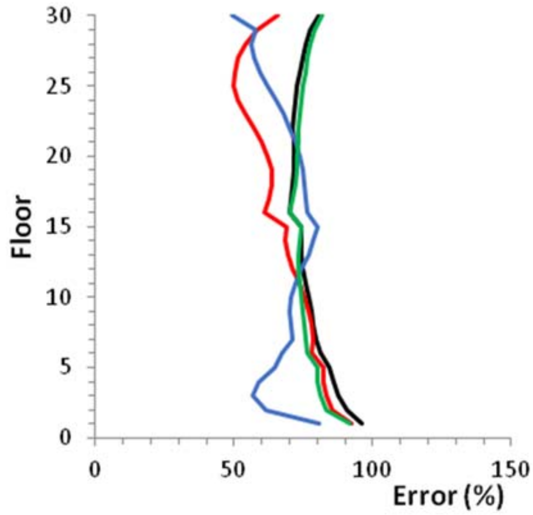
Figure 8. Interstory drift and horizontal relative displacement of the prototype building B45



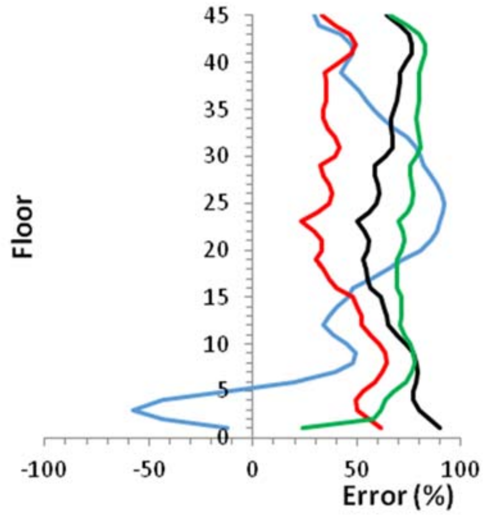
(a) Building B30. Loma Prieta



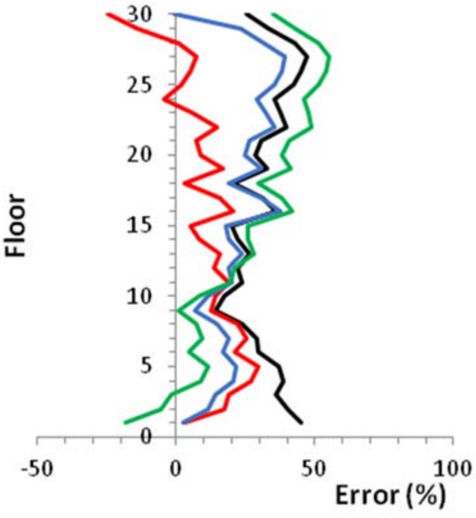
(b) Building B45. Loma Prieta



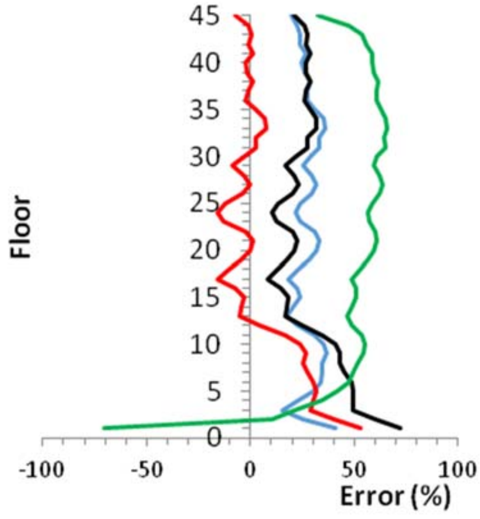
(c) Building B30. Friuli



(d) Building B45. Friuli

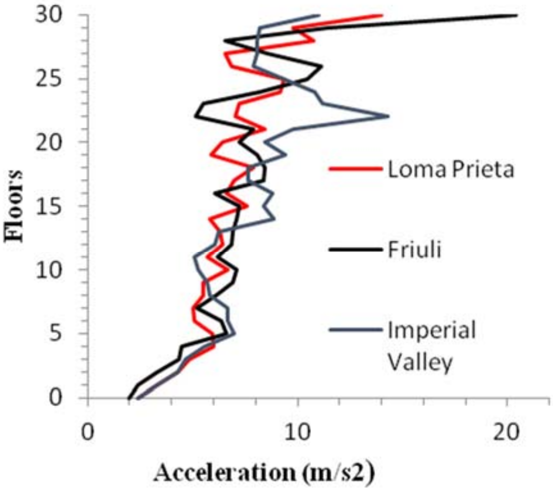


(e) Building B30. Imperial Valley

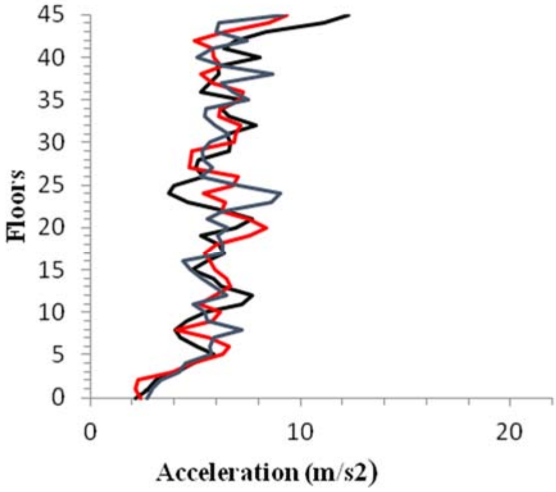


(f) Building B45. Imperial Valley

Figure 9. Interstory drift error for the B30 and B45 prototype buildings



(a) Building B30



(b) Building B45

Figure 10. Maximum absolute accelerations of the prototype buildings

CONCLUSIONS

This paper presents a numerical study about the reliability of nonlinear static analyses (conventional, modal, and adaptive pushover) for high-rise buildings. The study consists of comparing the results for two 30 and 45-storey representative prototype buildings with nonlinear dynamic analyses for a number of representative seismic inputs. Main preliminary conclusions are:

- The results of the time-history analyses for the selected inputs show high scattering.
- All the static procedures, except the modal pushover in few exceptional cases, underestimate the interstory drift, mainly in the top storeys.
- **Conventional pushover**. **Uniform**. The interstory drifts are clearly underestimated in the top storeys, and can be slightly overestimated in the bottom storeys.
- **Conventional pushover**. **Mode 1**. The interstory drifts are clearly underestimated, mainly in the top storeys.
- **Modal pushover**. Modal pushover yields better results than conventional pushover does; the approximation is better in the top storeys. The interstory drifts are slightly underestimated.
- **Adaptive pushover**. Adaptive pushover yields better results than conventional pushover does; the approximation is better in the bottom storeys. The interstory drifts are mainly underestimated. Comparison between modal and adaptive pushover shows that, in general, the results from modal pushover are closer to the dynamic results.

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