Pushover analysis in the evaluation of the seismic response of steel frames

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ABSTRACT: Nowadays pushover analysis has great importance among the methods for the evaluation of the seismic response of structures. Unfortunately, it provides results which sometimes strongly depend on the type of analysis of the generic step (static or modal) and on the load pattern. In the present paper the response of different typologies of seismic resisting frames is analyzed by means of pushover and step-by-step dynamic analyses at the aim of highlighting such differences. The results of the dynamic analyses of *Moment Resisting Frames, Eccentrically Braced Frames* and *Tied eccentrically Braced Frames* are compared with those obtained by means of pushover analyses in which different invariant and adaptive load patterns are used. The comparison of such results allows some interesting observations regarding the range in which the pushover analysis, with appropriate load patterns, provides a good estimation of the dynamic seismic response.

1 INTRODUCTION

Among the technical tools devoted to the prediction of the seismic response of structures the inelastic static analysis (commonly known as *pushover analysis*) has acquired in the last decade a gradually increasing importance. It is considered by different building codes and technical documents (e.g. NEHRP Guidelines for the Seismic Rehabilitation of Buildings - FEMA 273-4 and Methodologies for Post-Earthquake evaluation and repair of concrete and masonry buildings - ATC40) within methodologies aiming at the correct evaluation of the seismic response and vulnerability of structures. The importance of a study focused on the calculation of its reliability in approximating the results of the dynamic response in correspondence of different levels of the seismic action proceeds from its simplicity of use with respect to dynamic analysis.

The present paper starts from the results of previous studies where some qualities and defects of the pushover analysis have been underlined (e.g. Mwafy & Elnashai, 2001; Bracci et al., 1997; Requena & Ayala, 2000; Yang & Wang, 2000) and focuses the attention on the importance of the pattern of the static equivalent forces with respect to the above-mentioned target of pushover analysis. It represents the development of a previous study carried out with reference to eccentrically braced frames only. The range of investigation has now been extended to other typologies of steel frames in order to draw more general conclusions.

2 STRUCTURAL MODELS

An eccentrically braced frame, a tied braced frame and a moment resistant frame have been examined as representative of the most common structural typologies used at present for seismic resistant steel buildings. In such plane structures the mechanisms resistant to the seismic actions are obviously different. Indeed, while in braced frames horizontal forces are essentially resisted by means of axial internal actions, in moment resisting frames the same forces are resisted by means of a structural behavior which is essentially governed by shear and flexure. In addition, in order to investigate the reliability of pushover analysis in systems in which higher modes of vibration may even strongly influence the structural behavior, the eccentrically braced frames have been considered twelve stories high and the moment resisting frame has been supposed as constituted by nine stories.

2.1 Eccentrically braced frames

Within the seismic design, a global mass of 146.8 kNsec^2/m has been considered present at each story of the frames.

Following the suggestions of the most of building codes short links have been adopted for both the eccentrically braced frames. Furthermore, at the aim of nullifying the interaction between deck and link, two members have been introduced at each level instead of the traditional single section (Perretti 1999).

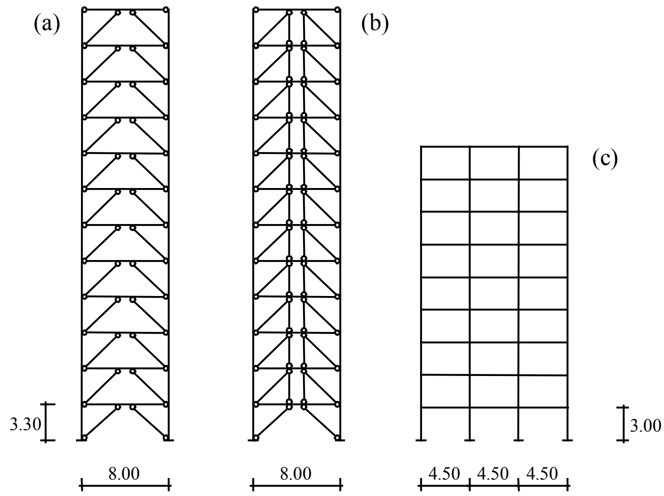

Figure 1. Schemes of the examined frames: EBF (a), TBF (b), MRF (c)

While the first sustains the vertical loads transmitted by the deck, the second resists the horizontal actions and constitutes the link itself. Such technical choice also allows, both in the phase of design and of evaluation of the structural response, a more reliable determination of the actions which apply to the link, otherwise affected by the imprecision due to the interaction between the deck and the link.

In order to simplify the construction of the joints of the eccentrically braced frame and at the aim of reducing the stress state at the lower story columns, hinged joints have been considered in such structures between beams and columns, columns and foundation and at the ends of the braces.

Furthermore, according to the capacity design criterion, columns, braces and ties (the last ones in the TBF only) have been designed so as to remain in the elastic field till the attainment of the ultimate plastic rotation of a whatever link. The shear V_u transmitted by links in correspondence of their failure is supposed to be 1.5 times the plastic shear V_p, owing to the hardening of steel. In order to take account of other uncertainties related to geometrical and mechanical characteristics, this value is further increased by means of a factor 1.2 so as to effectively grant that yielding or buckling will never occur in braces and ties before links have reached their ultimate rotational capacity.

The design value of the axial force in columns is generated by the presence of the vertical loads and by the action of yielded links. With reference to the evaluation of such last element in traditional eccentrically braced frames, an estimate of the probability that links reach their ultimate shear contemporarily at all the stories has suggested the designer decreasing the amplification factor previously described as a function of the level. Therefore, in the EBF the overall amplification factor of the share of the axial force of columns deriving from the yielding links has been assumed equal to 1.8 for the upper four stories; owing to the above-mentioned considerations, it has been fixed equal to 1.5 for the mid-height four stories and to 1.2 for the lower four stories. The reduction of the same amplification factor has not been taken into account for the tied braced frame owing to the better behavior of such systems.

Further information on the design of the eccentrically braced frame and on the tied braced frame may be found in (Ghersi & al. 2000) and (Ghersi et al. 2003), respectively.

2.2 Moment resisting frame

The moment resisting frame (Fig. 1) is endowed with rigid connections. The vertical loads acting on the beams within the seismic condition are equal to 16.6 kN/m. The mass is 22.8 kNsec²/m at each story.

The present frame has been designed according to a particular procedure (Neri 1999, Ghersi et al. 1999) aiming at reaching the structural failure by means of a global collapse mechanism. The cross-section of the beams has been designed so as to bear the vertical loads and to verify the limits defined by Eurocode 8 for the design interstory drifts. Indeed, while the maximum value of the bending moment caused by the vertical loads in the non-seismic condition is equal to 62.0 kNm the flexural strength of the beams (IPE 330 - Fe430) is equal to 201.0 kNm.

On the basis of a parametric analysis the design bending moment of the bottom cross-section of the first order columns has been evaluated as a function of the number of stories, number of bays and flexural strength of the beams (Ghersi et al. 1999).

Once selected the cross-sections of the beams and those of the first order columns, the design bending moment of the columns of the other stories has been fixed so as to favor the structural collapse into a global mode. In order to reach such a goal the ultimate multiplier of the horizontal forces has been determined by means of the limit analysis theory. Hence, the sum of the bending moments acting in the collapse configuration at the bottom or at the top of the columns of the generic story has been evaluated by rotational equations of equilibrium. At this end it is to be noticed that the axial force of columns has been evaluated by adding the shear forces transmitted by yielded beams at collapse to the effect of the vertical loads. Finally, the design value of the flexural strength required by the single column has been estimated at each story by uniformly dividing the previous sum of moments between the columns.

2.3 Dynamic properties of the structures

Although the frames under examination are characterized by non-negligible higher modes of vibration (Tables 1-2) no one of them has been designed by modal analysis. In all cases equivalent static forces have been considered as linearly increasing with the height of the buildings. Owing to such aspect of the design and to its own low redundancy, the eccentrically braced frame is selected so as to be representative of systems having structural defects and very undesirable seismic behavior. On the contrary, the response of the other two structures is typical of systems quite well designed and characterized by a widespread yielding of members before failure.

Table 1. Fundamental periods of vibration

T	EBF	TBF	MRF
(sec)	1.83	1.59	1.18

Table 2. Modal participation factors

Mode	EBF	TBF	MRF
1 st	0.659	0.679	0.785
2 nd	0.207	0.214	0.099
3 th	0.067	0.058	0.042
4 th	0.027	0.022	0.025
5 th	0.008	0.011	0.017
6 th	0.012	0.006	0.012
7 th	0.007	0.004	0.009
8 th	0.005	0.002	0.007
9 th	0.003	0.002	0.003
10 th	0.002	0.001	-
11 th	0.002	0.001	-
12 th	0.001	0.001	-

3 NUMERICAL ANALYSES

P-Δ effect has been considered both in dynamic and in pushover analyses. In such analyses the collapse has been conventionally individuated by the attainment of the ultimate plastic rotation of links (when present), beams and columns. In virtue of some code provisions the ultimate plastic rotation of the links (which have been designed to be short according to the definition of most of the building codes) has been fixed equal to 0.09 rad. Consequently, the shear hardening ratio of link cross-sections has been calculated so as to reach the ultimate shear in correspondence of the ultimate plastic rotation. Instead, the rotational capacity of beams and columns has been fixed to 0.03 rad. The hardening ratio of column cross-sections has been assumed equal to 0.03.

3.1 Dynamic analyses

In order to carry out dynamic analyses by means of accelerometric signals consistent with a unique elastic response spectrum, ten accelerograms matching the elastic response spectrum proposed by Eurocode 8 (1994) for soil C and characterized by a damping factor equal to 0.05 have been artificially generated. A Rayleigh damping has been considered in the analyses: the related coefficients have been assigned so as to have a damping factor equal to 0.05 in correspondence of two periods of vibration of the structures properly selected. The dynamic analyses have been carried out by means of the program DRAIN-2DX.

3.2 Pushover analyses

Static pushover analyses have been carried out by using two renowned classes of load patterns, named *invariant* and *adaptive* load patterns. As generally recognized, load patterns are defined invariant if they cannot change within the structural analysis. Instead, they are defined adaptive if some their updating (analytically specified) is performed in the attempt of following the modification of the stiffness properties of the system caused by damage, and consequently, the variation of the structural response to the examined accelerometric signal.

Within all the load patterns a further classification has been considered. Load patterns not directly connected to the dynamic properties of the structures or seismic events (e.g. constant and inverted triangular load patterns) will be called *semi-empirical* and identified later by the letter A. Differently, load patterns strictly related to the above mentioned properties will be referred as *theoretical.* They will be identified later by the letter B if invariant and by the letter C if adaptive.

3.2.1 Semi-empirical load patterns

– Load Pattern A1

The equivalent static forces F_i are proportional to the mass m of the i^{th} story:

$$F_i = \frac{m_i}{\sum_j^{n_s} m_j} V_b \qquad (1)$$

being n_s the number of levels and V_b the design value of the base shear.

– Load Pattern A2

The horizontal forces are proportional to the mass m and to the height h of the generic floor with respect to the base of the structure:

$$F_i = \frac{m_i \cdot h_i}{\sum_j^{n_s} m_j \cdot h_j} V_b \qquad (2)$$

In the investigated structures, having equal mass and interstory height at all floors, the load pattern A1 corresponds to the constant distribution while that defined as A2 individuates the inverted triangular distribution of horizontal forces.

3.2.2 Theoretical load patterns

– Load pattern B1 (C1)

The static forces are proportional to the eigenvector components of an equivalent mode of vibration (Requena & Ayala, 2000) evaluated by means of the relation:

$$\overline{\phi}_i = \sqrt{\sum_{j=1}^{n_m} \left(\phi_{ij} \cdot \Gamma_j \right)^2} \tag{3}$$

being Γ_j the participation factor and n_m the number of modes taken into account for the evaluation of the equivalent mode of vibration.
The equivalent static forces may be calculated by means of the expression:

$$F_i = \frac{m_i \cdot \overline{\phi}_i}{\sum_{k=1}^{n_s} m_k \cdot \overline{\phi}_k} V_b \tag{4}$$

– Load pattern B2 (C2)
The intensity of forces F_i (Freeman et al., 1998) depends on the dynamic properties of the system and, differently from B1, on the spectral pseudo-accelerations S_a of the modes of vibration taken into account for the targeted evaluation:

$$F_i = \sqrt{\sum_{j=1}^{n_m} \left(\Gamma_j \ \phi_{ij} \ S_{a\,j} \ m_i \right)^2} \tag{5}$$

– Load pattern B3 (C3)
The equivalent static forces are derived as difference of modal story shears Q of contiguous levels:

$$F_i = Q_i - Q_{i+1} \tag{6}$$

– Load pattern B4 (C4)
Analogously to model B3, the horizontal forces of this load pattern are obtained as a function of the difference of the modal story bending moments $\mathfrak{M}$ of contiguous levels:

$$F_i = \frac{\mathfrak{M}_i - \mathfrak{M}_{i+1}}{h} \tag{7}$$

In the load patterns B3-B4 (C3-C4) modal story shears and bending moments are calculated by means of the SRSS combination rule of the quantities related to the modes of vibration under examination.

4 RESULTS

4.1 Dynamic analyses

An *incremental dynamic analysis* has been carried out by considering increasing values of the peak ground acceleration from zero to 1.50 g with step 0.05 g. Hence, the obtained maximum values of the base shear and those of the top displacement have been plotted (Fig. 2) so as to describe the reference *dynamic pushover* (Mwafy & Elnashai 2001)

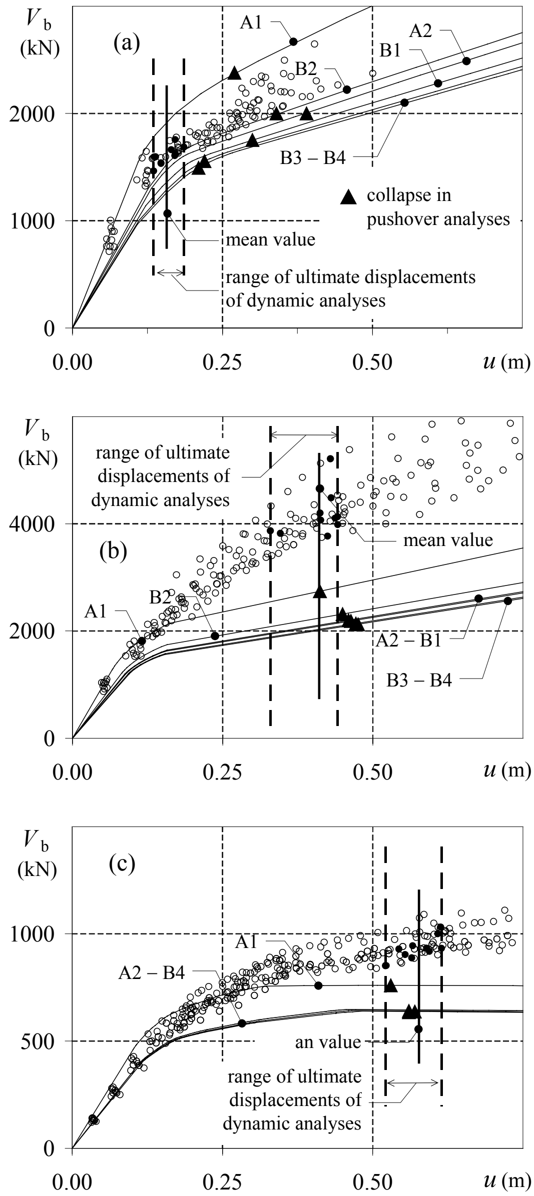

Figure 2. Dynamic pushover and capacity curves corresponding to invariant load patterns: EBF (a), TBF (b), MRF (c)

4.2 Pushover analyses

Pushover analyses have been carried out with reference to all the load patterns described in section 3.2. The superimposition of the capacity curves to the dynamic pushover (Fig. 2) highlights the different degree of reliability of pushover analysis as a function of the load pattern and of the structural scheme under examination. Indeed, in the EBF a great scattering in the values of the ultimate top displacement predicted by pushover analyses may be noticed. In addition, the entire pushover graphs seem to be rather different from each other. Between the experimented invariant load distributions only patterns B3-B4 lead to reliable results. Obviously, the reliability of pushover analysis depends, when using theoretical models, on the number of modes of vibration taken into account. Greater is the number of modes of vibration into account better is the reliability of the results. In particular, the curves shown in Figure 2 derive by considering a number of modes equal to the number of the floors. Differently equal to the number of the floors. Differently from the EBF, the compared analysis of dynamic pushover and pushover curves of either TBF or MRF shows that all theoretical load patterns other than the inverted triangular distribution lead to a good evaluation of the ultimate top displacement.

Unfortunately, in the present numerical analyses the base shear is almost always underestimated. Such discrepancy in the results has to be found in the specific desired target of the pushover analysis, in the effect of the higher modes of vibration and in the structural typology. Indeed, in structures the response of which is governed by a unique mode of vibration a correct distribution of equivalent static forces may catch the maximum values of whatever quantity of the structural response, because such maximum values occur simultaneously. On the contrary, when higher modes of vibration are important the maximum values of the response occur in different instants of the time history and correspond to different patterns of the equivalent static forces. For this reason, in such structures, whatever the load pattern is, only one aspect of the response may be pursued and possibly caught. Even if very questionable, if the structural damage is generally put is some connection with the absolute displacements of stories, primary aim of the pushover analysis may seem that of catching the shapes of the lateral displacements corresponding to different levels of the global deformation (top displacement). In virtue of their analytical expression and of their physical meaning load patterns B3-B4 (C3-C4) seem to be able to pursue such target in systems the seismic response of which is even strongly influenced by higher modes of vibration. Indeed, according to such patterns the static forces are defined so as to catch at each story the most probable value of the maximum story shears or overturning moments (which may be thought to be responsible of the absolute displacements if the structural behavior is governed by shear or overturning moment). As above-mentioned, other aspects of the response may be not predicted with the same precision by the same load pattern. Such inaccuracy may involve the distributions and levels of more local deformations (e.g. interstory drifts or plastic rotations of cross-sections), which may have, however, great importance for engineers.

At the aim of demonstrating how much the structural typology may influence the reliability in the prediction of the story shear, the response of the EBF is analyzed. As evident in Figure 2 also the base shear seems to be caught with good approximation by the load patterns B3-B4 because the low redundancy of the structure makes the story shear practically limited by the ultimate shear of links. In TBFs and in MRFs this does not happen. In such cases the amplification of the story shear with respect to the value caused by the first mode of vibration or by any other distribution of forces aiming at catching the absolute story displacements is very high.

The analysis of the story lateral displacements corresponding to the structural collapse (Fig. 3) confirms the impression given by Figure 2. In case of systems characterized by a poor seismic behavior and thus by the formation of partial collapse mechanisms only load patterns B3-B4 are able to follow the development of the structural deformation. In systems in which the global collapse mechanism is strongly pursued by means of a proper structural design, all the load patterns seem practically to be apt to predict with good reliability the structural response at collapse. Owing to the shape of the lateral displacements corresponding to the global collapse mechanism also the inverted triangular distribution of static forces shows acceptable results, comparable to those of the theoretical load patterns. Hence, some difference in the results of the pushover analyses may be expected if the overstrength of the elements which are devoted to remain in the elastic field until the structural collapse is reduced.

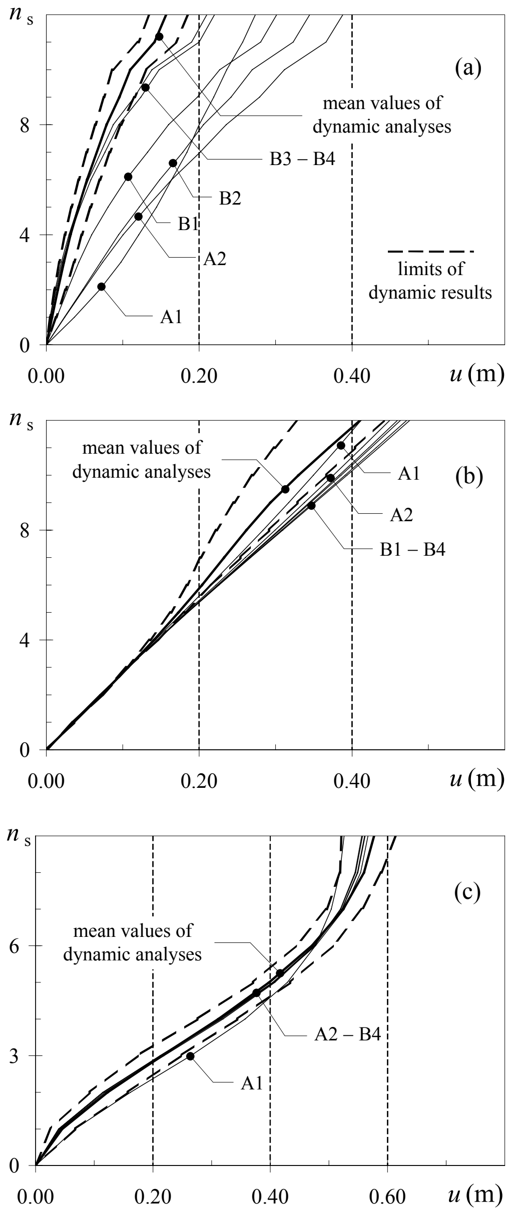

Figure 3. Lateral story displacements for invariant load patterns: EBF (a), TBF (b), MRF (c)

Differences between exact results and pushover analyses are very evident when referred to the collapse distribution of the maximum plastic rotations. At the aim of justifying such observation the ratio of the maximum demanded plastic rotations to their ultimate value is reported for some elements in Figure 4. The plots show the normalized plastic rotations of the links when referred to the EBF and TBF or those of the beam cross-sections where failure has firstly occurred if referred to the MRF. In almost all cases, the examined load patterns are able to predict the location of the cross-section that firstly collapses. But, where partial collapse mechanisms develop (e.g. the EBF under examination) only load patterns B3-B4 estimate with good approximation the spread of plastic deformations within the structure. In systems characterized by global collapse mechanisms, instead, quite all load patterns seem to catch the distribution of the normalized plastic rotations.

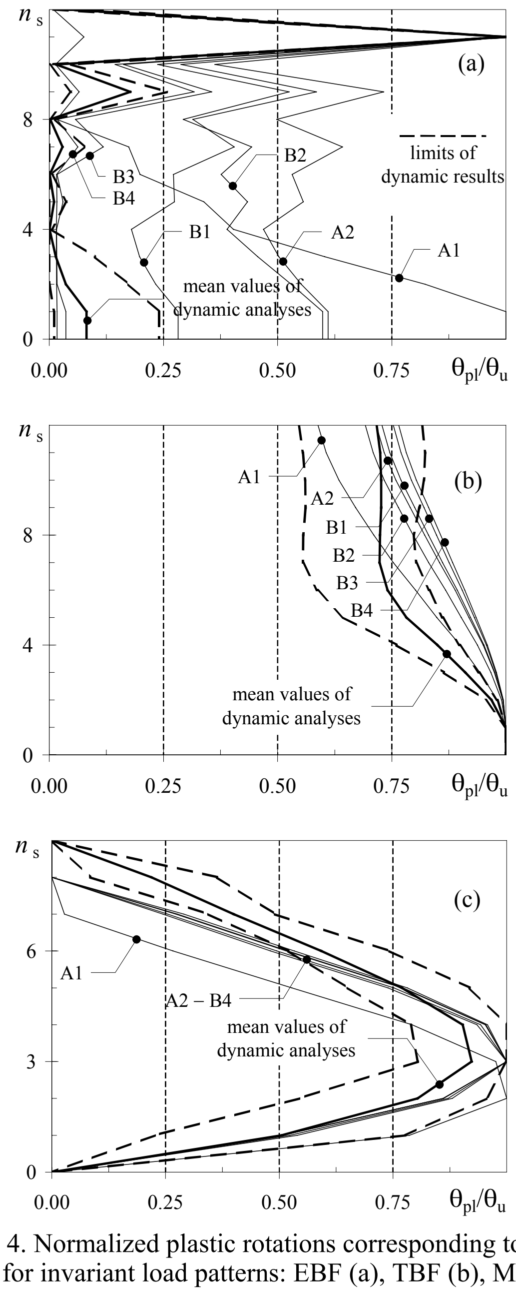

Figure 4. Normalized plastic rotations corresponding to the collapse for invariant load patterns: EBF (a), TBF (b), MRF (c)

Indeed, in the MRF the distribution of the maximum plastic rotations predicted by the presented pushover analyses is always quite accurate, with the exception of that produced by the uniform load pattern (A1). Some exception is found in the TBF, and it is not explainable yet. Finally, adaptive load patterns (not shown in the present paper for problems of space) produce slight improvements in the prediction of the dynamic response of structures.

5 CONCLUSIONS

The paper shows the analysis of the seismic response of some steel structures by means of pushover analyses. Attention is particularly focused on the influence of the load pattern on the reliability of the results in systems characterized by important higher modes of vibration.

In the paper a poor agreement has been particularly highlighted between the seismic response of structures which develop partial collapse mechanisms and the results related to the adoption of some of the most common invariant load patterns, suggested by building codes and researchers. Nevertheless, some other invariant load patterns have been individuated which give reliable results. Furthermore, adaptive load patterns have shown at present slight improvements in the reliability of the response.

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