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Simplified procedures for the seismic assessment of structural component demands

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Abstract. The implementation of performance-based design and assessment procedures in seismic codes leads to the need for an accurate estimation of local component demands. In the case of framed structures, these are usually defined by plastic or chord rotations. A rigorous estimation of these parameters is not straightforward, requiring not only the adoption of complex nonlinear structural models, but also of time-consuming numerical integration calculations. Moreover, the majority of existing codes and guidelines do not provide any guidance in terms of how these response parameters should be estimated. Therefore, the aim of this work is to evaluate and propose simplified procedures for the evaluation of structural component demands using both linear and nonlinear methods of analysis. To this end, four different steel buildings, designed according to different criteria, are analysed and their component demands assessed for increasing levels of seismic intensity.

Keywords: Seismic Assessment; Local Demands; Steel Structures

1 INTRODUCTION

The implementation of performance-based design concepts and assessment procedures in seismic codes requires from the analyst the need for an accurate estimation of local component demands in order to verify compliance criteria. In the case of framed structures, local component demands are commonly expressed in terms of plastic rotations in the case of steel members, and in terms of chord rotations in the case of RC members. The accurate estimation of these parameters is not straightforward, requiring not only the adoption of complex nonlinear structural models, but also of time-consuming numerical integration calculations. Moreover, the majority of existing guidelines and codes do not provide any guidance in terms of how these response parameters should be estimated. This situation arises, for example, when applying the linear analysis procedures prescribed in Part 3 of Eurocode 8 (CEN, 2005a) to steel structures. Despite the limitation of this type of analysis in providing reliable predictions of inelastic response parameters, the safety checks prescribed in the European code are largely based in the control of plastic member rotations.

Recently, a number of studies addressing this issue have been conducted (Günay and Sucuoglu 2010; Romão et al. 2010; Browning et al. 2008; Kosmopoulos and Fardis 2007) in which the accuracy of linear elastic analysis has been evaluated. On the basis of the results of more than a thousand non-linear dynamic analysis, Panagiotakos and Fardis (1999) developed rules for the estimation of mean and upper-characteristic peak inelastic member chord rotations from linear analysis. These mainly consist on the use of conversion factors on elastic chord rotations derived from modal response spectrum analysis or linear static analysis with inverted triangular equivalent lateral forces. For the mean value the authors proposed a value of about 1.0, while those for the upper-characteristic values the same authors proposed a value of about 1.5 over the height of the buildings. As referred by the authors, this proposal is essentially a generalization of the well-known equal-displacement rule of SDOF systems and its effectiveness is due to the fact that the fundamental period of the cracked elastic

structures considered is beyond the corner period of the input motion. However, the previous results were only derived from planwise regular and symmetric RC buildings, which raises some questions regarding its range of applicability.
As a result, Kosmopoulos and Fardis (2007) extended the previous work to asymmetric multi-storey RC buildings, having again concluded that elastic modal response spectrum analysis provides, on average, unbiased and fairly accurate estimates of member inelastic chord rotations. These conclusions were drawn from cases violating the linear analysis applicability criteria proposed by both EC8-3 and ASCE41-06 (ASCE, 2007), thus suggesting that there is room for re-examination and possible relaxation of the criteria, as already discussed in previous works (Romão et al. 2010; Fardis and Kosmopoulos 2007; Pinto and Franchin 2008).

Additionally, Günay and Sucuoglu (2010) proposed an improved linear elastic analysis procedure based on reducing the stiffness of structural members that are expected to respond in inelastic range in a single global iteration step, wherein inelastic chord rotations are determined on the basis of the equal displacement rule. The results obtained revealed that linear analysis can be effectively used, with such a simple modification, to predict the nonlinear seismic performance of structures, being at least as accurate as the prediction of nonlinear static procedures.

Local deformation demands can be alternatively measured in terms of story drifts (Gupta and Krawinkler 1999). In this case, the safety of the structure is assessed by comparing those values with indicative limit values proposed by guidelines and codes for various performance levels (e.g., and Bertero 2002; Gioncu and Mazzolani 2002; Grecea et al. 2004). Based on a study of a set of RC buildings, Browning et al. (2008) observed that, on average, the magnitude of the maximum story drift ratio calculated using nonlinear analysis is 1.5 times larger than that estimated using linear modal analysis, with a coefficient of variation of approximately 0.39. The location of the maximum inter-storey drift was also seen to vary significantly when using nonlinear and linear analysis. Several works proposing approximate methods to estimate these maximum lateral deformation demands can be found in literature (e.g., Ruiz-García and Miranda 2006; Akkar and Miranda 2005; Miranda and Reyes 2002).

In spite of the broad agreement that nonlinear-based procedures are a better tool to assess existing structures, linear elastic methods are, and will continue to be, used due to its relative simplicity and familiarity to practitioners, as confirmed in a survey conducted by Paret et al. (2011). Hence, further studies on linear analysis and proposals of new and more reliable procedures are required (Toranzo-Dianderas 2009). Therefore, and noting that the majority of studies conducted mainly focused on RC buildings, the aim of this work is to evaluate and propose procedures for the estimation of structural component demands using both linear and nonlinear methods of analysis. To this end, four steel buildings are analysed and the local component demands assessed for increasing levels of seismic intensity.

2 METHODS OF QUANTIFYING LOCAL DEFORMATION DEMANDS

From a code perspective, the assessment of existing buildings should be carried out by verifying the safety of each individual member of the structure. If any primary member does not verify safety, then the building fails its assessment. In steel moment-framed structures, these individual safety checks are typically based on the control of plastic hinge rotations. The quantification of this demand parameter in a beam is typical beam is usually carried out by assuming that the member can be analysed as a set of two independent cantilevers. This simplification results from the fact that under dominant lateral loading, the contraflexure points are localized somewhere close to the mid-span of the beam. However, other approaches have been proposed for comparison purposes with experimentally tested elements (Gioncu and Mazzolani 2002; Gioncu and Petcu 1997). Figure 1 represents a decomposition example of a simple structural system into standard cantilever beams.

As curvature demands increase locally, cross-sectional plastification begins to take place at all points where the yielding moment (M_y) is exceeded. Plastification spreads down the flanges, into the web, and eventually a fully plastified cross-section is reached at M_{pl} over some length of member designated as plastic hinge length L_{ph}. The accurate quantification of the member plastic rotation θ_{pl} can be defined as follows (Bruneau et al. 1998, Priestley et al. 2007):

$$\theta_{pl} = \int_{L-L_{ph}}^{L} \phi_{pl}(x)dx \tag{1}$$

where Φ_{pl} represents the plastic curvatures, which verify the condition $\Phi(x)> \Phi_y$, developed over L_{ph}, L is the length of the equivalent cantilever beam, defined as the distance from the end node to the point of contraflexure and Φ_y is the cross-sectional yielding curvature defined as M_y/EI.

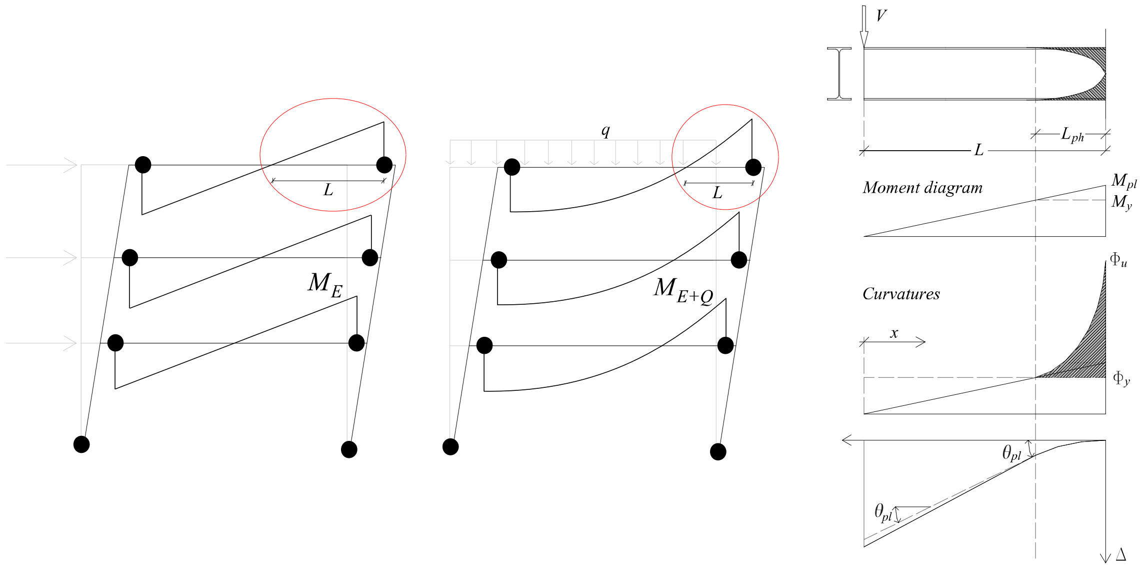

Figure 1. Decomposition of structure into standard cantilever beams and the concept of plastic rotation definition.

Alternatively, the member local deformation demands can be quantified in terms of chord rotations, which are defined as the angle between the chord connecting the end sections of the member to the contraflexure point and the tangent to the member axis at the end section (Figure 2). This is the approach followed by EC8-3 to assess RC members (Mpampatsikos et al. 2008, Romão et al. 2010).

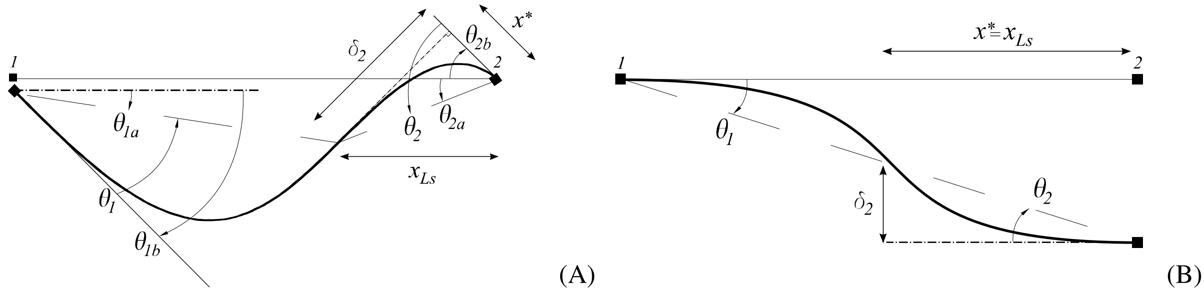

Figure 2. Examples of chord rotation definition: (A) typical beam and (B) simplified column.

The chord rotations θ_1 and θ_2 of the two structural member ends can be analytically quantified by means of the *Exact Integral Method* (*EIM*), as follows:

$$\theta_1 = \int_{x_{Ls}}^{L} \phi(x) \left(\frac{x_{Ls} - x}{L - x_{Ls}} \right) dx \tag{2}$$

$$\theta_2 = \int_{0}^{x_{Ls}} \phi(x) \left(\frac{x_{Ls} - x}{x_{Ls}} \right) dx \tag{3}$$

where x_{Ls} is the abscissa of the point of contraflexure and L the member length. However, the alternative *Exact Geometrical Method* (*EGM*) is commonly adopted due to its prompt applicability. In this case, chord rotations are defined in a geometric way. It can be seen from Figure 2 that $tan(\theta_2)=\delta_2/x^*$ which, under the hypothesis that θ_2 is small, leads to $x^* = x_{Ls}$ and hence $tan(\theta_2)= \theta_2$, which can be then simplified as follows:

$$\theta_2 = \delta_2 / x_{Ls} \tag{4}$$

Since the calculation of δ_2 may not be straightforward, θ_2 is defined by:

$$\theta_2 = \theta_{2a} - \theta_{2b} \tag{5}$$

where θ_{2a} represents the contribution of the deflection at x_{Ls} with respect to the initial member configuration and θ_{2a} corresponds to the nodal rotation, considering clockwise rotations as positive. Equally, θ_1 is defined as:

$$\theta_1 = \theta_{1a} - \theta_{1b} \tag{6}$$

having θ_{1a} and θ_{1b} the same meaning of θ_{2a} and θ_{2b}, respectively. It has been found (Romão et al. 2010) that in cases of frame elements under large deformation demands, θ_{1a} is approximately equal to θ_{2a}. In these situations, an *Approximate Geometrical Method* (*AGM-DR*) that considers member drift and nodal rotations for beams and columns can be used to compute the chord rotation without evaluating x_{Ls} by setting:

$$\theta_{1a} = \theta_{2b} = d_y / L \tag{7}$$

where d_y represents the relative transversal displacements of sections 1 and 2, neglecting the contribution of the axial deformation of the member. Assuming these approximations, θ_1 and θ_2 can be quantified without further difficulties from Equations (5) and (6). More detailed information on the quantification of chord rotation demands can be found elsewhere (Romão et al. 2010).

3 CASE STUDY DESCRIPTION

As mentioned above, the study presented herein was conducted considering four different steel buildings. The plan layout and the elevation view of a moment-resisting frame of the building are illustrated in Figure 1. Each building was designed according to different criteria. The first building, denoted as GB, was designed accounting only for gravity loads following the rules prescribed in Eurocode 3 (CEN, 2005b). The remaining three buildings were seismically designed according to Part 1 of EC8 adopting a value for the behaviour factor (q) equal to 4.0. The difference between the three buildings is found on the different limits considered for the inter-storey drift sensitivity coefficient (θ), which is the parameter used to check for the need to consider second-order effects in the analysis and design process. Thus, the SB1 building was designed assuming 0.2< θ_{max} < 0.3, the SB2 building was designed considering 0.1< $\theta_{max} \leq 0.2$, being the second-order effects taken into account by multiplying the relevant seismic action effects by a factor equal to 1/(1 - θ), and finally the SB3 building was designed in order to minimize the relevance of second-order effects ($\theta \leq 0.1$).

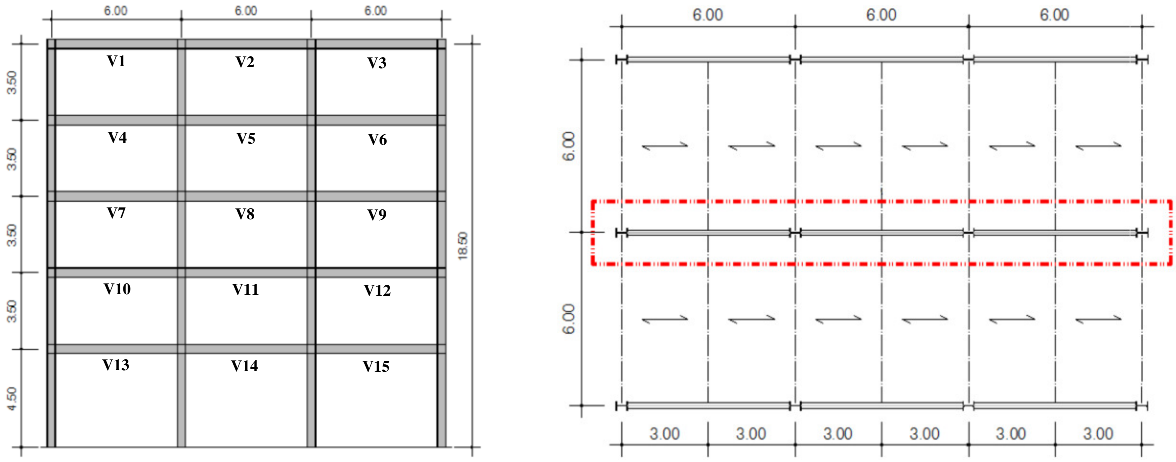

Figure 3. General features of the analysed structures.

The analyses of the frames were performed with the open source software OpenSees (PEER, 2011). Regarding the models used for nonlinear analysis, force-based beam-column elements were adopted considering 10 Gauss-Lobatto integration points along its length. Also, a cross-section discretization solution by fibers was followed and a bilinear elasto-plastic material model with 0.5% hardening was adopted for structural steel. The effect of the panel zones was neglected in this study. Modal analysis was firstly carried out for each frame with the aim of obtaining the dynamic characteristics of the buildings. The first three vibration periods of each frame are listed in Table 1.

Table 1. Dynamic characteristics of the buildings

Building	Periods of Vibration (s)		
	Mode 1	Mode 2	Mode 3
GB	1.63	0.50	0.26
SB1	1.50	0.48	0.25
SB2	1.20	0.39	0.20
SB3	0.90	0.38	0.13

The goal of this case study is not only to evaluate the effectiveness of each of the previously exposed methods of quantifying member deformation demands but also to address the issue of how to estimate plastic deformation demands in the context of linear elastic analysis. A key question on how to compute deformation demands from linear analysis which can be directly compared with the plastic rotation limits prescribed in EC8-3 should naturally be placed. The most suitable answer was found to be the determination of chord rotations, quantified on the basis of the *EGM* and *AGM-DR* methods described before. While the application of the *AGM-DR* is simple, based on the manipulation of nodal displacements and rotations, the *EGM* requires the calculation of the deflection at x_{Ls} to obtain θ_{1a} and θ_{2a}. This can be carried out by dividing each member into two cantilever beams with lengths equal to L- x_{Ls} and x_{Ls} with reference to nodes 1 and 2, respectively. Each cantilever beam is then treated individually and the deflection at its free node, equal to the deflection at x_{Ls}, calculated using the elastic integration method. In the present case study, since gravity loads were simply defined as point loads applied at the beam mid-span and the diagrams of bending moments are linear, the deflection can be calculated from node 2 as:

$$\delta = \delta_2 + \theta_{2b} x_{Ls} - V\, x_{Ls}^{\;3} \big/ 3EI \tag{8}$$

where δ_2 is the vertical displacement of node 2 and V the shear developed in the member, equal to M_2/x_{Ls}, being M_2 the bending moment at node 2.

Therefore, in order to evaluate the effectiveness of each method for quantifying chord rotation demands *per se*, thus excluding the variability associated with the type of method of analysis considered, pushover analyses were performed assuming both linear and nonlinear material behaviour. A fixed force pattern was defined as proportional to both the mass and height of each storey. The results were computed for total drift ratio (top displacement divided by building height) values of 1.0%, 2.5% and 4%. The following aspects will be considered in the discussion of results:

- The accuracy of $AGM\text{-}DR_{NL}$ comparing to the reference EIM_{NL} in the quantification of chord rotation demands when nonlinear material behaviour is considered;
- The ability of linear analysis to provide reasonable estimates of chord rotation by means of EGM_L and $AGM\text{-}DR_L$;
- The feasibility of using chord rotations to quantify plastic rotations.

4 RESULTS

Some of the main results obtained from this study are illustrated in Figure 4 to Figure 6. It is worth noting the remarkable influence of gravity loads in the distribution of plasticity in the members and hence the expected impact on the quantification of the chord rotations. A simplified expression to determine the rotation of the plastic hinge region, defined as $\theta_p=\delta/0.5L$, where δ is the beam deflection at midspan and L the beam span, is commonly found in seismic codes and guidelines (e.g. EC8, ASCE 41, etc.). This expression assumes that plasticity is symmetrically distributed along the beam length or, in other words, that equal plastic rotation demands develop at both ends of a beam. However, it is known (Castro et al. 2008) that in most cases this assumption is not valid, particularly when the level of gravity moments represent an important fraction of the beam flexural strength. Hence, it may be seen that when gravity loads are excluded, not only the chord rotation demands are equal at both beam ends (positive chord rotation values refer to node 1 and negative values to node 2), but also the maximum chord rotation value decreases about 30%, 23% and 20% for total drift values of 1.0%, 2.5% and 4.0%, respectively.

The results also indicate that as the lateral stiffness of the buildings increase (SB1 to SB3), an increasingly uniform distribution of plasticity along the building height is observed. This behaviour reflects the development of a plastic mechanism largely composed by beam hinges which results in a symmetrical distribution of chord rotation demands at the beam ends. Bojórquez et al. (2011) found similar results when assessing a set of buildings seismically designed according to the Mexico City Building Code, although referring that this equal distribution of rotation demands over the beams located at a particular story is due to the presence of rigid diaphragms.

Concerning the methods of quantifying local deformation demands, it may be seen that plastic behaviour tends to concentrate at the first and second stories of the building and that beams V13, V14 and V15 appear as critical and most demanded. Looking to the inter-storey drifts developed at those stories and the chord rotations at the referred beams, one may conclude that the $AGM\text{-}DR_{NL}$ method underestimates the demands comparing to the $EIMN_L$ method, particularly at the right node (node 2) of the beams, and derives values quite similar to those of inter-storey drifts. Yet, when gravity loads are neglected a perfect match is observed between both methods. In contrast, the $AGM\text{-}DR_L$ and the EGM_L yielded similar results, although underestimating the demands at lower stories of the building in comparison to EIM_{NL}. These results are in agreement with the distribution of inter-storey drifts depicted in Figure 5(A). Similar findings were observed in the first storey columns (P5, P10, P15 and P20), where linear analysis increasingly underestimated the chord rotation demands as the lateral deformation of the structure increased (Figure 5(C)).

As far as the use of chord rotations is concerned, Figure 5(D) and (E) demonstrate that it performs well in the prediction of plastic rotation demands, albeit some differences were observed at the left node (node 1) of the beams, which progressively diminished as the global drift increased. The yielded component of chord rotations was deducted to its total value so as to realistically reproduce plastic rotations. Two approaches were adopted in the definition of the chord rotation at yielding: the ASCE41-06 simplified approach, according to which the chord rotation at yielding is given for beams as:

$$\theta_y = M_{pl} L / 6EI \tag{9}$$

and for columns as,

$$\theta_y = M_{pl} L / 6EI \ \left(1 - N_{Ed} / N_{pl}\right) \tag{9}$$

where N_{ED} is the actual axial force in the element and N_{pl} the expected axial capacity of the element; and the accurate approach, which consists in the determination of the actual value of the chord rotation at yielding. It was observed that the ASCE41-06 simplified approach leads to unconservative estimates of rotation demands.

From the linear analysis applicability point of view, consistent estimates of deformations demands using linear elastic analysis seem to be only attained at building SB3, despite EC8-3 and ASCE41-06 allow its use to all buildings for both the Significant Damage and Damage Limitation limit states and EC8-3 enables its use to buildings SB1 and SB2 at the Near Collapse limit state (Araújo et al. 2012).

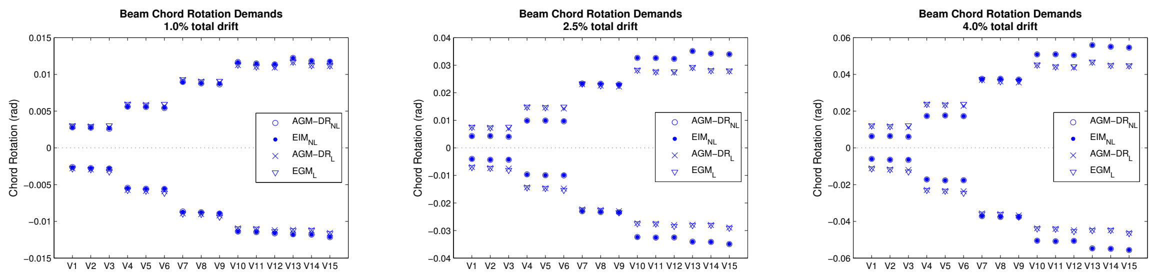

Figure 4. Beam chord rotations excluding the influence of gravity loads for building GB. The positive and negative values are referred to nodes 1 and 2 of the various elements, respectively.

From the results obtained one can conclude that linear analysis does not provide reliable estimates of chord rotation as levels of inaccuracy in the order of 30 to 50% were found for both beams and columns. This was verified even in buildings SB1 and SB2, which were seismically designed and which, according to EC8-3, could be assessed using linear analysis for all limit states. As a result, the approach proposed by EC8-3 to verify the seismic safety of existing steel buildings through linear elastic procedures seems to be unreliable and hence needs reassessment. A possible solution could consist in considering different compliance criteria, based for example in terms of interstorey drift ratios instead of chord rotations. The results obtained in this study allow concluding that linear analysis provides more reliable estimates of this demand parameter. Alternatively, the linear analysis applicability criteria could be restricted to buildings governed largely by weak beam-strong column mechanisms, as it is the case of building SB3, for which the local and global chord rotation errors were below 20%.

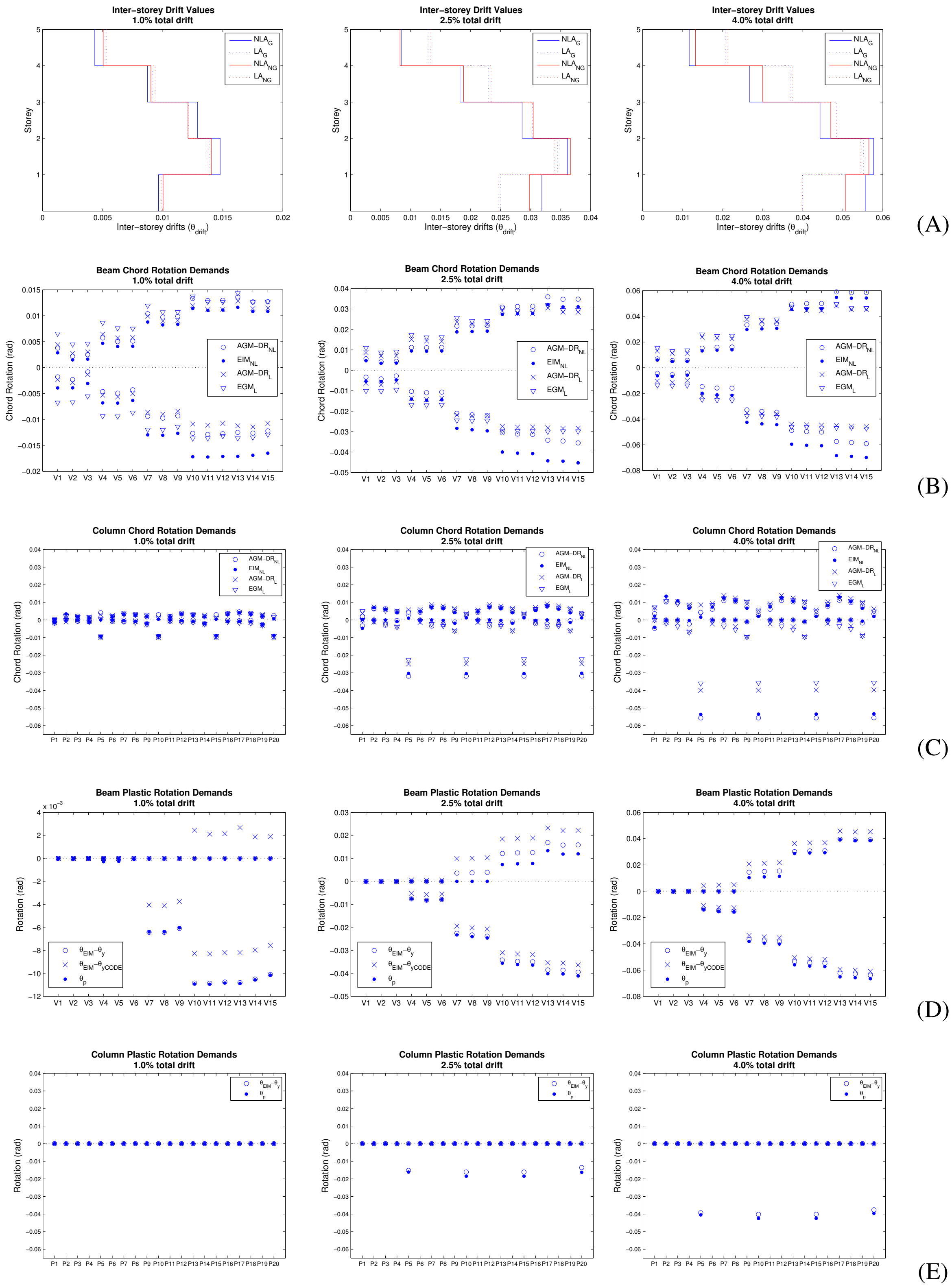

Figure 5. Local deformation demand estimates for building GB and different levels of global drift: (A) inter-storey drifts; (B) beam chord rotations; (C) column chord rotations; (D) beam plastic rotations; (E) and column plastic rotations. The positive and negative values of the deformation demands are referred to nodes 1 and 2 of the various elements, respectively.

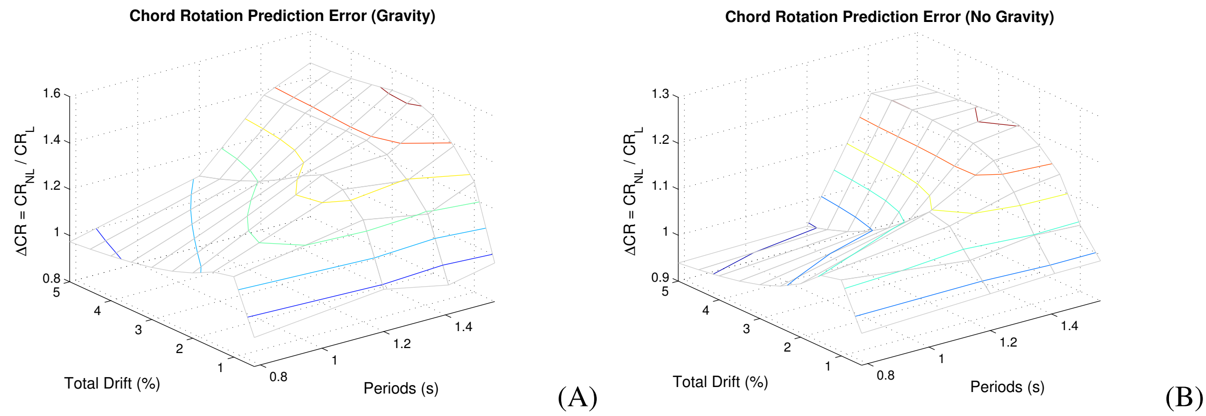

Figure 6. Evolution of the local chord rotation prediction error of beams with the period of vibration of buildings: (A) gravity loaded case; (B) gravity unloaded case.

5 CONCLUSIONS

In this paper, various methods of quantifying local deformation demands for the seismic assessment of structural components were presented. Its application to a set of steel buildings was carried out and its effectiveness evaluated. Additionally, the issue of how to compute local deformation demands using linear analysis was addressed.

The results obtained from the study of four steel buildings allow concluding that chord rotations evaluated with the Exact Integration Method (EIM_{NL}) provided more reliable estimates of plastic rotations, regardless of the lateral stiffness of the building. The Approximate Geometrical Method (AGM_{NL}) led to some considerable misestimates of deformation demands in buildings SB1 and SB2, with differences of about 40% with respect to chord rotations obtained using the EIM_{NL} method. In contrast, differences lower than 20% were observed in building SB3. Likewise, the estimation of chord rotations using linear analysis (EGM_L) led to inaccurate results comparing to the ones obtained from nonlinear analysis, despite the fact that, according to EC8-3, linear procedures could be applied at least to buildings SB1 and SB2. Errors in the order of 30% to 50% were found in these cases, with the exception of building SB2 where the error was lower than 20%.

It becomes clear from the study that further research should be carried out in order to improve the assessment procedures prescribed in the European seismic assessment code, particularly in terms of the procedures applicable to steel buildings.

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