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LINEAR ANALYSES FOR SEISMIC SAFETY ASSESSMENT OF EXISTING STEEL BUILDINGS: ARE THEY TRUSTWORTHY?

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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 and, in fact, the application of Part 3 of Eurocode 8 (EC8-3) requires the quantification of plastic rotations even when linear methods of analysis are used. The aim of the present study is thus to answer the question of how reliable are the local deformation demand estimates obtained using linear analysis procedures in comparison to more accurate nonlinear methods of analysis.

INTRODUCTION

The recent widespread awareness of earthquake experts and public authorities for the need of an accurate evaluation of the seismic vulnerability of the existing building stock, as well as the recognition of the existence of codified seismic assessment procedures with varying levels of accuracy, raised the attention for the need of a revision of normative documents that specifically address this topic. In Europe, Part 3 of Eurocode 8 (CEN, 2005a), EC8-3, which is the document that deals with this particular subject, is recognizably far from having a degree of maturity comparable with that of modern seismic design codes. It is therefore expected that important changes will be introduced in a second generation of Eurocodes (Pinto, 2005; Pinto and Franchin, 2008).

Some comparative applications of the EC8-3 procedures have been performed up to date (Romão et al., 2008 and 2010; Mpampatsikos et al., 2008; Masi et al., 2008), having shown that linear analysis tends to lead to demand values systematically larger than nonlinear analysis and that the prescribed stringent linear analysis applicability criteria is not expected to be satisfied by many existing buildings. On the basis of the results of more than a thousand non-linear dynamic analysis conducted on asymmetric multi-storey RC buildings, Kosmopoulos and Fardis (2007) concluded that elastic modal response spectrum analysis provides, on average, unbiased and fairly accurate estimates of member inelastic deformation demands. These conclusions were drawn from cases violating the linear analysis applicability criteria proposed by EC8-3, suggesting that there is room for a re-examination and possible relaxation of these criteria.

However, in spite of the very few studies have been carried out addressing the specific case of existing steel buildings (Araújo et al., 2012 and 2013a), the use of linear analysis in the assessment of this type of structures according to EC8-3 has been demonstrated by Araújo et al. (2012) to require

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particular caution, as it leads the analyst to quite an ambiguous scenario. Despite the well-known limitation of linear analysis in providing reliable predictions of inelastic response parameters, particularly at the member level, EC8-3 prescribes that the safety checks should be conducted in terms of plastic member rotations, rather than forces as it is proposed, for instance, by ASCE41-06 (ASCE, 2007). Consequently, a question may be naturally raised: how can the analyst perform safety checks when using linear analysis in the seismic assessment of existing steel buildings if plastic rotations cannot be obtained from this type of analysis method? Aiming to address this issue, Araújo et al. (2013a) found that the most suitable answer would be the use of chord rotations as an alternative to plastic rotations, quantified based on the geometry of the deformed shape of the member. The study presented herein gives continuity to the work initiated by Araújo et al. (2013a), including now a larger number of steel buildings with different structural characteristics and providing a comprehensive evaluation of the adequacy of the linear analysis applicability criteria proposed by EC8-3.

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 (Paret et al., 2011). In order to assess the degree of acquaintance of the civil engineering community with EC8-3 and their seismic assessment practices, a survey was conducted among Portuguese practitioners, which had 79 respondents. The responses obtained were found to be quite revealing, demonstrating that although 45% of the respondents confirmed having already dealt with a seismic safety assessment problem, only 8% used EC8-3 (Figure 1 (A)) and 55% still use the former Portuguese normative documents (RSA, 1983; REBAP, 1984). Among the reasons for not using EC8-3, 35% of the respondents said that they are unaware of its existence, while 36% considered it too complex to be applied in practice. With regard to the level of knowledge and familiarization of the civil engineering community with nonlinear static and dynamic procedures, almost half of the engineers (42%) said that they are not familiar with these type of procedures and only 20% of those who responded “yes” have already applied them in practice (Figure 1 (B)).

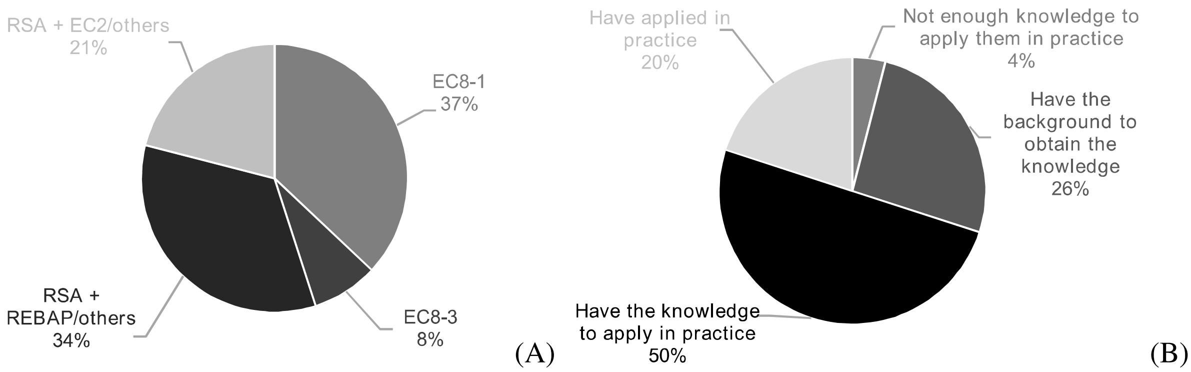


Figure 1. (A) Normative documents adopted in the seismic safety assessment of existing structures and (B) Knowledge of nonlinear methods of analysis to apply them in practice.

It is thus evident the unawareness of the Portuguese engineering community with respect to EC8-3 and an apparent resistance to use Eurocode 8 in detriment of the former Portuguese normative documents, which are clearly oriented towards the use of linear elastic methods of analysis. The promotion of future workshops and training programs that may help demystifying and acquainting practitioners with Part 3 of Eurocode 8 is deemed necessary, as well as further studies on the evaluation of the accuracy of the linear analysis procedures proposed by EC8-3, which is the scope of the present study.

CASE STUDY DESCRIPTION

The study presented herein was conducted considering ten 5-storey steel buildings composed by regular moment-resisting frames, as illustrated in Figure 2. Each building was designed according to different criteria. The first building, denoted as GB, was designed according to Eurocode 3 (CEN, 2005b) to resist gravity loads. The remaining nine buildings were seismically designed according to EC8-1 (CEN, 2004) assuming medium ductility (DCM class), with a behaviour factor q of 4.0, and to comply with different limits for the inter-storey drift sensitivity coefficient $\theta_{P\text{-}\Delta}$, which is defined in the code to address the treatment of second-order effects. The design seismic action was set for Zone 3 of the Portuguese territory and assuming soil type B, characterized by a soil factor, S, of 1.29 according to the Portuguese National Annex of Eurocode 8 (CEN, 2010). The main structural characteristics of the analysed buildings are summarized in Table 1. Grade S275 was considered for structural steel

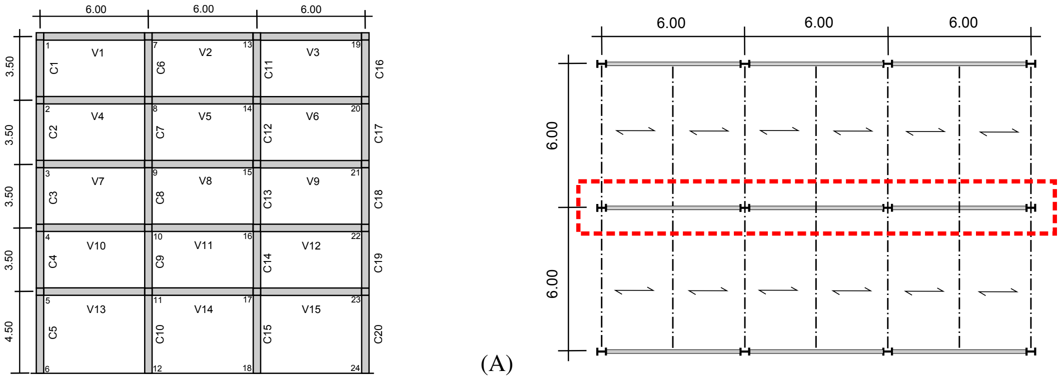


Figure 2. Frame dimensions, elements and node numbering: (A) elevation view; (B) and plan view

Table 1. Characteristics of the buildings analysed

Storey	Building GB (T_1 = 1.63s)							
	External Columns	Internal Columns	Beams	$\theta_{P\text{-}\Delta}$	Weak-beam / strong-column condition check			
5	HE240B	HE280B	IPE300	0.12	1.68	1.22	1.22	1.68
4	HE240B	HE280B	IPE300	0.23	3.35	2.44	2.44	3.35
3	HE240B	HE280B	IPE300	0.36	3.34	2.36	2.36	3.34
2	HE260B	HE300B	IPE300	0.45	3.62	2.46	2.46	3.62
1	HE260B	HE300B	IPE300	0.37	3.82	2.56	2.56	3.82

Storey	Building SB1 (T_1 = 1.50s)							
	External Columns	Internal Columns	Beams	$\theta_{P\text{-}\Delta}$	Weak-beam / strong-column condition check			
5	HE280B	HE280B	IPE300	0.12	2.44	1.22	1.22	2.44
4	HE280B	HE280B	IPE300	0.21	4.88	2.44	2.44	4.88
3	HE280B	HE280B	IPE300	0.29	4.88	2.36	2.36	4.88
2	HE300B	HE300B	IPE330	0.33	4.21	1.93	1.93	4.21
1	HE300B	HE300B	IPE330	0.28	4.51	2.01	2.01	4.51

Storey	Building SB2 (T_1 = 1.20s)							
	External Columns	Internal Columns	Beams	$\theta_{P\text{-}\Delta}$	Weak-beam / strong-column condition check			
5	HE300B	HE300B	IPE300	0.10	2.97	1.49	1.49	2.97
4	HE300B	HE300B	IPE300	0.16	5.95	2.97	2.97	5.95
3	HE400B	HE400B	IPE300	0.19	8.12	4.06	4.06	8.12
2	HE400B	HE400B	IPE330	0.20	8.04	3.96	3.96	8.04
1	HE450B	HE450B	IPE330	0.12	8.97	4.31	4.31	8.97

Storey	Building SB3 (T_1 = 0.90s)							
	External Columns	Internal Columns	Beams	$\theta_{P\text{-}\Delta}$	Weak-beam / strong-column condition check			
5	HE500B	HE500B	IPE300	0.06	7.66	3.83	3.83	7.66
4	HE500B	HE500B	IPE400	0.07	7.37	3.68	3.68	7.37
3	HE500B	HE500B	IPE400	0.09	7.37	3.68	3.68	7.37
2	HE500B	HE500B	IPE450	0.10	5.66	2.83	2.83	5.66
1	HE500B	HE500B	IPE450	0.08	5.66	2.80	2.80	5.66

Storey	Building SB4 (T_1 = 1.43s)							
	External Columns	Internal Columns	Beams	$\theta_{P\text{-}\Delta}$	Weak-beam / strong-column condition check			
5	HE200B	HE220B	IPE270	0.13	1.33	0.85	0.85	1.33
4	HE200B	HE220B	IPE300	0.24	2.04	1.28	1.28	2.04
3	HE220B	HE260B	IPE300	0.22	2.29	1.54	1.54	2.29
2	HE220B	HE260B	IPE450	0.21	0.91	0.65	0.65	0.91
1	HE220B	HE280B	IPE450	0.27	0.84	0.66	0.66	0.84

Storey	Building SB5 (T_1 = 1.47s)							
	External Columns	Internal Columns	Beams	$\theta_{P\text{-}\Delta}$	Weak-beam / strong-column condition check			
5	HE200B	HE220B	IPE270	0.15	1.33	0.85	0.85	1.33
4	HE200B	HE220B	IPE300	0.28	2.04	1.28	1.28	2.04
3	HE240B	HE260B	IPE300	0.27	2.68	1.54	1.54	2.68
2	HE240B	HE260B	IPE400	0.27	1.55	0.85	0.85	1.55
1	HE240B	HE280B	IPE400	0.30	1.45	0.87	0.87	1.45

Storey	Building SB6 (T_1 = 1.49s)							
	External Columns	Internal Columns	Beams	$\theta_{P\text{-}\Delta}$	Weak-beam / strong-column condition check			
5	HE220B	HE260B	IPE300	0.12	1.32	1.02	1.02	1.32
4	HE220B	HE260B	IPE300	0.23	2.63	2.03	2.03	2.63
3	HE220B	HE260B	IPE300	0.30	2.59	1.93	1.93	2.59
2	HE260B	HE280B	IPE360	0.30	1.99	1.22	1.22	1.99
1	HE260B	HE280B	IPE360	0.31	2.34	1.24	1.24	2.34

Storey	Building SB7 (T_1 = 1.45s)							
	External Columns	Internal Columns	Beams	$\theta_{P\text{-}\Delta}$	Weak-beam / strong-column condition check			
5	HE220B	HE260B	IPE300	0.12	1.32	1.02	1.02	1.32
4	HE220B	HE260B	IPE300	0.23	2.63	2.03	2.03	2.63
3	HE220B	HE260B	IPE300	0.30	2.59	1.93	1.93	2.59
2	HE260B	HE280B	IPE360	0.27	1.98	1.22	1.22	1.98
1	HE260B	HE280B	IPE400	0.28	1.82	0.96	0.96	1.82

Storey	Building SB8 (T_1 = 1.41s)							
	External Columns	Internal Columns	Beams	$\theta_{P\text{-}\Delta}$	Weak-beam / strong-column condition check			
5	HE220B	HE260B	IPE300	0.12	1.32	1.02	1.02	1.32
4	HE220B	HE260B	IPE300	0.23	2.63	2.03	2.03	2.63
3	HE220B	HE260B	IPE300	0.29	2.59	1.93	1.93	2.59
2	HE260B	HE300B	IPE360	0.26	1.98	1.39	1.39	1.98
1	HE260B	HE300B	IPE400	0.25	1.82	1.23	1.23	1.82

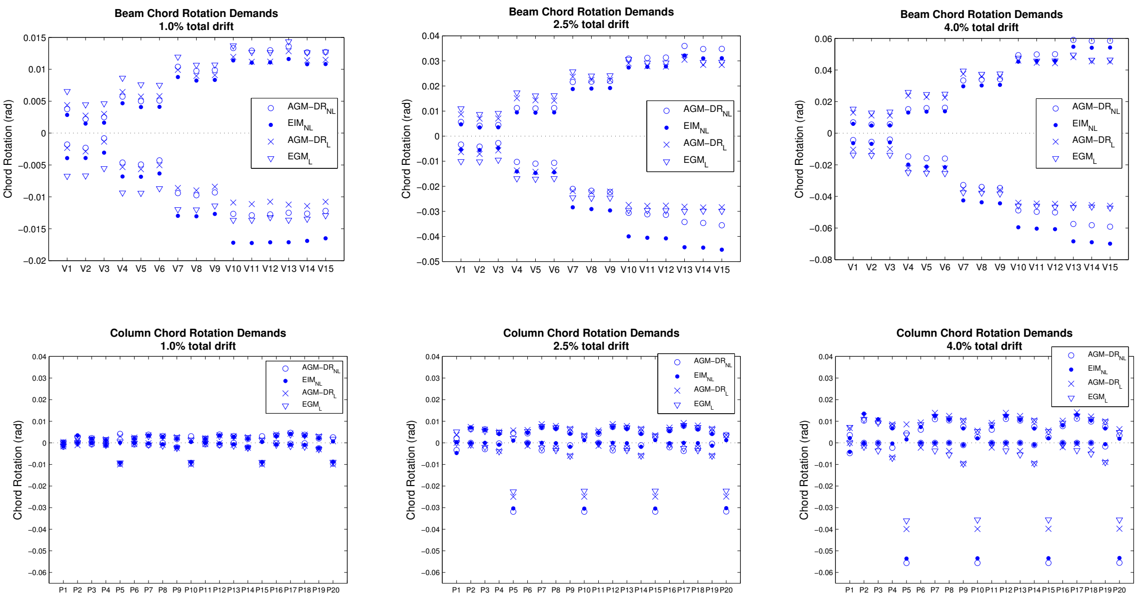
Storey	Building SB9 (T_1 = 1.46s)							
	External Columns	Internal Columns	Beams	$\theta_{P\text{-}\Delta}$	Weak-beam / strong-column condition check			
5	HE220B	HE260B	IPE300	0.12	1.3	1.0	1.0	1.3
4	HE220B	HE260B	IPE300	0.23	2.6	2.0	2.0	2.6
3	HE220B	HE260B	IPE300	0.30	2.6	1.9	1.9	2.6
2	HE260B	HE300B	IPE360	0.29	2.0	1.4	1.4	2.0
1	HE260B	HE300B	IPE360	0.28	2.3	1.6	1.6	2.3

Special attention was paid to the control of the weak-beam / strong-column criterion proposed by EC8-1 (CEN, 2004), which reflects the potential of each building to develop a soft-storey mechanism or plastic hinges at columns. The analyses were carried out using the open source software OpenSees (PEER, 2011) and two sets of structural models were developed in line with the analysis performed: linear and nonlinear. For the development of the numerical models for linear analysis, the previously presented data is sufficient. With regard to the models used for nonlinear analysis, force-based beam-column elements were adopted considering 10 Gauss-Lobatto integration points along the length, which offers a superior solution to the classical Gauss integration method when it is important to include in the integration the end points of the element. Additionaly, 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. Particular attention was given to the modelling of the viscous Rayleigh damping, having the mass proportional damping been neglected, since it does not have a real physical meaning, and a tangent-stiffness proportional damping assumed, which greatly reduces the elastic damping force when the structural stiffness drops to the post-yield level. A fraction of critical damping equal to 2% was considered.

The goal of this case study is thus to address the issue of how reliable are the plastic deformation demand estimates obtained when using linear elastic analysis, being the deformation demands defined in terms of chord rotations (Araújo et al., 2012). Recall that, according to EC8-3, the safety checks should be conducted in terms of plastic rotations, regardless the type of analysis considered, and that the use of linear analysis is restricted to those cases complying with certain applicability criteria. In order to evaluate the accuracy of linear analysis in the quantification of local deformation demands, pushover analyses were performed on both linear and nonlinear structural models developed for each building. The aim was to exclude the dispersion in the deformation demand estimates associated with the method of analysis adopted. A fixed load pattern was defined as proportional to the mass and height of each storey and the results were computed to various levels of global drift ratios (1%, 2.5% and 4%).

COMPARISON BETWEEN THE LOCAL DEFORMATION DEMAND ESTIMATES OBTAINED USING LINEAR AND NONLINEAR ANALYSIS

Figures 3 to 8 depict the results obtained for the majority of the analysed buildings. The chord rotation demands were quantified from linear analysis using the *Approximate Geometrical Method* ($AGM\text{-}DR_L$) and the *Exact Geometrical Method* (EGM_L) and from nonlinear analysis using the *Exact Integral Method* (EIM_{NL}) and, again, the *Approximate Geometrical Method* ($AGM\text{-}DR_{NL}$). More information on each one of these methods may be found in the work of Araújo et al. (2012).



(A)

(B)

Figure 3. Local deformation demand estimates for building GB subjected to different levels of global drift: (A) beam chord rotations; (B) and column chord rotations. The positive and negative values of the deformation demands are referred to nodes 1 and 2 of the various elements, respectively.

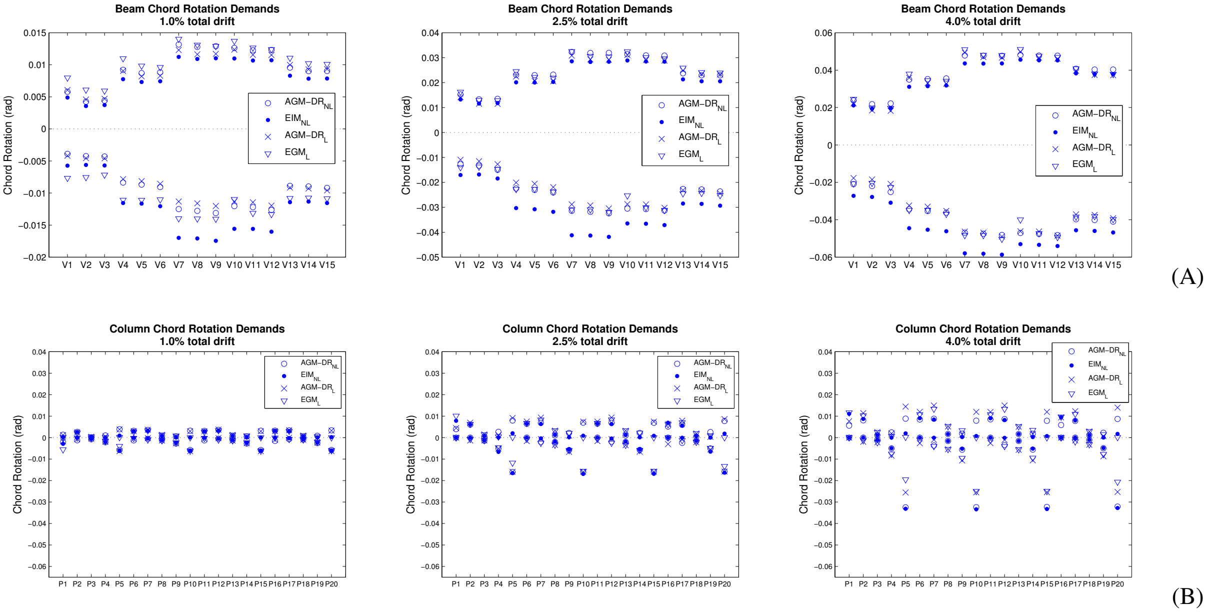


Figure 4. Local deformation demand estimates for building SB2 subjected to different levels of global drift: (A) beam chord rotations; (B) and column chord rotations. The positive and negative values of the deformation demands are referred to nodes 1 and 2 of the various elements, respectively.

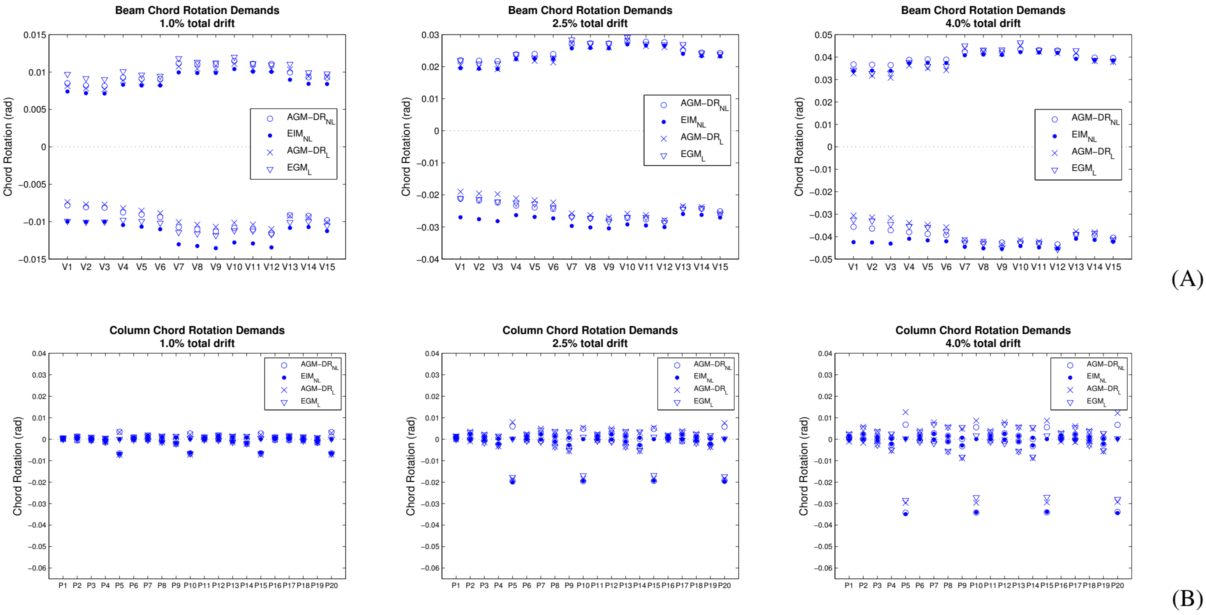
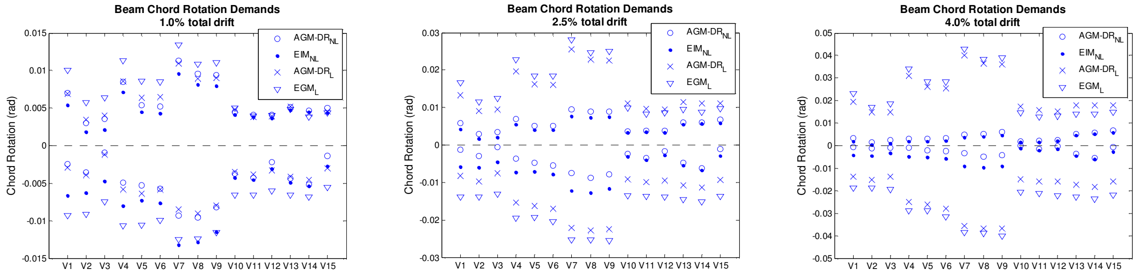
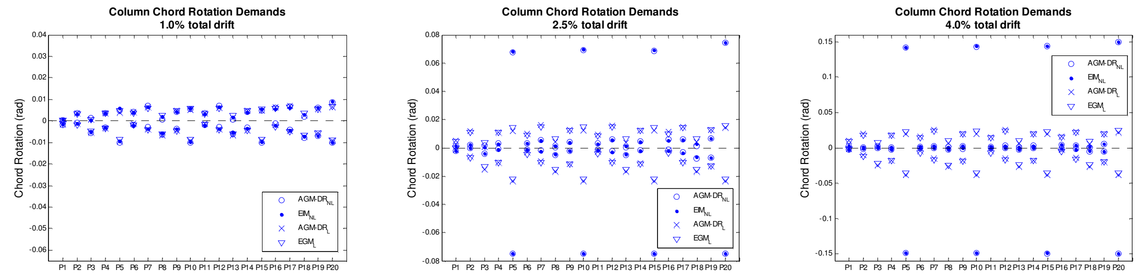


Figure 5. Local deformation demand estimates for building SB3 subjected to different levels of global drift: (A) beam chord rotations; (B) and column chord rotations. The positive and negative values of the deformation demands are referred to nodes 1 and 2 of the various elements, respectively.

From the observation of Figures 3 to 5, two conclusions may be readily drawn: (i) structures exhibiting a beam/column capacity ratio (CBMR) greater than 2.0 seem to ensure the development of a stable beam-sway mechanism, which results in a distribution of deformation demands along the structure that remains constant as the global drift ratio increases; (ii) as the lateral stiffness and strength of the structures increases, not only a more uniform distribution of the deformation demands over the height of the building is observed, but also linear analysis are seen to provide better chord rotation estimates in comparison to nonlinear analysis. In fact, the error between the chord rotation estimates obtained using nonlinear and linear analyses was found to be around 50% in the most demanded beam of building GB (node 1 of beam V15), which reduced to about 10% in building SB3 (node 1 of beam V12).

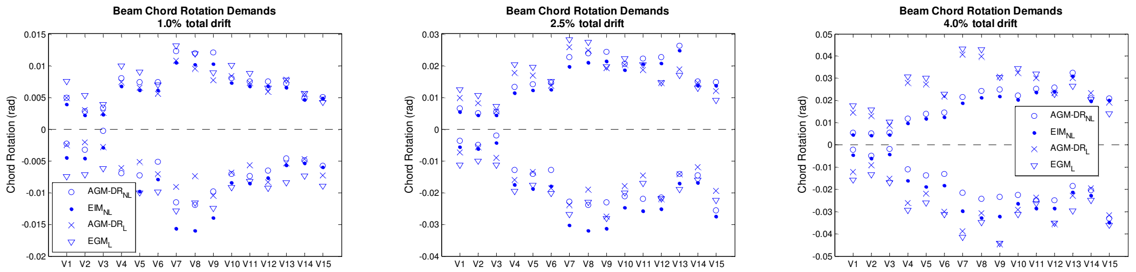


(A)

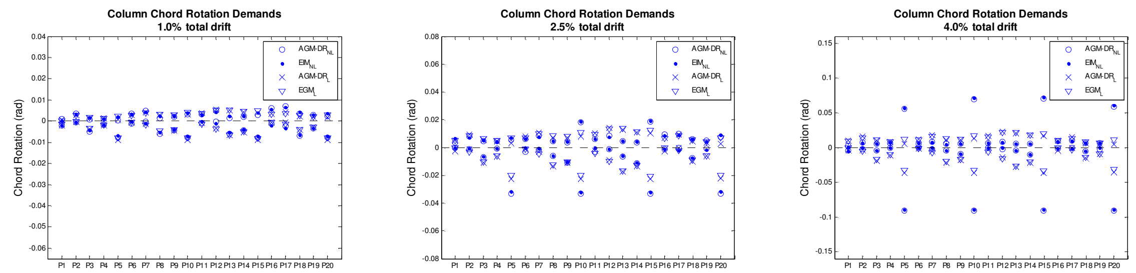


(B)

Figure 6. Local deformation demand estimates for building SB4 subjected to different levels of global drift: (A) beam chord rotations; (B) and column chord rotations. The positive and negative values of the deformation demands are referred to nodes 1 and 2 of the various elements, respectively.



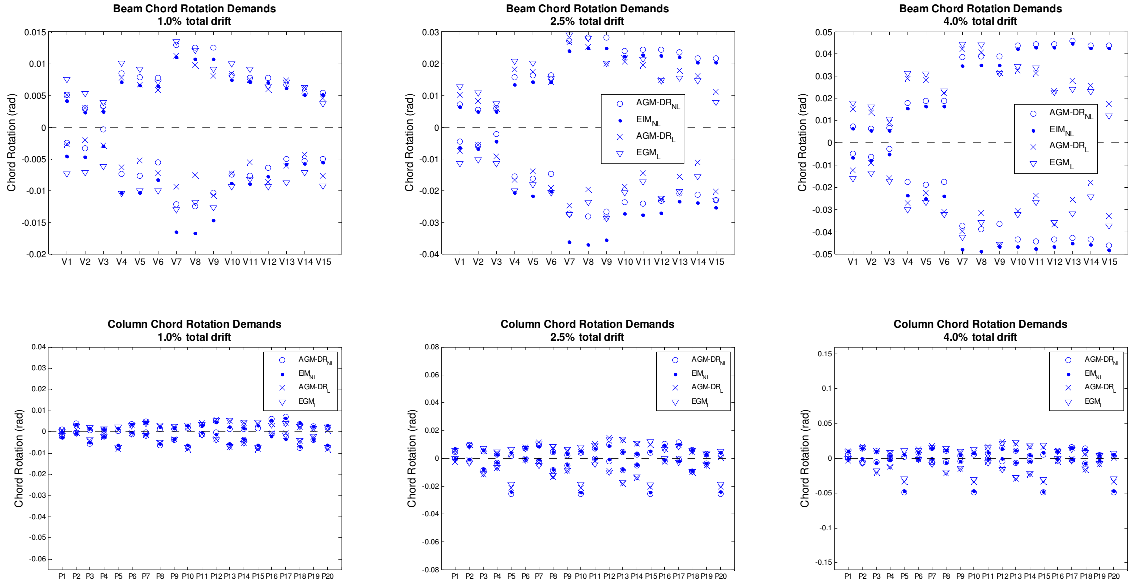
(A)



(B)

Figure 7. Local deformation demand estimates for building SB7 subjected to different levels of global drift: (A) beam chord rotations; (B) and column chord rotations. The positive and negative values of the deformation demands are referred to nodes 1 and 2 of the various elements, respectively.

In the case of buildings exhibiting a beam/column capacity ratio lower than 1.0 (buildings SB4, SB5 and SB7), a column-sway mechanism at the first storey was developed, as it would be expected. Nevertheless, whilst building SB4 initiated the development of plastic incursions at column P10 for a global drift ratio of about 1%, exhibiting a complete soft-storey mechanism for a global drift ratio of about 1.5% (Figure 6 (B)), the onset of the soft-storey mechanism in building SB7 occurred for a global drift ratio of 2.5% (Figure 7 (B)). In fact, linear analysis was seen to provide reasonable estimates (lower than 20%) of the chord rotations at the most demanded member of building SB7 (node 1 of beam V9) until a global drift ratio of about 2% was reached. As the global drift ratio continued increasing, the maximum deformation demands concentrated at the first storey columns.



(A)

(B)

Figure 8. Local deformation demand estimates for building SB8 subjected to different levels of global drift: (A) beam chord rotations; (B) and column chord rotations. The positive and negative values of the deformation demands are referred to nodes 1 and 2 of the various elements, respectively.

Finally, despite buildings SB6, SB8 and SB9 being characterized by beam/column capacity ratios greater than 1.0, more precisely greater than the EC8-1 limit of 1.30, all buildings developed plastic behaviour at the top end of first storey columns and second storey columns, which in some cases contribute to the formation of a soft-storey mechanism, initiated for a global drift ratio of approximately 2.5% in building SB6 and 4% in buildings SB8 and SB9. In fact, building SB6 developed a soft-storey for a global drift ratio of 4%. This concentration of deformation demands at columns, verified regardless of the assurance of the weak-beam / strong-column criterion, may be due to *P*-Δ effects, which are naturally amplified as the relative lateral displacements of the storey increase. As a result, an evident transition of the maximum deformation demands from the third floor beams (V7, V8 and V9) to the first floor beams (V13, V14 and V15) was observed in the nonlinear response of the buildings (Figure 8 (A)), reflecting the limitation of linear procedures to conveniently capture the chord rotation demands in buildings with these features. The error in the chord rotation estimates was seen to be particularly significant at the most demanded node of beam V14, reaching about 80% to 90% in buildings SB8 and SB9 for a global drift ratio of 4%.

Nonetheless, according to EC8-3, linear analysis may be just applied if verified a set of applicability criteria, and hence the actual impact of the predicted local deformation demand errors in the seismic safety assessment process has to be assessed by confronting these errors with the applicability of linear analysis.

CONFRONTATION OF THE CHORD ROTATION PREDICTION ERRORS WITH THE APPLICABILITY OF LINEAR ANALYSIS ACCORDING TO EC8-3

Part 3 of Eurocode 8 establishes that the applicability of linear analysis should be restricted to structures that comply with specific criteria related to the distribution of inelastic demands within a structure. In other words, EC8-3 allows the use of linear analysis for the estimation of local deformation demands in existing buildings if the ratio between the maximum and minimum demand-to-capacity ratios (*DCR*), defined in terms of bending moments in moment frames, over all primary elements, does not exceed a limit value, in the range between 2 and 3. Only elements with *DCR* values higher than 1.0, say elements expected to behave inelastically, should be considered. Additionally, EC8-3 prescribes that, around beam-column joints, the *DCR* ratios need only to be evaluated at the sections where plastic hinges are expected to form on the basis of the comparison of the sum of the beam flexural capacities to that of the columns. The assumption underlying the EC8-3 criteria is that if a structure goes into inelastic range with a uniform distribution of demands, which results from a regular distribution of stiffness, mass and strength, its response, in terms of displacements, is found to be acceptably accurate on the basis of the equal displacement rule, which is approximately valid for a single-degree-of-freedom oscillator. Indeed, this assumption seems to be perfectly validated by building SB3 (Figure 5), which, as depicted in Figure 9, not only verifies the EC8-3 linear analysis applicability criteria, independently of the global drift ratio of the structure, but is also associated to an error in the chord rotation estimates lower than an admissible level of 20%. However, the same conclusions are not valid for the remaining buildings featuring a beam/column capacity ratio greater than 2.0, particularly buildings GB and SB1, which although verifying the linear analysis applicability criteria for a global drift ratio up to about 2.5%, errors around 40 to 50% were observed in the chord rotation estimates when using linear analysis.

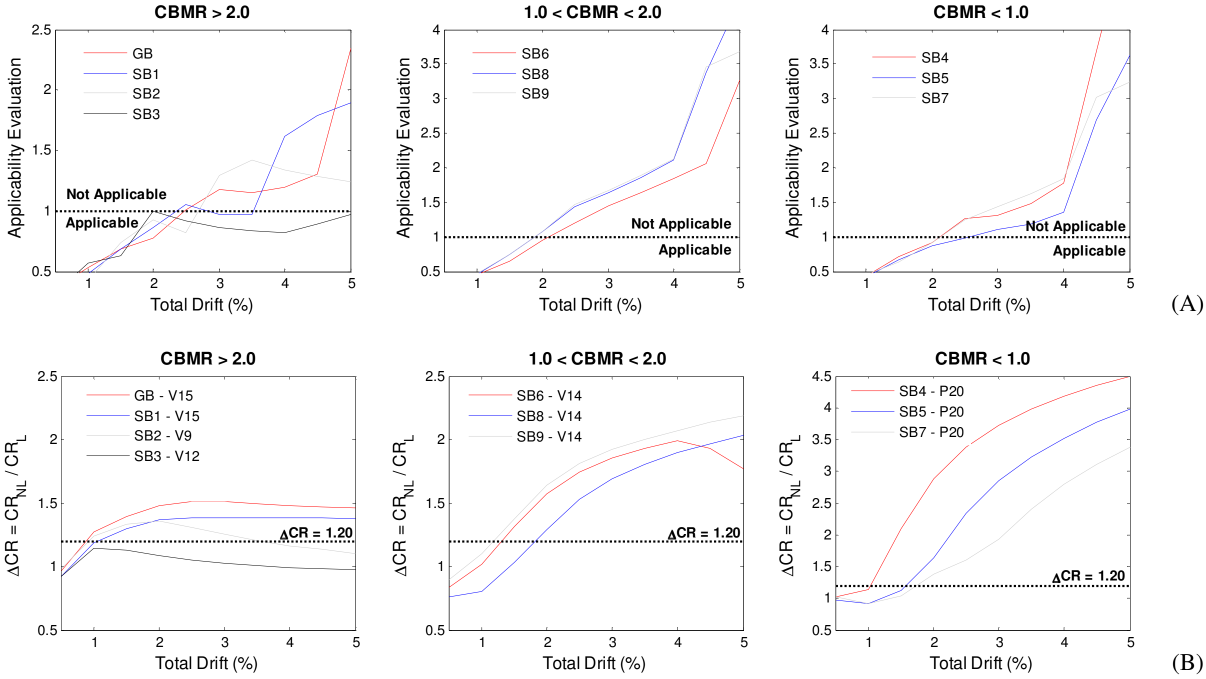


Figure 9. Confrontation of (A) the applicability of linear analysis according to EC8-3 with (B) the chord rotation prediction errors at the critical member of each analysed building.

The feasibility of the EC8-3 linear analysis procedure should be also questioned when applied to buildings with characteristics identical to buildings SB4 to SB9. In these cases, the error in the chord rotation estimates was seen to progressively increase with the global drift ratio, with linear analysis leading to chord rotation values in the order of 2 to 4 times the values of the nonlinear chord rotations. Still, EC8-3 allowed the applicability of linear analysis for each one of these buildings until a global drift ratio of about 2% was reached. Hence, the linear analysis applicability criteria proposed by EC8-3 do not seem to be associated to reliable levels of local deformation demand prediction errors, and thus appear to require further revision. Eventually, a linear elastic procedure identical to that of ASCE41-06 should be adopted, which not only relies on forces, rather than plastic rotations, to conduct the safety verifications, but also adopts more stringent applicability criteria, as already discussed by Araújo et al. (2012).

For the sake of completeness, the errors in the estimation of local deformation demands using current linear and nonlinear dynamic methods of analysis were also examined so as to understand how meaningful the previously predicted errors are. The Modal Response Spectrum method of analysis was adopted for the linear case (LDA), while a time-history analysis was adopted for the nonlinear one (THA). In the latter case, 15 groups of 7 ground motion records, selected and scaled with and without controlling the mismatch of each individual record spectrum with respect to the target spectrum, designated as G7I and G7, respectively, were considered. More information on the selection of the group of ground motion records may be found in Araújo et al. (2013b) and Macedo et al. (2013).

Figure 10 depicts the results obtained for buildings GB and SB3, demonstrating that, on average, the previously prediction errors seem to be somehow conservative, yet one may conclude that they are adequate and representative of the level of error incurred when using actual linear methods of analysis.

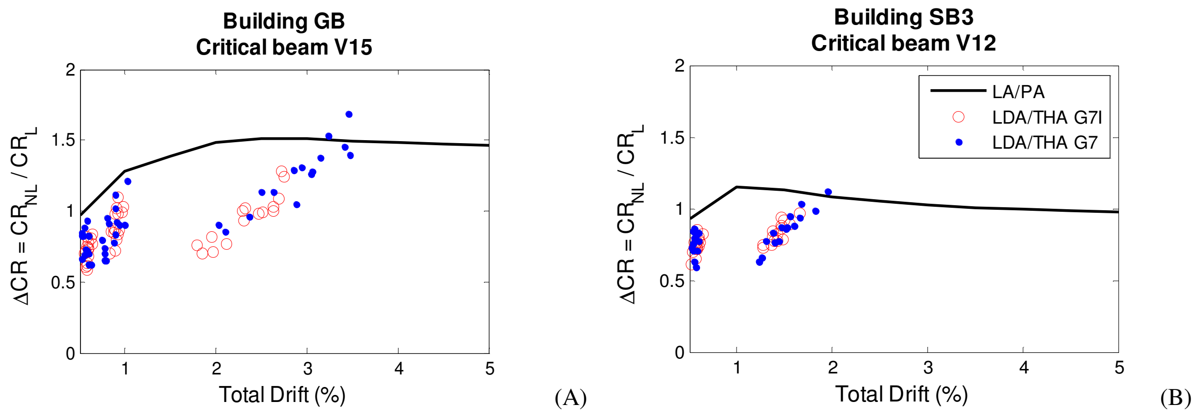


Figure 9. Errors in the chord rotation estimates considering actual linear and nonlinear methods of analysis obtained for buildings: (A) GB; (B) and SB3.

CONCLUSIONS

In the present work, a study aiming to address the issue of how reliable are the local plastic deformation demands obtained using linear analysis was carried out. This subject assumes particular relevance due to the fact that Part 3 of Eurocode 8 (EC8-3) prescribes that member safety checks should be always conducted in terms of plastic rotations, regardless of the type of analysis adopted. Hence, in order to conduct such study, chord rotations were considered as an alternative to plastic rotations, as they can be readily quantified from linear analysis and were demonstrated by Araújo et al. (2013a) to provide good estimates of the latter deformation parameter. According to EC8-3, the use of linear analysis procedures in the assessment of existing structures is, however, conditioned to certain applicability criteria. Therefore, the error associated with the use of linear analysis was confronted with the verification of such applicability criteria so as to provide an idea of the actual level of error incurred when using linear elastic methods of analysis.

With the exception of building SB3, which is characterized by a high and regular distribution of stiffness and strength over its height, linear analysis was seen to systematically provide unreliable estimates of chord rotation when comparing to those obtained from nonlinear analysis. Errors in the order of 30% to 50% were found in the case of buildings exhibiting a beam/column capacity ratio greater or around 2.0, which increased to about 70% in buildings featuring values of the same ratio between 1.0 and 2.0. Still, it was in the case of building SB4 that the EC8-3 linear analysis applicability criteria critically failed, as it allowed the use of this type of analysis even when a column-sway mechanism had already formed.

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 aiming at the development of alternative linear analysis applicability criteria.

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