Comparative Study of the European and American Seismic Safety Assessment Procedures for Existing Steel Buildings

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SUMMARY:
The concerns regarding the seismic assessment of existing buildings are becoming an important issue for public authorities and the scientific community. Whilst in the US procedures for seismic safety assessment were made available few years after the 1994 Northridge earthquake, in Europe the introduction of a specific Eurocode for seismic assessment only took place few years ago with the publication of Part 3 of Eurocode 8 (EC8-3). The experience with the application of the European code is therefore very limited. Few studies related to the application of the EC8-3 procedures to RC structures have already been conducted, but none have been performed for the case of steel structures. In this paper a comparative study of the European and American seismic safety assessment procedures as defined in EC8-3 and in the ASCE41-06 is carried out. To this end, the two standards are employed in the seismic assessment of four different steel buildings, designed according to different criteria. Special attention is devoted to the use of linear elastic analysis methods, both in terms of their applicability criteria and corresponding safety checks. The main results in the study are discussed and the difficulties in the application of the European code are identified.

Keywords: Eurocode 8, Seismic Assessment, Steel Structures

1. INTRODUCTION

The remarkable economic growth and widespread urbanization witnessed in the mid-to-late 20th century led to a worldwide rise of substandard constructions, generally designed with no concern for seismic resistance. Simultaneously, important and devastating earthquakes, as the 1989 Loma Prieta or the 1906 San Francisco earthquakes that struck in the US and the 1995 Kobe and 1999 Izmit earthquakes, resulted in the increasing awareness of the potential seismic risk arising from the existing building stock.

As a result, in August 1991 the American National Institute of Building Sciences entered into a cooperative agreement with the Federal Emergency Management Agency for a comprehensive seven-year program that led to the development of a set of national guidelines for the seismic rehabilitation of existing buildings (FEMA273 and FEMA356) and, later on, to the nationally recognized ASCE 41-06 standard.

In turn, in Europe the work in this area has started much later, with the recently published Part 3 of Eurocode 8, which took only about three years to be completed (Pinto, 2005). Hence, this document is far from possessing a degree of maturity comparable with that of the modern seismic design codes, being expected that its limitations and difficulties will reduce with the future editions of the document, thanks to the progress made by the intense research activity devoted to the subject.

Although few comparative applications of the code procedures have been performed to date (Romão *et al*., 2010a and 2010b; Mpampatsikos *et al*., 2008; Masi *et al*., 2008), none have addressed the case of steel structures. Thus, further research regarding the evaluation and validation of the code assessment procedures is needed, and special attention should be devoted to the particular case of steel structures.

The scope of this paper is, not only, to perform an application of the EC8-3 (CEN 2005a) procedure for safety assessment of existing steel buildings, from the common structural designer's point of view, but also to establish a comparison with the equivalent procedure as defined in the ASCE 41-06 (ASCE, 2007). To this end, the two standards are employed in the seismic assessment of four different moment-resisting framed (MRF) steel buildings, designed according to different criteria. The structures were defined in order to be representative of this type of construction without being excessively complex, thus facilitating the presentation of the results and the drawing of conclusions.

2. COMPARISON OF THE EC8-3 AND ASCE 41-06 PROCEDURES

2.1 Performance requirements and rehabilitation objectives

Although both documents are part of the last generation of codes, and so are performance- and displacement-based documents, wherein the direct analysis and verification quantities are generally structural displacements and the corresponding deformations induced by the seismic action in ductile components, their assessment procedures follow conceptually different approaches.

According to EC8-3, the appropriate levels of protection are considered to be achieved once satisfied a number of limit states (LSs), which are generally defined as: Near Collapse (NC), Significant Damage (SD) and Damage Limitation (DL). However, it will be the national authorities' responsibility to decide whether all three LSs shall be checked, or two, or just one of them. Also, different return periods may be ascribed to the various LSs to be checked in a country, which may be found in its National Annex.

On the other hand, the ASCE 41-06 considers that a certain rehabilitation objective, defined in accordance with the intended requirement goals, will be achieved once verified a set of performance levels: Operational (OP), Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP). In other words, depending on the economical, architectural or historical impact of the building and on its lifetime and rehabilitation costs, the structural engineer is allowed to select one of the following rehabilitation objectives: Limited, Basic Safety (BSO) or Enhanced, which are related to the various performance levels as illustrated in Figure 1. As a result, the American standard allows for greater flexibility in the decision-making and in the rehabilitation requirements definition, being the Enhanced Objective defined by the k, p and f performance requirements indicated in Figure 1, the one that better matches the EC8-3 performance requirements recommended for ordinary buildings.

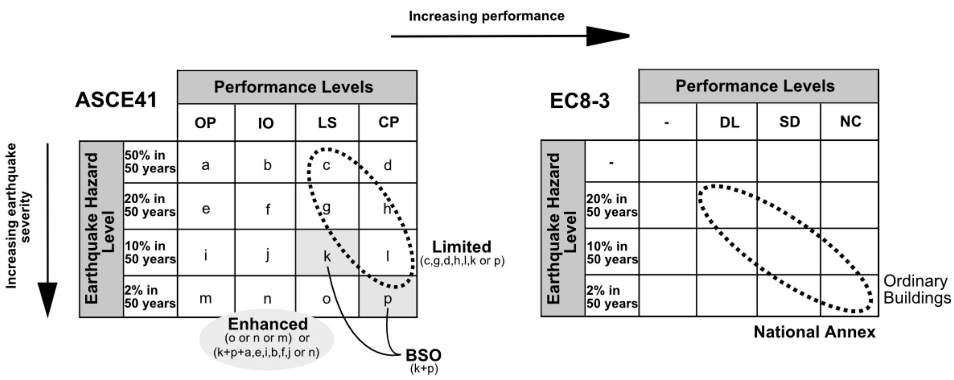


Figure 2.1. ASCE 41-06 Rehabilitation Objectives vs EC8-3 Performance Requirements.

Both EC8-3 and ASCE 41-06 formulate the requirements of each limit state and performance level in qualitative terms referring to more or less severe states of damage involving the structural system as a whole. However, when turning to the verification stage, both documents appear to require the analyst to satisfy all checks for all individual elements, which in fact, could lead to consider a building as seismically deficient even in the extreme case where only a single element would be found as nonconforming. This issue has already been discussed by Pinto & Franchini (2008) and an alternative procedure, based on a fault-tree representation, has been proposed.

2.2 Data collection requirements and the treatment of uncertainty

An important distinctive feature of the existing structures when compared to new ones is the fact that their structural properties can be obtained during the assessment process. Nevertheless, it is often difficult to obtain the correct data that defines the real material, geometrical and detailing characteristics of a certain building, being this incompleteness of knowledge always present in the process of evaluating existing structures.

Hence, both American and European documents try to account for this type of epistemic uncertainties through the definition of different knowledge levels, which are related to the amount and quality of the usable information, and the use of one single factor that covers all types of uncertainties, denoted as Confidence Factor (CF) in EC8-3 and Knowledge Factor (k) in ASCE 41-06. In the two standards these factors are used to reduce the material strength properties adopted in the evaluation of the capacity of each individual element. Despite recognizing the controversial features of these single factors (Franchin *et al* 2009; Romão *et al* 2008), their values will not be a matter of discussion in the present work.

Figure 2.2 depicts the comparison between the different approaches proposed by ASCE 41-06 and EC8-3 for the treatment of uncertainty. Whist in the latter document the level of knowledge only determines the admissible methods of analysis and the CF values, in the former the level of knowledge also influences the selection of the rehabilitation objective to be considered. Thus, an important conceptual difference between EC8-3 and ASCE 41-06 can be identified in the case of minimum or limited levels of knowledge, where the ASCE 41-06 not only considers that simplified elastic methods of analysis (Linear Static Procedures and Linear Dynamic Procedures) should be adopted, similarly to EC8-3, but also that the IO performance level check may be neglected. Such an approach could be due to the fact that in the IO performance level, as in the DL limit state of EC8-3, no yielding is expected to occur in any structural element, both ductile and brittle, and hence the verification of this performance level could lead to a poor estimate of the actual response of the building.

ASCE41

Data	Knowledge Levels			
	Minimum	Usual		Comprehensive
Rehabilitation Objectives	BSO or Lower LS+CP	BSO or Lower	Enhanced	Enhanced
Analysis Procedures	LSP, LDP	ALL	ALL	ALL
Knowledge Factor (k)	0.75	1.00	0.75	1.00

EC8-3

Data	Knowledge Levels		
	Limited (KL1)	Normal (KL2)	Full (KL3)
Rehabilitation Objectives	-	-	-
Analysis Procedures	LF, MRS	ALL	ALL
Confidence Factor (CF)	1.35	1.20	1.00

Figure 2.2. Comparison between the ASCE41-06 and the EC8-3 different ways of treating uncertainty.

2.3 Analysis procedures and safety verifications

In the assessment of an existing building, the accuracy of the method of analysis that will be employed is of crucial importance, since a conservative method may indicate unnecessary expensive interventions, while a non-conservative one may leave the building exposed to an excessive risk.
Both EC8-3 and ASCE 41-06 recommend the use of similar methods of analysis in the assessment of existing buildings. These can range from simpler linear elastic methods, as the well known lateral force method, designated by Linear Static Procedure (LSP) in ASCE 41-06 and by Lateral Force method (LF) in EC8-3, or the modal response spectrum (MRS) method, designated by Linear Dynamic Procedure (LDP) in ASCE 41-06 and MRS in EC8-3, to more complex nonlinear methods,

as the pushover or the time-history dynamic analysis methods. Additionally, EC8-3 allows for the use of the q-factor approach, the basic design method prescribed in Part 1 of EC8 (CEN 2004), with a default value of q equal to 2.0 for steel structures. This value can be increased by about one-third to check the NC limit state or to higher values if analytically justified. Moreover, in this case both analysis procedures and safety checks should be performed according to EC8-1. However, the EC8-3 itself refers that this type of approach is generally not suitable to check the NC limit state and with such small values of q, the method is in most case, if not in all, conservative. Therefore, it should only find application in the case of buildings with an apparent overcapacity and/or located in low seismicity regions.

The application of linear analysis methods is restricted to structures that comply with specific criteria related with the distribution of inelastic demands in the structure. In order to evaluate such distribution, the ratio between the elastically evaluated demand resulting from the unreduced seismic action and the corresponding capacity, denoted by the parameter $\rho_i = D_i/C_i$ in EC8-3 and DCR$_i$ in ASCE 41-06, should be determined over all i-th ductile primary elements of the structure and compared with the limits prescribed by the standards. Thus, while according to EC8-3, the linear methods of analysis may be applied if the ratio ρ_{max}/ρ_{min}, defined over all ductile primary elements with $\rho_i>1$, does not exceed a maximum acceptable value in the range between 2 and 3, being 2.5 the recommended value, according to ASCE 41-06 the linear analysis methods may be employed if all component DCRs $\leq$ 2.0. In addition to this limit, the American code extends the applicability of linear analysis procedures to structures with one or more DCRs greater than 2.0 if they comply with a set of criteria related to structural irregularity.

With regard to the safety verifications of the structural elements, a distinction is made by both documents between ductile and brittle elements. For ductile elements, the safety checks are based on the evaluation of plastic deformations (θ_{Demand}) which are then compared with the corresponding deformation capacities defined in the codes ($\theta_{Capacity}$). For example, in the case of steel moment frames the safety checks are performed in terms of plastic hinge rotations. Concerning the brittle elements, both codes prescribe safety checks based on the comparison of element internal forces (Q_{Demand}) with the corresponding element strength capacity ($Q_{Capacity}$). Figure 2.3 summarizes the safety checks proposed by both codes, from which two main differences should be pointed out. Firstly, while in the case of ASCE 41-06 the acceptance criteria for the applicability of linear procedures is always based in the check of the inequalities in terms of the components strengths, for both ductile and brittle elements, the EC8-3 considers that, for ductile elements, the safety checks should be carried out in terms of deformations.

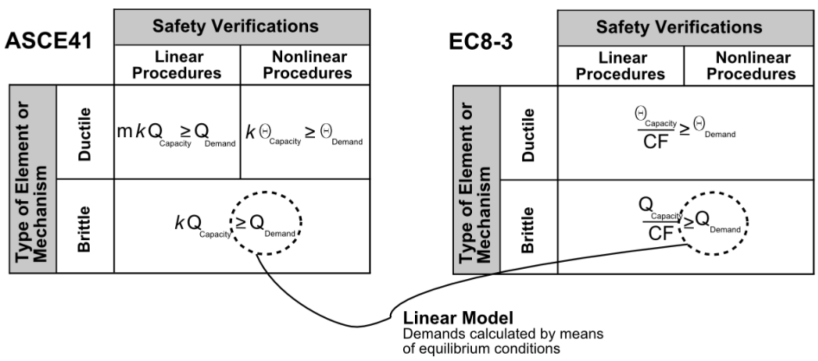


Figure 2.3. Summary of the safety checks proposed by ASCE41-06 and EC8-3.

Some questions can be raised regarding the EC8-3 approach, as it requires the analyst to evaluate the safety of each ductile element by checking its plastic rotation capacity based on the demand obtained from a linear elastic structural model. Secondly, unlike the EC8-3, according to which only beams are treated as ductile members, ASCE 41-06 considers that, in the case of typical moment-resisting frames, both beam and column elements may be treated as ductile.

3. CASE STUDY

As mentioned above, the study presented herein was conducted considering four 5-storey MRF steel buildings with the configuration in plan and elevation illustrated in Figure 3.1. 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 three buildings were seismically designed according to Part 1 of Eurocode 8 assuming a value of 4.0 for the behaviour factor (q) and considering the elastic response spectrum for seismic actions of type 1 and 2. The PGA was taken equal to 0.15g and it was assumed soil of type B according to the EC8 definition. The three buildings were designed to comply with different limits for the inter-storey drift sensitivity coefficient (θ) which is defined in the code to address the treatment of second-order effects. Thus, the SB1 building was designed to comply with $0.2< \theta <0.3$. In this case the second-order effects were directly included in the numerical analysis. The SB2 building was designed assuming $0.1< \theta \leq 0.2$, being the second-order effects taken into account by multiplying the relevant seismic action effects by a factor equal to $1/(1 - \theta)$, and finally, the SB3 building was designed in order to neglect second-order effects ($\theta \leq 0.1$).

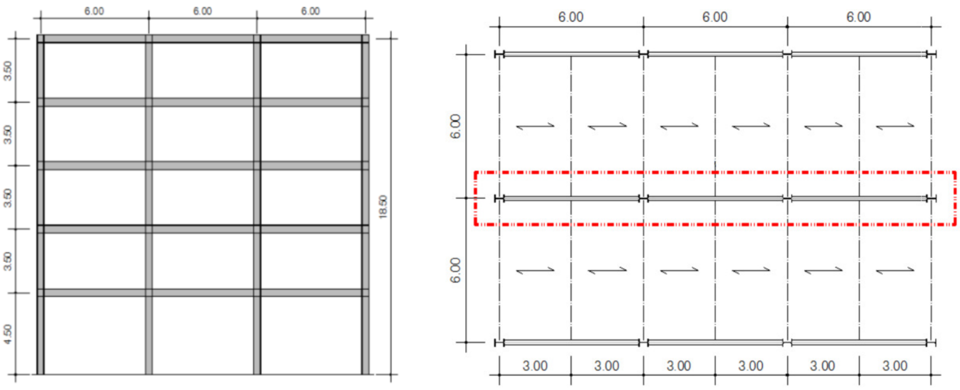


Figure 3.1. Elevation and plan views of the 5-storey frame

This study focused uniquely on the assessment of the structures based only on linear analysis procedures following the prescriptions of EC8-3 and ASCE 41-06.

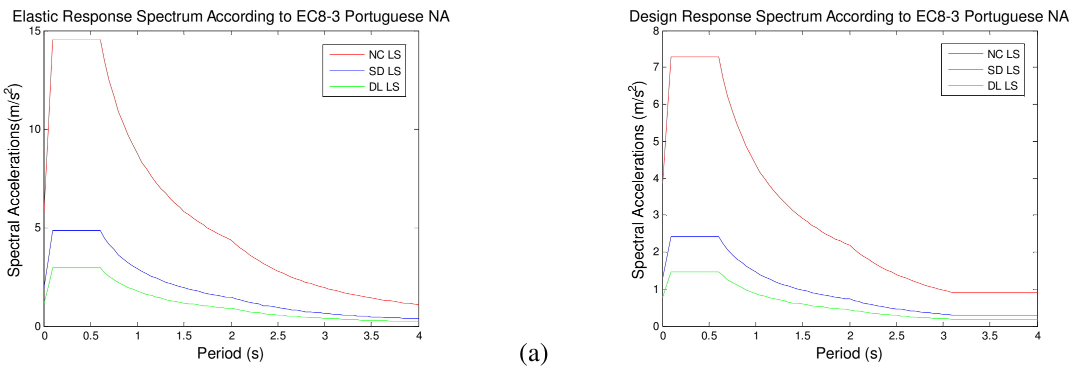


Figure3.2. Adopted seismic action as defined in EC8-3: (a) Elastic response spectrum; (b) Design response spectrum for q=2.0 for DL and SD LSs and q=2.0(1+1/3) for NC LS.

The analyses of the frames were performed with the open source software OpenSees (PEER 2011). Modal analysis was 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 4.1.

Table 4.1. Dynamic characteristics of the buildings

Building	Periods of Vibration (s)		
	Mode 1	Mode 2	Mode 3
GB	1.56	0.48	0.25
SB1	1.41	0.45	0.24
SB2	1.20	0.37	0.19
SB3	0.78	0.26	0.12

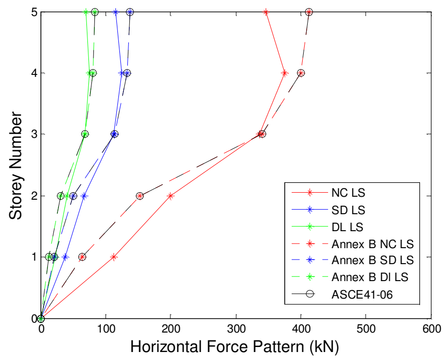
4. APPPLICATION OF EC8-3

4.1. Lateral Force analysis (LF)

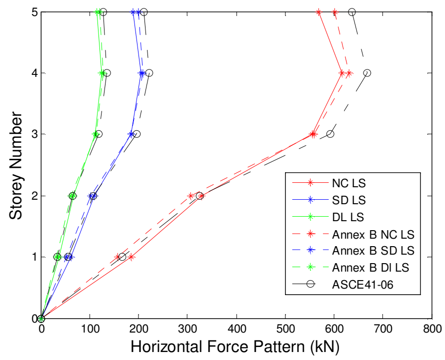
According to EC8-3 the assessment of existing steel buildings using LF analysis should be performed considering two different patterns of horizontal forces: (i) one similar to the pattern proposed in EC8-1, which is proportional to the displacements of the fundamental mode or the height of the storey masses, $h_{x,i}$, and the storey masses, $W_{x,i}$; (ii) and an additional one, proposed in Annex B of EC8-3, which is given by:

$$F_{x,i} = \frac{W_{x,i} \cdot h_{x,i}^{\delta}}{\sum W_{x,i} \cdot h_{x,i}^{\delta}} F_b \tag{4.1}$$

where δ is an exponent that is function of the period of vibration of the structure and F_b the base shear. The ASCE 41-06 suggests a similar expression to the latter. Thus, only the F_b value will differ between both codes. Figure 4.1 displays the horizontal force patterns for each limit state defined in EC8-3. A comparison with the ASCE 41-06 horizontal force pattern is also provided in the figure. It may be observer that, not only the horizontal load pattern proposed in the EC8-3 Annex B, for both linear and nonlinear static analysis, led to values more similar to the ones obtained using ASCE41-06, but also it seems to better capture the real horizontal stiffness distribution of the structure.



(a)

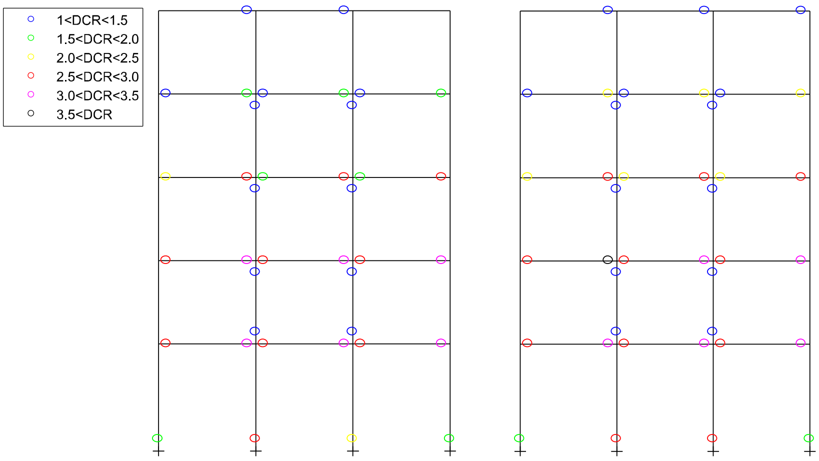


(b)

Figure 4.1. EC8-3 pattern of horizontal forces: (a) GB building; (b) SB3 building.

As it has already been discussed above, before the assessment of any existing building using linear methods of analysis, its applicability may be firstly checked. To this end, the distribution of the inelastic demands along the structure has to be evaluated, defined in terms of Demand-to-Capacity Ratios (DCRs) or ρ_i in EC8-3. Figure 4.2 represents this distribution of DCRs along the GB building for both EC8-3 horizontal force patterns and for the NC limit state. As expected the Annex B load pattern led to more severe structural response results, and so to higher DCR values. Moreover, since the GB building was not designed to resist the seismic action, the distribution of inelastic demands is highly variable within the structure, being the ratio ρ_{max}/ρ_{min} equal to approximately 3.30 for both EC8-3 load patterns. As a result, the lateral force analysis method could not be used in the assessment of this existing building for the case of the NC limit state.

The same analysis was repeated for the remaining buildings in study, being the final results presented in Table 4.2. For the cases in which the linear method of analysis is applicable, difficulties arise regarding the procedure to adopt in checking the plastic rotations developing in the ductile members with the limits defined in EC8-3.



(a) (b)

Figure 4.2. DCR distribution along the GB steel building for the NC limit state and using the LF method of analysis: (a) EC8-1 force pattern; (b) EC8-3 Annex B force pattern.

4.2. Modal Response Spectrum analysis (MRS)

The MRS method is, generally, a more accurate method in comparison with the lateral force method. However, the percentage of buildings complying with them is anticipated being not very large. As previously performed for the LF analysis method, the applicability of the MRS method was assessed, being the results obtained for the GB building and NC limit state presented in Figure 4.3. It is worth noting that in the adopted nomenclature the letter V refers to beams and the following two numbers define their location, counting from left to right, in terms of bay and storey. In turn, the letter A refers to each vertical alignment of columns, P, also counting from left to right, and the last value defines the column's storey number. Once again, the linear methods of analysis, in this case the MRS, fail on their applicability requirements in the GB building and for the NC limit state.

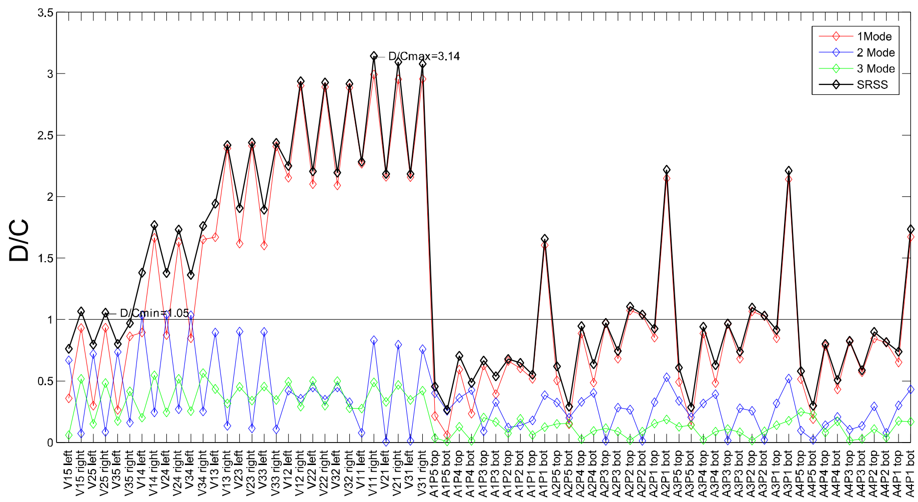


Figure 4.3. DCR values for each structural element of the GB steel building for the NC LS and using the MRS method of analysis.

Table 4.2 summarises the applicability of each analysis method defined in EC8-3 for each building and for the various limit states. As shown in the table, the SB3 building is the only one to comply with the requirements for the applicability of both linear elastic methods of analysis and for all limit states. This fact was already expected, due to the high lateral stiffness and strength of SB3 building, which was designed in order for second-order effects to be neglected. In the case of the SB2 building, although the MRS method fails in terms of its applicability requirements, according to EC8-3 the safety assessment of this building could be carried out using the LF method of analysis. Thus, a major issue arise from the fact that this building could be assessed using the simpler lateral force method, but not using the supposedly more accurate modal response spectrum analysis. Finally, assuming that for ordinary buildings the safety assessment has to be performed for all three limit states, nonlinear methods of analysis would have to be employed to assess the GB and SB1 buildings.

Table 4.2. Applicability of the linear methods of analysis according to EC8-3

Building	Lateral Force Analysis			Modal Response Spectrum Analysis		
	NC LS	SD LS	LD LS	NC LS	SD LS	LD LS
GB	Not Applicable	Applicable	Applicable	Not Applicable	Applicable	Applicable
SB1	Not Applicable	Applicable	Applicable	Not Applicable	Applicable	Applicable
SB2	Applicable	Applicable	Applicable	Not Applicable	Applicable	Applicable
SB3	Applicable	Applicable	Applicable	Applicable	Applicable	Applicable

4.3. *q*-Factor approach

Despite referring the *q*-factor approach as generally not suitable for checking the NC limit state, EC8-3 allows its use. The results obtained with the use of this alternative approach are presented in Table 4.3, for each building and limit state. Once again, the SB3 building verified all safety requirements and the admissible inter-storey drifts imposed by the DL limit state, being these results in agreement with the ones previously obtained with the LF and MRS analysis methods. However, as expected, this method led to very conservative results, especially in the case of the SB1 and SB2 buildings, which were designed according to EC8-1 considering a behaviour factor (q) equal to 4.

Table 4.3. Safety of the buildings according to the EC8-3 *q*-factor approach

Building	*q*-Factor Approach		
	NC LS	SD LS	LD LS
GB	Not Safe	Safe	Verify
SB1	Not Safe	Safe	Verify
SB2	Safe	Safe	Safe
SB3	Safe	Safe	Safe

5. APPPLICATION OF ASCE 41-06

With the aim of identifying conceptual differences, and their implications, between the applicability requirements of linear methods proposed by EC8-3 and ASCE 41-06, only the Linear Static Procedure, which is equivalent to the Lateral Force method defined in EC8-3, was carried out in the assessment of the structures. The results obtained for the SB2 building and for both EC8-3 NC limit state and ASCE 41-06 CP performance level are presented in Figure 4.4. In this case, it should be noted that the applicability requirements will not be verified, both according to EC8-3, once the ratio ρ_{max}/ρ_{min} is approximately equal to 2.73, and according to ASCE41-06, due to the maximum DCR value of 2.82 which is greater than the limit of 2.0 defined in the standard. Table 4.4 presents the applicability of the Linear Static Procedure defined in ASCE 41-06 for each building and for the various performance levels. It is interesting to note that the method cannot be applied to verify the CP performance level, being only suitable to check the remaining two performance levels prescribed in ASCE 41-06. It should be recalled that the satisfaction of the Basic Safety Objective requires the check of both CP and LS performance levels. The reason for the non-applicability of the Linear Static Procedures is related to the fact that, neither the structural regularity criteria, nor the $DCR_i \leq 2.0$ condition were verified. In other words, while EC8-3 admits that the ratio ρ_{max}/ρ_{min} should be lower than a certain value ranging from 2 to 3 which, in practical terms, corresponds to impose that the inelastic demands are distributed within the structure regardless of the intensity of the seismic action, the $DCR_i \leq 2.0$ condition prescribed in ASCE 41-06 is easily violated for increasing levels of seismic intensity as the capacity is constant for a given member while the demand, in the context of a linear elastic analysis, is proportional to the level of seismic action.

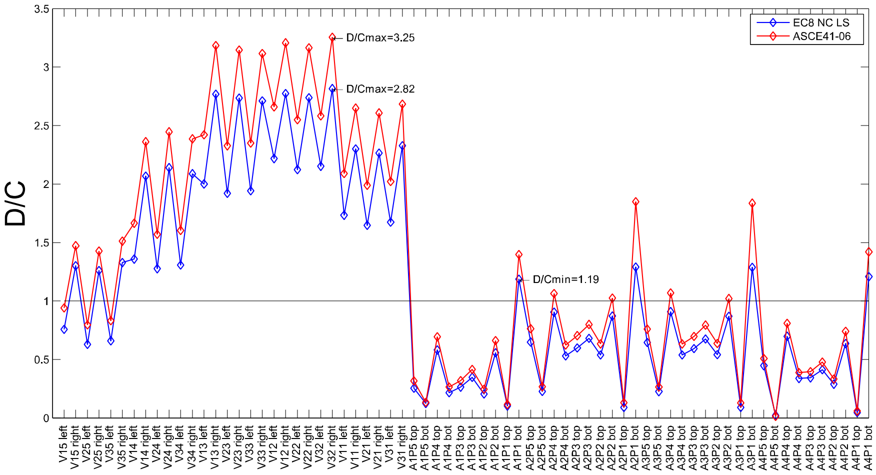


Figure 4.4. DCR values for each structural element of the SB2 building for the NC limit state and using both EC8-3 and ASCE 41-06 linear static analysis methods.

Table 4.4. Applicability of the Linear Static Procedure according to ASCE 41-06

Building	Linear Static Procedure		
	CP Performance Level	LS Performance Level	IO Performance Level
GB	Not Applicable	Applicable	Applicable
SB1	Not Applicable	Applicable	Applicable
SB2	Not Applicable	Applicable	Applicable
SB3	Not Applicable	Applicable	Applicable

Finally, Table 4.5 provides a comparison of the aapplicability of the linear static procedures according to ASCE 41-06 and EC8-3 for the various limit states (or performance levels in ASCE 41-06) established in the two documents.

Table 4.5. Applicability of the linear static procedures according to ASCE 41-06 and EC8-3

Building	ASCE 41-06 Performance Levels			EC8-3 Limit States		
	CP	LS	IO	NC	SD	DL
GB	Not Applicable	Applicable	Applicable	Not Applicable	Applicable	Applicable
SB1	Not Applicable	Applicable	Applicable	Not Applicable	Applicable	Applicable
SB2	Not Applicable	Applicable	Applicable	Applicable	Applicable	Applicable
SB3	Not Applicable	Applicable	Applicable	Applicable	Applicable	Applicable

5. CONCLUSIONS

In this paper a comparison between the European (EC8-3) and American (ASCE 41-06) provisions for the seismic safety assessment of buildings was presented. The performance objectives stipulated in the two codes were discussed along with the approaches in each code to treat the uncertainty related with the knowledge of the structure. Furthermore, the types of analysis prescribed in the codes were discussed with special focus given to the criteria defined for the applicability of linear elastic procedures.

The application of the two codes to four steel buildings designed according to different criteria allowed identifying some difficulties and inconsistencies related with the application of the European code. Concerning the criteria defined in the code to check the applicability of linear elastic methods, the study has shown that, according to EC8-3, one of the buildings (SB2) could be assessed using the simpler lateral force method, but not using the supposedly more accurate modal response spectrum analysis. Furthermore, in the cases for which the linear elastic analysis was applicable, no safety checks could be performed as EC8-3 requires the analyst to evaluate the safety of each ductile element by checking its plastic rotation capacity based on the demand obtained from a linear elastic structural model.

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

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