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ON THE APPLICABILITY OF CONVENTIONAL SEISMIC DESIGN METHODOLOGIES TO HYBRID RC-STEEL SYSTEMS

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Abstract: This paper analyses the adequacy of the q-factor approach when applied to hybrid RC-steel systems. This approach is prescribed by EC8-1 as the basic design method, and usually referred to as the most conservative. However, due to its simplicity and popularity for the design of new structures, practitioners are more likely to resort to it, than to the more complex nonlinear static and dynamic procedures, when dealing with the seismic assessment and strengthening of existing RC buildings. A case-study application is thus presented to analyse and discuss the difficulties a practitioner will face when assessing the efficiency of a steel-brace retrofitting system designed within the framework of EC8-1.

1. Introduction

Seismic design is currently codified by structural codes and standards of practice using the so-called "force-based design" (FBD). This is mainly due to historical reasons and related to how design is carried out for other actions, such as dead and live loads [1]. This procedure involves the consideration of a behaviour factor (q-factor) as a simple mean to account for the nonlinear behaviour of the structure while performing linear static analysis. However, in structures designed according to old seismic codes (or even non-seismically designed), the uncertainties about the nonlinear behaviour are relevant (for instance, the location and ductility capacity of the potentially inelastic regions are not fully known). It is therefore very difficult to define a direct correlation between the real internal forces that develop in structural members during the seismic excitation, and those experienced by an equivalent indefinitely elastic structure. Consequently, several authors (e.g., Mpampatsikos *et al* [2]) have argued that the force-based seismic assessment of such structures will not yield, in general, satisfactory results.

With the purpose of providing information about the seismic performance of RC buildings strengthened with steel braces, several experimental and analytical studies have been conducted over the last years. Unfortunately, only a few (e.g., Maheri and Akbari [3]; TahamouliRoudsari *et al* [4]) have resulted in proposals for *q*-factors to be applied in the design process and, moreover, limited confidence should be assigned to them due to the discrepancy between results. In fact, the latter show that the improvement in the seismic behaviour of a RC structure retrofitted with steel braces is not proportional to the corresponding increase in lateral strength, which suggests that the *q*-factor based design process might not be an efficient approach to achieve retrofitted structures with good seismic performance. As such, the nonlinear behaviour of the existing and retrofitted structures should be faced directly, despite the considerable increase in complexity of the assessment and design procedures. However, due to the simplicity and popularity of the *q*-factor approach for the design of new structures, practitioners are more likely to resort to it, than to the more complex nonlinear static and dynamic procedures, when involved in situations requiring the seismic assessment and strengthening of existing RC buildings.

The application of conventional seismic design methodologies to hybrid RC-steel systems is thus addressed in the following sections. The *q*-factor based EC8-3 [5] seismic assessment procedure is first reviewed, followed by a case-study application intended to analyse and discuss the difficulties a practitioner will face when assessing the efficiency of a steel-brace retrofitting system designed within the framework of EC8-1 [6]. Afterwards, the obtained retrofitted structure is re-evaluated using nonlinear methods of analysis. The obtained results are discussed in light of the EC8-3 performance requirements, and conclusions are drawn about the adequacy of the FBD methodology (and associated *q*-factors) to such situations.

2. The *q*-factor approach in EC8-3

Clause 4.1(3) on EC8-3 states that the assessment procedure should be carried out by means of the general analysis methods specified in Section 4.3 of EC8-1, as modified per the former standard to suit the specific problems encountered in the assessment. For a chosen performance requirement, the effects of the seismic action (combined with the other permanent and variable loads) can thus be evaluated by means of linear or nonlinear methods of analysis, depending on the characteristics of the structure under evaluation and the choice of the analyst. Each of these methods involve different levels of complexity, accuracy, computational effort, as well as of requirement for specialized knowledge in the field. The *q*-factor approach – a linear static design method with reduced input seismic demand – is prescribed by EC8-1 as the basic design method and usually referred to as the most conservative.

The seismic action to be adopted when using the *q*-factor approach within the context of EC8-3 is referred in its clauses 2.2.1(4) and 4.2(3). The design spectra for linear analysis are the ones defined in sub-section 3.2.2.5 of EC8-1, scaled to the values of the design ground acceleration established for the verification of the different limit states (LS's). A default value of $q = 1.5$ is proposed for RC structures, regardless of structural type. Higher values may eventually be adopted if suitably justified with reference to the local and global available ductility (evaluated according to the provisions of EC8-1), but this is usually not easy to do. Clause 2.2.2(3) on EC8-3 does however refer that the value of $q = 1.5$ (or a duly justified higher one) corresponds to the fulfilment of the LS of Significant Damage (SD). If the LS under evaluation is that of Near Collapse (NC), that value may be increased by about one-third, although it is also referred that this approach is generally not suitable for checking this LS.

Regarding structural modelling, Clause 4.3(2) of EC8-3 states that all provisions of sub-sections 4.3.1 and 4.3.2 of EC8-1 should be applied without modifications. In particular, member stiffness should be simulated according to paragraphs 4.3.1(6) and 4.3.1(7). The former states that the effect of cracking should be considered by evaluating the stiffness at the time when the reinforcement starts to yield. The latter states that, unless a more accurate analysis of the cracked elements is performed, their flexural and shear stiffness properties may be taken equal to one-half of those of the uncracked elements. This simplified 50% stiffness reduction is widely used and generally accepted for the design of new structures, but it is difficult to justify for the assessment of existing structures, especially when applied to members that are prone to early cracking [7]. However, within the straightforward context of developing a linear elastic analysis model to be used with the *q*-factor design approach, this simplification is thought to be acceptable enough.

Concerning safety verifications, clauses 2.2.1(4) to (7) on EC8-3 state that all structural elements should be verified by checking that demands due to the reduced seismic action do not exceed the corresponding capacities in terms of strength. For the calculation of the latter of ductile or brittle elements, mean value properties of the existing materials should be used as directly obtained from *in-situ* tests and from additional sources of information, appropriately divided by the applicable confidence factors. For new or added materials, nominal properties should be used. In the case of brittle elements, material strengths should be further divided by the partial factor of each material when calculating the corresponding strength capacities. For the verification of the LS's of NC and SD, clauses 2.2.2(3) and 2.2.3(3) state that demands shall be based on the reduced seismic action relevant for each LS, and capacities evaluated as for non-seismic design situations. On the other hand, for the LS of Damage Limitation (DL), Clause 2.2.4(3) refers that demands and capacities shall be compared in terms of mean inter-storey drift. Two shortcomings can thus be pointed out to the EC8-3 safety verification procedure when using the *q*-factor design approach: (i) RC member capacities are the same for the LS's of NC and SD (given the installed axial force), and (ii) no limit values are recommended for the inter-storey drifts to be observed when checking for the LS of DL.

The above referred criteria are summarized in Table 4.3 on EC8-3, including the values of the material properties to be adopted when evaluating the demands and capacities of ductile and brittle elements, for all types of analysis, as well as the criteria that shall be followed for the corresponding safety verifications. However, inconsistencies seem to exist between the contents of Table 4.3 regarding the *q*-factor approach and what is stated in the precedent text: (i) it is said that demands on brittle elements should be determined in accordance with the relevant section of EC8-1, which appears to be a reference to the capacity design rule (sub-section 5.2.3.3) and related others (no reference to this procedure exists in the before text concerning the *q*-factor approach); (ii) it is said that the mean values of material properties should be divided by both the confidence factor and the material partial factor when calculating the capacities of elements (the before text only applies the material partial factor to the case of the capacities of brittle elements).

3. Application to a retrofitting case-study

Sub-section 5.1.2 on EC8-3 states that the selection of the "type, technique, extent and urgency" of the retrofitting intervention should be based on the structural information collected during the assessment of the building. The following aspects should be taken into account: (i) all identified local gross errors should be appropriately remedied; (ii) structural regularity should be improved as much as possible, both in elevation and in plan; (iii) increase in the local ductility supply should be effected where required; (iv) the increase in strength after the intervention should not reduce the available global ductility. The required characteristics of regularity and resistance can be achieved by either the modification of the strength and/or stiffness of an appropriate number of existing components (local modification), or by the introduction of new structural elements (global modification). The procedure to design the retrofitting system should include the following steps: (a) conceptual design; (b) analysis; (c) verifications. The conceptual design stage should cover the following: (i) selection of techniques and/or materials, as well as of the type and configuration of the intervention; (ii) preliminary estimate of dimensions of additional structural parts; (iii) preliminary estimate of the modified stiffness of the retrofitted elements. Structural analysis should then be performed considering the modified characteristics of the building and safety verifications should be carried out for existing, modified, and new structural elements. Finally, the description of the expected effect of the retrofitting solution on the structural response of the building should be included in the design documentation. The following sub-sections go through these steps during a case-study that tests the approach described in the previous section.

3.1 Structural characterization

The chosen RC structure is considered to be representative of the design and construction common practice in southern European countries, such as Italy, Portugal and Greece, until the late 1970's. As such, it was designed to withstand only vertical loads. The reinforcement details were specified according to the available codes and construction practice at that time. Hence, no specific seismic detailing was considered, no preferential inelastic dissipation mechanisms were assumed, and no specific ductility or strength provisions were considered [8]. The full details on the geometry of the structure, material properties, and vertical loading can be found in Falcão Moreira [9], as well as the results of the seismic assessment, performed according to the provisions of EC8-3 running nonlinear static and dynamic analyses, considering a moderate-high European seismic hazard scenario. The results of the latter show that, for the structure to become compliant with the performance requirements of the LS of NC, a global retrofitting intervention is necessary. In order to be effective, the retrofitting system will have to be capable of reducing floor displacements, eliminating the irregular response of the third storey, and reducing the shear demand on columns. A global strengthening intervention using concentric X-diagonal steel braces is proposed herein. Fig. 1 shows the layout of the bracing system. The diagonals are composed by hot-rolled circular hollow section (CHSH) steel profiles, directly connected to the RC beam-column nodes of the central bay (5.00 *m* long by 2.70 *m* high). The connection is thought out to behave as a "nominally pinned joint" (as defined in EC3-1-8), i.e., capable of transmitting the internal forces without developing significant moments. At the points where the braces cross, no structural connections exist.

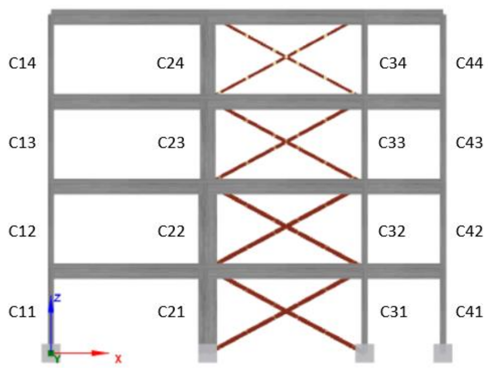

Fig. 1: Layout of the proposed retrofitting system

3.2 Design of the retrofitting system

As no specific rules for the design of hybrid RC-steel systems exist in EC8-1, its provisions concerning steel frames with concentric braces were taken as reference and starting point for the design of the retrofitting system. Beginning with the braces' layout, Clause 6.7.1(2) requires diagonal elements to be placed in such a way that the structure, under load reversals, exhibits similar load deflection characteristics at each storey, in opposite senses of the same braced direction. To that end, the rule provided by Clause 6.7.1(3) should be met at every storey. The geometry shown in Fig.1 meets this requirement, so no changes were necessary. A linear elastic analysis model was then developed according to the requirements of sub-section 4.3.1 (on EC8-1), and the frame was analysed under the effect of its vertical loads combined with the seismic demand defined by the NC elastic response acceleration spectrum. The effect of the compressed braces was neglected during the analysis, as required by Clause 6.7.2(2), and behaviour factors q corresponding to two different ductility classes (DCL: q=1.5; DCM: q=3.0) were considered to obtain the seismic forces on the retrofitted structure.

Concerning the detailed design of the steel braces, the applicable provisions of sub-section 6.7.3 on EC8-1 were taken as reference: (i) the non-dimensional slenderness $\bar{\lambda}$, as defined in EC3-1-1 [10], should be limited to $1.3 < \bar{\lambda} \leq 2.0$; (ii) the yield resistance $N_{pl,Rd}$ of the gross section should be such that $N_{pl,Rd} \geq N_{Ed}$, where N_{Ed} is the design axial force on the tensioned brace; (iii) the maximum overstrength ratio $\Omega_i = N_{pl,Rd,i}/N_{Ed,i}$ over all braces should not differ more than 25% from the minimum value $\Omega = \min(\Omega_i)$. The lower limit to the non-dimensional slenderness $\bar{\lambda}$ is adopted so that, during the pre-buckling stage (when both compression and tension braces are active), the RC frame's columns are not overloaded beyond the action effects obtained from the analysis at the ultimate stage (when only the tension braces are taken as active). Regarding the imposition of a maximum difference of 25% between the overstrength ratios Ω_i, it means to ensure a homogeneous dissipative behaviour of the steel braces. The definition of a retrofitting solution that fulfilled these conditions required an iterative analysis and design procedure that was carried out considering the two previously referred q-factors. Tables 1 to 3 summarize the obtained results, considering a design yield stress f_y equivalent to that of steel grade S275 (EC3-1-1).

Table 1: Steel braces design results: q = 1.5 (DCL)

						N_{Ed} (kN)		$\Omega_i = \frac{N_{pl,Rd,i}}{N_{Ed,i}}$	
Storey	**Cross-section**	L (m)	f_y (MPa)	$\bar{\lambda}$	$N_{pl,Rd}$ (kN)	**XX+**	**XX-**	**XX+**	**XX-**
4	CHSH 88.9x3.2	5.68	275.00	2.16	237.05	166.94	169.11	1.42	1.40
3	CHSH 139.7x4.0	5.68	275.00	1.37	470.25	419.19	422.84	1.12	1.11
2	CHSH 139.7x5.0	5.68	275.00	1.37	583.00	564.19	567.67	1.03 = Ω	1.03
1	CHSH 139.7x5.0	5.68	275.00	1.37	583.00	561.51	575.20	1.04	1.01 = Ω

Table 2: Steel braces design results: q = 3.0 (DCM)

						N_{Ed} (kN)		$\Omega_i = \frac{N_{pl,Rd,i}}{N_{Ed,i}}$	
Storey	**Cross-section**	L (m)	f_y (MPa)	$\bar{\lambda}$	$N_{pl,Rd}$ (kN)	**XX+**	**XX-**	**XX+**	**XX-**
4	CHSH 88.9x3.2	5.68	275.00	2.16	237.05	85.49	89.15	2.77	2.66
3	CHSH 88.9x3.2	5.68	275.00	2.16	237.05	172.61	176.28	1.37	1.34
2	CHSH 114.3x3.2	5.68	275.00	1.67	308.00	238.99	243.48	1.29 = Ω	1.26 = Ω
1	CHSH 114.3x3.2	5.68	275.00	1.67	308.00	222.47	228.35	1.38	1.35

Table 3: Base shear values for the bare (BF) and retrofitted (DCL; DCM) frames

	BF	DCL	DCM
Fundamental period T_1 (sec)	0.94	0.36	0.45
Spectral corner period T_c (sec)	0.60	0.60	0.60
NC spectral acceleration $S_{e,NC}$ (T_1)	$0.46768 \cdot g$	$0.73269 \cdot g$	$0.73269 \cdot g$
Total mass m (ton)	173.93	173.93	173.93
Elastic base shear V_e (kN)	683.72	1062.63	1062.63
Behaviour factor q	2.0 (*)	1.5	3.0
Assessment / design base shear V_b (kN)	341.86	708.42	354.21

(*) q-factor for the seismic assessment of the BF under the LS of NC, as per EC8-3: 2.2.2(3)

The design axial forces on the tensioned braces, as well as the fundamental period and base shear values shown above, were obtained considering the effective (secant-to-yield) stiffness EI_{eff} of the RC elements equal to one-half of their gross stiffness EI. As referred in Section 2, this is the default procedure according to Clause 4.3.1(7) on EC8-1 to consider the effect of cracking when performing seismic design using linear elastic analysis models. Concerning the shaded values of $\bar{\lambda}$ and Ω_i, those indicate situations for which the design provisions of sub-Section 6.7.3 on EC8-1 were not fulfilled. However, the maximum value of the non-dimensional slenderness is only slightly exceeded, and it is well known that the condition regarding the maximum difference between overstrength ratios is often hard to fulfil on top storeys [9]. Moreover, it is recalled that these are design provisions that apply to concentric-braced steel frames, therefore not being mandatory for the design of steel-braced RC structures.

3.3 RC member safety checks

For the retrofitting system to be deemed adequate, the RC members must be proven safe by checking that demands due to the reduced seismic action do not exceed their (ductile and brittle) capacities in terms of strength, while duly considering the axial forces induced by the steel braces. As referred in section 2., clauses 2.2.2(3) and 2.2.3(3) on EC8-3 state that capacities shall be evaluated as for non-seismic design situations. However, to allow for the possibility that the actual yield strength of the steel braces is higher than their nominal yield strength f_y, the safety checks were performed herein according to the capacity design rule specified in sub-section 6.7.4 on EC8-1, adapted to RC members and reproduced below in Eq. (1):

$$N_{Rd}\ (M_{Ed}) \geq N_{Ed,G} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot N_{Ed,E} \qquad (1)$$

where $N_{Rd}\ (M_{Ed})$ is the design axial resistance of the beam or column according to EC2-1-1 [11], considering the interaction of the axial resistance with the bending moment M_{Ed}, the latter being defined by its value in the seismic design situation; $N_{Ed,G}$ is the axial force in the beam or column due to the non-seismic actions included in the combination for the seismic design situation; $N_{Ed,E}$ is the axial force in the beam or column due to the design seismic action; $\gamma_{ov} = 1.25$ is the overstrength factor as defined in EC8-1: 6.1.3(2); Ω is the minimum overstrength ratio Ω_i over all steel braces, as already defined above. Additionally, the shear force demands V_{Ed} in the seismic design situation were checked against the shear force capacity values V_{Rd} obtained with the provisions of EC2-1-1.

The results of the safety checks are given below in terms of demand-to-capacity ratios (D_i/C_i) for each control section i, using a graphical representation to provide a better view of the outcome provided by each retrofitting system. For comparison purposes, the D/C ratios referring to the seismic assessment of the BF according to the q-factor approach are also included. Fig. 2 and Fig. 3 thus show, respectively, the NC axial force and NC shear force D/C

ratios on the columns of the bare (BF) and retrofitted (DCL; DCM) frames. As for the beams, the D/C ratios were found to be consistently below 1.0, therefore the correspondent graphical representation is deliberately omitted.

Fig. 2: BF, DCL, and DCM: NC axial force D/C ratios on columns (q-factor approach results)

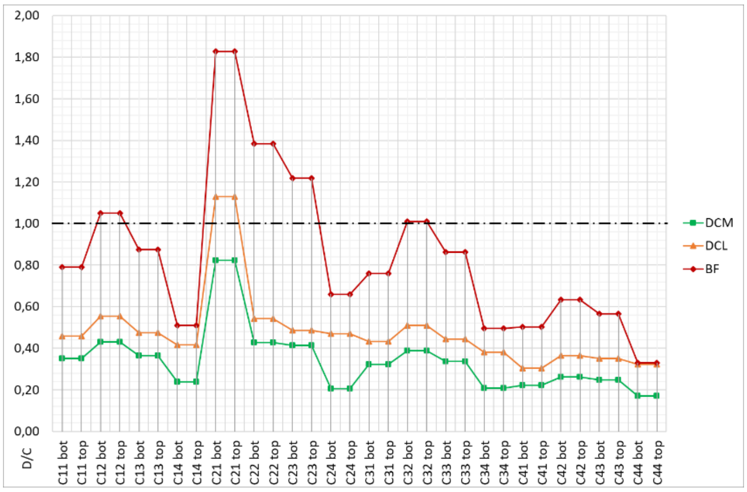

Fig. 3: BF, DCL, and DCM: NC shear force D/C ratios on columns (q-factor approach results)

The D/C ratios comparison shows the drastic improvement in the seismic behaviour of the frame caused by the inclusion of the steel braces. Even though some control sections remained unsafe when considering the DCL retrofitting system, the global assessment scenario is completely different from that of the BF which revealed generalized excessive flexural demands and clearly excessive shear demands on the first, second and third storeys of column C2. The fact that the axial force D/C ratios on the first storey of column C3 are higher in the DCL than they were in the BF indicates that the global axial force induced by the tensioned steel braces is more than that column can resist. Therefore, the DCL retrofitting system should be deemed inadequate for being too robust. On the other hand, when considering the DCM retrofitting system, both the axial force and shear force D/C ratios were found to be below 1.0 on all control sections, thus allowing the retrofitted frame to be deemed safe for the LS of NC according to the q-factor approach.

4. Evaluation of the designed retrofitting system

The previous section presented a case-study application of force-based design (FBD) to the global strengthening of an older-type RC building using X-diagonal steel braces. As no specific rules for the design of hybrid RC-steel systems exist in EC8-1, its provisions concerning steel frames with concentric braces were taken as reference. In parallel, to obtain the seismic forces on the retrofitted structure, behaviour factors q corresponding to two different ductility classes were considered. Lastly, to assess the efficiency of the strengthening intervention, the safety of RC members was checked combining the requirements of the q-factor approach on EC8-3 with the capacity design rule specified in sub-section 6.7.4 on EC8-1 – adapting it to the case of RC members – to allow for the possibility that the actual yield strength of the steel braces is higher than their nominal yield strength. The outcome of this process was a retrofitted structure that seems to be compliant with the requirements of the LS of NC according to the q-factor approach on EC8-3. However, due to the several adjustments that were introduced in the referred code provisions in order to try to adapt them to the structural typology at hand, the performance of the obtained retrofitted structure will be evaluated in the following sub-sections using nonlinear static and dynamic methods of analysis. The results will then be discussed in light of the applicable EC8-3 performance requirements for the LS of NC, and conclusions will be drawn about the effective seismic safety of the retrofitted structure.

4.1 Numerical modelling

The numerical model of the structure was developed using software platform *SeismoStruct*, developed by Antoniou and Pinho [12]. A distributed plasticity model was considered, combined with fibre discretization to represent cross-section behaviour. A force-based FE formulation was implemented, considering five integration sections per element. The full details on the nonlinear modelling options can be found in Falcão Moreira [9], as well as the definition of the target displacements for the nonlinear static analyses, scaling of accelerograms for the nonlinear dynamic analyses, and computation of the RC members' capacities. The latter were defined according to the capacity models included in EC8-3, for deformation- and strength-controlled mechanisms.

4.2 Performance evaluation

The performance evaluation results obtained with nonlinear dynamic analysis are given below, in terms of demand-to-capacity ratios (D_i/C_i) for each control section i, using the same graphical representation as above. These ratios are the mean values over the most unfavourable results obtained within the scaled set of ground acceleration records. In the case of the nonlinear static (pushover) analyses, the demand values on structural members are obtained at the NC target displacements corresponding to each lateral load pattern and seismic action sense. The associated D/C ratios thus correspond to the most unfavourable results among those obtained on each control section. However, being very similar to those provided by the nonlinear dynamic analyses, their representation was deliberately omitted herein. Fig. 4 shows the chord rotation D/C ratios on the columns of the retrofitted (DCM) frame, while Fig. 5 shows the corresponding shear force D/C ratios.

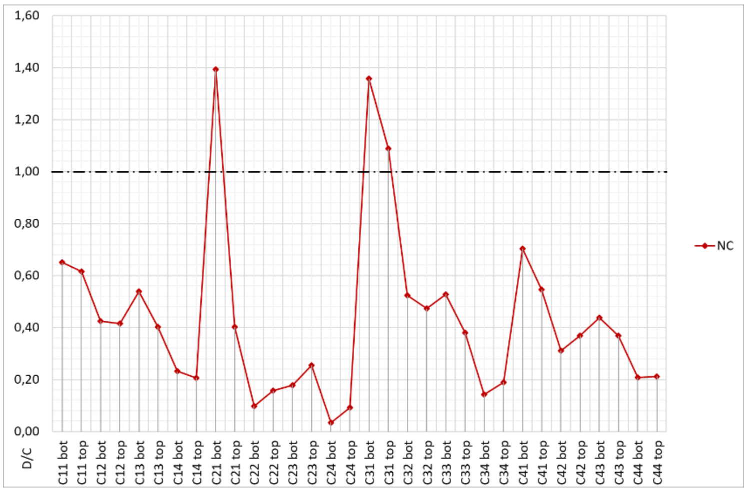

Fig. 4: DCM: NC chord rotation D/C ratios on columns (nonlinear dynamic analysis results)

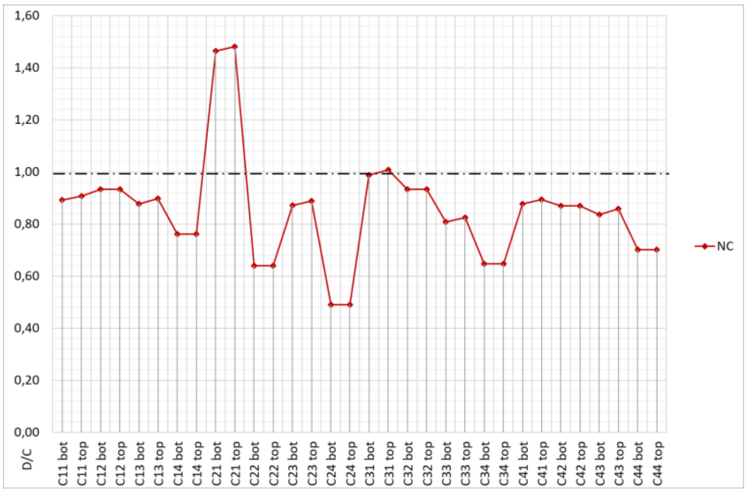

Fig. 5: DCM: NC shear force D/C ratios on columns (nonlinear dynamic analysis results)

4.3 Discussion

The D/C ratios presented above show a different picture than the one provided by the results given in sub-section 3.3 (i.e., the outcome of the q-factor approach). Effectively, values above 1.0 were obtained on two columns (C2 and C3) at the first storey, both for deformation- and strength-controlled collapse mechanisms. This shows that the considered strengthening system is, in fact, unable to reduce the seismic demands on all structural members to values below the corresponding capacities, hence leading to the conclusion that the DCM retrofitted frame should, after all, be deemed unsafe for the LS of NC according to EC8-3.

Given the circumstances, a decision was made to also evaluate the other retrofitted frame (DCL) under nonlinear static conditions. However, convergence difficulties in the numerical models quickly confirmed the conclusions given in sub-section 3.3 regarding this bracing system, i.e., column C31 is not capable of enduring the induced additional axial forces. Therefore, the DCL retrofitted frame is confirmed to be unsafe for the LS of NC according to EC8-3.

In conclusion, the performance evaluation process has shown that both bracing systems are unable to make the retrofitted structure fulfil the EC8-3 requirements for the LS of NC, which raises several questions about the adequacy of the FBD methodology presented in Section 3, when applied to the design of such hybrid RC-steel structural systems.

5. Conclusions

The application of conventional seismic design methodologies to hybrid RC-steel systems was addressed. The conducted study demonstrated that the FBD process gives no guarantees in terms of adequate seismic behaviour of the retrofitted structures. Further research on this matter is thus needed to develop effective performance-based design methodologies that takes explicit consideration of the interaction between both structural systems (RC structure and steel braces), namely the influence of the steel braces' resistance on the deformation capacity of the RC members. The authors hope that these findings will help researchers and practitioners become more aware of the likely error associated with the several adjustments that need to be introduced in conventional seismic code provisions in order to try to apply them to a hybrid RC-steel system.

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