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SEISMIC PERFORMANCE OF NEW EUROCODE 8-COMPLIANT STEEL ECCENTRICALLY BRACED FRAMES

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Abstract

In the present work, the effectiveness of seismic design rules introduced in the second generation of Eurocode 8 is investigated with reference to medium- and high-ductile steel eccentrically braced frame (EBF) structures. For this purpose, dynamic non-linear analyses (DNLAs) were performed on a set of 6 + 6 six-story buildings, i.e., designed accounting for the variation of i) link length, ii) seismic intensity and iii) ductility class. Derived seismic response was hence compared against outcomes from DNLAs on corresponding EBF structures conceived in compliance with the previous version of EN1998. Obtained results show the effectiveness of new design provisions, i.e., especially for medium-ductile EBFs, which can now be suitably designed by means of more simplified rules.

Keywords: Eccentrically Braced Frames (EBFs), Seismic Design, Steel Structures, 2nd Generation of Eurocodes.

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1 INTRODUCTION

When addressing the ductile seismic design of steel structures, multiple solutions can be adopted to withstand horizontal actions and dissipate incoming energy [1-6], i.e., *i*) moment resisting frames (MRFs) which rely on flexural dissipation at beams' ends [7-9], *ii*) concentrically braced frame (CBFs) in which axial hinges are expected to form in diagonal braces [10-11] and *iii*) eccentrically braced frames (EBFs), which dissipate seismic energy through the coupled bending-shear regime occurring in the so-called seismic links, i.e., the beam portions included among adjacent braces [12-13]. Among such solutions, EBFs represent a promising option, i.e., as they are characterized by larger stiffness with respect to MRFs, while they do not completely prevent the realization of openings as in the case of X-shaped CBFs [11].

On the other hand, seismic performance of EBFs is influenced by several key factors that need to be accounted for in the design process [12-16]. For instance, European seismic provisions in force [17] prescribe an explicit ductility check on dissipative zones for lone EBF structures, i.e., as seismic links may endure very large plastic rotations (up to 0.08 rad) during a disruptive earthquake. Moreover, the dissipation regime occurring in links is not a-priori established, as it rather depends on both their geometrical and mechanical features [12-16].

To this end, it is noteworthy reporting that the upcoming 2nd generation of EN1998 [18-19] – whose 1st generation (EN1998-1:2004 [17]) currently provides seismic design rules for European countries since 2004 – has collected results from very recent experimentations and theoretical studies to improve design rules for seismically resistant, ductile steel EBFs, i.e., addressing some criticalities and incongruences emerging from twenty years of use of EN1998-1:2004 while attempting at simplifying and clarifying the entire design process [18-19].

Within this framework, the present paper discusses the effectiveness of seismic design rules introduced in [18-19] with reference to both medium- and high-ductile steel EBF structures. For this purpose, dynamic non-linear analyses (DNLAs) were performed on a set of 6 + 6 six-story archetypal buildings, i.e., designed according to both EN1998-1:2004 and prEN1998-1-1:2024 + prEN1998-1-2:2024 and accounting for the variation of link type, seismic intensity and ductility class.

In light of the above, this work is mainly divided in three parts. In the first section, main changes among both generations of EN1998 related to seismic design of EBFs are illustrated. Subsequently, the archetypal building selected as case study is thoroughly described both in terms of geometrical features and relevant numerical modelling. Finally, results from DNLAs are presented and compared to outline the effectiveness of new rules included in [18-19].

2 MAIN VARIATIONS IN EUROPEAN SEISMIC DESIGN RULES FOR DUCTILE STEEL EBFS

2.1 Generality

Ductile steel EBFs are designed to dissipate incoming seismic energy through cyclic plasticization of seismic links, i.e., beam portions enclosed among adjacent braces [12-16]. The relevant internal actions regime for dissipation purposes (i.e., bending moment M, shear V or combined M-V) depends on the ratio among the link physical length e and its "mechanical" length e_0. The mechanical length is in turn expressed as $e_0 = M_{p,link}/V_{p,link}$, with $M_{p,link}$ and $V_{p,link}$ being the link plastic bending and plastic shear capacity, respectively. As it descends from basic equilibrium conditions, large e/e_0 ratios will result in the link dissipating in bending, while shear dissipation is associated with small e/e_0 ratios. As first highlighted in the pioneering works of Popov [14-16], thresholds can be identified to distinguish "short" (SL-EBFs, V dominated), "intermediate" (IL-EBFs, M-V combined) and "long" link (LL-EBFs, M dominated) structures.

In particular, accounting for cyclic strain hardening effects, the threshold among SL- and IL-EBFs was estimated as 1.6 e_0, while the value 3.0 e_0 separates IL- and LL-EBFs [12-16]. Such values are explicitly included in EN1998-1:2004. As a result, geometrical proportioning of links is not a-priori separable from their mechanical response. Typically, SL-EBFs are preferred due to larger lateral stiffness, although the ductility demand on links highly rises as e decreases [14].

Other structural components (non-link containing beam parts, diagonals, columns, joints) are conceived as non-dissipative, and should hence be over-resistant in compliance with the well-known capacity design philosophy [20-22].

Geometrical arrangement of diagonals as respect to link-containing beams enables a further distinction among D-shaped – one brace, link placed at the beam end, Y-shaped – two braces, link placed vertically – and K-splitted EBFs – two braces, link placed at the beam midspan – with the latter solution being the most popular in technical practice. According to European provisions, each configuration can be either designed as medium- or high-ductile, and relevant rules for dissipative and non dissipative zones are detailed to achieve such structural behavior. Key insights about these design rules are reported in the following subSections.

2.2 Medium ductile EBFs

The comparison among seismic design rules for medium ductile steel EBFs (i.e., DCM and DC2 according to 1st and 2nd generation of Eurocode 8, respectively) is reported in Figure 1. Namely, main changes between EN1998-1:2004 [17] and prEN1998-1-1:2024 + prEN1998-1-2:2024 [18-19] can be summarized as follows:

- The behavior factor q for force based design was reduced from q = 4 to q = 3. Moreover, q is now expressed as the product of three factors accounting for the dissipation capacity of the EBF (q_D = 1.8), the plastic redistribution capability (q_R = 1.1) and all other overstrength sources (q_S = 1.5), respectively. This has relevant implications while checking P-Delta effects;
- While DCM design was always possible in high seismicity countries, DC2 design of EBFs is now enforced for a plateau value of spectral acceleration $S_\delta \geq 0.51\ g$ and not allowed anymore for $S_\delta \geq 0.66\ g$;
- prEN1998-1-2:2024 now allows the use of hollow structural sections (RHS or SHS) as seismic links in place of I-shaped profiles. Relevant formulas to estimate the plastic bending and shear resistance have been provided accordingly;
- Ductility checks for seismic links are now only enforced in DC2 if design is based on non-linear structural analyses;
- With reference to non-dissipative zones, an hybrid formulation for design was prescribed by EN1998-1:2004. Accordingly, seismic demand in terms of axial force $N_{Ed,E}$ had to be incremented by 1.1 γ_{ov} Ω times to account for strain hardening, material randomness and link overstrength, respectively. Remarkably, Ω mistakenly accounted for strain hardening as well, i.e., as proved by the coefficient 1.5 in its definition. prEN1998-1-2:2024 now includes a revised hybrid formulation only for high ductile EBFs, while DC2 ones can be now designed with a simplified procedure, i.e., by doubling $N_{Ed,E}$. Indeed, a flat-rate value Ω = 2 is now prescribed for medium-ductile EBFs;
- Overstrength checks to ensure uniform links yielding along the building height are not required anymore in DC2 design;
- Lateral deformability check in ultimate conditions have been now introduced. Namely, the maximum interstorey drift ratio $IDR_{max} = \delta_{max}/H_{is}$ should not exceed 0.015 at significant damage (SD) limit state.

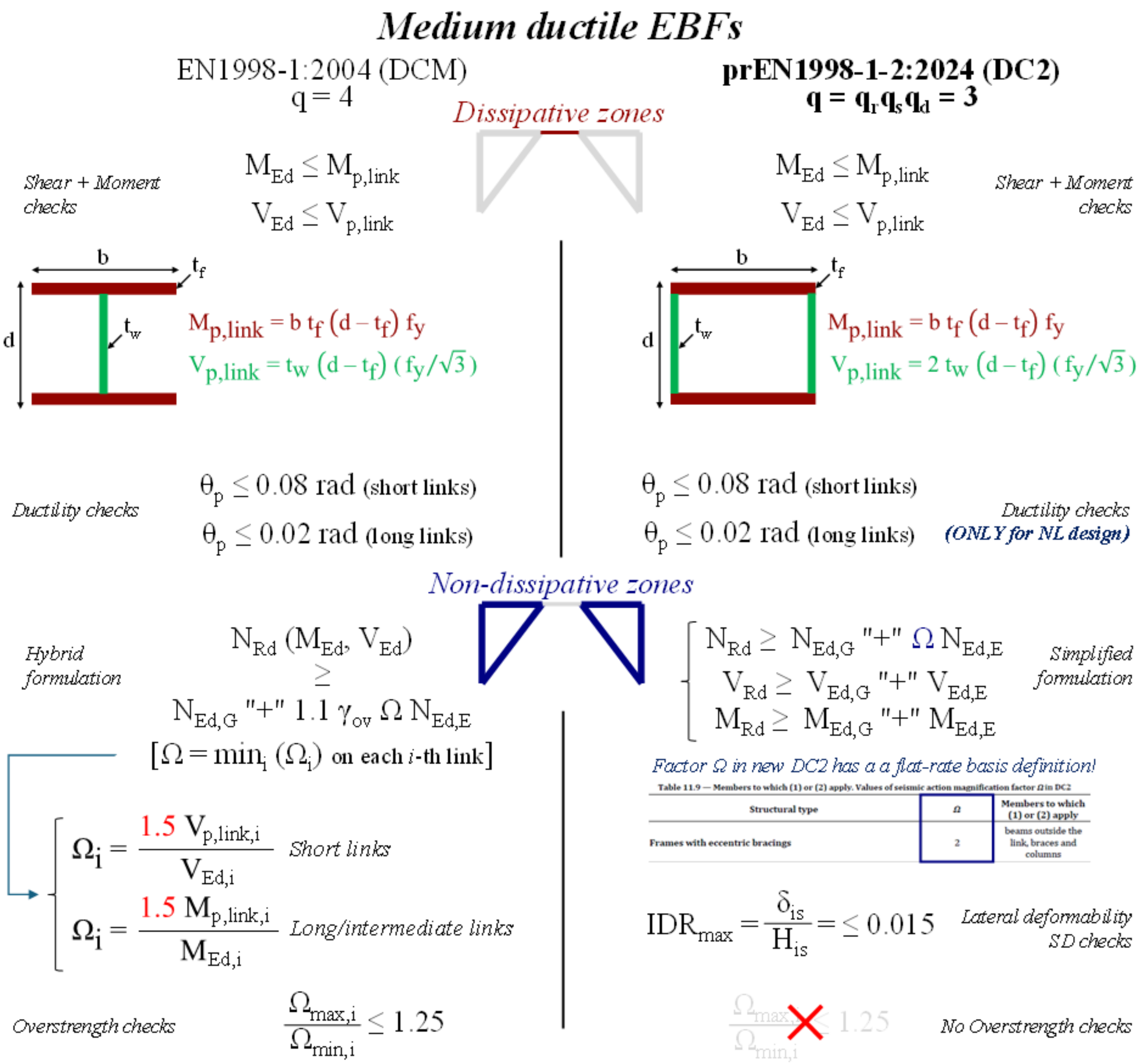

Figure 1: Seismic design rules for medium ductile EBFs according to EN1998-1:2004 and prEN1998-1-2:2024.

2.3 High ductile EBFs

The comparison among seismic design rules for high ductile steel EBFs (i.e., DCH and DC3 according to 1st and 2nd generation of Eurocode 8, respectively) is reported in Figure 2. Namely, main changes between EN1998-1:2004 [17] and prEN1998-1-1:2024 + prEN1998-1-2:2024 [18-19] can be summarized as follows:

- The same value of behavior factor q = 6 is prescribed in both DCH and DC3. Nevertheless, q is now expressed as the product of $q_D = 3.1 \times q_R = 1.1 \times q_S = 1.5$;
- DC3 rules are applicable for any S_δ level, but are is now mandatory for $S_\delta \geq 0.66\, g$;
- Link design is identical as respect to DC2. Nevertheless, ductility checks are always mandatory in DC3;
- Revised hybrid formulations for non-link containing beam segments, diagonals and columns were introduced. Strain hardening and links overstrength are now correctly accounted for in relevant formulas;
- The e/e_0 threshold among V- and M-dominated links has been slightly reduced (2.6 in place of 3.0), i.e., with some implications for non-dissipative zones design;
- Both overstrength and lateral deformability checks are mandatory at SD limit state.

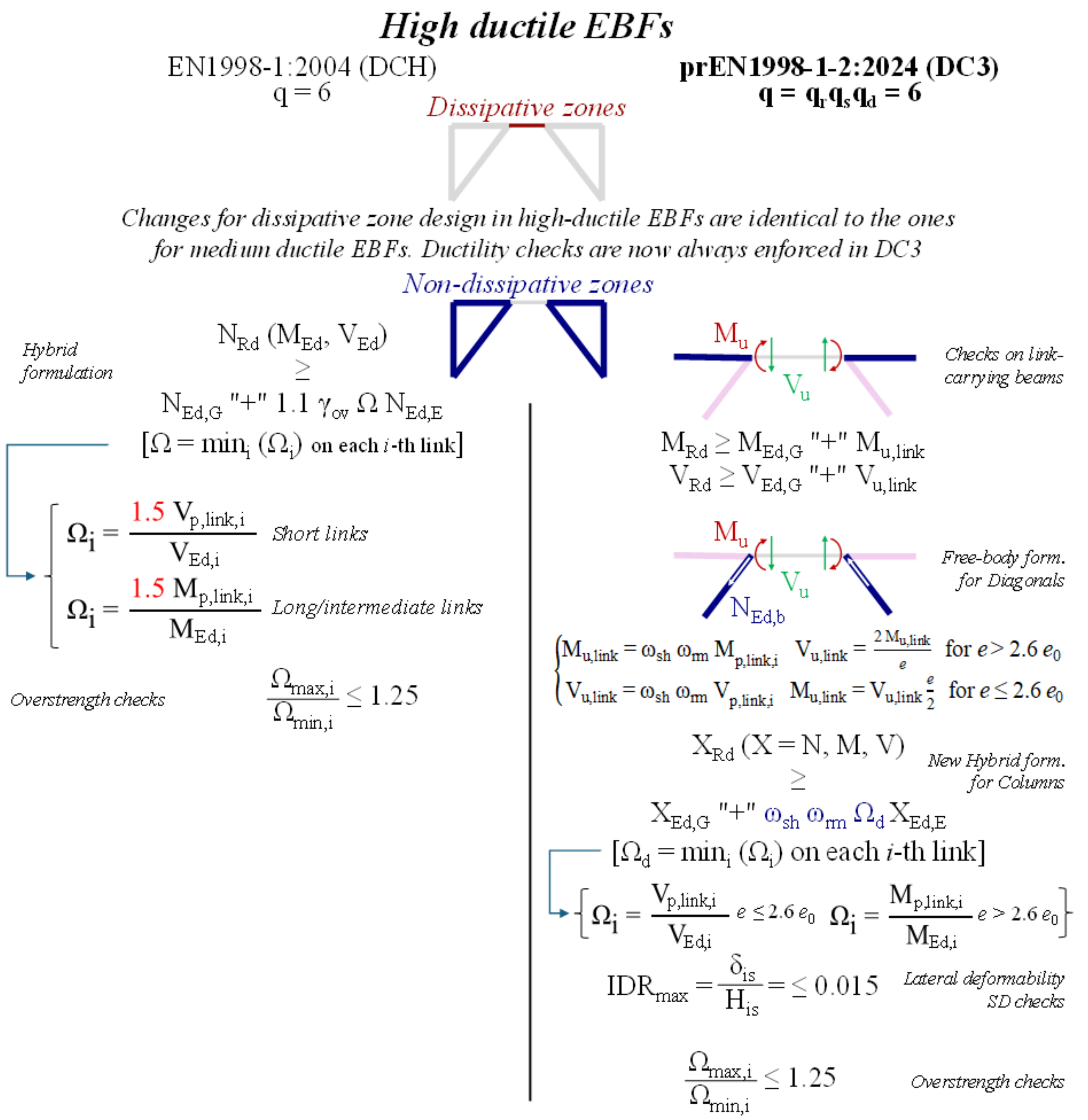

Figure 2: Seismic design rules for high ductile EBFs according to EN1998-1:2004 and prEN1998-1-2:2024.

3 DESCRIPTION AND MODELLING OF THE CASE STUDY

3.1 Main geometrical features of the building

The selected case study is the worked example of archetypal steel building included in the seismic design manual of the European Convention for Constructional Steelwork (Figure 3 [23]). The structure is articulated in six levels having a constant inter-story height H_{is} = 3.5 m, i.e., with the only exception being represented by the 1st floor which has H_{is} = 4 m. The rectangular building plan (24 × 31 m) features five uneven spans (L = 5÷7 m) along the horizontal direction, four identical spans (L = 6 m) in vertical direction and two central staircases.

2+2 Seismically resistant systems are placed in perimetral spans. For the present work, they were conceived as either SL or LL K-splitted EBFs. A residential destination of use (q_k = 2 kN/m^2) was considered while designing floors, for which a typical *composite slab* + *secondary beams* + *main beams* arrangement was assumed.

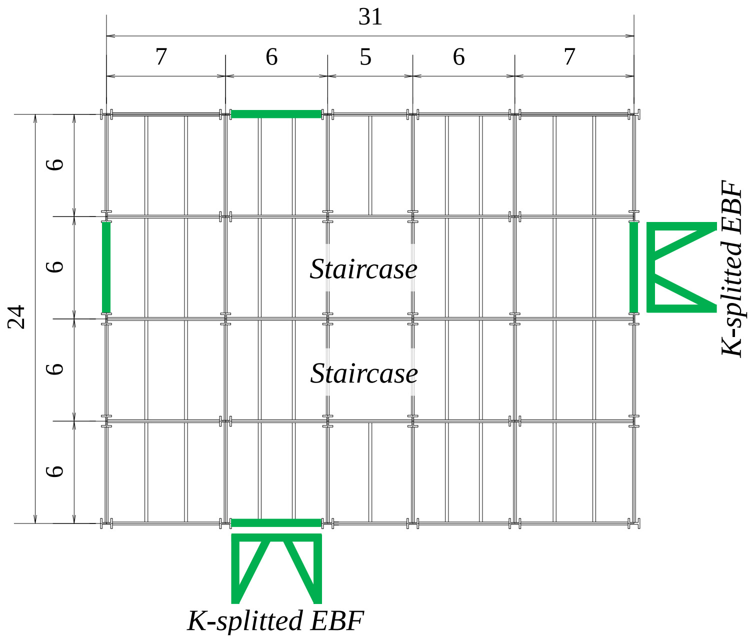

Figure 3: Building plan of the selected case study (*adapted from* [23]).

3.2 Modelling features for DNLAs

Finite element modelling of the case-study was developed in SAP 2000 v.23 ([24]). Each structural member was modelled trough a mono-dimensional frame element. Permanent and live loads deriving from floors, claddings and crowding were modelled as equivalent loads applied on relevant members. Equivalent restraints or releases were introduced at relevant members' ends to account for column bases' fixity and joints' flexural stiffness, respectively [25].

Non-linear behavior of seismically-resistant systems was accounted for by means of lumped plastic hinges placed in links, diagonals and columns. Hinges response was modelled according to prEN1998-1-1:2024 prescriptions [18]. In-plane rigidity of floors was taken into account via diaphragm constraints at each level.

Seismic response of each designed configuration was investigated trough non-linear dynamic analyses (DNLAs). For this purpose, 22 ground motion signals were drawn from the FEMA P-695 catalog of natural earthquake events [26] and hence suitably scaled to match the design normative spectrums of interest.

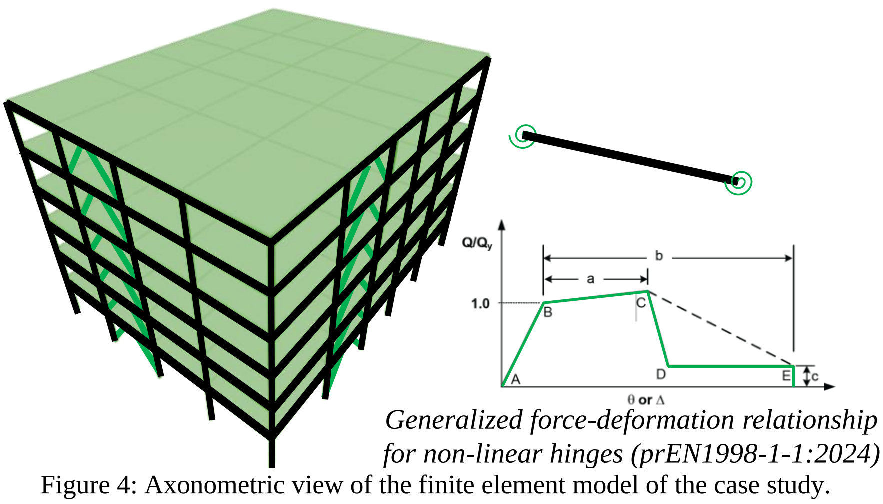

Figure 4: Axonometric view of the finite element model of the case study.

3.3 Parametric study

In order to assess the effectiveness of new seismic design rules for medium- and high-ductile EBFs, 6 + 6 configurations were designed with reference to the case study, i.e., accounting for the variation of the following parameters:

- Link length (either short – SL – or long – LL);
- Seismic design intensity (S_δ = 0.66 *g* or 0.76 *g*);
- Adopted provisions (EN1998-1:2004 or prEN1998-1-1:2024 + prEN1998-1-2:2024)
- Ductility class (medium – DCM, DC2 or high – DCH, DC3).

To identify each configuration, a proper labelling "XL-S_δ-DCX" was used. For example, LL-0.76-DCH configuration features high-ductile long links designed according to EN1998-1:2004 DCH rules to withstand a maximum seismic intensity equal to 0.76 *g*.

4 RESULTS AND DISCUSSION

Results from DNLAs are summarized in Figure 5 in terms of median values of largest inter-story drifts attained at each level (solid black lines). Relevant drift limits as per prEN1998-1-2:2024 (IDR_{max} = 0.015) are also represented via solid green lines. For the sake of brevity, only results referred to short link (SL) EBFs are depicted in the Figure.

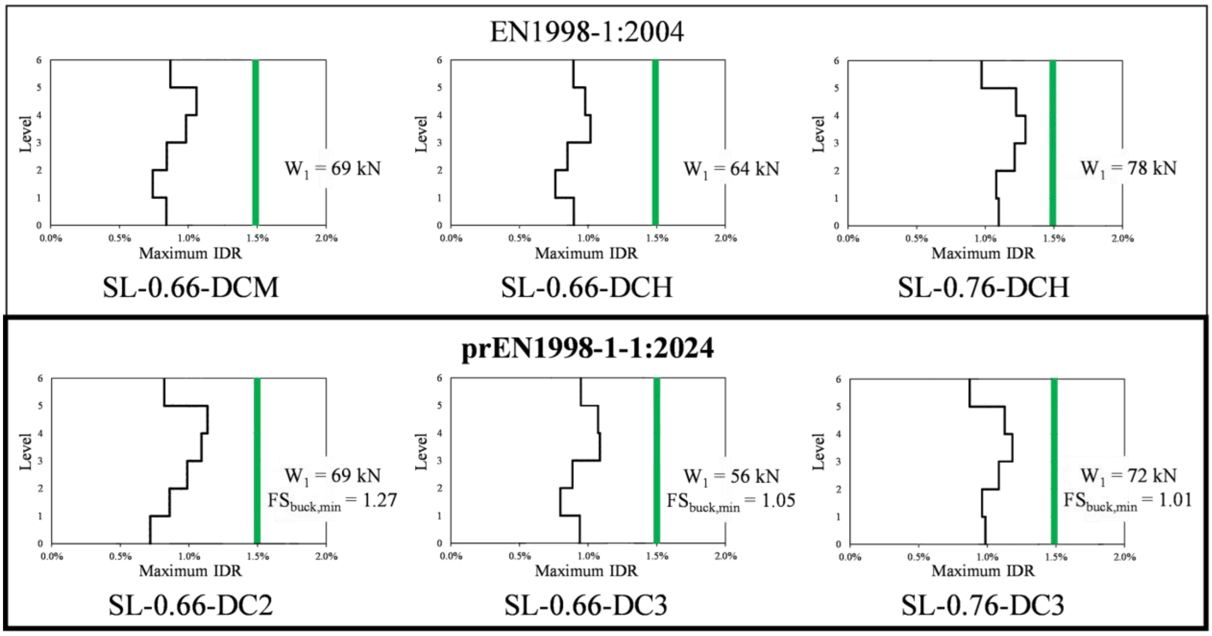

Figure 5: Results from DNLAs for SL configurations.

It can be easily noticed that median values of IDR_{MAX} for structures designed according to EN1998-1:2004 would meet new SD deformability requirements from prEN1998-1-2:2024. Notably, very similar IDR_{MAX} trends are achieved for DC2/DC3 structures as respect to DCM/DCH ones, i.e., with very limited IDRs at each story. This result occurs both in case of SL and LL EBFs, although LL-0.66-DCH/DC3 configurations very slightly exceed the IDR_{MAX} threshold at the lone last floor. This is a consequence of the larger deformability and sensitivity to P-Delta effect shown by long link EBFs.

In order to investigate more deeply the effectiveness of new seismic provisions, minimum safety factors for buckling checks $FS_{buck,min}$ were estimated for columns and diagonals in DC2/DC3 structures. Remarkably, exceedance of buckling strength was never detected

($FS_{buck,min} > 1$ for all cases). This is particularly relevant for new DC2 approach, as very simplified rules for non dissipative zones can be hence deemed suitable.

Moreover, configuration designed according to prEN1998-1-1:2024 + prEN1998-1-2:2024 are characterized by a slightly lower weight per single EBF system W_1. This plausibly descends from corrections made to formulations for non dissipative zones design.

5 CONCLUSIONS

In the present work, the seismic performance of medium- and high-ductile steel EBFs designed according to the upcoming version of Eurocode 8 was assessed through DNLAs and compared with analogous structures complying with EN1998-1:2004. For this purpose, an archetypal medium-rise building was selected as case study. On the basis of derived outcomes, the following conclusions can be pointed out:

- Most relevant changes in prEN1998-1-2:2024 with respect to medium-ductile EBFs (DCM → DC2) refer to a reduction of behavior factor *q* and a simplified criterion for brace and column design, i.e., based on a prescribed increment of seismic axial forces;
- With reference to high-ductile EBFs (DCH → DC3), the inconsistency arising while accounting for strain hardening was solved. Moreover, more accurate criteria were provided for the design of non-dissipative zones;
- New design rules for medium-ductile (DC2) EBFs, although strongly simplified, result in a suitable performance of non dissipative elements;
- DC3 design outcomes are basically comparable with DCH ones, although corrections brought to the hybrid formulations for columns and diagonals result in slightly lighter structures;
- Further studies will be carried out with reference to other representative configurations.

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