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COMPARATIVE ASSESSMENT OF STEEL MOMENT FRAMES DESIGNED TO EUROCODE 8

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Abstract: *This paper investigates various changes in the seismic design of steel structures introduced in the upcoming second generation of Eurocode 8. The main objective is to compare the provisions in the current and revised version of the code, with a particular focus on the assessment of steel columns. For this purpose, two typical high-ductility steel moment-resisting frames of 3 and 7 storeys are employed as case studies. The frames are analysed in OpenSees according to the lateral force method, and the relevant design checks are then performed based on both code versions. More specifically, the axial force, shear force, bending moment, buckling, and lateral torsional buckling utilisation ratios of columns are compared, in addition to the stability coefficients. Special attention is given to the overstrength ratio, which is determined using a more realistic approach in the revised code version, accounting for the effect of gravity loading. The seismic performance of both frames is also compared using nonlinear analysis. The results show that higher column design loads are obtained with the upcoming provisions, and hence, higher utilisation ratios. In fact, this increase is more profound in bending-related checks, as the design bending moments of columns are considerably higher following the revised code. The revised provisions also lead to about half the values of the stability coefficients obtained with the current guidelines. Overall, when the new version of Eurocode 8 is adopted, it becomes more demanding to satisfy the column design requirements, while the second-order effects are usually less critical.*

Introduction

Adequate seismic design of steel moment frames is of vital importance to ensure their satisfactory performance in the event of an earthquake. In Europe, although the application of Eurocode 8 (CEN, 2004) has greatly improved the seismic performance of buildings, there is a need to update the current form of the code by incorporating recent research findings that can contribute to safer designs (Elghazouli, 2010; Silva et al., 2020; Lignos and Hartloper, 2020). Thus, a revised, second generation, version of the code is being prepared, which introduces significant changes in the design and assessment procedures (CEN, 2020).

The revised version of the code improves several aspects of the current provisions for steel moment-resisting frames. One key aspect is a more realistic consideration of the overstrength of beams, by accounting for the influence of gravity loads, as suggested by Elghazouli (2010). By distinguishing between gravity and seismic loads, the revised overstrength factor can offer more appropriate estimations and lead to a more accurate implementation of the capacity design principles in steel moment frames. The revised definition of overstrength is adopted to define the column design loads and is also incorporated in the definition of the interstorey drift sensitivity coefficient and the drift limitation check (CEN, 2020).

The revised Eurocode 8 provisions have been applied in a series of preliminary studies to highlight the differences with the current guidelines (e.g., de Almeida et al., 2022). Focusing on steel moment-resisting frames, Lemma et al. (2022) compared the seismic performance of 96 frames optimally designed to both code versions. It was shown that with the upcoming code provisions, the stability requirements did not govern the design, and therefore, lighter solutions were possible. The frames designed with both the current and revised Eurocode 8 showed adequate performance in the damage limitation, significant damage, and near collapse limit states.

This paper compares the provisions of the current and revised Eurocode 8 for the seismic assessment of two typical steel moment-resisting frames of 3 and 7 stories. Seismic analyses of the frames according to the Lateral Force Method are carried out, in addition to static analyses

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under gravity loading and nonlinear static pushover analyses. The axial force, shear force, bending moment, buckling, and lateral torsional buckling utilisation ratios of steel columns, as well as the overstrength factors and the interstorey drift sensitivity coefficients at each floor, are evaluated for both frames, based on the current and updated code provisions. In addition, the pushover curves of the frames are compared. Based on the results, the changes introduced in the revision of Eurocode 8 are highlighted and the main implications are discussed.

Modelling procedures

Structural frames

Two steel moment-resisting frames of 3 and 7 stories, respectively, are employed as case studies in this paper. Both frames comprise 3 bays of 6.0m each and exhibit a typical storey height of 3.5m, except for the first floor, where the height is 4.5m. The steel yield strength is assumed as 275 MPa and the characteristic values of gravity loads are given in Table 1. The frame sections are selected such that the relevant design requirements of both the current and revised Eurocode 8, as well as the Eurocode 3 provisions, are satisfied. A behaviour factor of 4 and high ductility class (DCH in the current Eurocode 8, or, equivalently, DC3 in the revised version) is assumed in the design. The seismic hazard is defined based on Type 1 response spectrum, for a reference peak ground acceleration of 0.36g, which corresponds to the highest seismic zone in Greece, according to the National Annex to Eurocode 8. The structural models represent residential buildings, belonging to Importance Class II, or, equivalently, Consequence Class CC2 in the revised Eurocode 8. The geometry and member sizes are illustrated in Figure 1, while the first three modal periods are presented in Table 2.

	Dead load (kN/m^2)	Imposed load (kN/m^2)
Floor	5.75	2.00
Roof	4.75	1.00

Table 1. Characteristic values of gravity loads.

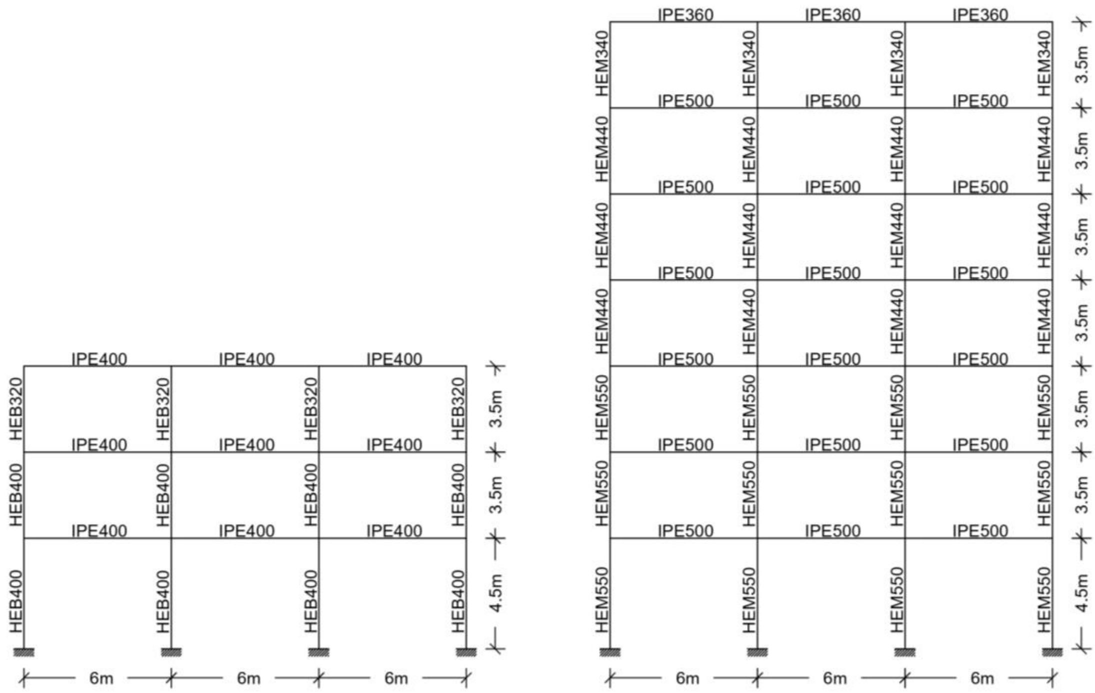


Figure 1. Elevation views of 3- and 7-storey frames.

	T$_1$(s)	T$_2$(s)	T$_3$(s)
3-storey frame	0.70	0.20	0.10
7-storey frame	1.04	0.34	0.18

Table 2. Modal periods of steel frames.

The frames are modelled in OpenSees (McKenna et al., 2011), employing the lumped plasticity approach. A schematic of the numerical model created for the 3-storey frame is illustrated in Figure 2. The beams and columns are simulated through elastic beam-column elements with zero-length springs at their ends, where their nonlinearity is concentrated. The modified Ibarra-Medina-Krawinkler (IMK) model (Ibarra et al., 2005), calibrated according to Lignos and Krawinkler (2011), is assigned to the springs. The panel zones are modelled as a rectangular assembly of rigid elastic elements, pinned-connected at the three corners and with a rotational spring (zero-length element) at the top right corner. The spring properties are defined based on the Gupta and Krawinkler (1999) approach. Rayleigh damping of 2% at the 1st and 3rd modes is assigned to all elements, except for the zero-length springs, following the recommendations by Zareian and Medina (2010). The structural models are fixed at the column bases and diaphragm action is assumed at each floor.

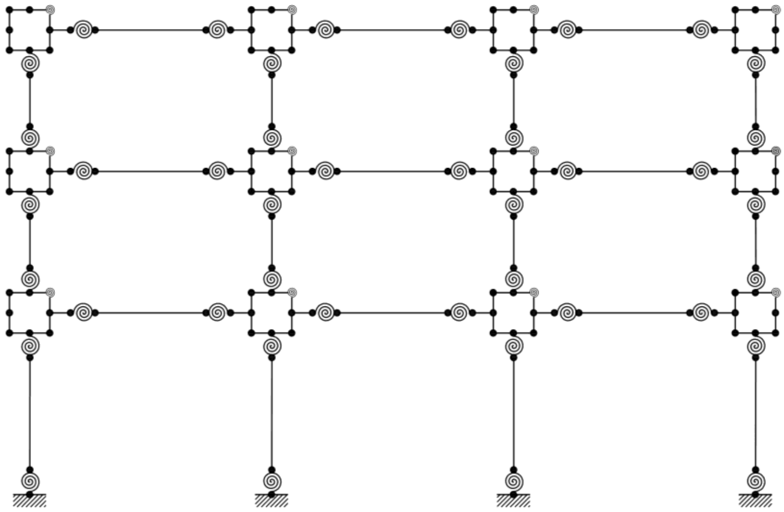


Figure 2. Numerical model in OpenSees, shown for the 3-storey frame.

Analysis approaches

Linear static analyses of the frames are conducted in OpenSees (McKenna et al., 2011), considering the gravity and seismic loading cases. The gravity load is determined as a combination of the permanent and 30% of the variable load (CEN, 2004; CEN, 2020), applied uniformly at the beams of each floor. For the seismic case, the Lateral Force Method is adopted, distinguishing between the current and revised Eurocode 8 provisions. Accordingly, horizontal loads with a first-mode proportional distribution are applied at each floor level. It is noted that the Lateral Force Method is applicable herein, as both frames satisfy the relevant code requirements.

Nonlinear static pushover analyses are also conducted, by employing a lateral force distribution according to the Lateral Force Method. The displacement at the roof is chosen as the controlled displacement and is increased uniformly until a roof drift level of 0.05 or numerical instability is reached.

Comparative assessment

The most critical design checks for the two frames, based on the current and revised EC8, are compared in this section. Firstly, the overstrength factors are estimated, and the differences between the two versions of the code are highlighted. Next, member design checks are carried out, and the second-order stability coefficients are estimated. It is noted that the interstorey drift limitation check is not shown herein, as there was almost no difference between the two code versions for the examined buildings. Finally, the pushover curves of the two frames are presented.

Overstrength factor

The revised EC8 draft introduces significant changes in the seismic design, mostly based on a more realistic estimation of the lateral overstrength. An important source of overstrength is the capacity design of columns. To ensure that the beams yield before the columns, the column design loads are increased so that they correspond to the first plasticity in beams. More specifically, the column seismic design loads are multiplied by an overstrength factor Ω that represents the increase in beam seismic design loads until their plastic capacity is reached. The current EC8 provisions do not differentiate between gravity and seismic actions, assuming that both are scaled equally during loading. Accordingly, Equation 1 is used for the overstrength.

$$\Omega = \min\left(\frac{M_{pl,Rd,i}}{M_{Ed,G+E,i}}\right) \tag{1}$$

where $M_{pl,Rd,i}$ is the design plastic moment of resistance of beam i, $M_{Ed,G+E,i}$ is the design bending moment of beam i due to the combination of gravity and seismic loading, and the minimum refers to all beams in dissipative zones.

However, if a distinction is made between gravity and seismic loading, the overstrength factor can be more accurately estimated from Equation 2, proposed by Elghazouli (2010), which is adopted in the revised EC8.

$$\Omega_{rev} = \min\left(\frac{M_{pl,Rd,i} - M_{Ed,G,i}}{M_{Ed,E,i}}\right) \tag{2}$$

where $M_{Ed,G,i}$ is the design bending moment of beam i due to gravity loading, and $M_{Ed,E,i}$ is the design bending moment of beam i due to seismic loading.

The overstrength factors of the two frames under consideration are reported in Table 3, based on both code provisions. Evidently, the revised formula results in considerably higher overstrength (54% and 26% increase for the 3- and the 7-storey frame, respectively), as the multiplier of the seismic bending moment of the critical beams, which is required to achieve the beam plastic capacity, is more accurately estimated. This has a significant effect on the design of columns, as discussed in the following section.

	Current EC8	**Revised EC8**
3-storey frame	1.90	2.92
7-storey frame	2.14	2.69

Table 3. Overstrength factors of the frames according to the current and revised EC8.

Member design checks

The design of the two frames is assessed herein, by performing the main checks according to both EC8 versions. The most significant discrepancy was noted in the design column loads, mainly due to the new definition of overstrength. Hence, the comparison that follows focuses on the design checks for columns, while those for beams are omitted since no notable dissimilarity was observed.

In the context of capacity design and to account for additional sources of overstrength, such as material overstrength or hardening, the code prescribes multipliers for the seismic design loads of columns in steel moment frames. The total multiplier is defined as $1.1\gamma_{ov}\Omega$ and $\gamma_{rm}\gamma_{sh}\Omega_{rev}$ in the current and updated code, respectively, where γ_{ov} is the material overstrength factor, γ_{rm} is the randomness material factor, and γ_{sh} is the overstrength factor accounting for hardening of the dissipative zone. As shown in the previous section, $\Omega_{rev} > \Omega$, and for steel S275: $1.1\gamma_{ov} > \gamma_{rm}\gamma_{sh}$; hence, the revised EC8 provisions are associated with higher column seismic demands. The column design axial force N_{Ed}, shear force V_{Ed}, and bending moment M_{Ed} are determined by combining the corresponding gravity and seismic loads, as shown in Equations 3-5.

$$N_{Ed} = N_{Ed,G} + \lambda N_{Ed,E} \tag{3}$$

$$V_{Ed} = V_{Ed,G} + \lambda V_{Ed,E} \tag{4}$$

$$M_{Ed} = M_{Ed,G} + \lambda M_{Ed,E} \tag{5}$$

where $N_{Ed,G}$, $V_{Ed,G}$, and $M_{Ed,G}$ are the axial force, shear force, and bending moment, respectively, due to gravity loading, $N_{Ed,E}$, $V_{Ed,E}$, and $M_{Ed,E}$ are the axial force, shear force, and bending moment, respectively, due to seismic loading, and λ is the multiplier for the seismic loads ($1.1\gamma_{ov}\Omega$ and $\gamma_{rm}\gamma_{sh}\Omega_{rev}$ based on the current and revised EC8, respectively).

Figures 3 and 4 compare the utilisation ratios of columns at each storey, as obtained from the current and revised EC8, for the 3-storey and the 7-storey frame, respectively. The axial, shear, bending, buckling, and lateral torsional buckling (LTB) utilisation ratios are computed as the ratio of the corresponding design load over the column resistance.

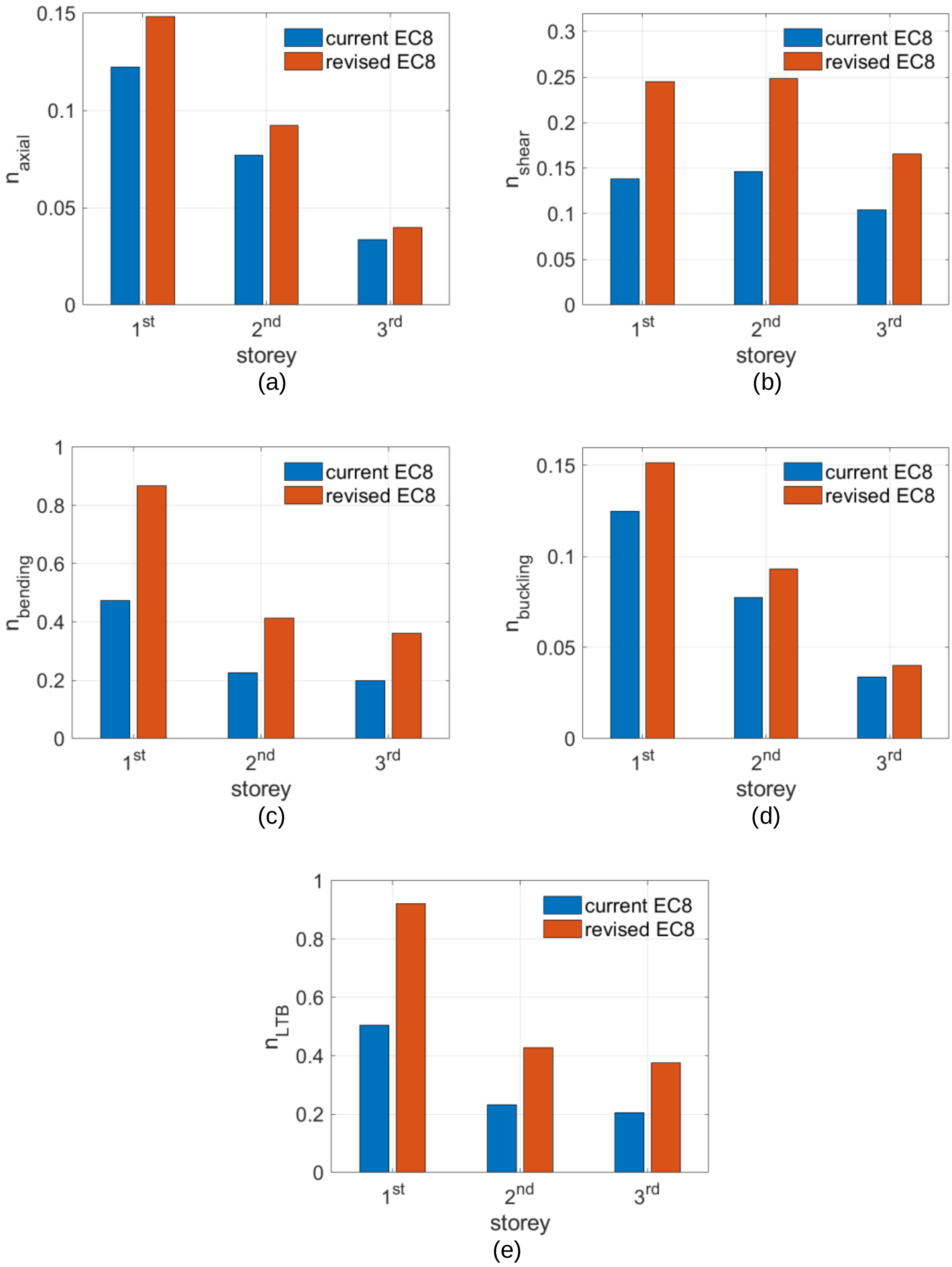
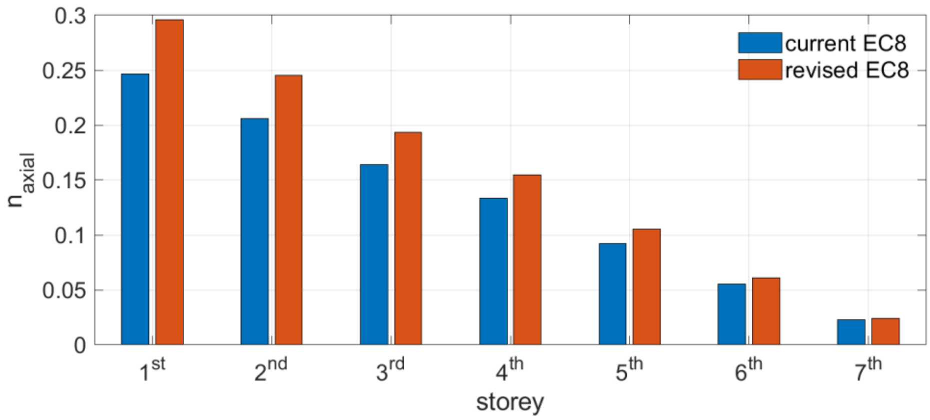


Figure 3. Utilisation ratios of the 3-storey frame according to the current and revised EC8: (a) axial force, (b) shear fore, (c) bending moment, (d) buckling, and (e) lateral torsional buckling.

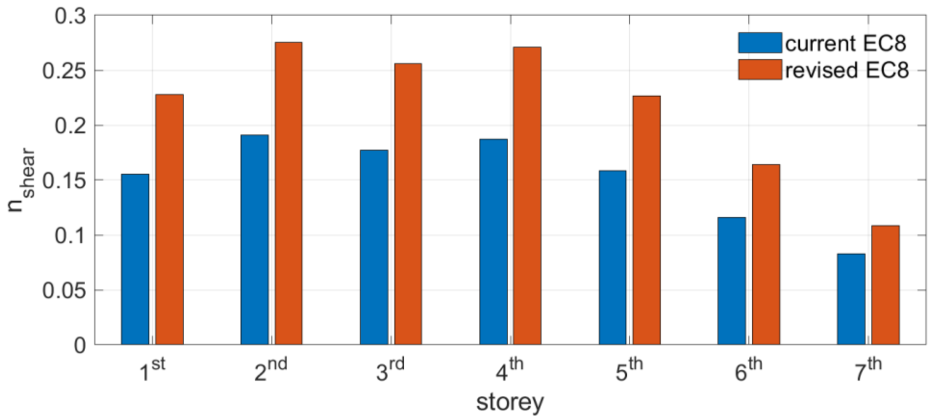
Higher utilisation ratios are obtained when employing the revised code, as a direct result of the higher column design loads. In fact, this increase is more profound in bending-related checks (i.e., bending and lateral torsional buckling), since the design bending moments of columns are considerably higher in the revised EC8 compared to the current code version. The differences between the current and revised utilisation ratios tend to decrease at higher stories of a given building. The seismic design loads are generally reduced on upper floors, and therefore the effect of the revised code provisions is relatively minor. Based on the above discussion, the most notable differences between the two code versions are observed for the bending moment and

LTB resistance checks on the first floor, with the upcoming provisions leading to 81% higher utilisation ratios for the 3-storey frame and 49% higher ratios for its 7-storey counterpart.

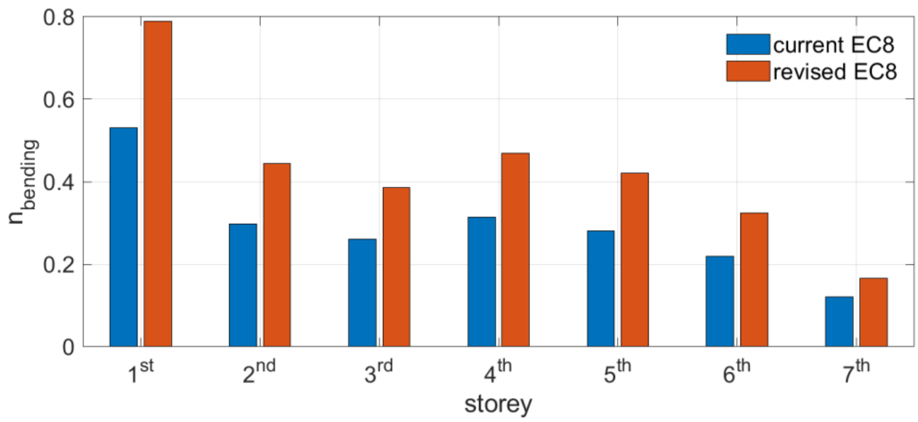
All column design checks are satisfied for both examined frames and with both code provisions, as the utilisation ratios are consistently lower than unity and the shear utilisation ratios are lower than 0.5. It is noted that the member sizes were intentionally selected such that the design requirements according to both code versions are fulfilled. However, with the revised EC8, it becomes more demanding to satisfy the column design requirements, particularly those related to the bending resistance, as discussed above.



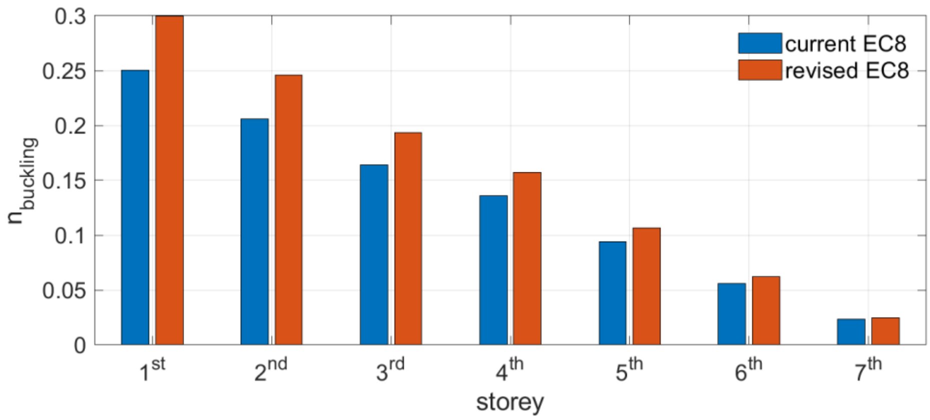
(a)



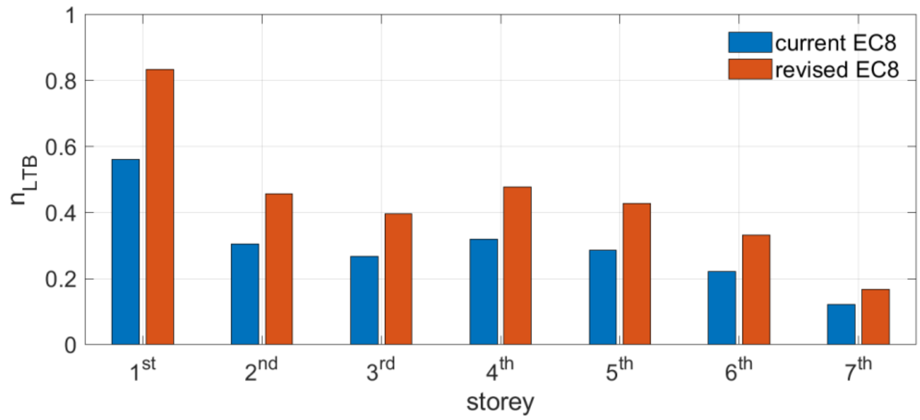
(b)



(c)



(d)



(e)

Figure 4. Utilisation ratios of the 7-storey frame according to the current and revised EC8: (a) axial force, (b) shear fore, (c) bending moment, (d) buckling, and (e) lateral torsional buckling.

Stability coefficient

The influence of second-order effects is also examined, by calculating the stability coefficient θ. If θ<0.1, second-order effects need not be accounted for in the analysis, according to both code versions. The stability coefficient is currently estimated from Equation 6, which is updated to Equation 7 in the revised EC8. This allows a more accurate estimation of the inelastic stiffness, which is significantly underestimated in the current code, resulting in unrealistically large θ values.

$$\theta = \frac{P_{tot} d_r}{V_{tot} h} \tag{6}$$

$$\theta = \begin{cases} \dfrac{P_{tot} d_r \gamma_{SD,CC}}{V_{tot} h \gamma_{rm} q_r \Omega_{rev}}, & if\ DC3\ and\ q_s < \gamma_{rm}\Omega_{rev} \\ \dfrac{P_{tot} d_r \gamma_{SD,CC}}{V_{tot} h q_r q_s}, & otherwise \end{cases} \tag{7}$$

where P_{tot} is the total gravity load due to the seismic design situation at and above the storey under consideration, d_r is the design interstorey drift, V_{tot} is the total seismic storey shear, h is the storey height, $\gamma_{SD,CC}$ is the performance factor for the significant damage limit state, and q_r and q_s are components of the behaviour factor.

Figures 5 and 6 show the stability coefficient based on the current and revised provisions, computed at each storey of the 3- and 7-storey frames, respectively. The revised EC8 leads to about half the values obtained with the current code. This trend is consistent at all floor levels and for both examined structures. The lower levels of the stability coefficient, obtained from Equation 7, are due to the adoption of a more realistic definition that accounts for the available overstrength. It should be noted, however, that this depends on the level of the behaviour factor employed in the design. For lower behaviour factors, lower lateral frame overstrength would be expected, resulting in reduced differences between the design base shear and the actual frame capacity, and hence, a less notable effect of the revised code provisions on the stability coefficient.

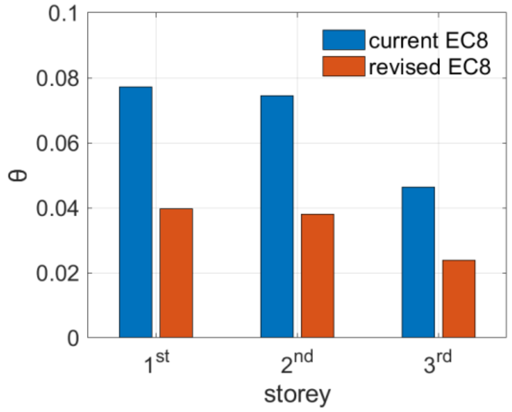


Figure 5. Stability coefficients of the 3-storey frame according to the current and revised EC8.

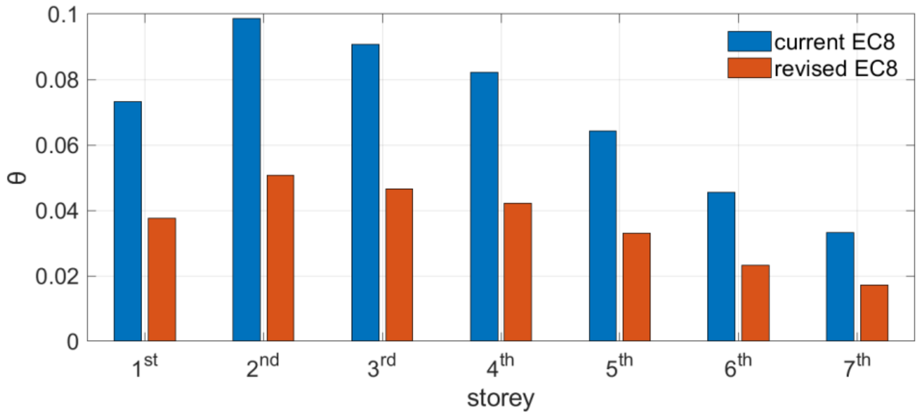


Figure 6. Stability coefficients of the 7-storey frame according to the current and revised EC8.

Pushover curves

The capacity curves of the two frames are plotted in Figure 7. The curves were obtained from pushover analyses with a modal load pattern, proportional to the force distribution from the Lateral Force Method. The maximum base shear is about 1100kN for the 3-sorey frame, while it is increased by approximately 40% to 1800kN for the 7-storey frame. In both structures, a hardening part follows the initial elastic region of the pushover curves, as the beam and column hinges yield, until a plateau is reached. A decrease in stiffness is also notable for the 7-storey frame, due to the accumulation of degradation. At the range of roof drift considered herein, no degradation is detected in the case of the 3-storey frame.

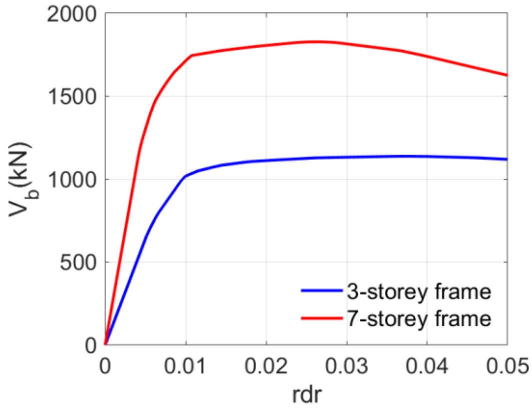


Figure 7. Pushover curves of the 3-storey and 7-storey frame.

Conclusions

This paper investigated the effect of the revised Eurocode 8 provisions on the seismic design of steel moment-resisting frames, focusing on the highest ductility class. Two typical frames of 3 and 7 stories were selected as benchmark structures. After creating numerical models for the frames in OpenSees, analyses based on the Lateral Force Method and the nonlinear static pushover method were carried out. The overstrength factors, column utilisation ratios, and stability coefficients of the two frames were evaluated based on both the current and revised Eurocode 8. The results showed that the updated code provisions lead to higher overstrength factors and hence higher column design loads. Accordingly, the bending and lateral torsional buckling utilisation ratios were up to 80% higher with the revised version of the code, suggesting that it may be more demanding to satisfy the column design requirements. The stability coefficients, on the other hand, were reduced to approximately half the corresponding values obtained with the current code provisions. This implies that the P-Δ effects are not expected to govern the design when the revised code is adopted, at least for the ductility class and behaviour factor level considered in this paper.

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