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COMPARATIVE ANALYSIS OF REINFORCED CONCRETE FRAMES ACCORDING TO FIRST AND SECOND GENERATION EUROCODE 8

Summary: A comparative analysis was conducted on reinforced concrete frame structures designed according to the first and second generations of Eurocode 8, focusing on the DCM and the DC3 ductility classes. The results show similar global behaviour and identical seismic forces, but differ in required ductility and in the amount of longitudinal and transverse reinforcement in critical regions, particularly in columns. Interstorey drift is checked for the Damage Limitation limit state in the first generation of Eurocode 8 and for the Damage Limitation and Significant Damage limit states in the second generation. Compared with DCM frames, DC3 frames exhibit a lower interstorey drift sensitivity coefficient, stricter local ductility requirements, and higher behaviour factors from pushover analysis. The DC3 ductility class requires more rigorous design and detailing rules, which may complicate its application in practice.

Keywords: Eurocode 8, ductility, seismic design, reinforced concrete frames, pushover

KOMPARATIVNA ANALIZA RAMOVSKE KONSTRUKCIJE PREMA PRVOJ I DRUGOJ GENERACIJI EUROKODA 8

Rezime: U radu je izvršena komparativna analiza ramovske konstrukcije projektovane prema prvoj i drugoj generaciji Eurokoda 8, sa fokusom na klase duktilnosti DCM i DC3. Rezultati pokazuju slično globalno ponašanje i identične seizmičke sile, dok su razlike izražene u obezbjeđenju zahtijevane duktilnosti i količini podužne i poprečne armature u kritičnim oblastima, posebno u stubovima. Međuspratni drift se prema prvoj generaciji Eurokoda 8 provjerava u graničnom stanju ograničenja oštećenja, a prema drugoj generaciji u graničnim stanjima ograničenja oštećenja i značajnog oštećenja. U odnosu na DCM ramove, DC3 ramovi imaju manji koeficijent osjetljivosti na međuspratni drift, strožije zahtjeve za lokalnu duktilnost i veće vrijednosti faktora ponašanja dobijene pushover analizom. Primjena klase duktilnosti DC3 zahtijeva rigoroznija pravila dimenzionisanja i obrade detalja, što njenu primjenu može učiniti složenijom u projektantskoj praksi.

Ključne reči: Eurokod 8, duktilnost, seizmičko projektovanje, AB ramovi, pushover

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1. INTRODUCTION

Continuous progress in the seismic design of reinforced concrete structures, based on theoretical, experimental, and numerical research, has driven the revision of Eurocode 8 and the development of its second generation. To assess the impact of these changes, this paper presents a comparative analysis of two ductility classes defined within Eurocode 8: the medium ductility class (DCM) of the first generation [1] and ductility class DC3 according to the provisions of the second generation [2] and [3]. These ductility classes were selected for comparison because the same behaviour factor values are prescribed for the corresponding structural systems, so similar global seismic behaviour can be expected. Particular attention is given to the design procedures and to the changes introduced in the new generation, which are most pronounced in satisfying the local ductility requirements of elements.

One of the objectives of this paper is to assess whether the rules of the new generation lead to more efficient use of material, considered through the required amount of reinforcement. To capture the nonlinear response of the structure, nonlinear static (pushover) analyses were performed for both frames, based on which the plastic hinge formation mechanism and the actual behaviour factor were analysed.

2. STRUCTURE DESCRIPTION AND ANALYSIS METHODOLOGY

Two reinforced concrete frames of identical geometry were designed. Both frames have six storeys and two bays. The beam and column cross – sections remain constant over the height. The geometry, loading, and all other input data used in the analysis are given below:

- the building belongs to importance class II,
- frame span (axis to axis): l = 5 m,
- storey height: h = 3 m,
- beam cross – section: b/h = 40/50 cm,
- column cross – section: b/h = 50/50 cm,
- additional permanent load: Δg = 25 kN/m,
- variable load: q = 10 kN/m,
- ground type: C,
- reference peak ground acceleration: $a_{gR} = 0.375g$,
- spectrum type 1,
- ductility class: DCM / DC3
- relative viscous damping: ξ = 5 %,
- concrete class: C 25/30,
- reinforcement steel class: B400C

This choice of geometry, i.e., the identical configuration of the two frames, makes it possible to highlight the similarities and differences between the two ductility classes considered, thereby excluding all external influences other than those resulting from the provisions of the first and second generation Eurocodes themselves.

The gravity loads considered include the self – weight of the structural elements, as well as the additional permanent and variable loads, applied as line loads with the values given above. The flexural and shear stiffness of the cross – sections were taken as 50 % of the total stiffness of the uncracked concrete section, in accordance with [1]. Since the frames were modelled as planar (2D) models, the floor slab was not explicitly modelled; instead, equal horizontal displacements were assigned to all nodes at the same storey level to approximate the in – plane stiffness effect of the floor diaphragm.

One of the significant changes introduced by the second generation of European standards concerns the way the seismic action is defined. According to the first generation of Eurocode 8 [1], the seismic action is defined using a single parameter – the maximum horizontal ground acceleration a_{gR}. Unlike the previous approach, in the second generation [2] the elastic response spectrum for a given return period is defined using two parameters: the reference spectral acceleration $S_{\alpha,ref}$, which describes the acceleration level at the spectral "plateau", and $S_{\beta,ref}$, which corresponds to the spectral acceleration for a vibration period of 1 s.

The procedure used in the second generation is not presented here. It is important to note that the authors of this paper chose to define the seismic action using the procedures of the first generation of Eurocode 8 [1]. The main reason for this choice was to ensure that all input data for both reinforced concrete frames were identical, so that any differences in the seismic response of the frames would result solely from the differences in the design and detailing provisions of the different Eurocode generations.

After defining all the required input data and frame characteristics, the analysis was carried out. Linear static analysis, as well as modal response spectrum analysis, were performed using the software package CSI ETABS [4]. Based on the resulting internal forces and load combinations, the frames were designed according to the provisions of Eurocode 2 [5] [6] and Eurocode 8 [1] [2] [3], of the first and second generation, respectively. Both frames were designed following the capacity design principles, so that plastic hinges are expected to form at the beam ends, in accordance with the strong column – weak beam principle [7].

After completing the design and processing the results of the linear analysis, a nonlinear static (pushover) analysis was performed using the software package SeismoStruct (Seismosoft) [8]. The analysis methodology and its results are presented in detail in the remainder of the paper.

3. DESIGN CRITERIA

This chapter compares selected design and detailing rules for reinforced concrete beams and columns according to the first generation of Eurocode 8 ([1], DCM ductility class) and the second generation of Eurocode 8 ([2] and [3], DC3 ductility class), presenting only the criteria for which a difference exists between the two ductility classes considered.

3.1. BEHAVIOUR FACTOR

The behaviour factor for the DCM frame is q = 3.9. For the bare moment – resisting frames considered in this study, the behaviour factor is calculated according to the modified procedure introduced in the second generation of the Eurocode standards, [2] and [3], as expressed by the following equation:

$$q = q_R q_S q_D \tag{1}$$

where:

- the overstrength factor q_R for multi – storey, multi – bay buildings is taken as 1.3,
- the component of the behaviour factor q_S related to overstrength from other sources is taken as 1.5,
- the component of the behaviour factor q_D related to the deformation and energy – dissipation capacity is taken as 2.0.

Finally, the behaviour factor for the DC3 frame, according to the second generation [3], is:

$$q = 1.3 \cdot 1.5 \cdot 2.0 = 3.9 \tag{2}$$

The second generation of the Eurocode standards [3] specifies a lower value, q=2.3, for DC3 frames with interacting masonry infills.

3.2. LOCAL DUCTILITY REQUIREMENT FOR BEAMS

A comparison of the design and detailing criteria for beams for the DCM and DC3 ductility classes is presented in Table 1.

Table 1: Beam design criteria

Parameter	DCM	DC3
Design shear force based on the capacity design method	$V_{Ed,max,1} = 1.0 \cdot \frac{M1^{-} + M2^{+}}{l_{cl}} + V_{(g+0.3 \cdot p)}$	$V_{Ed,max,1} = 1.15 \cdot \frac{M1^{-} + M2^{+}}{l_{cl}} + V_{(g+0.3 \cdot p)}$
Maximum bar diameter through interior joints	$\frac{d_{bL,int}}{h_c} \leq \frac{7.5 \cdot f_{ctm}}{1.0 \cdot f_{yd}} \cdot \frac{1 + 0.8 \cdot \nu_d}{1 + 0.75 \cdot (2/3) \cdot \rho_{'}/\rho_{max}}$	$\frac{d_{bL,int}}{h_c} \leq \frac{7.5 \cdot f_{ctm}}{1.2 \cdot f_{yd}} \cdot \frac{1 + 0.8 \cdot \nu_d}{1 + 0.75 \cdot (2/3) \cdot \rho_{'l}/\rho_{l,max}}$
Maximum bar diameter through exterior joints	$\frac{d_{bL}}{h_c} \leq \frac{7.5 \cdot f_{ctm}}{1.0 \cdot fyd} \cdot (1 + 0.8 \cdot \nu_d)$	$\frac{d_{bL}}{h_c} \leq \frac{7.5 \cdot f_{ctm}}{1.2 \cdot fyd} \cdot (1 + 0.8 \cdot \nu_d)$
Maximum longitudinal reinforcement ratio	$\rho_{max} = \rho_{'} + \frac{0.0018}{\mu_{\varphi} \cdot \varepsilon_{sy,d}} \cdot \frac{f_{cd}}{f_{yd}}$	$\rho_{max} = \rho_{'1} + 0.5\ \%\ (C25\ and\ B400)$
Beam shear resistance without shear reinforcement	$V_{Rd,c} = [C_{Rd,c} \cdot k \cdot (100 \cdot \rho 1 \cdot fck)^{1/3} + k_1 \cdot \sigma_{cp}] \cdot b_w \cdot d$	$\tau_{Rd,c} = \frac{0.66}{\gamma_V} \cdot (100 \cdot \rho 1 \cdot fck \cdot d_{dg}/d)^{1/3} \geq \tau_{Rdc,min}$

For the DC3 ductility class, the amplification factor adopted in the shear design, $\gamma_{Rd} = 1.15$, is higher than the corresponding value for the DCM ductility class, $\gamma_{Rd} = 1.0$, which directly leads to increased design shear forces.

Likewise, the factor $\gamma_{Rd} = 1.2$, used in the calculation of reinforcement anchorage, is higher than the corresponding value for the DCM ductility class, $\gamma_{Rd} = 1.0$, which allows smaller bar diameters to be adopted through the beam – column joints.

For both ductility classes, the maximum tension reinforcement ratio consists of the compression reinforcement ratio increased by an additional term. For the DC3 class, this additional term reduces to a fixed value, depending on the steel and concrete class, which is about 0.2 % lower than the corresponding value for the DCM class, simplifying the calculation while retaining a conservative approach.

The first generation [5] formulation directly expresses the shear resistance without shear reinforcement as a shear force, $V_{Rd,c}$ using the coefficients $C_{Rd,c}$, k, v_{min} and $k_1\sigma_{cp}$ with the requirement $V_{Rd,c} \geq V_{Rd,c,min}$. The second generation [4] formulation initially expresses the shear resistance as a shear stress, $\tau_{Rd,c}$ using the coefficient $0.66/\gamma_V$, the size dependent factor $(d_{dg}/d)^{1/3}$ and the lower bound stress resistance $\tau_{Rd,c,min}$. The parameter d_{dg} represents the roughness of the failure zone and depends on the type of concrete and the properties of the aggregate. For the seismic design considered in this study, $d_{dg} = 16$ mm was adopted in accordance with the second generation provisions [2]. In this study, the beam shear resistance without shear reinforcement obtained from the second generation expression was approximately 30 % lower than that obtained from the first generation expression.

3.3. LOCAL DUCTILITY REQUIREMENT FOR COLUMNS

A comparison of the design and detailing criteria for columns for the DCM and DC3 ductility classes is presented in Table 2.

Table 2: Column design criteria

Parameter	DCM	DC3
Maximum normalized axial force	$\nu_d = \frac{N_{Ed}}{A_c \cdot f_{cd}} < 0.65$	$N_{Ed,i} = N_{Ed,G,i} + \Omega \cdot N_{Ed,E,i}$; $0.10 < \nu_d < 0.55$
Length of the critical column region	$l_{cr} = max(h_c; \frac{l_{cl}}{6}; 45cm)$	$l_{cr} = max(b_{max}; \frac{l_{cl}}{6}; 0.45m)$
Check of the mechanical volumetric ratio of transverse reinforcement	$\omega_{wd} \geq \frac{1}{\alpha} \cdot [30 \cdot \mu_{\varphi} \cdot \nu_d \cdot \varepsilon_{sy,d} \cdot \frac{b_c}{b_0} - 0.035]$	

For the DCM ductility class, the limitation of the normalized axial force applies directly to the value of N_{Ed} obtained from the linear (elastic) analysis. For the DC3 ductility class, the axial force $N_{Ed,i}$ is still determined from the internal forces of the linear analysis – the gravity component $N_{Ed,G,i}$ and the seismic component $N_{Ed,E,i}$ – except that the seismic component is amplified by the coefficient $\Omega = 2.0$, resulting in a higher design axial compressive force compared to the DCM ductility class. In addition, for the DC3 class, besides the stricter upper limit $\nu_d < 0.55$, a lower limit on the normalized axial force $\nu_d > 0.10$ is also prescribed, which does not exist for the DCM ductility class. The length of the critical column region for the DC3 class is determined based on the larger of the two cross – sectional dimensions b_{max}, instead of the dimension in the direction considered, h_c used for the DCM class. Finally, while the first generation of Eurocode 8 [1] requires checking the mechanical volumetric ratio of transverse reinforcement at the column base, the second generation [3] does not require this check for the DC3 ductility class, other than satisfying the minimum value.

For DCM frames, no dedicated analytical check is prescribed for beam – column joints; the transverse reinforcement adopted at the column ends is simply continued through the joint [1]. On another side, for DC3 frames, joint verification is required, following one of the two approaches defined in [3].

3.4. CHORD ROTATION

According to [3], sufficient ductility and plastic rotation capacity must be ensured in all critical regions of beams and columns. This requirement is expressed through the product of the behaviour factor components q_R and q_D, and the chord rotation at yield, θ_y, at each end of the element, which must not exceed the chord rotation at the significant damage (SD) limit state, θ_{SD}.

$$q_D \cdot q_R \cdot \theta_y \leq \theta_{SD} \tag{3}$$

The chord rotation at yield, θ_y, for rectangular beams and columns, can be determined using expression (4), as proposed in [2].

$$\theta_y = \varphi_y \cdot \frac{L_V + a_l}{3} + \frac{\varphi_y \cdot d_{bL} \cdot f_y}{8 \cdot \sqrt{fc}} + 0.0019 \cdot (1 + \frac{h}{1.6 \cdot L_V}) \tag{4}$$

In this paper, the beam shear span L_V was calculated for each seismic load combination. For columns, the shear span L_V is taken as 50 % of the clear height, in accordance with [9].

The parameter φ_y represents the yield curvature, obtained from a cross – section analysis at the element end using the software XTRACT [10]. The tension – shift term due to the slope of the bending moment diagram a_l is defined in the second generation of Eurocode 2 [6] using the same expression as in the first generation [5]. The parameters f_y and f_c represent the mean values of the reinforcement yield strength and the concrete compressive strength, respectively.

For the verification of the SD limit state, the resistance of ductile mechanisms is determined using expression (5), as defined in [2].

$$\theta_{SD} = \frac{1}{\gamma_{Rd}} \cdot (\theta_y + \alpha_{SD,\theta} \cdot \theta_u{}^{pl}) \tag{5}$$

The parameter $\alpha_{SD,\theta}$ represents the fraction of the plastic part of the ultimate deformation ($\theta_{u,pl} = \theta_u - \theta_y$) corresponding to the attainment of the SD limit state, while γ_{Rd} is the partial resistance factor for that limit state. In the current version [2], the reference value for $\alpha_{SD,\theta}$ is taken as 0.5. In this paper, the coefficient γ_{Rd} was adopted as 1.35, in accordance with [11].

4. RESULTS OF THE LINEAR ANALYSIS

The results obtained from the analyses performed are presented and discussed below, with reference to the influence of the parameters considered on the structural behaviour.

Table 3 presents the total gravity and total seismic shear force values for the DCM and DC3 frames,, along with the maximum values of the interstorey drift ratios and sensitivity coefficients:

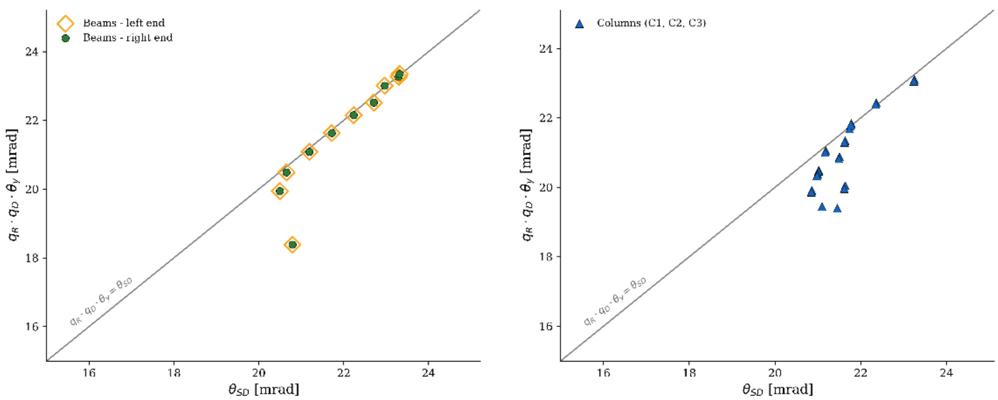
Table 3: Results of the linear analyses

	P_{tot} [kN]	V_{tot} [kN]	$d_{r,DL}$ / h_s	$d_{r,SD}$ / h_s	max Θ
DCM	2215.0	334.4	0.00768	n/a	0.088
DC3			0.00920	0.01535	0.045

Note: n/a = not applicable

Unlike the first generation of Eurocode 8 [1], which primarily controlled interstorey drift ratios through the damage limitation (DL) verification associated with a return period of 95 years, the second generation [3] introduces separate DL and SD interstorey drift ratio checks associated with return periods of 115 and 475 years, respectively. The maximum interstorey drift ratios obtained for DCM and DC3 frames are reported in Table 3. The DCM frame satisfies the DL drift check requirement of the first generation [1], while the DC3 frame satisfies the SD drift check requirement of the second generation [3]. For the bare DC3 frame, the DL drift limit from the first generation [1] was adopted for comparison. In addition, the maximum values of the sensitivity coefficient are lower for the DC3 frame than for the DCM frame. This is mainly because, according to [3], the parameters q_R and q_S appear in the denominator of the sensitivity coefficient expression. Consequently, assuming the same interstorey drift, gravity loads, and total seismic shear force, the sensitivity coefficient is lower for the DC3 frame.

The chord rotation capacity was verified for the left and right ends of all 12 beams, for the seismic force acting from right to left and from left to right, as well as for the column sections C1, C2, and C3, in accordance with Equation (3).



(a) *(b)*

Figure 2: Chord rotation capacity verification: (a) beams; (b) columns

The results are shown in Figure 2. All the elements analysed, beams and columns, satisfy the specified condition, with a negligible exceedance of up to 1 %.

The quantities of concrete and the adopted reinforcement for beams and columns, together with the percentage differences, are given in Table 4.

Table 4: Material quantities and percentage differences

	Concrete		Reinforcement							
	Beams	Columns	Beams				Columns			
			Longitudinal		Transverse		Longitudinal		Transverse	
	m^3	m^3	kg	kg/m^3	kg	kg/m^3	kg	kg/m^3	kg	kg/m^3
DCM	10.8	13.5	1068.5	98.9	271.1	25.1	1293.7	95.8	298.3	22.1
DC3	10.8	13.5	1068.5	98.9	405.6	37.6	1496.1	110.8	462.7	34.3
Increase in percentage (%)			0	0	49.6	49.6	15.6	15.6	55.1	55.1

Table 4 shows that the amount of longitudinal beam reinforcement for the DC3 ductility class remains the same as the corresponding reinforcement of the DCM frame. For the columns of the DC3 frame, an increase in longitudinal reinforcement of about 16 % is observed. The higher longitudinal column reinforcement results from the factor Ω, which, according to [3], multiplies the axial forces in the seismic design combination, leading to higher axial tensile forces and, consequently, a greater required amount of design reinforcement.

In addition to the longitudinal reinforcement, an increase of 50 % to 55 % in the transverse (shear) reinforcement is observed for both beams and columns of the DC3 frame. This is due to the need to verify the chord rotation according to the second generation of Eurocode 8, [2] and [3], which represents a stricter requirement than the check of the mechanical volumetric ratio of transverse reinforcement of the first generation of Eurocode 8 [1].

5. PUSHOVER ANALYSIS

Modern structural analysis software can perform nonlinear analysis relatively efficiently, taking into account the requirements of current codes. The pushover, or nonlinear static, analysis is carried out by loading the structure with monotonically increasing horizontal forces under constant gravity load, up to the target displacement. The target displacement represents an estimate of the maximum roof displacement of the building under the design earthquake. It is determined from the elastic response spectrum (EC8 – Annex B, [1]) or in accordance with the SeismoStruct 2025 program [8].

The main goal of the analysis is to obtain the "capacity curve", i.e., the relationship between the base shear force and the roof displacement.

According to the first generation of Eurocode 8 [1], at least two lateral force distributions must be applied, while according to the second generation of Eurocode 8 [2], it is necessary to apply at least the modal distribution. In this paper, two distributions were applied, namely:

- the "uniform distribution", in which the lateral forces are proportional to the masses regardless of their height. This simulates the inertial forces in a potential storey mechanism.
- the "modal distribution", in which the lateral forces are distributed according to the shape of the first vibration mode of the structure, or according to a linear triangular distribution over the height of the building. This simulates the inertial forces that produce a beam mechanism under nonlinear behaviour.

For further calculation, the distribution that yields the less favourable results should be adopted as governing, i.e., the one giving the higher target displacement and the lower overstrength factor.

In the structure considered, the results given in Tables 5 and 6 show that the modal distribution produces larger target displacements for both ductility classes, while the uniform distribution yields the lower overstrength factor. For this reason, the results for both lateral load distributions are presented.

Table 5: Target displacement and overstrength factor for DCM

Parameter	Uniform distribution	Modal distribution
Target displacement d_t	0.22 m	0.26 m
Overstrength factor α_u/α_1	1.27	1.36

Table 6: Target displacement and overstrength factor for DC3

Parameter	Uniform distribution	Modal distribution
Target displacement d_t	0.23 m	0.26 m
Overstrength factor α_u/α_1	1.31	1.39

5.1. ASSESSMENT OF THE BEHAVIOUR FACTOR COMPONENTS

The assessment of the behaviour factor and its components (q_R, q_S and q_D) is based on the mathematical model proposed by Plumier [12], also adopted in [2]. The analysis starts from the capacity curve of the structure, on which four characteristic points are identified:

- (V_e, d_e) – the elastic base shear force obtained from the design spectrum reduced by the behaviour factor, with the corresponding roof displacement;
- (V_1, d_1) – the base shear force and displacement at the formation of the first plastic hinge in the structure;
- (V_u, d_y) – the effective yield point of the idealized bilinear curve, obtained from the condition of equal elastic stiffness (K_e=V_e/d_e); and
- (V_u, d_r) – the base shear force and displacement at the formation of the global plastic mechanism.

The displacement at the formation of the global mechanism (d_r) is determined according to one of two criteria [13]:

- if a global plastic mechanism forms, d_r is read directly at that point;
- if a global plastic mechanism does not form, the displacement at which the strength drops by 20 % relative to the maximum value is adopted.

The behaviour factor and its components q_R, q_S and q_D are determined according to the expressions given in Table 7.

Table 7: Behaviour factor components

Component of behaviour factor	Expression
q_R	$q_R = \frac{V_u}{V_1}$
q_S	$q_S = \frac{V_1}{V_e}$
q_D	$q_D = \frac{d_r}{d_y}$
d_y	$d_y = \frac{V_u \cdot d_e}{V_e}$

By applying the expressions given in Table 7, the behaviour factors are obtained for the uniform and modal distributions, for the DCM and DC3 ductility class frames (Tables 8 and 9).

Table 8: Nonlinear analysis parameters for the uniform distribution

Ductility class	**V_e (kN)**	**V_1 (kN)**	**V_y (kN)**	**V_u (kN)**	**d_e (m)**	**d_y (m)**	**d_r (m)**	**q**
DCM	334.4	437.9	556.3	556.3	0.052	0.086	0.285	5.51
DC3		447.3	585.4	585.4	0.052	0.091	0.300	5.79

Table 9: Nonlinear analysis parameters for the modal distribution

Ductility class	**V_e (kN)**	**V_1 (kN)**	**V_y (kN)**	**V_u (kN)**	**d_e (m)**	**d_y (m)**	**d_r (m)**	**q**
DCM	334.4	350.4	475.3	475.3	0.052	0.074	0.300	5.78
DC3		356.2	495.9	495.9	0.052	0.077	0.360	6.95

6. CONCLUSIONS

The aim of this paper was to compare the design provisions and requirements for different ductility classes according to the first and second generation of Eurocode 8. The main conclusions of the linear and nonlinear analyses are summarized below. The conclusions of the linear analysis are as follows:

- both frames exhibit similar global behaviour and identical seismic design forces, since the same behaviour factor (q = 3.9) was obtained for both the DCM and DC3 ductility classes;
- the required longitudinal beam reinforcement remained the same, while the longitudinal column reinforcement of the DC3 frame increased by 16 %, as a consequence of the factor Ω, which amplifies the axial forces in the seismic design combination;
- the transverse (shear) reinforcement of the DC3 frame increased by 50 % to 55 % for both beams and columns compared to the DCM frame, since the chord rotation verification for the DC3 ductility class proved to be a stricter criterion than the check of the mechanical volumetric ratio of transverse reinforcement for the DCM ductility class;
- the sensitivity coefficients of the DC3 frame are lower than the corresponding coefficients of the DCM frame, confirming the lower sensitivity of the DC3 ductility class to interstorey drift effects;

The conclusions of the nonlinear analysis are as follows:

- the actual behaviour factors obtained from the pushover analysis range from 5.51 to 6.95, considerably above the design value of q = 3.9, with the DC3 frame consistently showing a higher behaviour factor than the DCM frame (5.79 versus 5.51 for the uniform distribution, and 6.95 versus 5.78 for the modal distribution), confirming that the stricter local ductility detailing rules result in genuinely greater structural ductility;
- the modal lateral load distribution in the pushover analysis yields larger target displacements (0.26 m versus 0.22 – 0.23 m), but also higher overstrength factors α_u/α_1 (1.36 – 1.39 versus 1.27 – 1.31) compared to the uniform distribution;
- the design value of the behaviour factor, q=3.9, was found to be conservative for the investigated frame configurations, but this conclusion should not be generalised to all reinforced concrete moment – resisting frames.

In summary, compared with the first generation, the second generation of Eurocode 8 provides a more detailed design framework that may allow a more precise assessment of structural response, but it requires additional checks and is therefore not necessarily simpler for the designer.

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