LOCAL AND GLOBAL DUCTILITY OF WIDE-BEAM RC FRAMES

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Abstract

Italian and Spanish seismic codes prescribe lower behaviour factor (*q*) for wide-beam reinforced concrete moment resisting frames (WBF) with respect to conventional *q* factors adopted for deep-beam frames (DBF). Conversely, other relevant seismic codes worldwide, such as Eurocode 8, consider WBF capable of high ductility performances, provided that design rules related to (i) stress transfer in connections, (ii) lateral stiffness and (iii) energy dissipation are complied. On the other hand, local ductility of wide beams (WB) appears to be systematically lower than that for deep beams (DB). A parametric comparative numerical analysis of deformations of DB and WB, following the current approach, shows that WB have larger ultimate chord rotation but lower ductility in terms of chord rotation than DB despite the similar ductility in curvature, which is mainly due to a lower plastic hinge lengths in WB. Aimed at verifying whether such disadvantage is overcome by modern codes or not, several archetype European RC residential buildings are designed alternatively with DB or WB. Seismic performances are assessed with different degree of detail. Results suggest that WBF provide at least similar global seismic capacities than DBF, especially in frames whose design is ruled by damage limitation limit state. Hence, any reduction of *q* in Mediterranean codes for WBF appears to be at least obsolete.

Keywords: Wide beams, deep beams, chord rotation ductility, behaviour factor, seismic codes.

1. Introduction

Eurocode 8 [1] and other seismic codes worldwide do not make any explicit difference between wide-beam RC frames (WBF) and conventional deep-beam frames (DBF) as lateral load-resisting system except for specific prescriptions on beam-column connections. However, some codes of the Mediterranean area (the Italian NTC [2] and the Spanish NCSE-02 [3]), in which WBF are quite common as typology [4], prescribe lower behaviour factors (q) for WBF with respect to the value prescribed for DBF.

Experimental and analytical studies [5-11], mainly focused on beam-column connections, show that WBF and DBF have similar local performances if modern code's provisions on local geometry are fulfilled, especially limitation on maximum beam width. Other studies [12] suggest that limitations on beam width and prescriptions related to the stiffness of the building (damage limitation and second-order effects) can lead to similar performances of WBF and DBF in case of High Ductility Class (DCH). Notwithstanding the availability of valuable studies on the topic, the limitations prescribed by Mediterranean codes on WBF's q still suggest a lack of systematic analytical comparison between global seismic capacities of WBF and DBF.

Herein, the drawbacks of WBF with respect to DBF are discussed in the light of modern codes' provisions. Firstly, local performances of wide beams (WB) are compared to those of deep beams (DB) in a parametric study considering 16 Eurocode-compliant design and assessment scenarios for WB and DB, respectively. Then, seismic performances of a typical European 5-storey RC building designed alternatively with DB and WB according to EC8 and NCSE-02 are comparatively assessed through nonlinear static analyses, assuming different hypotheses of fulfilment of damage limitation limit state (DLS). Finally, simplified parametric assessment of a set of 72 frames designed to EC8, NTC and NCSE-02, adopting similar q for WBF and DBF, is conducted.

2. Code provisions on wide-beam frames

Historically, more restrictive provisions (typically hazard restrictions or reduction of q have been proposed for WBF with respect to DBF, similarly to flat-slab structures. Still, some current codes reduce q of WBF with respect to DBF. In particular, the Italian NTC applies a reduction to 2/3 of q for WBF, downgrading them to Medium Ductility Class (DCM), typically resulting in q=3.9. Spanish NCSE-02 applies a reduction to 1/2 of q, downgrading WBF to Low Ductility Class structures, resulting in q=2. However, Eurocode 8 (EC8 in the following) and most current codes only impose geometric and mechanical limitations to wide-beam column connections, mostly regarding the restriction of beam width (b_w), and no restriction or reduction of q is suggested for WBF. In [12,13] a fulfilling overview of worldwide code prescriptions for WBF is provided and the experimental and analytical results underpinning code provisions are analyzed.

A preliminary comparison of WBF and DBF shows five potential disadvantages in WBF when compared to DBF: (i) lower lateral stiffness; (ii) deficient stress transfer in beam-column connections; (iii) higher stress demand in joint panels; (iv) poorer energy dissipation in connections; and (v) poorer local ductility of beams.

Wide beams (WB) show lower cross-sectional stiffness than deep beams (DB). However, as long as codes provide interstorey drift (IDR) limitations for DLS design (NCSE-02 does not) and second order corrections, the stiffness of WBF has to be rather similar to that of DBF because larger columns sections have to be implemented. Regarding wide beam-column stress transfer, the equilibrium of the fraction of the beam section outside the column core requires sufficient transverse torsional behaviour and proper bond in longitudinal reinforcement bars; otherwise, full capacity cannot be attained. However, modern codes limit b_w in order to make it agree with the effective width, i.e. the fraction of the total beam width which satisfies flexural equilibrium of forces when framing a narrower column through strut-and-tie load paths (see Fig. 1a and b); thus proper stress transfer is guaranteed regardless of the properties of other elements. Moreover, joint strut design is not an issue anymore since seismic codes specify the geometry of effective joint panel, which may extend laterally to both sides thus expanding the compressed area (see Fig. 1c).

With respect to cyclic energy dissipation, some experimental tests [6,11] suggest that hysteretic behaviour is systematically poorer for WBF rather than for DBF due to deficient bond of reinforcement in columns and

beams, leading to approximately 9% increase of displacement demand if equivalent viscous damping is considered. However, those tests show values of b_w larger than the requirements of EC8. Conversely, in a EC8-compliant sub-assemblage [14], cyclic behaviour with wide beams cannot be considered poorer. In [8,9] it is suggested that the higher initial cracking in WB rather than in DB, due to gravity loads, leads to higher pinching, but very high column diameter is used, thus results may not be considered fully conclusive. Hence, modern code's provisions should overcome the potential issue related to poorer hysteretic behaviour.
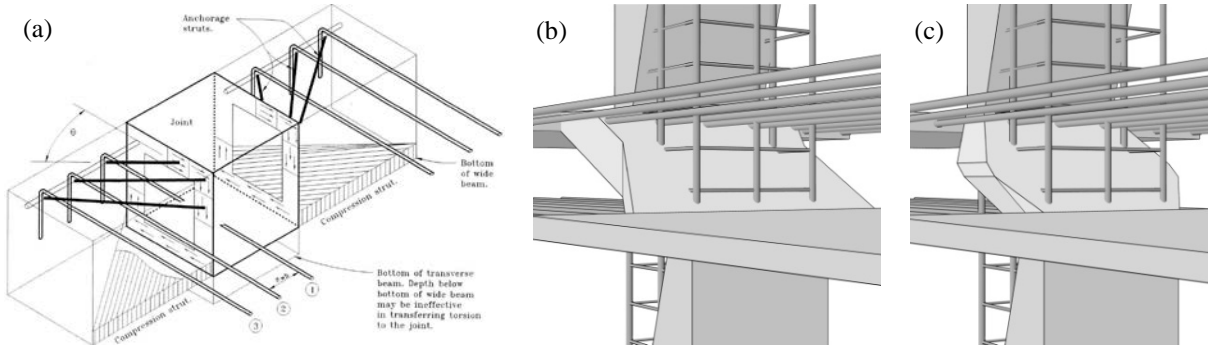

Fig. 1 –Strut-and-tie models for stress transfer in wide beam-column connections, with struts independent from the joint pannel (a, from [5]) or integrated within the pannel (b); and joint effective shear width (c, from [12])

3. Local ductility of wide and deep beams

The poorer local ductility of WB with respect to DB is herein investigated through a parametric study. Inelastic flexural deformation of members is typically characterised by chord rotation θ at yielding and ultimate (θ_y and θ_u, respectively). Within lumped plasticity framework, macroscopic value θ is related to local variables, i.e. cross-section curvatures at yielding and ultimate (ϕ_y and ϕ_u, respectively), through shear span (L_V) and plastic hinge length (L_{pl}). In the following, b_w and h_b are cross sectional width and height, respectively; c_n, concrete covering; d, effective depth; d', distance from extreme fibres to the barycentre of reinforcement; z, internal lever arm; x, neutral axis depth; d_{bL} and d_{bt}, mean diameter of longitudinal and transverse bars; A_{s1} and A_{s2} the tensioned and compressed reinforcement area; ω, ω' and ω_{tot}, bottom, top and total mechanical reinforcement ratio; ρ_w, transverse reinforcement ratio; f_c, resistance of concrete; f_y and f_u, yielding and maximum steel tension; and, finally, M_y and M_u, yielding and ultimate bending moment. Any parameter A, ratios between values corresponding to WB and DB are indicated as $A_{W/D}$ (rather than using the heavier notation A_{WB}/A_{DB}).

In general, elastic stiffness of WB is lower than for DB due to $h_{b,W/D}\leq1$, although $b_{w,W/D}\geq1$; thus, also post-cracked deformability may be expected to be higher for WB rather than for DB. In terms of curvature ductility (μ_ϕ), traditionally WB are considered to provide lower values than DB [15], i.e. $\phi_{u,W/B}\leq\phi_{y,WB}$. Such statements are based on generic considerations: when h_b is reduced, higher A_{s1} is required, thus large compressed concrete area is needed in order to satisfy equilibrium, which, sometimes, can be only attained by means of higher x, likely causing higher ϕ_y and lower ϕ_u, thus lower μ_ϕ. However, such statements do not take into account that $b_{w,W/D}$ can be quite large and, almost all, that sections designed in High Ductility Class (DCH) perform as confined ones. On the other hand, it cannot be found any explicit and systematic comparative analysis in literature for WB and DB regarding chord rotations and ductility in terms of chord rotation ($\mu_{\theta,W/D}$).

Preliminary considerations aimed at estimating $\mu_{\phi,W/D}$ and $\mu_{\theta,W/D}$ for design in DCH are carried out. Two generic beams (one DB and one WB, $h_{b,W/D}\leq1$) with similar M_y, ω'/ω and L_V are considered. M_y is approximately proportional to d, given that similar z are expected both for DB and WB because $b_{w,W/D}\geq1$. Considering that $M_y=A_{s1}\cdot z$, then $A_{s1,W/D}\approx1/d_{W/D}$.

Regarding curvatures, it can be assumed that ϕ_y is attained at yielding of tensioned reinforcement, and distribution of tensions in concrete can be considered almost triangular, thus $\sigma_c=E_{sec}'\cdot\varepsilon_c$, being σ_c and ε_c the concrete maximum stress and strain (at top fibre in positive bending) and E_{sec}' the equivalent secant Young modulus corresponding to triangular distribution. At a first step, it is assumed that $\omega'=0$, and x is considered

negligible compared to ratios of d. Based on geometric compatibility, the approximated expression for $\phi_{y,W/D}$, for both cases (confined and unconfined) is provided in Eq. (1) depending only on relative geometry between WB and DB. The last expression can be considered as representative also when $\omega'\neq0$, because top reinforcement may be subjected stresses quite lower than f_y for amounts of ω_{tot} corresponding to design in DCH.

For the evaluation of $\phi_{u,W/D}$, similar reasoning can be made. In this case, constant concrete stress distribution (i.e. rectangular stress block) is considered. If confinement is not taken into account, ultimate state in the section may correspond to failure of concrete in compression, and consequently $\phi_{u,W/D,unconf}$ is obtained as in Eq. (2) and corresponding ductility $\mu_{\phi,W/D,unconf}$ is expressed as in Eq. (4). Hence, if their cross-sectional areas are similar, WB and DB are expected to show similar ultimate curvature, and consequently the lack of ductility of WB respect to DB is proportional to their effective depths. However, design in DCH provides a very important confinement of the concrete core, so a different reasoning must be developed. In this case, ultimate state in the section may correspond to excessive deformation of tensioned steel [16]. Therefore, $\phi_{u,W/D,conf}$ can be expressed as in Eq. (3), which is similar to the relationship between yielding curvatures. Hence, rather similar ductilities are expected for WB and DB for confined sections designed in DCH (Eq. (5)), conversely to what is usually found in literature.

$$\phi_{y,W/D} = 1/d_{W/D} \tag{1}$$

$$\phi_{u,W/D,unconf} = b_{W/D} \cdot d_{W/D} \tag{2}$$

$$\phi_{u,W/D,conf} = 1/d_{W/D} \tag{3}$$

$$\mu_{\phi,W/D,unconf} = b_{W/D} \cdot d_{W/D}{}^2 \tag{4}$$

$$\mu_{\phi,W/D,conf} \approx 1 \tag{5}$$

Regarding chord rotations, Eurocode 8 part 3 –EC8-3 in the following— [17] is followed. θ_y expression depends mainly on ϕ_y. For θ_u, two approaches are proposed: one with a more fundamental basis, depending on constant $\phi_{pl}=\phi_u-\phi_y$ alongside L_{pl}, and a pure empirical expression for θ_u. $\theta_{y,W/D}$ can be expressed as in Eq. (6). $\theta_{u,W/D}$ following fundamental approach could be estimated as $L_{pl,W/D}\cdot\phi_{u,W/D}$ (i.e. proportional to ultimate curvatures through plastic hinge length) if L_{pl} is considered as negligible respect to L_V and especially if yielding deformations are assumed to be negligible respect to ultimate ones when compared between WB and DB. Proposed expressions of L_{pl} increase with h_b, thus $L_{pl,W/D}\leq1$. Hence, $\theta_{u,W/D}$ could be approximated as in Eq. (7) and (8) for unconfined and confined cases, respectively. Subsequent ductility ratios are shown in Eq. (10) and (11), which are similar to those corresponding to curvatures.

$$\theta_{y,W/D,EC8} \leq 1/d_{W/D} \tag{6}$$

$$\theta_{u,W/D,EC8\_fun,unconf} \leq b_{W/D} \cdot d_{W/D} \tag{7}$$

$$\theta_{u,W/D,EC8\_fun,conf} \leq 1/d_{W/D} \tag{8}$$

$$\theta_{u,W/D,EC8\_emp} \tag{9}$$

$$\mu_{\theta,W/D,EC8\_fun,unconf} \approx b_{W/D} \cdot d_{W/D}{}^2 \tag{10}$$

$$\mu_{\theta,W/D,EC8\_fun,conf} \approx 1 \tag{11}$$

$$\mu_{\theta,W/D,EC8\_fun,conf} = d_{W/D}{}^{0.65} \tag{12}$$

For pure empirical approach, ratios of θ_u can be expressed as in Eq. (9), thus always higher for WB rather than DB regardless on b_w. Similar confinement contribution is considered for WB and DB, although it may be higher for WB when design in DCH and EC2 provisions are considered [12]. Subsequent ductility ratio is obtained in

Eq. (12). Hence, for confined sections, lower $\theta_{u,W/D}$ and $\mu_{\theta,W/D}$ are expected according to experimental approach rather than fundamental one. It is possible to obtain equivalent implicit values of $L_{pl,eq,W/D} \approx \theta_{u,W/D}/\phi_{u,W/D}$; WB may show shorter $L_{pl,eq}$ than DB for confined sections but higher values in the unconfined case, which is contrary to the trend observed in most of the expressions proposed for plastic hinge length [15-18].

Hence, within the limitations of these preliminary simplified considerations, in general WB designed in DCH are expected to provide similar curvature ductilities but lower-to-similar chord rotation ductilities rather than DB. Aimed at a proper assessment of those preliminary conclusions, a systematic analysis is required. In this section, the set of 16 WB and 16 DB already used in [12] is adopted aimed at a comparative numerical analysis of deformations. The actual comparison between DB and WB is based on both magnitudes of ϕ and θ, and also the corresponding μ_ϕ and μ_θ are obtained. The characteristics of the set of beams are presented in Table 1, assuming L_V=2.5m, c_n=20mm, d_{bL}=14mm, d_{bt}=8mm, f_c=33MPa and f_y=630MPa. Five parameters are assumed: (i) class (DB or WB); (ii) cross-sectional aspect ratio (h_b/b_w) for each class (types A and B, providing higher or lower M_y, respectively); (iii) ω'/ω=1 or 1.5, which satisfy the requirements of EC8 for DCH; (iv) ω_{tot} (high and low, which makes top and bottom reinforcement, respectively, correspond to code's upper and lower limit when ω'/ω=1.5); and (v) effectiveness of transverse reinforcement on confinement (yes or no). DB and WB show similar M_y for each case, and higher reinforcement case provides approximately three times the flexural strength provided by low reinforcement case. Stirrup arrangements satisfy the requirements for DCH of EC8 and also the limitations provided by EC2 [19] regarding the number of transverse legs.

Table 1 – Characteristics of the analysed set of beams (from [12])

Class of beam	Section type (A/B)	b_w [mm]	h_b [mm]	Hoops [mm]	ρ_w [%]	ω_{tot} (Low)		Reinf. ratio [%] (ω'/ω=1.5)	M_y [kNm]	[%] (ω'/ω=1)	M_y [kNm]	ω_{tot} (High)		Reinf. ratio [%] (ω'/ω=1.5)	M_y [kNm]	[%] (ω'/ω=1)	M_y [kNm]
DB	A	300	600	2φ8/70	0.48	0.10	ρ'	0.30	-181	0.25	±152	0.29	ρ'	0.90	-524	0.75	±442
							ρ	0.20	+122				ρ	0.60	+357		
	B	300	500				ρ'	0.30	-124	0.25	±104		ρ'	0.90	-357	0.75	±301
							ρ	0.20	+84				ρ	0.60	+244		
WB	A	650	300	4φ8/70	0.44	0.19	ρ'	0.60	-177	0.50	±149	0.60	ρ'	1.89	-513	1.50	±446
							ρ	0.40	+120				ρ	1.26	+362		
	B	500	300		0.57	0.17	ρ'	0.54	-123	0.45	±103	0.53	ρ'	1.65	-355	1.38	±301
							ρ	0.36	+83				ρ	1.10	+244		

3.1 Curvatures

Full moment-curvature (M-ϕ) relations are obtained through a fibre model for all the cases. Eurocode-based strain-stress models are assumed. For concrete, EC2 parabolic envelope and confinement model proposed in EC8 are adopted. For steel, bilinear envelope with hardening is considered, with f_u/f_y and ultimate stresses ε_{su} according to characteristic values suggested in EC2 for steel type B.

Results for confined cases with asymmetric longitudinal reinforcement are shown in Fig. 2. Post-elastic hardening of reinforcement causes that in most cases $M_u>M_y$. In almost all the cases there is spalling of concrete cover before the attainment of ε_{su} in the tension reinforcement, causing an instantaneous slight drop of M. Hence, ϕ_u of WB reach larger values than DB, as predicted. It is worth noting that the increment of secant-to-yielding stiffness for high-reinforced section respect to low-reinforced ones is rather similar to such increment of total reinforcement. In most cases, simplified assumptions made in the preliminary considerations (pre-emptive yielding of steel, concrete and steel failure in unconfined and confined sections, respectively, quasi-linear behaviour of concrete until ϕ_y, negligible values of x with respect to d, similar top reinforcement stresses at ϕ_y, etc.) are confirmed, and estimated values of ϕ and μ_ϕ are predicted with error lower than 10%. In Fig. 3, one of the couple DB-WB is studied in detail. It corresponds to a case in which WB presents approximately half depth and double width than DB, thus cross-sectional area is rather similar. Results confirm predictions: WB shows double ϕ_y, similar $\phi_{u,unconf}$ and more than double $\phi_{u,unconf}$ than DB.
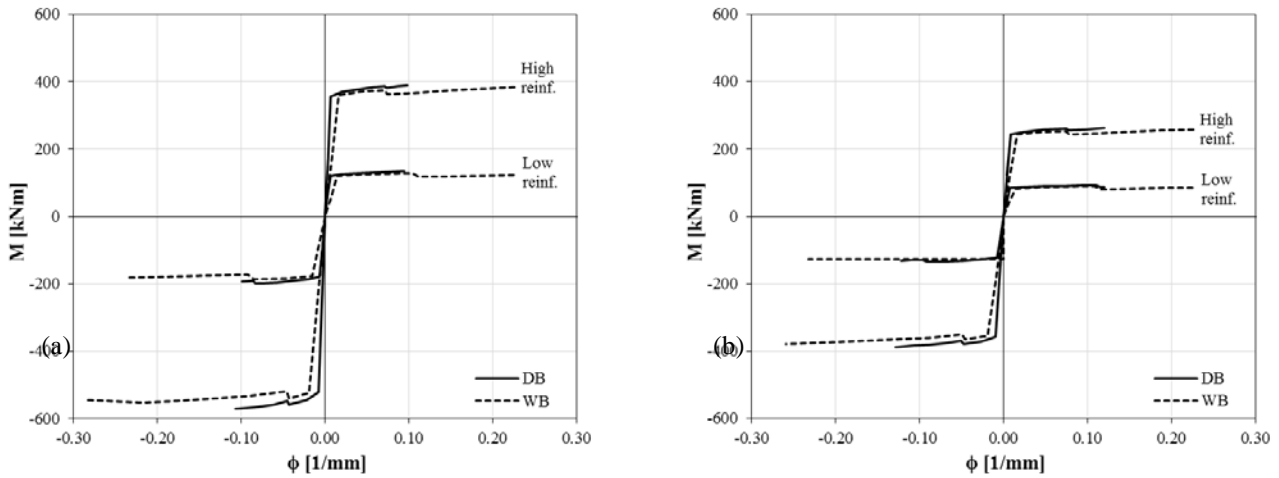
Fig. 2 – M-ϕ relations for the confined cases corresponding to ω'/ω=1.5, for section types A (a) and B (b)
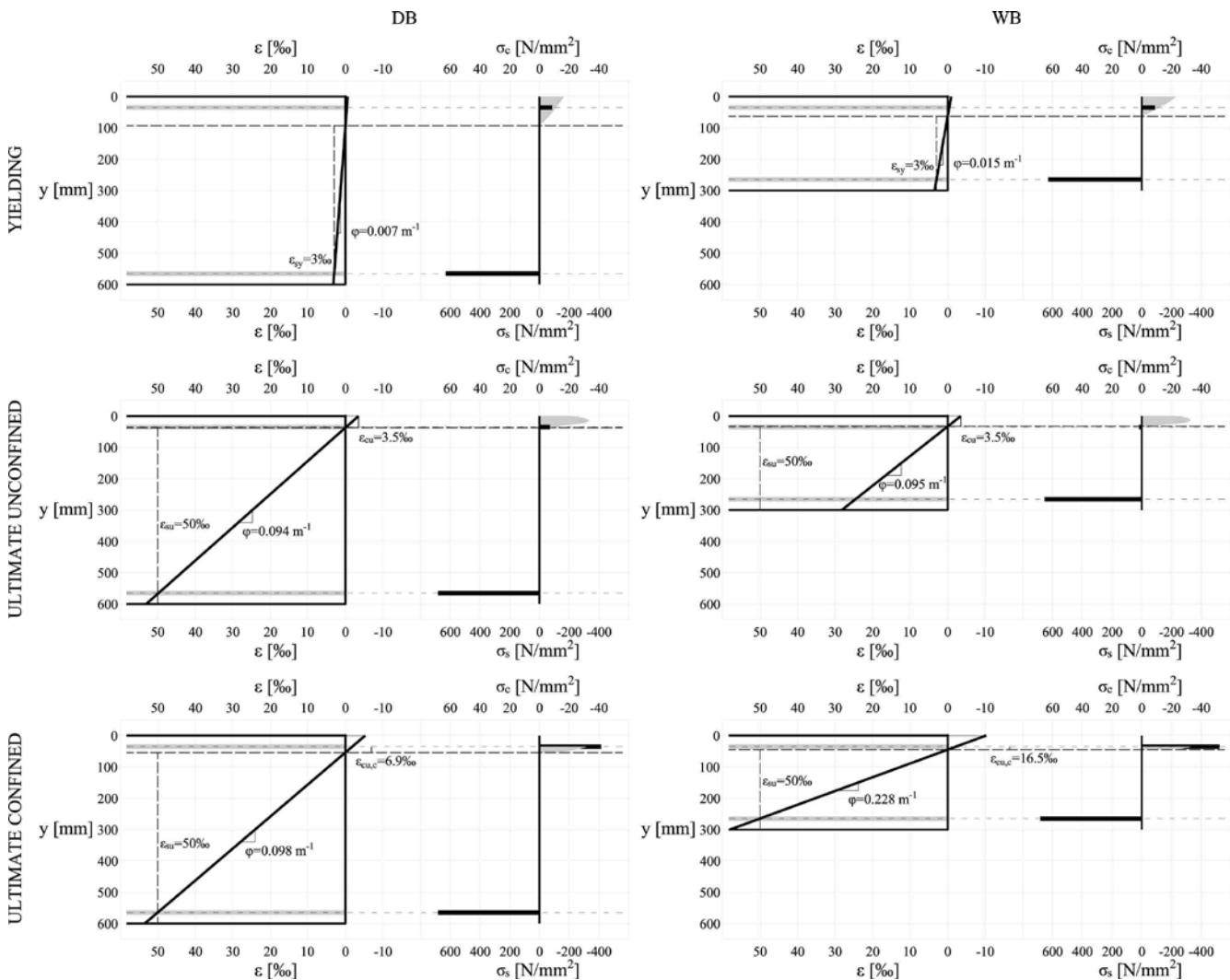

Fig. 3 – Cross-sectional strain plains and stresses for positive flexure of DB and WB type A, high reinforcement and ω'/ω=1

Results for all the cases are shown in Fig. 4a to 4e, in which mean ratios between WB and DB are indicated as W/B at the bottom of each sub-plot. In general, more satisfactory values are obtained for asymmetric reinforced sections in negative bending than in the rest. For unconfined cases, high reinforced sections show much poorer performances in terms of μ_ϕ than low reinforced ones (almost half values) while for confined sections the bias is much lower. It is worth noting that provisions of EC2 regarding distribution of stirrup legs within the width of the section causes quite higher contribution of confinement in WB rather than in DB: in terms of μ_ϕ, DB get multiplied by 1.5 while for WB the factor is almost 3.0. Even in the cases of asymmetric-high-reinforced WB to negative bending, which do not satisfy DCH provisions on longitudinal reinforcement, high confinement causes similar values of μ_ϕ than in the rest of the cases.

3.2 Chord rotations

EC8-3 procedures are carried out for all the cases. In Fig. 4f, θ_y has similar values than ϕ_y, although $\theta_{y,W/D}$ is 15% lower than $\phi_{y,W/D}$ on average due to the different shear contribution at yielding, independent from curvature. Mean secant-to-yielding stiffness are on average 9% and 23% of the uncracked gross stiffness for low- and high-reinforced DB, respectively, and 15% and 38% for WB, respectively, due to the higher reinforcement ratios in WB rather than in DB. Mean global value is 21%, coherent with [20].

In Fig. 4g and 4h, θ_u for unconfined and confined cases, respectively, obtained following the EC8-3 fundamental approach, are shown. L_{pl} of WB is 0.86 times that of DB, on average, which is exactly the ratio between mean values of $\theta_{u,W/D}/\phi_{u,W/D}$ for confined beams (see Fig. 4h and 4c); however, for unconfined beams, still larger θ_u for WB rather than for DB are shown notwithstanding the lower L_{pl} for WB, because in this case large yielding deformations are not negligible respect to ultimate ones. Consequently, μ_θ is 40% lower for WB rather than for DB for unconfined section, while similar ductility is expected for confined beams (see Fig. 4i and 4j, respectively).

Fig. 4k and 4l correspond to θ_u for unconfined and confined cases, respectively, obtained following the EC8-3 empirical approach. The relative positive influence of confinement on WB is quite lower than for the fundamental approach: the mean increment of θ_u is only 16% instead of 125%. For unconfined beams, notwithstanding the similar ϕ_u for WB and DB, higher values of θ_u are observed for WB rather than for DB; in fact, implicit equivalent plastic hinge length are 32% higher for WB, on average. Instead, for confined cases, mean $L_{pl,eq,W/D}=0.62$, which is more consequent with explicit values within the fundamental approach. Such lower influence of confinement on WB causes that, even on confinement beams, μ_θ is 25% lower for WB rather than for DB (see Fig. 4m and 4n).

4. Compared seismic assessment of deep- and wide-beam frames

Hence, the strongest reason for any *q* reduction on WBF may be the lower local ductility of WB with respect to DB. However, the extrapolation from local ductility to *q* can be inappropriate, because *q* refers to overall capacity, and global ductility does not depend on local ductility of beams only. In [12], based on spectral consideration and assuming simplified collapse mechanisms (see Fig. 5a), it is suggested that similar global seismic capacities can be expected for WBF and DBF especially if DLS limitation is the critical situation of design, because rather similar lateral stiffness of both frames is expected. Even when WBF show larger periods rather than DBF, higher rotational capacities of WBF with respect to DBF could lead to similar performances.

In order to evaluate whether a reduction of *q* could be justified or not in modern seismic codes, seismic performances of WBF and DBF are comparatively assessed through nonlinear static analyses [13]. Firstly, a typical European 5-storey RC housing unit is designed as WBF and DBF according to Eurocode 8 and NCSE-02. Current *q* values are considered: 5.85 or 4.68 for EC8 (depending on the regularity in elevation), and 4.0 or 2.0 for NCSE-02 (for DBF and WBF, respectively). In order to cover a wide range of design choices, two versions of design according to EC8 are considered, depending on the members' stiffness for DLS fulfilment: in "EC8$_{50-50}$", both design stiffness at DLS and Ultimate Limit State (ULS) are assumed as 50% of the gross uncracked one, while in "EC8$_{100-50}$", 100% of uncracked stiffness is employed for DLS, and 50% for ULS.
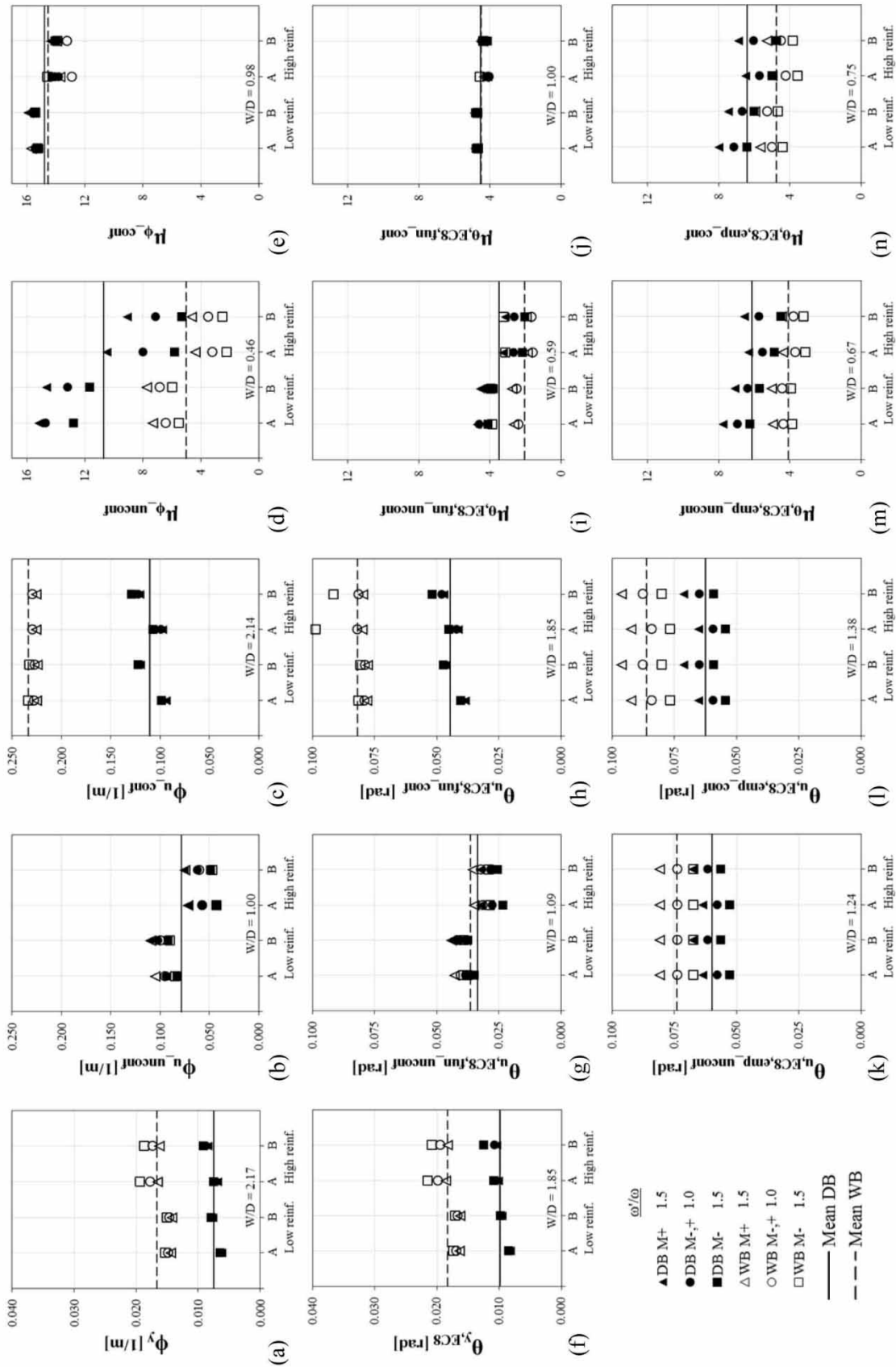
Fig. 4 – ϕ_y (a), ϕ_u for unconfined (b) and confined (c) beams, and μ_ϕ for unconfined (d) and confined (e) beams; θ_y (f), θ_u for unconfined (g) and confined (h) beams, and μ_θ for unconfined (i) and confined (j) beams, following EC8-3 fundamental approach; θ_u for unconfined (k) and confined (l) beams, and μ_θ for unconfined (m) and confined (n) beams, following EC8-3 empirical approach.
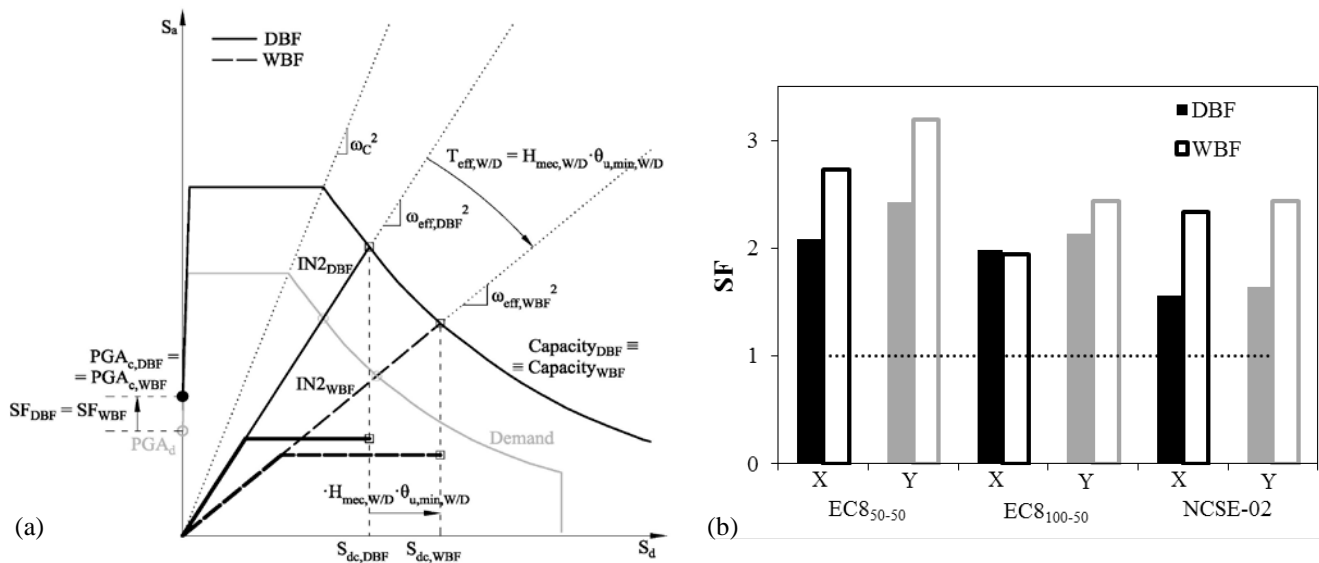
Fig. 5 – Acceleration-displacement response spectrum condition for equal performances of WBF and DBF (a, from [12]) and global $SF=PGA_c/PGA_d$ in each case (b, from [13])

DLS design is the critical condition in $EC8_{50-50}$ buildings. Especially for WBF, very large columns are required in order to compensate the lower stiffness of beams, causing important cantilever behaviour and very high storey shear overstrengths and column-to-beam capacity design ratios. In $EC8_{50-50}$ buildings, huge section dimensions of first and second storey columns may constitute a great shortcoming regarding architectural functionality. In $EC8_{100-50}$ buildings, the design to DLS is not so relevant, especially in DBF. NCSE-02 buildings are mainly force-based, so smaller sections and higher reinforcement ratios can be observed also in WBF design to DCL. Seismic capacities of all the models are assessed by means of nonlinear static analysis and N2 spectral method. The height involved in the mechanism (H_{mec}) depends mainly on column-to-beam capacity design ratios, which are higher for WBF than DBF, especially for $EC8_{50-50}$. Even when capacity design ratios are quite similar for both WBF and DBF (e.g., NCSE-02), a difference of one or two storeys favourable to WBF is observed.

In general, inelastic incursion at ULS demand is limited for all the buildings (see Fig 6), especially in NCSE-02 WBF (see Fig 6f), which remains in equivalent elastic field. WBF show similar or even greater global seismic capacities with respect to DBF in all the cases (Fig. 5b): *SF* of WBF are 31%, 6% and 49% higher than those for DBF on average, in the case of $EC8_{50-50}$, $EC8_{100-50}$ and NCSE-02, respectively. The causes of such good performances of WBF in comparison with DBF are: (i) higher H_{mec} (Fig. 6); (ii) higher $\theta_{u,min}$, due to lower h_b and higher L_{Vc} (Table 2); (iii) in EC8 buildings, sufficient stiffness of WBF (Fig. 5), and (iv) in NCSE-02 buildings, higher base shear due to lower design *q*. Such range of increase for *SF* of WBF with respect to that of DBF, in EC8 structures, may balance any possible rise of displacement demand due to poorer cyclic behaviour, which has shown to be likely limited for code-compliant structures.

In order to evaluate whether the previous results, favourable to WBF (even when similar *q* to that of DBF are adopted) could be generalised to RC-MRF residential building stock, a higher set of case studies is evaluated. A parametric assessment of 72 planar frames is performed, corresponding to 12 couples of WBF and DBF with different geometry and designed to low and high seismicity complying three different codes: EC8, NTC and NCSE-02. In each code, *q* corresponding to DCH is assumed also for WBF. Different values are adopted for parameters: number of storeys (*n*): 3, 6 and 9; spans (*L*): 3.5 and 5.5m, i.e., a representative range for residential buildings in Europe [4]; and design acceleration equal to 0.12g and 0.25g. The assessment of relative performances between WBF and DBF is carried out by means of the simplified approach proposed in [12].

Results of relative performances between WBF and DBF ($SF_{W/D}$), obtained without accounting for favourable influence of H_{mec} in WBF, are presented in Fig. 7. EC8 and NTC show mean values of $SF_{W/D}$ favourable to WBF (1.08 and 1.02, respectively), while for NCSE-02 mean performance is poorer for WBF than

for DBF (mean 0.91). In 83% of the EC8 buildings, WBF show better performance than DBF. For NTC the ratio decreases until 50%. Conversely, every single couple designed to NCSE-02 show $SF_{W/D}<1.0$. The cause of the satisfactory performance of WBF in EC8 and NTC (even without any consideration of $H_{mec,W/D}$) is that they often show sufficient stiffness, and whenever it is lower than the corresponding stiffness of DBF, the difference is so small that it gets largely overcome by the rest of the beneficial contributions to performances, which may also balance the possible decrease of capacity in WBF due to poorer hysteretic behaviour.
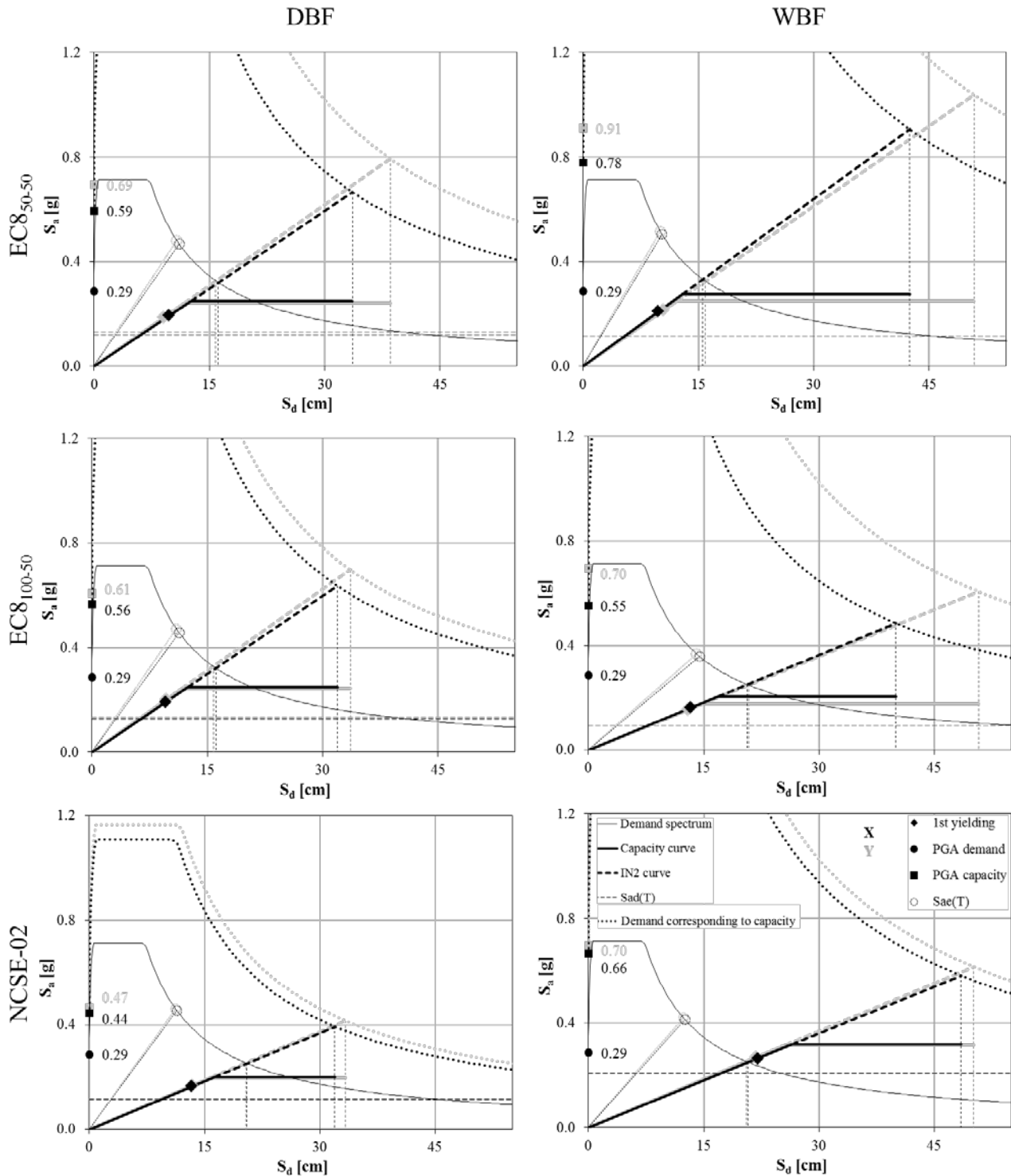

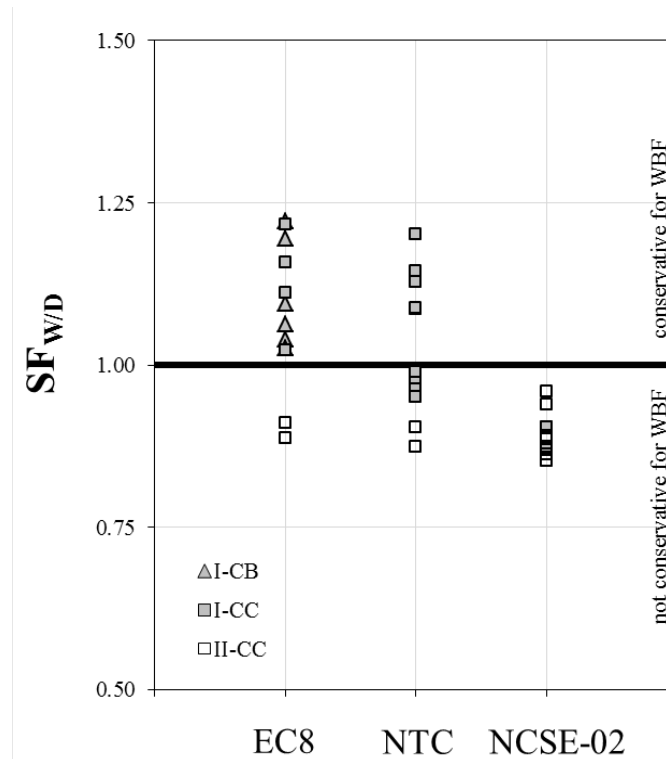Fig. 6 – N2 ULS spectral performance and maximum capacity of each model (from [13])

Fig. 7 – Estimated relative performance between WBF and DBF ($SF_{W/D}$) for each code (from [13])

5. Conclusions

A comparative numerical analysis of curvatures and chord rotations of a parametric set of 16 wide beams and 16 deep beams, with and without effective confinement, is carried out. Preliminary considerations confirm that cross-sectional aspect ratio plays a fundamental role in the local ductility of beams, and parametric results show that confined wide beams present larger ultimate chord rotation but lower chord rotation ductility than deep beams despite the similar curvature ductilities, due to lower plastic hinge lengths in wide beams.

Aimed at verifying whether such disadvantage is overcome by modern codes or not, several seismic performances of representative European RC residential buildings designed alternatively with deep beams or wide beams are assessed with different degree of detail. Results show that global seismic capacity of wide-beam frames get substantially improved thanks to different effects increasing both their effective stiffness and their maximum deformation capacity: higher cantilever behaviour, higher ultimate chord rotation at column bases and beams ends, lower shear deformability of joints; beam-to-column width limitation makes it hard to reduce column sections at upper storeys, and both design to Damage Limitation State and corrections due to second order effects lead to greater column sections in the mid-low part of the building, causing higher column overstrength.

Therefore, high-ductility wide-beam frames may provide similar or even better performances with respect to deep-beam frames when Damage State Limitation is among design criteria. Hence, any reduction of behaviour factor for wide-beam frames appears to be at least obsolete.

5. References

[1] CEN (2004). *Eurocode 8: design of structures for earthquake resistance – Part 1: general rules, seismic actions and rules for buildings*. European Standard EN 1998-1:2003 – Comité Européen de Normalisation, Brussels, Belgium.

[2] CS.LL.PP (2009). Instructions for the application of the Technique Code for the Constructions. *Official Gazette of the Italian Republic*, 47 (in Italian)

[3] CDSC (2002). *Seismic construction code, NCSE-02*. Committee for the Development of Seismic Codes, Spanish Ministry of Construction, Madrid, Spain (in Spanish)

[4] De Luca, F., Verderame, G.M., Gómez-Martínez, F., Pérez-García, A. (2014). The structural role played by masonry infills on RC building performances after the 2011 Lorca, Spain, earthquake. *Bulletin of Earthquake Engineering* **12**(5):1999-2026

[5] Gentry, T.R., Wight, J.K. (1992). Reinforced concrete wide beam-column connections under earthquake-type loading. *Report nº UMCEE 92-12*. Department of Civil and Environmental Engineering, University of Michigan, Ann Arbor, Michigan, USA

[6] LaFave, J.M., Wight, J.K. (1997). Behavior of reinforced exterior wide beam-column-slab connections subjected to lateral earthquake loading. *Report nº UMCEE 97-01*. Department of Civil and Environmental Engineering, University of Michigan, Ann Arbor, Michigan, USA

[7] Quintero-Febres, C.G., Wight, J.K. (1997). Investigation on the seismic behavior of RC interior wide beam-column connections. *Report nº UMCEE 97-15*. Department of Civil and Environmental Engineering, University of Michigan, Ann Arbor, Michigan, USA

[8] Benavent-Climent, A., Cahís, X., Zahran, R. (2009). Exterior wide beam-column connections in existing RC frames subjected to lateral earthquake loads. *Engineering Structures* **31**:1414-1424

[9] Benavent-Climent, A., Cahís, X., Vico, J.M. (2010). Interior wide beam-column connections in existing RC frames subjected to lateral earthquake loading. *Bulletin of Earthquake Engineering* **8**:401-420

[10] Li, B., Kulkarni, S.A. (2010). Seismic behavior of reinforced concrete exterior wide beam-column joints. *Journal or Structural Engineering (ASCE)* **136**(1):26-36

[11] Fadwa, I., Ali, T.A., Nazih, E., Sara, M. (2014). Reinforced concrete wide and conventional beam-column connections subjected to lateral load. *Engineering Structures* **76**:34-48

[12] Gómez-Martínez, F., Alonso-Durá, A., De Luca, F., Verderame, G.M. (2016). Ductility of wide-beam RC frames as lateral resisting system. *Bulletin of Earthquake Engineering* **14**(6):1545-1569

[13] Gómez-Martínez, F., Alonso-Durá, A., De Luca, F., Verderame, G.M. (2016). Seismic performances and behaviour factor of wide-beam and deep-beam RC frames. *Engineering Structures* **125**:107-123

[14] Masi, A., Santarsiero, G. (2013). Seismic tests on RC building exterior joints with wide beams. *Advanced Materials Research* **787**:771-777

[15] Paulay, T., Priestley, M.J.N. (1992). *Seismic design of concrete and masonry structures*. John Wiley and Sons, New York, USA

[16] Fardis, M.N. (2009). *Seismic Design, Assessment and Retrofitting of Concrete Buildings*. Ed. Springer, London, UK

[17] CEN (2005). *Eurocode 8: design of structures for earthquake resistance – Part 3: assessment and retrofitting of building*s. European Standard EN 1998-1:2005 – Comité Européen de Normalisation, Brussels, Belgium

[18] Biskinis, D.E., Fardis, M.N. (2010). Flexure-controlled ultimate deformations of members with continuous or lap-spliced bars. *Structural Concrete* **11**(2):93-108

[19] BSI (2004). *Eurocode 2: Design of concrete structures: Part 1-1: General rules and rules for buildings*. British Standards Institutions, London, UK

[20] Panagiotakos, T.B., Fardis, M.N. (2001). Deformations of reinforced concrete members at yielding and ultimate. ACI *Structural Journal* **98**(2):135-148 and Appendix 1 (69 pp.)