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. https://doi.org/10.1016/j.engstruct.2016.06.034

SEISMIC PERFORMANCES AND BEHAVIOUR FACTOR OF WIDE-BEAM AND DEEP-BEAM RC FRAMES

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ABSTRACT

Reinforced Concrete Wide-Beam Frames (WBF) are a common architectural solution in Mediterranean countries. On this structural typology there is not yet a uniform approach among European codes: while Eurocode 8, as other relevant seismic codes in USA and New Zealand, considers WBF capable of high ductility performances, still in recent versions of Spanish and Italian seismic codes there is cap to the maximum behaviour factor (*q*) for this structural system. In order to verify the appropriateness of such provisions, seismic performances of WBF and conventional deep beam frames (DBF) are comparatively assessed through nonlinear static analyses. The same architectural layout of a typical Mediterranean 5-storey RC housing unit is designed according to Eurocode 8, adopting different stiffness assumptions, and according to the Spanish seismic code NCSE-02. Based on detailed assessment results, a simplified parametric assessment of 72 frames designed according to Eurocode 8, Italian seismic code NTC and NCSE-02 is then considered assuming similar *q* for WBF and DBF. Results suggest that any reduction of behaviour factor prescribed for wide-beam frames is at least obsolete. In fact, even if wide beams show lower local ductility than deep beams, generally WBF provide at least similar global seismic capacities than DBF, especially in frames whose design is ruled by serviceability limit state (i.e., damage limitation).

KEYWORDS: Wide beams, deep beams, seismic codes, behaviour factor, chord rotation, ductility, effective period, collapse mechanism

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NOMENCLATURE

DB	Deep beams
DBF	Deep-beam reinforced concrete frames
DCH	High ductility class
DCL	Low ductility class
DCM	Medium ductility class
DLS	Damage limitation limit state
IDR	Interstorey drift ratio
ULS	Ultimate limit state
WB	Wide beams
WBF	Wide-beam reinforced concrete frames
a_g	Peak ground acceleration in soil type A
a_{gR}	Reference peak ground acceleration in soil type A
b_b	Beam gross section width
b_c	Column width
b_w	Beam web width
$C_{P\text{-}\Delta}$	Amplification factor accounting for P-Δ effects
C_s	Spectral acceleration capacity
d_{bi}	Maximum beam bar diameter passing through the joint
d_{bo}	Maximum beam bar diameter passing outside the joint
d_c	Maximum column bar diameter
D_u	Top displacement capacity
e	Beam-column eccentricity
E_cI_c	Member stiffness
f_{ck}	Concrete characteristic compressive strength
f_{conf}	Confinement contribution to θ_u
$f_{K,sec}$	Ratio between the stiffness degradation of connections in WBF with respect to DBF
f_{yk}	Steel characteristic yield strength
H	Building height
h_b	Beam depth
h_c	Column depth
h_f	Upper slab tension flange thickness
H_{mec}	Height of the building involved in the collapse mechanism
i	Number of the storey
K_{eff}/	Effective stiffness
K_{el}	Elastic stiffness
L	Member length
L_V	Shear span
M_{Rb}	Moment of resistance at beam end
M_{Rc}	Moment of resistance at column end
n	Number of storeys
PGA_c	Capacity peak ground acceleration
PGA_d	Demand peak ground acceleration
q	Behaviour factor
R_D	Spectral contribution to q
R_S	Structural overstrength
R_α	Structural overstrength from first yielding to global mechanism
R_μ	Ductility strength reduction factor
R_ω	Structural overstrength until first yielding
S	Soil amplification factor
$S_a(T_{eff})$	Effective spectral acceleration demand
$S_{ae}(T)$	Elastic spectral acceleration of design
$S_{ae}(T)$’	Equivalent elastic spectral acceleration of design
S_{du}	Maximum spectral displacement capacity
S_{dy}	Yielding spectral displacement
SF	Structure global safety factor (capacity/demand)
$T_{100\%EI}$	Design period for gross uncracked member stiffness
$T_{50\%EI}$	Design period for member stiffness 50% of the gross uncracked one
T_{code}	Simplified code design period
T_{eff}	Effective period
T_{el}	Elastic period
V_d	Storey shear demand
V_R	Storey shear strength
w	Outer cantilevered beam width respect to narrower column core
Γ	First mode participation factor
ΔK	Relative interstorey difference of stiffness
Δm	Relative interstorey difference of mass
θ_u	Ultimate chord rotation
$\theta_{u,min}$	Minimum θ_u between members involved in the collapse mechanism
θ_{ULS}	Maximum chord rotation attaining ULS
θ_y	Yielding chord rotation
λ	Normalised first mode participating mass
μ_θ	Chord rotation ductility
ν	Normalised axial load
ρ	Bottom longitudinal reinforcement ratio
ρ’	Top longitudinal reinforcement ratio
ρ_{tot}	Total longitudinal reinforcement ratio
ρ_w	Transverse reinforcement ratio

1. INTRODUCTION

Traditionally, seismic codes have been quite cautious in allowing the use of wide-beam reinforced concrete frames (WBF) as the only lateral resisting system of buildings [1][2][3][4][5][6][7][8][9][10] [11][12]. Conversely, more recent seismic codes do not make any explicit difference between WBF and conventional deep-beam frames (DBF) with the exception of some requirements on beam-column connections.

Still, some national seismic codes of the Mediterranean area, such us the Italian NTC [13] and the Spanish NCSE-02 [14], do not consider WBF as a system that can be designed in High Ductility Class (DCH). Thus, they prescribe lower behaviour factors (q, also called "strength reduction factor") for WBF with respect to DBF. On the contrary, Eurocode 8 part 1 [15] (EC8 in the following) does not prescribe any limitation to the behaviour factor of reinforced concrete (RC) WBF.

Reasons for limiting q in Mediterranean codes are not explicitly stated. Experimental and analytical background suggests that WBF may present some drawbacks when compared to DBF: (i) deficient stress transfer within connections, (ii) lower lateral stiffness and (iii) poorer energy dissipation of beams. However, recent literature studies [10][12] provide evidence that design provisions in modern seismic codes may overcome such deficiencies, directly or indirectly. Literature evidence on WBF is mainly based on experimental and analytical studies focusing on local structural behaviour [1][2][3][4][5][6][7][9][16][17][18][19]. Still, there is a lack of systematic studies addressing global performances of WBF against equivalent DBF fulfilling the requirements of different codes. Herein, a comparison of seismic assessment of both structural types is carried out. The final aim is to verify whether the whole framework of modern performance-based codes can balance the disadvantages of WBF with respect to DBF, and in which local context (if any) a reduction of q can be justified.

Diverse analytical studies regarding relative performances of WBF compared with DBF [1][3] show very similar performances for both types. However, these studies cannot be yet defined neither systematic nor generalizable. Firstly, they have been carried out within the American framework of codes and construction practice. In [1], planar frames are assessed, not buildings; and lower interstorey heights are used for WBF. In [3], the tested buildings have wide beams (WB) in the internal frames, deep beams (DB) in the external ones, and intermediate shear walls; thus, the collapse mechanism is not ruled by WB, making any comparison unfeasible.

Moreover, both works use chord rotation values obtained from mix lumped plasticity and fibre models matching with their own experimental results, but not fitted to any larger database in accordance to the common approach employed in the last ten years among the scientific community, and adopted by recent codes. Some other analytical studies, corresponding to the Spanish framework, have been carried out [20][21][22][23]. Unfortunately, the last three works only focus on WBF, while, in the first study, WBF and DBF are designed to different *q* values, thus preventing any comparison for DCH.

Hence, the scope herein is to provide a systematic and generalizable analytical comparison of WBF and DBF performances. The latter is carried out through nonlinear static analyses of a 5-storey building model designed alternatively with WB and DB, according to both EC8 and Spanish NCSE-02. The comparison is made for different design hypotheses and evaluating the consequences of the design assumptions on the nonlinear performances. Finally, simplified assessment of a parametric set of 72 frames representing residential buildings in Europe, corresponding to different codes (EC8, NTC and NCSE-02) is carried out in order to extrapolate and generalise the results obtained for the specific case study. Large-span WBF, as those typical in Australia and described in [7] or [16], are out of the scope of this paper.

2. CODE PROVISIONS ON WIDE-BEAM FRAMES

Due to historic uncertainties about the seismic performance of WBF, more restrictive provisions have been proposed for WBF with respect to DBF, such as limitations to their use in high seismicity areas, or reduction of the behaviour factor (*q*). The same restrictions are often referred also to flat-slab structures, to which seismic behaviour of WBF used to be assimilated. However, the vast majority of current codes only impose geometric and mechanical limitations to wide beam-column connections as a condition for the application of standard design procedures, in order to ensure proper stress transfer and the consequent exploitation of the full capacity of elements.

However, some national codes in the Mediterranean area, where the use of WBF is more widespread [20][21][22][23][24][25], still prevent WBF to be designed in High Ductility Class (DCH). Italian NTC reduces *q* of WBF to 2/3 of that provided for DBF, downgrading them to Medium Ductility Class (DCM). Spanish NCSE-02 reduces *q* to 1/2, downgrading WBF to Low Ductility Class (DCL).

Design provisions for both ultimate (ULS) and damage limitation limit states (DLS) regarding WBF and flat-slab structures corresponding to different Mediterranean codes –Italian NTC, Spanish NCSE-02, Turkish TSI [26] and Greek EAK 2000 [27]— together with other benchmark international codes such as EC8, American ACI 318-08 [28], or New Zealander NZS 3101 [29] are summarized in Table 1 and Fig. 1. For both structural systems, all the codes set maximum interstorey drift ratio (IDR) with the exception of NCSE-02. In the case of WBF, codes regulate local design of connections, especially concerning the restriction of beam width (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. Only Australian AS3600 [30] does not impose any restriction to beam width. Other provisions (e.g. amount of top reinforcement to be placed within column core, maximum effective width of upper slab flange and that of joint panel) underpin the same basic principles, while some geometrical restrictions are oriented to ensure adequate bond behaviour of the longitudinal reinforcement.

Code provisions have a counterpart in experimental and analytical studies showing different disadvantages of WBF with respect to DBF. The main issues related to WBF as the principal horizontal load carrying systems can be grouped as follows [12]:

1) Lower elastic lateral stiffness: WB have lower depth with respect to DB, when similar geometric and mechanical properties are assumed for the rest of the structure. It results in more significant non-structural damage and second order effects.
2) Deficient stress transfer in beam-column connection [1][2][3][4][5][6][7][9][16][19]: the equilibrium of the fraction of the beam section passing outside the column core ("outer" part) requires sufficient transverse torsional behaviour and proper bond in longitudinal reinforcement bars; otherwise, full capacity cannot be attained.
3) Higher stress demand in joint panels: lower depth of WB with respect to DB often causes higher compression in the diagonal struts within joint panels.
4) Poorer local ductility of beams [1][21]: the increment of ultimate chord rotation (θ_u) in WB with respect to DB is not as high as the increment of yielding chord rotation (θ_y), because plastic hinge length is lower for WB rather than for DB.
5) Poorer energy dissipation of connections [2][5][6][9]: deficient bond in reinforcement bars passing through the connection in WBF leads to higher "pinching" of hysteretic cycles.

Table 1: Prescriptions regarding flat-slab and wide-beam frames systems according to different codes (from [12])

CODE (seismic, RC, connections)	Max. IDR[(1)] [%]	BEAMLESS TWO-WAY FLAT SLAB: Max. a_g [g]	BEAMLESS TWO-WAY FLAT SLAB: Deforma-bility restric-tions	BEAMLESS TWO-WAY FLAT SLAB: Max. duct. class (q; q reduction from DCH)	WIDE BEAMS: Min. h_b [cm]	WIDE BEAMS: Max. duct. class (q; q reduction from DCH)	WIDE BEAMS: Min. h_c [cm]	Max. w (outer part of b_w) for: Member web	Max. w for: Reinf. (both sides)	Max. w for: Edge beam b_b from max. e	Max. w for: Joint shear	Max. w for: Upper slab tension flange reinforcement	Max. w for: Beam shear	% upper reinf. within column core
Greece: EAK (2000) [27], EKOS (2000) [31]	1.25	-	Stiffness required (5)	DCH (q=3.5)	Stiffness required (5)	DCH (q=3.5)	25	min{$0.25h_c$; $0.5b_c$}(8,9)	-	$0.66b_c$	-	$h_f\cdot\{0;2;2.5;4\}$ (17)	-(21)	-
New Zealand: NZS 1170.5 (2004) [32], NZS 3101 (2006) [29]	2.5(2)	-	IDR≤ 0.9%	DCL(q=1.25; -79%)	$\approx 27d_c \approx$ 43(6,7)	DCH (q=3.5)	$\approx 30d_{bi}$ ≈48(6,7)	-	$0.25h_c$(9,11)	$0.25h_c$	$0.25h_c$ (16)	min{$L/8$;$8h_f$;h_b; $h_c\cdot\{0.5;0.75\}$(18)}; min{$L/8$;$8h_f$;$3h_b$}(19)	$0.25h_c$ (22)	90%(24)
Spain: NCSE-02 (2002) [14], EHE-08 (2008)[a] [33]	-	-	-	DCL (q=2; -50%)	-	DCL (q=2; -50%)	25-30	-	0.0; $0.5h_b$(12)	$0.5b_c$	-	$h_f\cdot\{0;2;2;4\}$ *(17)	0.0 (23)	-
Italy: NTC (2008) [13]	≈1.3(3)	-	-	DCH (q=5.85)	-	DCM (q=3.9; -33%)	$\approx 36d_{bi}$ ≈55(6,7)	min{$0.5h_b$; $0.5b_c$(8)}(10)	-	$0.5b_c$(14)	$0.25h_c$ (8)	$h_f\cdot\{0;2;0;2\}$(17)	-(21)	75%
Europe: EC8 (2004) [15]	1.0	min{0.08; $0.1/S$}	-	DCL (q=1.5; -74%)[b]	-	DCH (q=5.85)	$\approx 36d_{bi}$ ≈55(6,7)	-	min{$0.5h_b$; $0.5b_c$}a(8,9,13)	$0.5b_c$	$0.25h_c$ (8)	$h_f\cdot\{0;2;2;4\}$(17)	-	-
Turkey: TSI (2007) [26]	2.0	0.20	H≤13m	DCM (q=4; -50%)	min{$3h_f$; 30}	DCH (q=8)	25	$0.5h_b$(8)	-	-	0.0	-	-(21)	-
USA: ASCE/SEI 7-10 (2010) [34], ACI 318-08 (2008) [28], ACI 352R-02 (2002)[a] [35]	1.0-2.5(4)	-	-	DCM (q=4; -38%)	-	DCH (q=8)	$20d_{bo}$ ≈32(7)	min{$0.75h_c$; b_c}(9)	-	-(15)	0.0	min{$L/20-b_w/2$;$8h_f$}; min{$L/8-b_w/2$;$8h_f$; h_c}$\geq 2b_b$ (19) (20)	-(21)	33%

[a] Recommendations, not mandatory
[b] Current version of EC8 does not cover flat slab, 1.5 is the basic assumption for elastic design; new version in progress
(1) For DLS but obtained from ULS displacements
(2) Specific for ULS
(3) Obtained from specific DLS demand spectrum
(4) Depending on a_g and number of storeys
(5) Sufficient stiffness to ensure frame –not cantilever— behaviour in all columns
(6) Formulation depending in most of the cases on ductility class, material strengths, axial load, reinforcement ratios and location of the joint
(7) Considering ϕ_w=16mm
(8) Edge beams not explicitly considered
(9) Not for low-ductility design
(10) Referred to gross section, not to web
(11) Referred to the 90% of the required flexural reinforcement; remaining 10% within (19)
(12) Required transverse beam for external connections or internal connections with moment inversion
(13) Not mandatory, only for taking advantage of the column compression on the bond behaviour
(14) Higher values only if proper perpendicular reinforcement is placed
(15) Further research is needed
(16) Also reciprocal requirement for columns in the case of wide column – narrow beam connection
(17) Exterior connection with and without transverse beam, and analogous for interior connection, respectively
(18) Exterior connection with and without transverse beam, respectively
(19) For beam flexural designing and for overstrength evaluation for column and joint designing, respectively
(20) Torsional evaluation of spandrel beam in external connections required
(21) Maximum b_w limitation may control both flexural and shear behaviour
(22) Uncertain, not explicitly indicated
(23) Value at the column face; $0.5h_b$ at distance of higher than $0.5h_b$ from the column face; intermediate values from linear interpolation
(24) Not column core but joint effective width; strut-and-tie analysis required for lower values

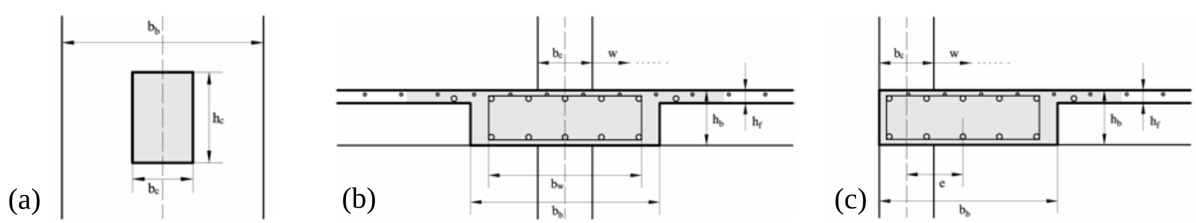

Fig. 1: Graphic description of variables used in Table 1, corresponding to: plan of interior connection (a), and elevation of connection belonging to central (b) and edge (c) frame (from [12])

All the issues, with exception of those related to local ductility, are overcome by specific design provisions in modern seismic codes [12][35]. As long as design to DLS –controlling IDR— and second order (P-Δ) corrections are implemented, the stiffness of WBF must be rather similar to that of DBF by means of the use of larger column sections. Specific design of joints prevents any compression failure of the diagonal strut. Stress transfer in connections is guaranteed if beam width restrictions and other detailing rules are observed (see Table 1). Those provisions may also overcome to a great extent pinching in WBF, according to [35] and also from results observed in [17][19]. Other evidences of decrease of global capacities of WBF with respect to DBF (around 9% [2][9] or higher [5][6]) due to pinching may have lower reliability because poorer local detailing is not consistent with European codes' framework.

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 global capacity, and global ductility does not depend on local ductility of beams only [10][12].

3. CASE STUDY: DESIGN

A case study building is designed to medium-high seismic level according to different codes and modelling assumptions. Then, their respective performances are assessed.

In Fig. 2, a typical Mediterranean 5-storey RC multi-family housing unit, according to [36] and [37], is presented. Design gravitational loads are similar for all the storeys: dead loads (6.2kN/m^2), live loads (2.0kN/m^2), and brick walls dead loads (7, 5 and 3kN/m for exterior, dividing, and parapet walls, respectively). The same NCSE-02 horizontal demand elastic spectrum is chosen for all the cases (Fig. 3).

3.1 Three design alternatives: EC8$_{50\text{-}50}$, EC8$_{100\text{-}50}$ and NCSE-02

Two different seismic codes are considered: EC8 and NCSE-02. The last one imposes a *q* reduction of 50% for WBF, while EC8 does not provide any cap to *q*.

Effective stiffness of WBF plays a very important role in their relative performance; thus, the assumption of certain design stiffness for members is a crucial decision. NCSE-02 does not suggest any reduction of stiffness. EC8 suggests a reduction of 50% both for ULS and Serviceability Limit State (i.e. DLS), while American ASCE/SEI 41-06 [38], up to a 70% for beams and 30-70% for columns; NTC from 0% to 50%; NZS 3101, 60-73% and 0-70% for beams and columns in ULS, respectively, and 0-65% for DLS.

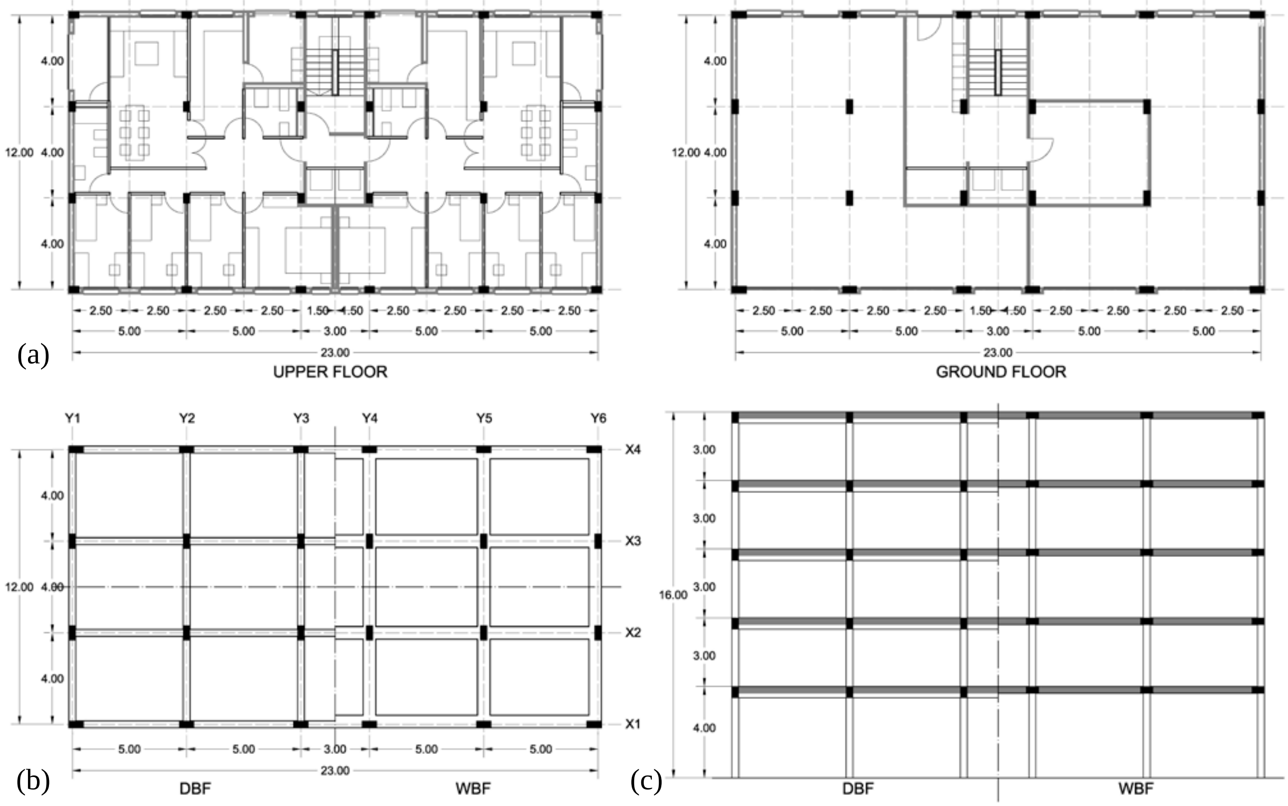

Fig. 2: Case study: distribution (a) and structural arrangement in plan (b), and in elevation (c).

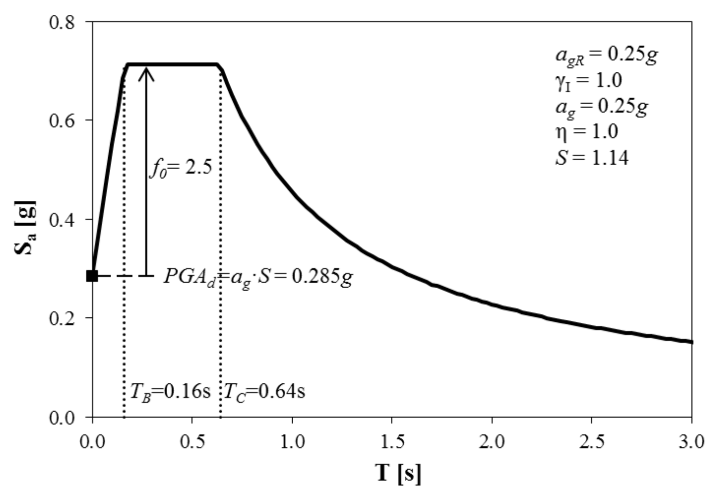

Fig. 3: Elastic horizontal demand acceleration spectrum; parameters follow EC8 terminology

In order to cover a wide range of design choices, two different versions of design according to EC8 are considered. A different assumption is made on elastic members' stiffness for DLS design. In the designed alternative "$EC8_{50\text{-}50}$", both elastic stiffness at DLS and ULS are assumed as 50% of the uncracked one. In "$EC8_{100\text{-}50}$", 100% of uncracked stiffness is employed for DLS, and 50% for ULS. Even if the design hypothesis $EC8_{100\text{-}50}$ can seem not realistic, it well represents an upper bound design version aimed at providing more robust conclusions with regard to personal choices of design. EC8 does not regulate some relevant assumptions regarding stiffness of the elements: contribution of member ends within joint panels, contribution of upper slab or joists [12], contribution of outstanding (b_b-b_w) in wide beams [12], and does not specify quantitative procedure for the evaluation of regularity of stiffness in elevation. Thus, $EC8_{100\text{-}50}$ could represent a feasible design result, less dependent on DLS limitation, and a capacity designed structure according to Mediterranean codes such as that of Italy and Spain. Finally, this design hypothesis is relevant in this study for further investigation of the effect of damage limitation prescriptions on the design of WBF structures.

3.2 Mechanical properties and design strategies

The design procedure is carried out in order to minimise cross section of members and thus avoiding unnecessary overstrength that could affect the relative performances of DBF and WBF.

Concrete f_{ck}=25MPa and steel f_{yk}=500MPa are used. Different stiffness contributions of joint regions are considered. NCSE-02 does not consider any contribution; conversely, EC8 suggests considering it, but no clear modelling strategy is proposed. Fardis [39] suggests placing only rigid offsets in beams, leading to a decrease of elastic stiffness which is proportional to beam depths, so it is higher for DBF rather than WBF. Such behaviour is coherent with the experimental evidences [40], and it is adopted herein for EC8 structures. Upper slab contribution is not considered in terms of stiffness, which is conservative for DLS design.

Values of q for EC8 buildings are 5.85 or 4.68 for structures regular in elevation or not, respectively, if default –non-explicit— values of overstrength factor are assumed. Regularity in elevation is evaluated through the quantitative criteria provided by NTC. In NCSE-02, q is 4.0 (DCH) and 2.0 (DCL), and DCH is only allowed if there is no bending moment inversion. Such restriction, rather than being based on ductility considerations, seems to compensate the absence of joint detailing rules and the low confidence in their capacity to alternate bending moments [41]. Thus, only within the scope of this paper, DCH with moment inversion is allowed. Storey amplification of seismic action due to reduction of masonry infills and due to P-Δ effect are adopted only in EC8 buildings, as NCSE-02 does not provide any quantitative provision.

Full cyclic flexural and shear capacities are considered for WB, provided that all the code prescriptions regarding geometric and mechanical restrictions in beam-column connections are satisfied (see section 2).

Design redistribution of bending moments in beams, which is allowed by EC8, is not considered in the design phase aimed at homogeneity between the different design cases. Negative moments in WB are higher than those in DB due to their lower relative stiffness with respect to columns. Thus, higher redistribution would be needed for WB with respect to that of DB in order to equalise maximum hogging and sagging moments, which would eventually lead to the attainment of pre-emptive yielding in WB ends.

Beam section dimensions are assumed to be similar in all the building, while column dimensions are assumed to be similar at each storey. They cannot be reduced considerably within two consecutive storeys, especially for WBF, because spliced bars from the lower column cannot separate significantly from the vertical configuration when passing through the joint. Sizing of columns in WBF is influenced also by beams width limitation and maximum eccentricity requirements in edge beams.

NCSE-02 capacity design rules have been demonstrated to be inefficient and sometimes impossible to be employed because it imposes the strength hierarchy by increasing the safety factor of the brittle element/mechanism (e.g., column with respect to beam) instead of dimensioning them based on the capacity of the ductile element/mechanism [10][25][41][42]. Thus, the same quantitative expressions established by EC8 for column-to-beam and shear-to-bending capacity design are taken into account for NCSE-02, but adopting lower prescribed capacity design ratios (1.1). Capacity design of joints is not considered for this code.

Regarding local detailing, DCL rules are more severe for NCSE-02 than those of EC8 in medium-high seismicity [41][42], while, in NCSE-02, DCH local ductility detailing rules are more relaxed than in EC8 [10][41]. Furthermore, NCSE-02 prescriptions regarding detailing of columns are mandatory depending on $a_g \cdot S$ (i.e., the anchoring acceleration value) of the design spectra, instead of depending on the ductility class.

3.3 Results of design

In Tables 2, 3 and 4, characteristics of the 6 design versions are summarised, being i the storey; L the member length; L_V the shear span; b_c and h_c the width and height of column sections; and b_w and h_b the width and height of beam sections.

Design results confirm the severity of the requirements of EC8 not related to the force-based design: base shear capacities are always larger than NCSE-02 ones even for higher q. In Fig. 4, deformed shapes of all the models are compared.

DLS design is the critical condition in $EC8_{50\text{-}50}$ buildings. Especially for WBF, very large columns are required in order to compensate the lower stiffness of beams, which together with minimum longitudinal ratio and minimum number of bars due to maximum spacing of stirrup legs, provide very high storey shear overstrengths (V_R/V_d) and column-to-beam capacity design ratios ($\Sigma M_{Rc}/\Sigma M_{Rb}$). Important cantilever behaviour (i.e. shear span in 1st storey column larger than half the interstorey height) is observed. In $EC8_{50\text{-}50}$ buildings, huge section dimensions of first and second storey columns may constitute a great shortcoming regarding architectural functionality. The WBF frame dimensions for $EC8_{50\text{-}50}$ design hypothesis represent an upper bound for the buildability of the structure

In $EC8_{100\text{-}50}$ buildings, the design to DLS is not so relevant, especially in DBF, resulting in smaller sections with reinforcement ratios slightly higher than the minimum. In this case, DBF and WBF present similar columns. On the other hand, NCSE-02 buildings are mainly force-based, so smaller sections and higher reinforcement ratios can be observed also in WBF design to DCL, and. lower local ductility of members is expected (see Fig. 5).

WBF induces lower relative demand in beams than DBF, especially in higher storeys (due to minimum reinforcement ratios) and in lower storeys, due to cantilever effect: L_V in columns 25-42% higher in WBF rather than in DBF (Table 2), which is also the cause of their regularity in elevation notwithstanding the greater interstorey height (Table 3, being Δm and ΔK the relative interstorey differences regarding storey mass and stiffness, evaluated according to NTC quantitative definitions).

Table 2: Geometric design properties of each model ((*)mean)

MODEL		i	b_c	h_c	$(L_V/L)_X$	$(L_V/L)_Y$	$(L_V/h_c)_{min}$	$\nu^{(*)}$	$\rho_{tot}^{(*)}$	$\rho_{w,b}$	$\rho_{w,h}$	b_w	h_b	$\rho'^{(*)}$	$\rho^{(*)}$	$\rho_{max}/\rho_{min}^{(*)}$	ρ_w
		Columns												Beams			
		-	[mm]	[mm]	-	-	-	-	[%]	[%]	[%]	[mm]	[mm]	[%]	[%]	-	[%]
EC8$_{50-50}$	DBF	5	300	300			10.0	0.08	1.8	0.67	0.67			0.28	0.37	1.37	
		4	300	300			10.0	0.11	1.4	0.67	0.57			0.32	0.37	1.24	
		3	400	350	≈0.50		7.5	0.14	1.1	0.54	0.57	300	500	0.43	0.38	1.13	0.48
		2	500	350			6.0	0.15	1.0	0.56	0.57			0.50	0.47	1.14	
		1	600	400	0.75	0.75	6.7	0.17	1.3	0.56	0.67			0.57	0.53	1.21	
	WBF	5	500	500			7.5	0.03	1.0	0.54	0.54			0.30	0.23	1.29	
		4	600	550			6.0	0.06	1.1	0.67	0.54			0.45	0.26	1.77	
		3	700	550	≈0.50		5.0	0.07	1.0	0.67	0.61	650	300	0.58	0.39	1.53	0.44
		2	800	550			4.4	0.08	1.0	0.67	0.61			0.62	0.45	1.41	
		1	800	550	0.95	0.97	5.0	0.11	1.0	0.67	0.61			0.59	0.41	1.47	
EC8$_{100-50}$	DBF	5	400	350			7.7	0.05	1.4	0.67	0.57			0.28	0.37	1.36	
		4	450	350			6.7	0.10	1.3	0.60	0.57			0.33	0.37	1.24	
		3	500	350	≈0.50		6.0	0.14	1.1	0.54	0.57	300	500	0.44	0.45	1.26	0.48
		2	550	350			5.5	0.17	1.3	0.61	0.57			0.57	0.54	1.19	
		1	600	350	0.55	0.60	6.7	0.20	1.1	0.56	0.57			0.64	0.63	1.14	
	WBF	5	400	350			7.7	0.05	1.4	0.67	0.57			0.33	0.25	1.34	
		4	450	350			6.7	0.10	1.3	0.60	0.57			0.50	0.31	1.59	
		3	500	350	≈0.50		6.0	0.14	1.1	0.54	0.57	500	300	0.61	0.42	1.46	0.53
		2	550	350			5.5	0.17	1.3	0.61	0.57			0.73	0.56	1.32	
		1	600	350	0.69	0.83	6.7	0.20	1.1	0.56	0.57			0.78	0.56	1.40	
NCSE-02	DBF	5	300	300			10.0	0.08	1.9	0.45	0.45			0.31	0.41	1.32	
		4	300	300			10.0	0.17	2.8	0.45	0.45			0.34	0.41	1.21	
		3	350	300	≈0.50		8.6	0.22	2.4	0.38	0.45	300	500	0.43	0.41	1.12	0.48
		2	400	300			7.5	0.27	2.1	0.50	0.45			0.49	0.46	1.20	
		1	450	300	0.53	0.55	8.9	0.30	2.3	0.45	0.45			0.61	0.57	1.22	
	WBF	5	300	300			10.0	0.08	2.8	0.48	0.48			0.54	0.50	1.11	
		4	350	300			8.6	0.14	3.0	0.38	0.67			0.94	0.68	1.42	
		3	400	300	≈0.50		7.5	0.19	3.1	0.50	0.67	450	300	1.36	1.02	1.41	0.45
		2	500	300			6.0	0.21	2.5	0.40	0.67			1.61	1.28	1.30	
		1	600	300	0.75	0.78	6.7	0.22	3.5	0.45	0.67			1.56	1.30	1.22	

Table 3: Mechanic design properties of each model ((*)mean)

MODEL		i	IDR_X	IDR_Y	$C_{P\text{-}\Delta,X}$	$C_{P\text{-}\Delta,Y}$	$(\Sigma M_{Rc}/\Sigma M_{Rb})_X^{(*)}$	$(\Sigma M_{Rc}/\Sigma M_{Rb})_Y^{(*)}$	Δm	ΔK_X	ΔK_Y	$(V_R/V_d)_X$	$(V_R/V_d)_Y$	q_d
			IDR limitation		P-Δ amplification		Capacity design ratio		Regularity in elevation					
		-	[%]	[%]	-	-	-	-	[%]	[%]	[%]	-	-	-
EC8$_{50-50}$	DBF	5	0.45	0.47	1.00	1.00	-	-	-	-	-	(3.73)	(3.61)	
		4	0.46	0.46	1.00	1.00	1.48	1.77	11.3	41.2	43.0	3.71	3.61	
		3	0.48	0.45	1.00	1.00	1.89	2.20	1.6	22.5	26.1	3.50	3.46	4.68
		2	0.49	0.45	1.00	1.00	2.19	2.28	1.5	14.8	17.6	3.99	3.97	
		1	0.44	0.42	1.11	1.00	2.67	2.68	5.2	-7.7	-13.9	3.43	3.40	
	WBF	5	0.30	0.30	1.00	1.00	-	-	-	-	-	(10.29)	(10.17)	
		4	0.45	0.44	1.00	1.00	4.50	5.93	10.1	17.4	17.0	8.69	8.56	
		3	0.50	0.50	1.12	1.11	5.39	6.23	3.5	15.7	15.7	9.51	9.37	5.85
		2	0.50	0.50	1.14	1.13	6.66	7.19	2.2	15.9	15.9	9.84	9.72	
		1	0.32	0.32	1.00	1.00	8.03	8.54	6.1	24.5	23.0	7.07	7.00	
EC8$_{100-50}$	DBF	5	0.17	0.18	1.00	1.00	-	-	-	-	-	(6.42)	(6.25)	
		4	0.30	0.29	1.00	1.00	1.81	2.21	9.7	13.8	15.8	4.07	3.97	
		3	0.36	0.35	1.00	1.00	1.89	1.90	0.8	7.3	10.0	3.50	3.43	4.68
		2	0.41	0.38	1.12	1.00	1.92	2.03	0.8	9.0	10.0	3.71	3.65	
		1	0.38	0.37	1.14	1.13	1.96	1.98	4.2	-15.8	-20.1	2.91	2.82	
	WBF	5	0.25	0.24	1.00	1.00	-	-	-	-	-	(9.29)	(9.14)	
		4	0.40	0.39	1.15	1.15	2.55	3.34	9.7	8.0	7.6	6.27	6.16	
		3	0.49	0.48	1.24	1.23	2.38	3.09	0.8	3.3	4.7	5.57	5.48	5.85
		2	0.50	0.50	1.32	1.29	2.51	3.01	0.8	7.7	8.8	5.97	5.89	
		1	0.40	0.40	1.26	1.25	2.97	3.38	4.2	10.1	8.8	4.56	4.53	
NCSE-02	DBF	5	0.21	0.22			-	-						
		4	0.40	0.42			1.36	1.72						
		3	0.56	0.57	-		1.39	1.71			-			4.00
		2	0.57	0.55			1.56	1.61						
		1	0.63	0.62			1.62	1.63						
	WBF	5	0.33	0.34			-	-						
		4	0.53	0.52			1.52	1.71						
		3	0.65	0.63	-		1.47	1.45			-			2.00
		2	0.64	0.62			1.53	1.57						
		1	0.43	0.42			2.32	2.25						

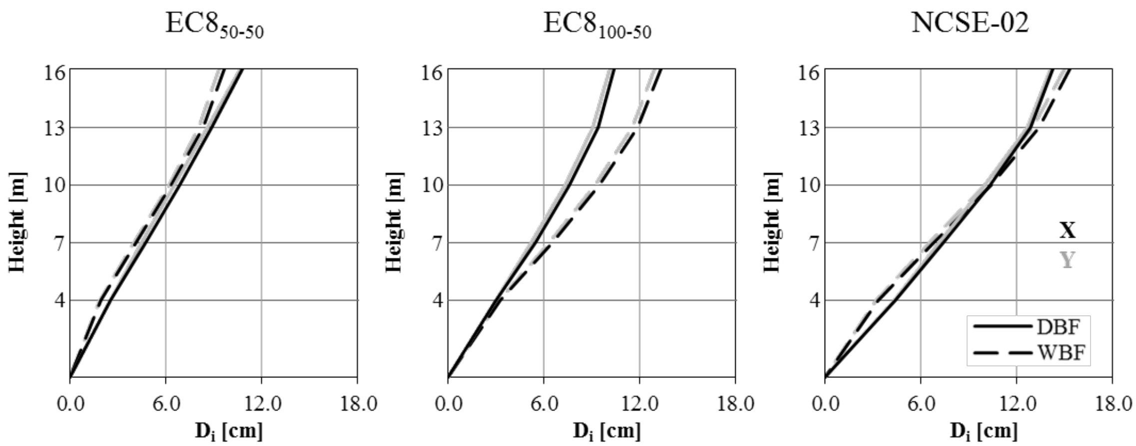

Fig. 4: Lateral deformed shape in both directions for all the models (adapted for comparison)

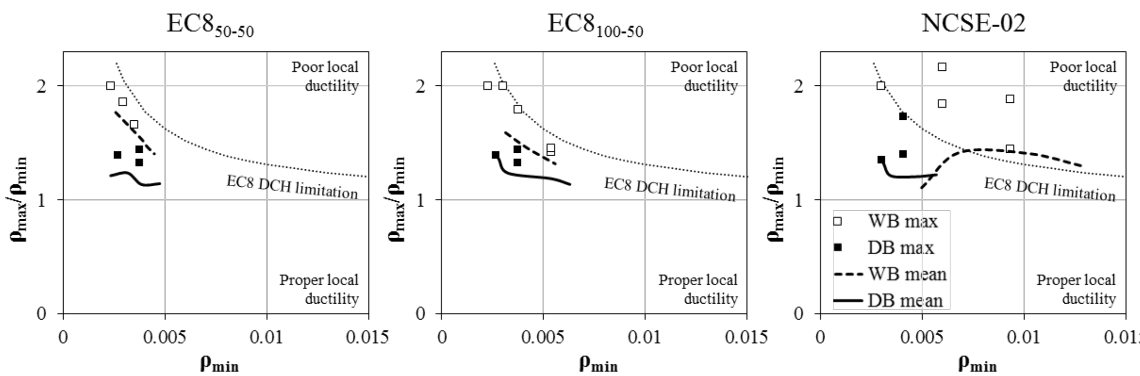

Fig. 5: Mean and maximum storey values of ρ_{max}/ρ_{min} in relation with the EC8 limitation for design to DCH, for all the cases

Design periods ($T_{50\%EI}$ or $T_{100\%EI}$, corresponding to EC8$_{50\text{-}50}$ and EC8$_{100\text{-}50}$, respectively) are quite higher than those suggested by codes (T_{code}) (Table 4). In EC8$_{50\text{-}50}$ and NCSE-02, stiffnesses of WBF and DBF are similar (Fig. 4). For NCSE-02, the latter is an indirect consequence of low q values for WBF rather than DBF. Conversely, in EC8$_{50\text{-}50}$ buildings, the cause is the strict IDR limitation. If similar IDR are required for both types, storey stiffnesses must be also similar, and global stiffness and elastic period (T_{el}) will likely do so, as well. Moreover, for EC8$_{50\text{-}50}$ frames, periods of design are even 7% lower for WBF, because column sections in upper storeys are slightly oversized with respect to the maximum IDR, due to limitation in the interstorey reduction of column sections and beam effective width requirements. Hence, in EC8$_{50\text{-}50}$ buildings also P-Δ requirements are fulfilled (through amplification factors $C_{P\text{-}\Delta}$, see Table 3), while in EC8$_{100\text{-}50}$ buildings such factors are considerably higher.

Table 4: Dynamic properties for each model (**bold**: design to ULS; *italics*: design to DLS)

MODEL			Modal properties					Spectral demand acceleration		
			T_{code} [s]	$T_{100\%EI}$ [s]	$T_{50\%EI}$ [s]	Γ -	λ -	$S_{ae}(T)$ [g]	$S_{ae}(T)$' [g]	$S_{ad}(T)$ [g]
$EC8_{50\text{-}50}$	DBF	X	0.60	0.69	***0.98***	1.36	0.83	0.466	0.597	0.128
		Y		0.67	***0.95***	1.39	0.82	0.480	0.552	0.118
	WBF	X		0.64	***0.90***	1.34	0.80	0.505	0.659	0.113
		Y		0.63	***0.89***	1.34	0.80	0.515	0.669	0.114
$EC8_{100\text{-}50}$	DBF	X	0.60	*0.71*	**1.00**	1.29	0.87	0.456	0.596	0.127
		Y		*0.69*	**0.97**	1.29	0.87	0.471	0.611	0.131
	WBF	X		*0.90*	**1.28**	1.30	0.85	0.357	0.544	0.093
		Y		*0.88*	**1.24**	1.30	0.85	0.368	0.548	0.094
NCSE-02	DBF	X	0.45	**1.00**	-	1.26	0.89	0.454	0.454	0.114
		Y		**0.99**	-	1.27	0.89	0.461	0.461	0.115
	WBF	X		**1.11**	-	1.31	0.83	0.412	0.412	0.206
		Y		**1.09**	-	1.32	0.83	0.419	0.419	0.210

The equivalent real elastic spectral acceleration of design $S_{ae}(T)$' (see Table 4) is obtained from the original value $S_{ae}(T)$ by considering $C_{P\text{-}\Delta}$ and equivalent amplification for accidental eccentricity of masses.

4. CASE STUDY: N2 ASSESSMENT

In this section, performances and capacities of all the models are assessed by means of nonlinear static analysis ("pushover", SPO) and N2 spectral method [44][45].

Lumped plasticity is adopted for nonlinear modelling of the structures. Chord rotations' capacity thresholds are based on EC8 part 3 [46] formulations, fitted to a large experimental database [47]. Chord rotation capacities (θ_{ULS}) correspond to the threshold of Limit State of Significant Damage according to EC8.

Consistently with the design assumptions, rigid offsets are only placed in beam ends, and plastic hinges are placed at the faces of joint panels. Values of L_V=0.5·L are assumed for all the members except for first storey columns (Table 2), for which the elastic moment distribution made this assumption unrealistic.

Mean values for material properties are adopted. For concrete, Eurocode 2 [48] provisions are assumed, while for steel, typical factors of around 1.26 between mean and characteristic yield strengths are observed [49], which is equivalent to a factor 1.45 between mean and design values.

In Fig. 6, the ranges of values for θ_y and θ_{ULS} in all the buildings are presented; in first-storey columns, values correspond only to the bottom of the elements. Results are similar for both directions. Beams in the X-direction have higher rotation variability than in the Y-direction, because in the first case beams present different spans. Rotations in first storeys are higher with respect to all other locations of the buildings, especially for WBF, due to their larger L_V.

Higher θ_y, lower θ_{ULS} and, in turn, lower chord rotation ductility (μ_θ) are shown for NCSE-02. θ_{ULS} of WB are on average 38% higher than DB ones, similar to the results obtained in [12]. Lower μ_θ for WB are obtained with respect to DB: 13% and 28% lower for EC8 and NCSE-02, respectively, consistent with the average 27% obtained in the parametric study in [12].

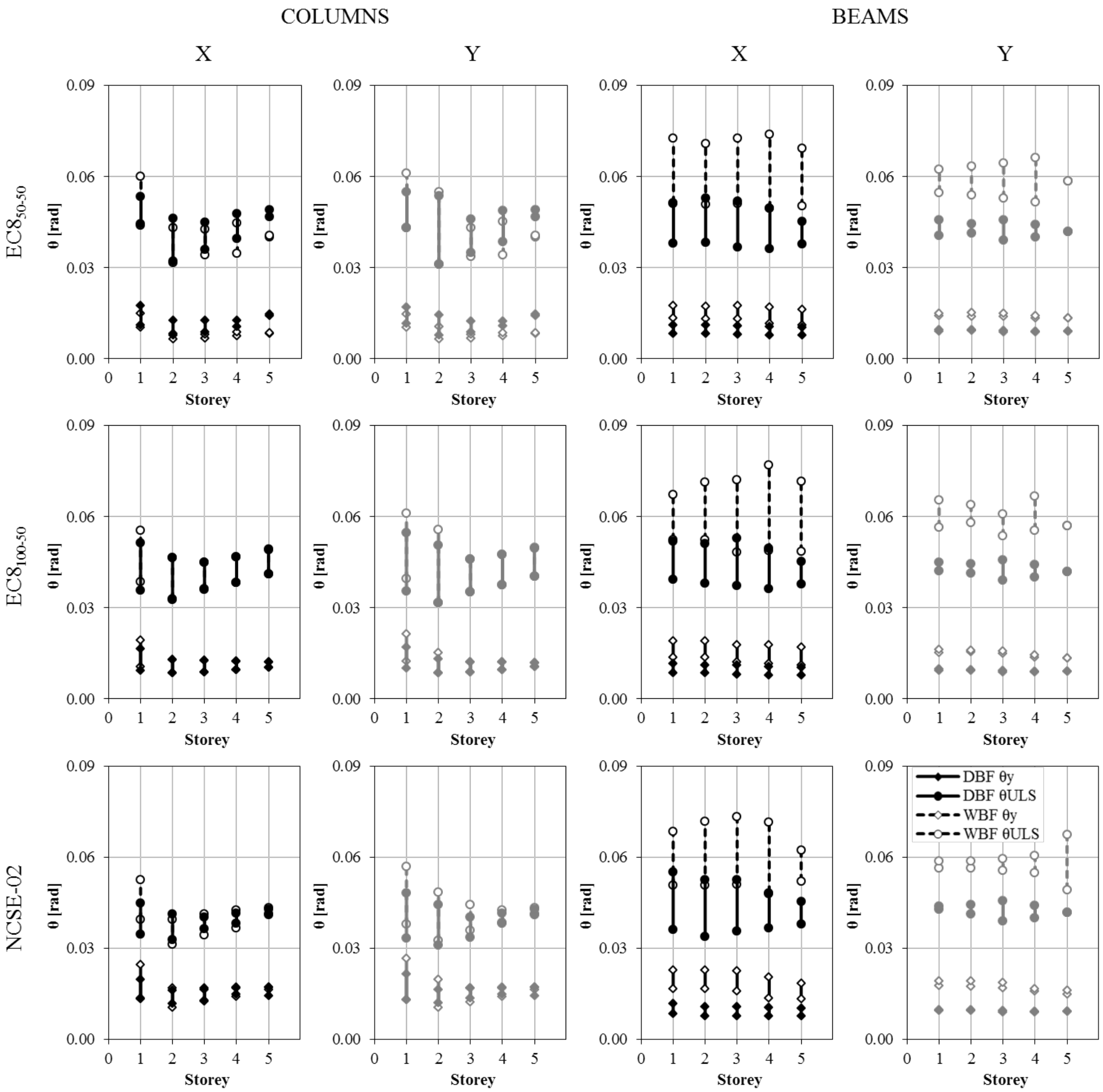

Fig. 6: θ_y and θ_u ranges in each storey of each model

4.1 SPO analyses

Nonlinear analyses are carried out with the commercial software SAP2000 v15 [50]. Two different lateral load patterns are considered for SPO: "MODE", proportional to modal displacement and masses, and "MASS", proportional to masses. Conventional collapse is attained when the first plastic hinge reaches θ_{ULS}. In Fig. 7, "MODE" mechanisms and top displacement capacities (D_u) of all the models are shown. The height involved in the mechanism (H_{mec}) depends mainly on column-to-beam capacity design ratios (Table 3), which is higher for WBF than DBF, especially for $EC8_{50\text{-}50}$. H_{mec} is higher in the Y-direction with respect to the X-direction. $EC8_{50\text{-}50}$, $EC8_{100\text{-}50}$, and NCSE-02 buildings show decreasing H_{mec}. Even in the case in which 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 $EC8_{50\text{-}50}$ buildings, first yielding occurs only in a beam end, while in the rest of the cases yielding is attained simultaneously at some columns bases and in some beams. Beams usually present lower design section overstrength with respect to columns, especially in $EC8_{50\text{-}50}$. Still, column bases, which are fixed to the foundation, increase their chord rotation demand more quickly than the surrounding hinges (also X-direction shorter bay beams experiment such behaviour). When this occurs, those column bases are also the first in attaining θ_{ULS}. Most demanded columns are usually central columns, thus the last plastic hinges forms in lateral columns heads of last storey involved in the mechanism.

In Fig. 8, pushover curves are plotted; bilinearization according to EC8 is performed even if it is proven not being the option guaranteeing the minimum error [51]. In most cases, ULS capacity of frames is attained before the complete formation of the collapse mechanism; local ductility capacity is exhausted beforehand. Maximum base shear in each case is consistent with storey overstrengths (see Table 3): higher for WBF rather than DBF in $EC8_{50\text{-}50}$ due to DLS design, and a similar trend is found in NCSE-02 due to lower q for WBF. Lower base shear in WBF rather than DBF is observed in $EC8_{100\text{-}50}$ due to lower demand (see Table 4) and less relevance of DLS design.

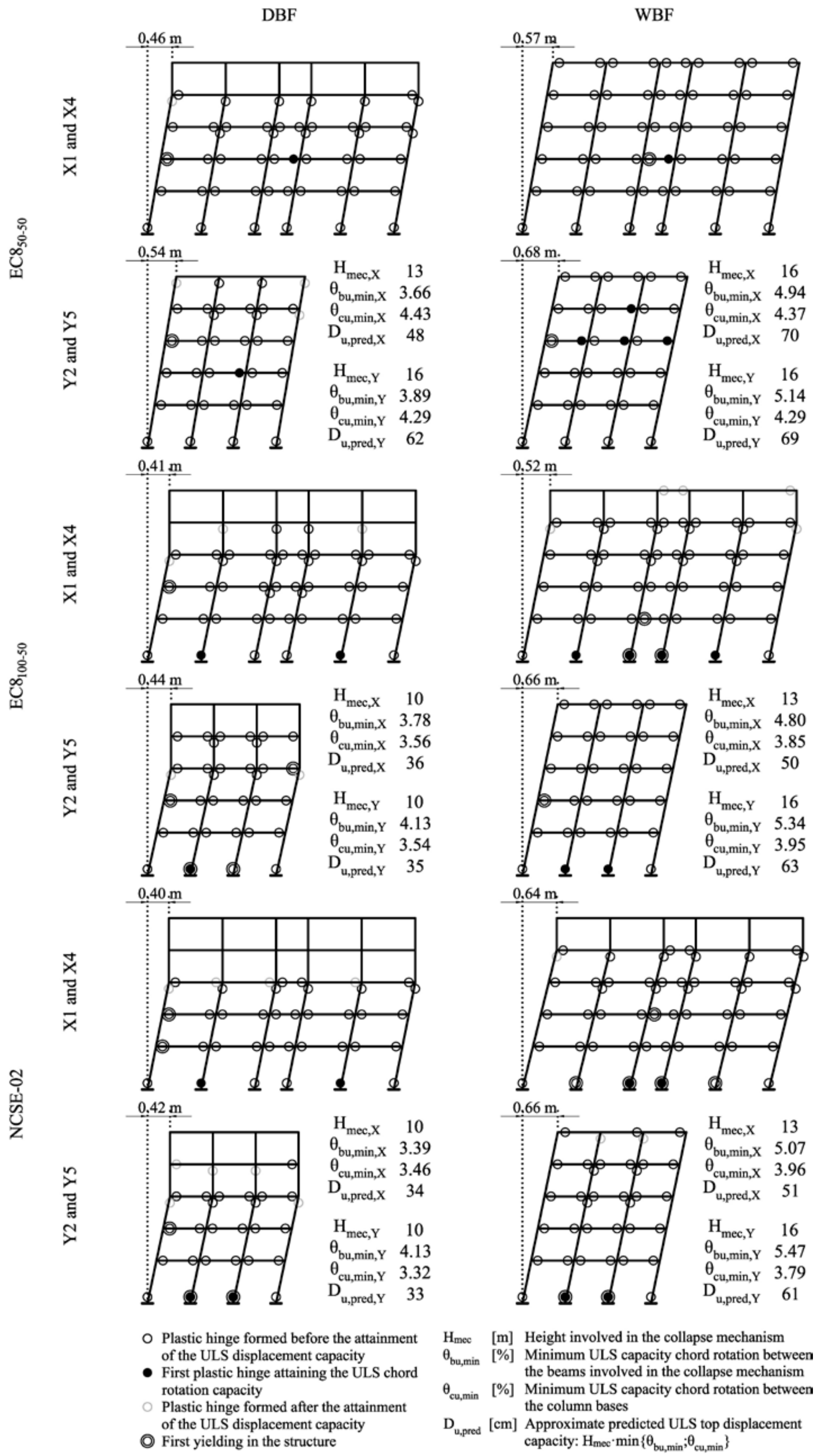

Fig. 7: Mechanism of collapse of each model for “MODE” lateral load distribution

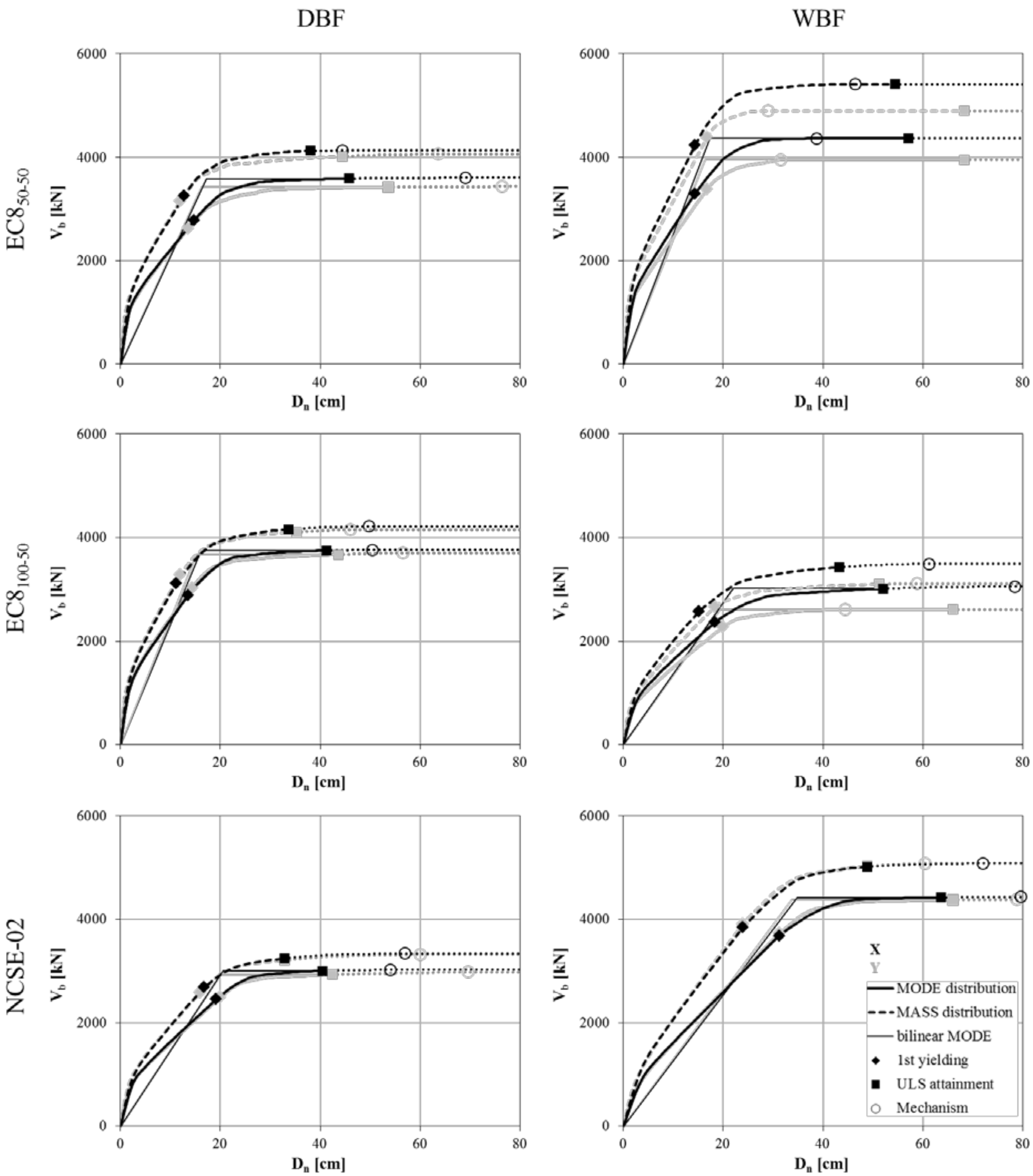

Fig. 8: Pushover curves and piecewise linear fits of each model

4.2 Assessment of capacities

N2 spectral method is used in order to assess performances and peak ground acceleration capacities (PGA_c) of all the structures. Bilinear pushover curves are expressed as capacity curves in the acceleration-displacement response spectrum (ADRS) format. Only "MODE" distribution results are considered, as rather similar relative capacities between both structural types are obtained for "MASS" cases. Results are shown in Table 5 and ADRS graphical format is shown in Fig. 9. Effective periods (T_{eff}) for the equivalent single degree of freedom of N2 method and their corresponding spectral effective acceleration demand ($S_a(T_{eff})$) are obtained.

Table 5: Performance properties for each model

MODEL			Bilinear capacity curve							Overstrength						
			T_{el} [s]	T_{eff} [s]	T_{eff}/T_{el} -	C_s [g]	S_{dy} [cm]	S_{du} [cm]	$S_a(T_{eff})$ [g]	R_ω -	R_α -	R_μ -	R_D -	q -	PGA_c [g]	SF -
$EC8_{50\text{-}50}$	DBF	X	0.65	1.43	2.19	0.249	12.6	33.6	0.320	1.51	1.29	2.67	1.87	9.73	0.59	2.08
		Y	0.63	1.40	2.21	0.242	11.8	38.6	0.326	1.57	1.30	3.28	1.69	11.38	0.69	2.43
	WBF	X	0.60	1.37	2.28	0.275	12.9	42.5	0.332	1.84	1.32	3.30	1.99	15.96	0.78	2.73
		Y	0.59	1.40	2.38	0.249	12.2	50.8	0.325	1.87	1.17	4.17	2.06	18.69	0.91	3.19
$EC8_{100\text{-}50}$	DBF	X	0.67	1.42	2.13	0.248	12.5	31.9	0.321	1.50	1.30	2.56	1.86	9.27	0.57	1.98
		Y	0.65	1.39	2.16	0.243	11.7	33.6	0.328	1.54	1.21	2.87	1.87	9.97	0.61	2.13
	WBF	X	0.85	1.82	2.14	0.205	16.9	40.0	0.250	1.73	1.27	3.27	2.17	11.33	0.55	1.94
		Y	0.83	1.84	2.22	0.177	14.9	50.8	0.248	1.65	1.15	3.42	2.21	14.27	0.70	2.44
NCSE-02	DBF	X	0.95	1.81	1.91	0.200	16.3	31.9	0.252	1.44	1.22	1.96	1.80	6.23	0.44	1.56
		Y	0.93	1.80	1.92	0.196	15.7	33.3	0.254	1.45	1.17	2.12	1.82	6.55	0.47	1.64
	WBF	X	1.05	1.83	1.75	0.317	26.5	48.5	0.249	1.28	1.20	1.83	1.65	4.67	0.67	2.33
		Y	1.03	1.81	1.77	0.314	25.6	50.1	0.252	1.28	1.17	1.96	1.66	4.88	0.70	2.44

Capacity curves are defined by three values: maximum spectral acceleration capacity (C_s) and yielding and maximum spectral displacement capacities (S_{dy} and S_{du}, respectively). Provided behaviour factor is calculated as $q=R_S{\cdot}R_D{\cdot}R_\mu$. Structural overstrength is obtained as $R_S=R_\alpha{\cdot}R_\omega$, where R_α is the ratio between C_s and the acceleration corresponding to first structural yielding, and R_ω is the ratio between the last value and the elastic spectral acceleration of design employed in the design ($S_{ae}(T)$', see Table 4). Spectral contribution is calculated as $R_D=S_{ae}(T)'/S_a(T_{eff})$. Ductility contribution (R_μ) is obtained by means of the R_μ-μ-T relationship suggested in EC8, from which IN2 curves are obtained [52]. This procedure does not account for any possible difference of cyclic energy dissipation between DBF and WBF, as hysteretic models for WBF designed to EC8 need further research (see section 2).

Aimed at a homogeneous comparison of global seismic capacities, PGA_c is obtained in all the cases through the adoption of proportional spectra (Fig. 9). Finally, safety factors $SF=PGA_c/PGA_d$ (Fig. 10) are obtained for each design alternative, being $PGA_d=a_g{\cdot}S$ the demand peak ground acceleration.

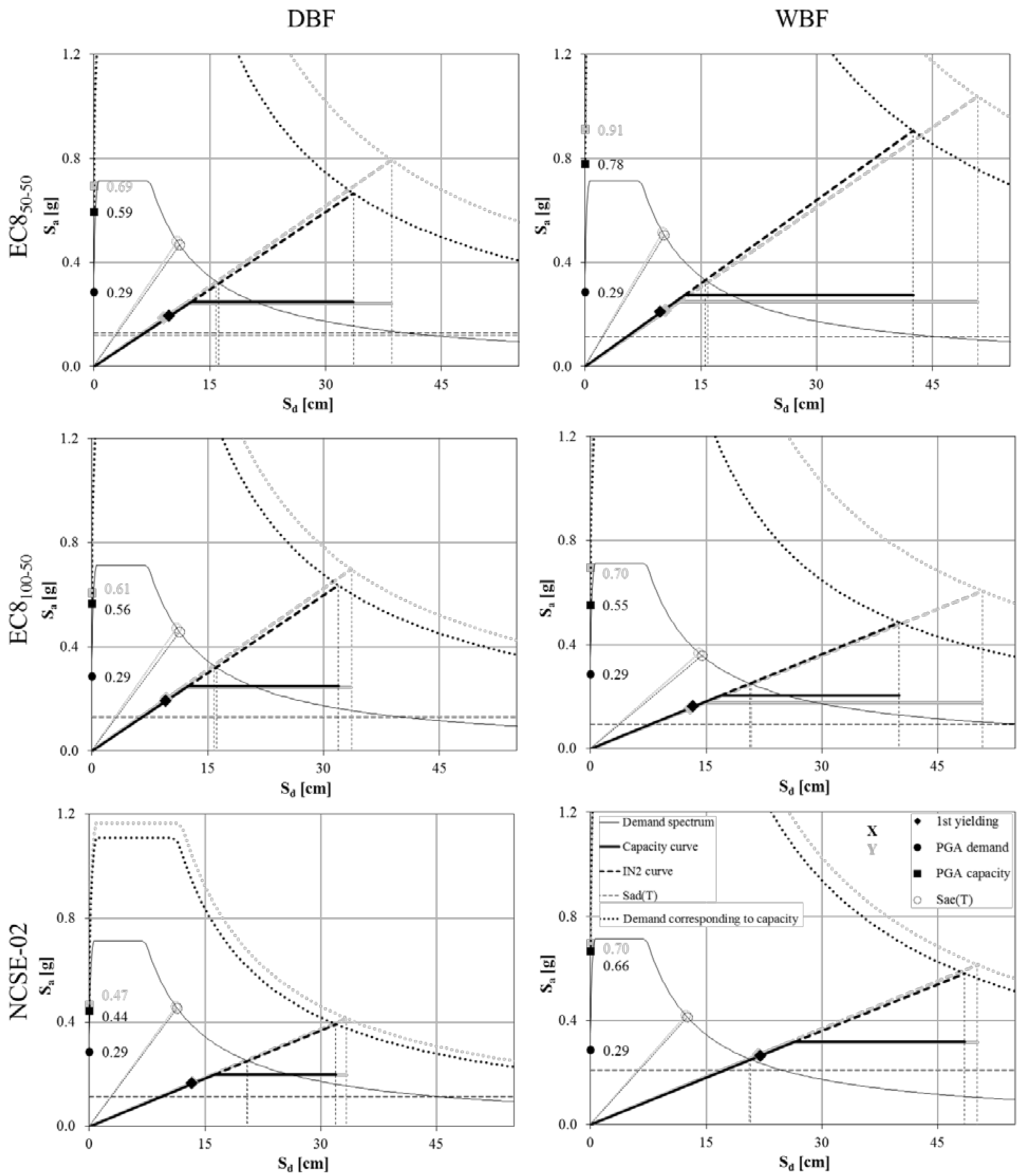

Fig. 9: ULS spectral performance and maximum capacity of each model, obtained with N2 method

Similar overstrengths (Table 5) are obtained for all the cases, while NCSE-02 frames show lower ductility. In general, inelastic behaviour at ULS demand is limited for all the buildings (Fig. 13), especially in NCSE-02 WBF, which remains in equivalent elastic field. The quantification of different sources of overstrength is coherent with the design assumptions. R_ω is 1.56 on average, slightly higher with respect to the steel overstrength of 1.45 (see section 3.1), while R_α is 1.23, slightly lower than α_u/α_1=1.30 proposed by EC8. R_D (mean 1.89) corresponds to mean period elongation, i.e., effective-to-elastic period ratio T_{eff}/T_{el}, equal to 2.21 for EC8 frames and 1.84 for NCSE-02 ones; such difference is caused by the higher reinforcement ratio of sections of NCSE-02 frames (Table 2). Thus, the corresponding mean effective-to-elastic stuiffness ratio (K_{eff}/K_{el}) is 0.20 (EC8) and 0.30 (NCSE-02), which in both cases is lower than the assumed value for ULS design (0.50). It is worth noting that 0.20 is also the mean value for secant-to-elastic stiffness ratio for members suggested in [53].

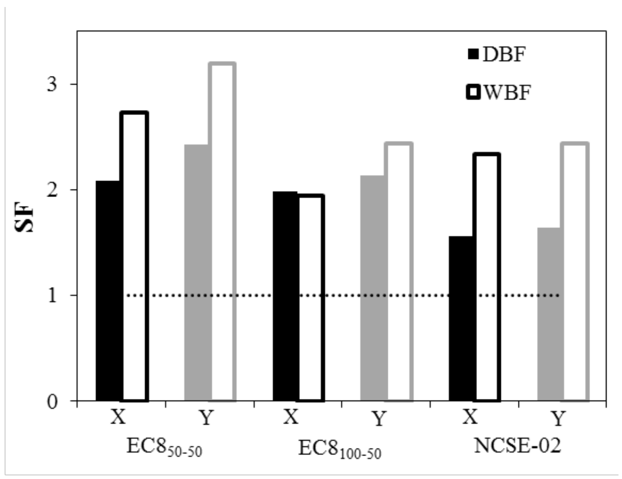

Fig. 10: Global $SF=PGA_c/PGA_d$ of all the models

In the following, ratios for any parameter A between WBF and DBF are indicated as $A_{W/D}$ (rather than using the heavier notation A_{WBF}/A_{DBF}), and analogously $A_{W\text{-}D}\equiv A_{WBF}\text{-}A_{DBF}$. Subscripts c and b refer to columns and beams, respectively

WBF show similar or even greater global seismic capacities with respect to DBF in all the cases (Table 5 and Fig. 9): SF of WBF are 31%, 6% and 49% higher than those for DBF on average, in the case of EC8$_{50\text{-}50}$, EC8$_{100\text{-}50}$ and NCSE-02, respectively. Moreover, if no reduction of q due to irregularity in EC8-DBF had been adopted, still better relative performances would have been expected for WBF. The causes of such good performances of WBF in comparison with DBF are: (i) higher H_{mec} (Fig. 8); (ii) higher $\theta_{u,min}$ (Fig. 6), due to lower h_b and higher L_{Vc} (Table 2); (iii) in EC8 buildings, sufficient stiffness of WBF, 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 (see section 2).

T_{eff} of WBF (Table 3) show similar values than DBF for $EC8_{50\text{-}50}$ and NCSE-02. In EC8 buildings it is an expected result, since DLS design is the critical condition; while for NCSE-02, such low difference in periods is likely a coincidence, because the increment of stiffness is not a target, but a secondary consequence of the increment of strength, in turn depending on whether such increment is achieved by means of bigger sections or higher reinforcements. In $EC8_{100\text{-}50}$ WBF show higher T_{eff} with respect to DBF.

Assessment of DLS performance of EC8 frames is carried out according to the stiffness assumptions exposed in section 3.2. DLS results (Fig. 11) confirm that, at least in this particular case study, assuming gross elastic stiffness for members might not be appropriate. However, DLS is not either fully satisfied in $EC8_{50\text{-}50}$, because the effective stiffness for DLS is, on average, 45% of the elastic one, which is slightly lower than the assumed value for design, 50%. Such results are more in accordance with estimation of stiffness degradation suggested by other codes (see section 3.1).

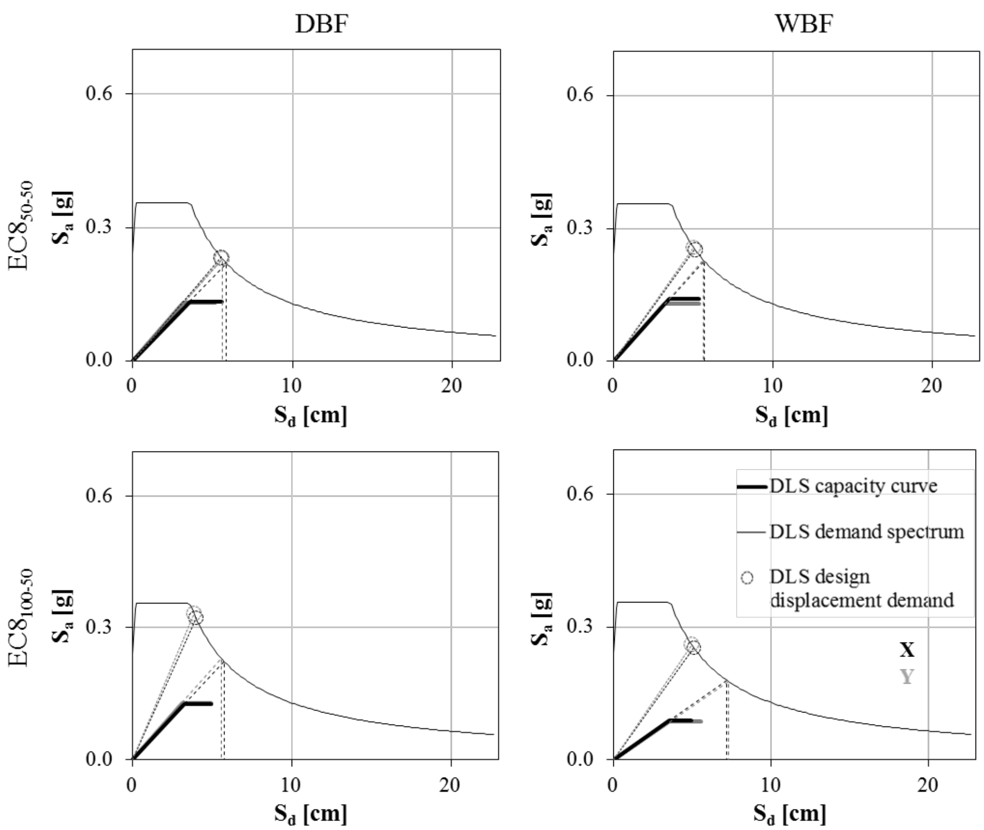

Fig. 11: DLS spectral performance of EC8 models

5. COMPLEMENTARY ASSESSMENT OF PARAMETRIC SET OF FRAMES

The previous results, corresponding to three different design alternatives and two directions of analysis, suggest that: (i) WBF designed for DCH, adopting similar *q* than DBF and satisfying different DLS limitations, may provide at least similar capacities than DBF; and (ii) WBF designed to DCL, adopting much lower *q* than DBF and without satisfying any DLS limitations, may provide much larger capacities than DBF. Thus, Mediterranean code limitations on *q* for WBF may not be justified in most cases. In order to evaluate whether such conclusion could be generalised to RC-MRF residential building stock designed according to different codes, a higher set of case studies is evaluated.

Hence, a parametric design is carried out, resulting in 72 different planar frames, 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. EC8 represents the most favourable code for WBF due to its strict reduction of member stiffness (E_cI_c) for DLS design (50% of the elastic one). Frames corresponding to NTC can be designed assuming uncracked stiffness of members (thus rather equivalent to design $EC8_{100\text{-}50}$, see section 2.2), which is the most unfavourable hypothesis for WBF. Frames corresponding to NCSE-02 have no design to DLS, but in this case similar *q* are adopted for WBF and DBF, in order to check if also in codes with no IDR limitation it is possible to remove the limitation of *q* for WBF.

Aimed at covering the widest possible range of situations of design, a parametric study based on relevant design features is carried out, assuming different realisations for each parameter: 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 [36][37]; and a_{gR}: 0.12g and 0.25g. Elastic spectra are obtained in analogy with Fig. 3, and similar material mechanical properties and design strategies are adopted. Geometry of the frames is shown in Fig. 12. All of them have four bays with similar spans, and interstorey heights are 3m with the exception of the ground storey, which is 4m.

Aimed at an agreement between accuracy and computational demand, the assessment of relative performances between WBF and DBF is carried out by means of the simplified approach proposed in [12], based on similar approaches already used in other studies [36][37][54][55][56][57]. It considers that collapse of structures is attained by means of a "rigid" mechanism of *n* storeys, without any pre-yielding contribution neither of the (*n*-1) upper storeys nor of the intermediate column ends, and assuming similar evolution of chord rotations in all the member ends involved. This approach is quite conservative from the point of view of WBF, i.e. unfavourable for WBF with respect to DBF, because higher H_{mec} for WBF rather than for DBF are not taken into account.

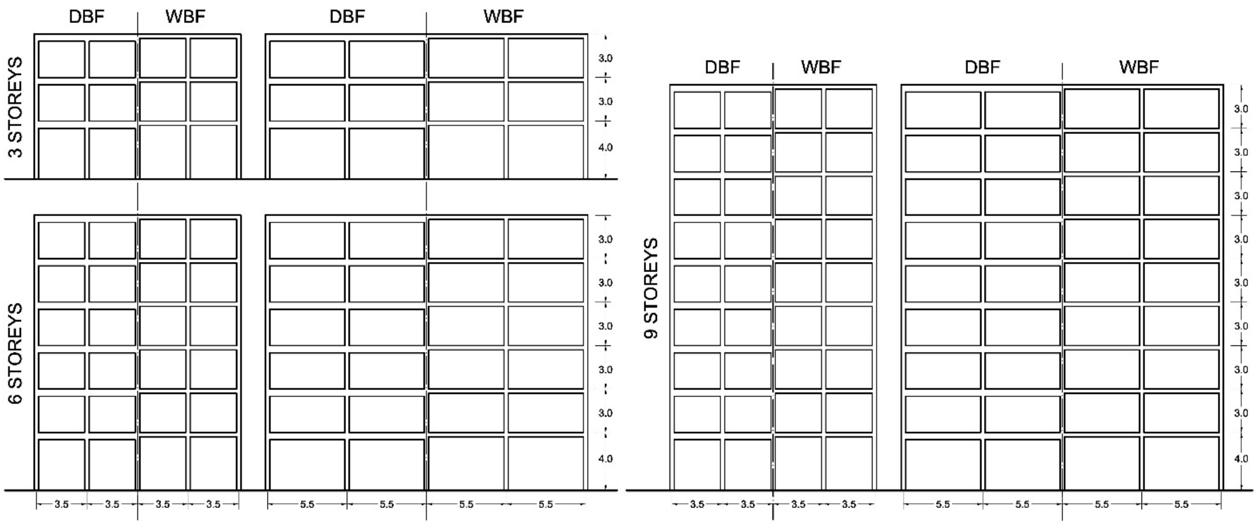

Fig. 12: Geometry of the different frames of the set

In all the following, subscript *1* and *n* refer to the storey of interest (first or last, respectively); $\theta_{u,min}$ is the minimum θ_u between members involved in the collapse mechanism; f_{conf} is the confinement contribution to θ_u; and $f_{K,sec}$ is the ratio between the stiffness degradation of connections in WBF with respect to DBF, see [10][12] for further details.

Results of design are presented in Table 6. They confirm the trends observed in the specific case studies. DLS is likely to be the critical condition of design for WBF with respect to DBF. For EC8 frames, this is the most frequent critical condition for both types, with the exception of low seismicity cases. For NTC frames, the critical conditions are DLS for high seismicity and gravity load combination for low seismicity cases. Conversely, for NCSE-02 frames, seismic situation is the critical one in most cases, due also to the lower q corresponding to DCH than in the other codes. In general, capacity design of columns in WBF does not often affect the dimensions because of the higher overstrength due to DLS design.

Table 6: Results of design and assessment of the whole set of frames, considering $H_{mec,W/D}$=$f_{K,sec}$=1.0
(G: gravitational, S: seismic, D: deformability; I: $T_{el,W/D}$≈1, II: $T_{el,W/D}$>1; C: first θ_u attainment in column, B: first θ_u attainment in beam)

Design parameters						WBF													DBF													W/D		
	Member stiffness						Geometric properties				Mechanic properties						Modal prop.			Geometric properties				Mechanic properties						Modal prop.				
Code	DLS [%E_cI_c]	ULS [%E_cI_c]	n	L [m]	a_{gR} [g]	Crit. condition (G/S/D)	b_w [mm]	h_b [mm]	$h_{c,1}$ [mm]	$h_{c,n}$ [mm]	$f_{conf,b}$ -	$f_{conf,c}$ -	$\nu_{c,1}$ -	$L_{Vc,1}$ [m]	$\theta_{ub,min}$ [rad]	$\theta_{uc,min}$ [rad]	T_{el} [s]	Γ -	Crit. condition (G/S/D)	b_w [mm]	h_b [mm]	$h_{c,1}$ [mm]	$h_{c,n}$ [mm]	$f_{conf,b}$ -	$f_{conf,c}$ -	$\nu_{c,1}$ -	$L_{Vc,1}$ [m]	$\theta_{ub,min}$ [rad]	$\theta_{uc,min}$ [rad]	T_{el} [s]	Γ -	Scenario (I/II)	Sub-scenario (CC/CB//BB)	$SF_{W/D}$ -
EC8	50	50	3	4.0	0.12	D	400	300	300	250	1.14	1.24	0.12	2.17	0.067	0.061	0.90	1.23	D	200	400	300	250	1.01	1.24	0.12	2.15	0.054	0.060	0.87	1.23	I	CB	1.09
					0.25	D	500	300	500	400	1.18	1.26	0.05	2.82	0.068	0.054	0.48	1.28	D	250	450	450	300	1.04	1.29	0.06	2.38	0.052	0.054	0.48	1.30	I	CB	1.06
				6.5	0.12	D	600	300	350	350	1.15	1.21	0.15	2.31	0.080	0.058	0.88	1.21	D	300	500	350	250	1.05	1.21	0.14	2.15	0.061	0.056	0.83	1.28	I	CC	1.02
					0.25	D	700	350	600	500	1.13	1.31	0.06	3.08	0.073	0.053	0.47	1.28	D	300	550	500	350	1.03	1.26	0.07	2.41	0.058	0.052	0.50	1.29	I	CC	1.11
			6	4.0	0.12	D	400	300	350	250	1.14	1.21	0.18	2.32	0.067	0.058	1.45	1.31	D	200	400	350	250	1.01	1.21	0.18	2.27	0.053	0.057	1.38	1.32	I	CB	1.04
					0.25	D	700	330	550	450	1.13	1.33	0.09	2.76	0.063	0.054	0.75	1.31	D	250	450	500	300	1.04	1.26	0.10	2.60	0.052	0.053	0.82	1.37	I	CB	1.20
				6.5	0.12	D	600	300	400	350	1.15	1.32	0.23	2.54	0.080	0.059	1.50	1.28	G	300	500	400	300	1.05	1.32	0.22	2.25	0.061	0.057	1.29	1.30	II	CC	0.91
					0.25	D	800	350	750	550	1.16	1.32	0.08	4.08	0.074	0.054	0.83	1.34	D	300	600	500	300	1.02	1.26	0.15	2.36	0.055	0.051	0.88	1.38	I	CC	1.16
			9	4.0	0.12	G	400	300	350	250	1.14	1.21	0.28	2.33	0.067	0.058	1.99	1.35	G	200	400	350	250	1.01	1.21	0.27	2.28	0.053	0.057	1.88	1.35	I	CB	1.03
					0.25	D	700	330	550	450	1.13	1.33	0.13	2.78	0.063	0.054	1.14	1.31	D	300	500	450	250	1.05	1.29	0.18	2.27	0.051	0.053	1.20	1.42	I	CB	1.22
				6.5	0.12	D	600	300	450	350	1.15	1.29	0.32	2.82	0.079	0.058	2.08	1.30	G	300	500	450	300	1.05	1.29	0.27	2.40	0.060	0.054	1.68	1.35	II	CC	0.89
					0.25	D	1000	350	900	750	1.12	1.32	0.09	5.03	0.071	0.055	1.13	1.34	D	300	650	500	300	1.01	1.26	0.23	2.29	0.053	0.051	1.24	1.38	I	CC	1.22
NTC	100	50	3	4.0	0.12	D	400	300	300	250	1.07	1.07	0.12	2.17	0.067	0.053	0.90	1.23	D	200	400	300	250	1.01	1.24	0.12	2.15	0.054	0.053	0.87	1.23	I	CC	0.97
					0.25	D	400	300	400	300	1.19	1.07	0.07	2.48	0.067	0.053	0.63	1.27	D	200	400	350	250	1.01	1.21	0.09	2.26	0.053	0.050	0.71	1.28	I	CC	1.20
				6.5	0.12	D	600	300	350	350	1.07	1.07	0.15	2.31	0.080	0.051	0.88	1.21	S	300	500	300	250	1.05	1.24	0.19	2.08	0.061	0.052	0.97	1.22	I	CC	1.09
					0.25	D	600	300	550	450	1.15	1.19	0.06	3.31	0.079	0.050	0.59	1.29	D	300	500	400	300	1.05	1.32	0.11	2.24	0.061	0.051	0.68	1.26	I	CC	1.09
			6	4.0	0.12	G	400	300	300	250	1.07	1.07	0.25	2.18	0.067	0.053	1.55	1.30	G	200	400	300	250	1.01	1.24	0.24	2.16	0.054	0.053	1.49	1.30	I	CC	0.96
					0.25	D	400	300	450	250	1.18	1.18	0.12	2.78	0.066	0.052	1.09	1.38	D	200	400	450	250	1.01	1.29	0.11	2.67	0.053	0.052	1.06	1.37	I	CC	0.98
				6.5	0.12	G	600	300	400	350	1.19	1.19	0.23	2.53	0.080	0.054	1.52	1.27	G	300	500	400	300	1.05	1.32	0.22	2.25	0.061	0.052	1.29	1.30	II	CC	0.91
					0.25	D	700	300	750	450	1.18	1.18	0.07	5.01	0.080	0.052	0.99	1.37	D	300	500	450	300	1.05	1.29	0.18	2.39	0.060	0.050	1.12	1.32	I	CC	1.14
			9	4.0	0.12	G	400	300	350	250	1.07	1.07	0.28	2.33	0.067	0.051	1.99	1.35	G	200	400	350	250	1.01	1.21	0.27	2.28	0.053	0.051	1.88	1.35	I	CC	0.95
					0.25	D	500	300	450	250	1.18	1.18	0.18	2.66	0.069	0.052	1.55	1.38	D	200	400	450	250	1.01	1.29	0.19	2.69	0.053	0.052	1.55	1.37	I	CC	1.00
				6.5	0.12	G	600	300	450	350	1.18	1.18	0.27	2.81	0.079	0.053	2.11	1.30	G	300	500	450	300	1.05	1.29	0.27	2.40	0.060	0.050	1.68	1.35	II	CC	0.87
					0.25	D	700	330	700	450	1.19	1.18	0.13	4.22	0.074	0.051	1.45	1.34	D	300	500	450	300	1.05	1.29	0.27	2.40	0.060	0.050	1.59	1.35	I	CC	1.13
NCSE-02	-	100	3	4.0	0.12	S	400	300	300	250	1.07	1.07	0.12	2.17	0.067	0.053	0.90	1.23	S	200	400	300	250	1.01	1.14	0.12	2.15	0.054	0.053	0.87	1.23	I	CC	0.97
					0.25	S	500	300	350	300	1.07	1.07	0.09	2.24	0.067	0.050	0.71	1.23	S	250	450	350	300	1.04	1.15	0.09	2.15	0.053	0.049	0.61	1.23	II	CC	0.87
				6.5	0.12	S	600	300	350	350	1.07	1.07	0.15	2.31	0.078	0.051	0.88	1.21	S	300	500	350	300	1.05	1.12	0.14	2.15	0.061	0.049	0.80	1.22	II	CC	0.94
					0.25	S	600	300	400	350	1.19	1.19	0.12	2.49	0.077	0.053	0.79	1.24	S	300	500	400	350	1.05	1.12	0.11	2.24	0.061	0.051	0.66	1.22	II	CC	0.85
			6	4.0	0.12	S	400	300	300	250	1.07	1.07	0.25	2.18	0.067	0.053	1.55	1.30	S	200	400	300	250	1.01	1.14	0.24	2.16	0.054	0.053	1.49	1.30	I	CC	0.96
					0.25	S	500	300	400	300	1.19	1.19	0.15	2.42	0.067	0.053	1.18	1.30	S	250	450	400	300	1.04	1.15	0.14	2.27	0.052	0.052	1.02	1.31	II	CC	0.89
				6.5	0.12	G	600	300	400	350	1.19	1.19	0.23	2.53	0.077	0.054	1.52	1.27	G	300	500	400	300	1.05	1.12	0.22	2.25	0.061	0.052	1.29	1.30	II	CC	0.91
					0.25	S	700	330	450	450	1.18	1.18	0.19	2.56	0.076	0.051	1.19	1.27	S	300	550	450	300	1.03	1.13	0.18	2.30	0.058	0.049	1.05	1.33	II	CC	0.96
			9	4.0	0.12	G	400	300	350	250	1.07	1.07	0.28	2.33	0.067	0.051	1.99	1.35	G	200	400	350	250	1.01	1.14	0.27	2.28	0.053	0.051	1.88	1.35	I	CC	0.95
					0.25	S	500	300	400	300	1.19	1.19	0.22	2.43	0.067	0.053	1.71	1.32	S	250	450	400	300	1.04	1.15	0.22	2.28	0.052	0.052	1.47	1.33	II	CC	0.89
				6.5	0.12	G	600	300	450	350	1.18	1.18	0.27	2.81	0.077	0.053	2.11	1.30	G	300	500	450	300	1.05	1.12	0.27	2.40	0.060	0.050	1.68	1.35	II	CC	0.87
					0.25	S	700	330	500	450	1.16	1.18	0.24	2.80	0.075	0.050	1.71	1.28	S	300	600	450	300	1.02	1.13	0.27	2.25	0.055	0.049	1.36	1.35	II	CC	0.86

Table 7: Mean values of ratios of significant parameters between WBF and DBF extracted from Table 6

Code	$h_{b,W/D}$	$h_{c,W/D}$	$f_{conf,b,W/D}$	$f_{conf,c,W/D}$	$\nu_{c,W-D}$	$L_{Vc,1,W/D}$	$\theta_{u,min,W/D}$	$T_{el,W/D}$	$\Gamma_{W/D}$	$SF_{W/D}$
EC8	0.66	1.16	1.11	1.02	-0.02	1.26	1.06	1.02	0.97	1.08
NTC	0.68	1.16	1.12	1.01	-0.03	1.24	1.02	1.00	1.00	1.02
NCSE-02	0.65	1.01	1.10	1.00	0.00	1.09	1.03	1.14	0.99	0.91

Beams dimensions are generally conditioned by gravitational deflection limitation in the case of large spans and by seismic situation for short spans. For frames with large spans designed according to EC8 and NTC in high seismicity conditions, high depths are required for WB: up to 350mm, which can be considered as a cost-effective limit for such beams [21]. Moreover, in these frames very large depths of columns are required (up to 900mm), and it may not be possible to reduce them very much in higher storeys, because WB have also large width due to DLS limitation, and thus large widths of columns are required in order to satisfy width limitation of WB. The last condition not only determines depth of columns but also widths, so it may not be possible in most of cases to place "wall-type" columns integrated within non-structural walls. Hence, WBF may not be a feasible cost-effective solution for such situations.

Regarding chord rotations, $\theta_{ub,min,W/B}$ (mean 1.29) is lower than that obtained in both the specific case study and in the numeric analysis carried out in [12]. This is only due to lower values of $h_{b,W/D}$ (see Table 7), given that confinement contribution is higher for WB, similar in all three studies. Such lower $h_{b,W/D}$ is caused by the adoption in the actual procedure of small depths of DB for low spans, which is not possible to be pursued in buildings with different spans because h_b is usually similar for all the beams and depends on the largest span of the building. On the other hand, $\theta_{uc,min,W/B}$ (mean 1.07 for EC8 and NTC and 1.02 for NCSE-02) is almost only proportional to L_V, which is larger for WBF than for DBF (mean 1.25 times for EC8 and NTC, and only 1.09 times for NCSE-02). Still, such values correspond to limited favourable influence of L_V in the performance of WBF with respect to DBF (not higher than 8% on average). Contributions of h_c, ν and $f_{conf,c}$ are limited and balanced between each other (Table 7). Notwithstanding the large values of L_V in columns of WBF, the critical element is always a beam, while for DBF it occurs only in 6 cases of 36, all of them corresponding to EC8. The last is due to higher transverse reinforcement in columns rather than in NTC, causing larger $f_{conf,c}$ in EC8 (1.28) rather than in NTC (1.14), on average.

Regarding T_{el}, WBF whose critical condition is DLS show similar or even lower T_{el} than DBF, especially in the case of high seismicity. Again, the reason is the greater cantilever behaviour and the required high dimensions in upper storeys, as it is shown in previous specific case study EC8$_{50\text{-}50}$ and in Fig. 13: DBF can be designed to show almost linear deformed shape, thus each storey show similar IDR, slightly lower than the limit; conversely, in WBF the storeys in the central part rules completely the design, thus top and bottom storeys show higher stiffness than corresponding storeys of DBF. The above effect causes lower T_{el} but not as much as it would correspond to the increment of global stiffness, because the requirement of greater structural members can increase total mass up to 25%. Hence, almost all the buildings designed according to EC8 and NTC show $T_{el,W/D}\approx 1$, even for some cases in which DLS design is not the critical condition. Only two out of 12 buildings for each code show periods up to 25% higher for WBF than DBF. Conversely, most NCSE-02 couples (nine out of 12) show $T_{el,W/D}>1$ (1.14 on average) (Table 7). On the other hand, such different deformed shape implies values of Γ slightly lower for WBF with respect to DBF, as observed in the specific case study.

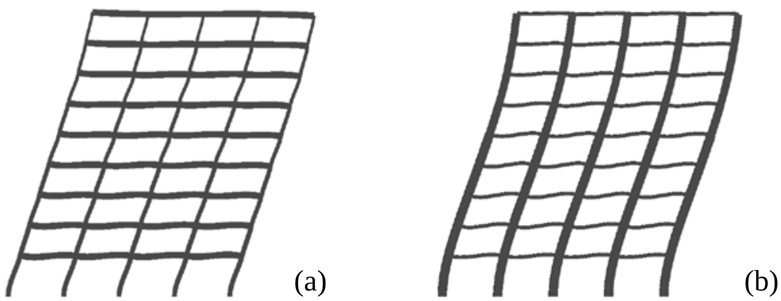

Fig. 13: Deformed shape for seismic situation of DBF (a) and WBF (b) with n=9 and L=5.5m designed to EC8 for a_{gR}=0.25g

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. 14. EC8 and NTC show mean values of $SF_{W/D}$ favourable to WBF (1.08 and 1.02, respectively, see Table 7), 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 $SF_{W/D}<1.0$. Dispersion of values is very low (coefficients of variation is approximately 10% for EC8 and NTC and only 5% for NCSE-02), thus likely correlations between q adopted in design and the use of one or another structural typology are suitable. In Fig. 14, “CB” or “CC” in the legend stands as reference for the first attainment of θ_u in WBF and DBF respectively, where C stands for first θ_u attainment in column, and B stands first θ_u attainment in beam, see also data in Table 6.

The cause of the satisfactory performance of WBF in EC8 and NTC (also 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.

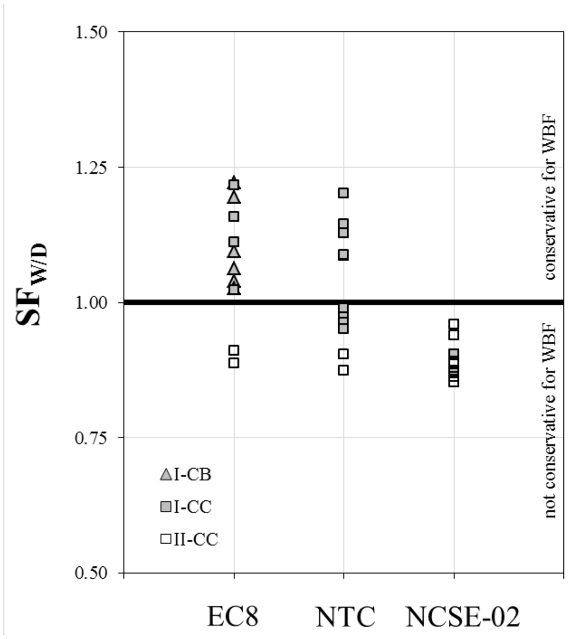

Fig. 14: Estimated relative performance between WBF and DBF ($SF_{W/D}$) for each code considering $H_{mec,W/D}$=$f_{K,sec}$=1.0 (I: $T_{el,W/D}$≈1, II: $T_{el,W/D}$>1; C: first θ_u attainment in column, B: first θ_u attainment in beam, in WBF anf DBF)

Thus, based on the results of the approximate assessment, within the actual framework of Italian NTC, the design of WBF for high ductility, adopting the corresponding q, can be allowed without any further design recommendation than the local geometric limitations in connections (already provided by the code in its present form). Regarding NCSE-02, it is not possible to state with sufficient confidence that q limitations for WBF can be removed within the actual framework, which does not provide any DLS design. It is worth noting that design strategies reflect always personal choices, and those conclusions are generalizable to the extent of the design choices (still reflecting common practice in European countries) adopted in this study.

Notwithstanding the mean satisfactory performance of WBF, even better than DBF, in some cases WBF may be not cost-effective or even a feasible structural system solution. However, the adoption of one or another system should be a decision of the designer, without any further penalisation of seismic code, as the reduction of q.

6. CONCLUSIONS

Different assessments of a typical Mediterranean 5-storey RC housing unit are carried out in order to evaluate the relative performance of wide-beam frames with respect to deep-beam frames, aimed at finding out whether the reduction of behaviour factor (*q*) for wide-beam frames, proposed by Italian and Spanish national seismic codes, are justified. Different design alternatives are considered: Eurocode 8, assuming diverse stiffness modelling approaches, and Spanish seismic code NCSE-02. The first code allows designing wide-beam frames in high ductility class, without any reduction of *q* with respect to deep-beam frames, while the Spanish code prescribes 50% reduction of *q* for wide-beam frames. Assessment is carried out by means of the N2 method. Finally, results are generalised through a simplified parametric assessment of a set of 72 high ductility frames corresponding to both wide and deep-beam frames designed according to Eurocode 8, Italian seismic code NTC, and NCSE-02, adopting similar *q* for both lateral load carrying systems.

Results show that, notwithstanding the lower local ductility of wide beams with respect to deep beams, global seismic capacity of wide-beam frames get substantially improved thanks to some effects increasing both their effective stiffness and their maximum deformation capacity. These causes, regarding wide-beam frames, can be organised in three groups:

1) <u>Mechanical causes.</u> Higher cantilever behaviour results in higher ultimate chord rotation at column bases, and beams show also higher ultimate chord rotation; both resulting in higher displacement capacity. Also, lower shear deformability of joints can result in higher effective stiffness.
2) <u>Code limitations.</u> Beam-to-column width limitation makes it hard to reduce column sections at upper storeys, and both design to Damage Limitation State (DLS) and corrections due to second order effects lead to greater column sections in the mid-low part of the building. These provisions cause higher overstrength in columns, leading to collapse mechanisms involving higher number of storeys, and cause also higher stiffness.
3) <u>Construction/executive practice causes.</u> As larger column sections are required in lower storeys, it is not possible for spliced bars to make important reduction of column sections when rising to the upper storeys.

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, within the design choices adopted in this study, it is suggested that design of wide-beam frames in high ductility class, adopting the corresponding *q*, could be allowed within the actual framework of NTC without any additional provision than local geometric limitations in connections (already prescribed in the current version). Regarding NCSE-02, it is not possible to state with sufficient confidence that *q* limitations for wide-beam frames can be removed within the actual framework as long as it does not provide any serviceability limit state design prescription (i.e. damage limitation). Further experimental research would be required aimed at a more accurate definition of the hysteretic behaviour of wide-beam frames in comparison with deep-beam frames, so that nonlinear dynamic analyses could be carried out and more reliable results of the relative performance between both types would be obtained.

ACKNOWLEDGEMENTS

The work presented has been developed in cooperation with Rete dei Laboratori Universitari di Ingegneria Sismica – ReLUIS – for the research program founded by the Dipartimento della Protezione Civile (2014-2018).

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