Engineering Structures 125 (2016) 107–123

Contents lists available at ScienceDirect

Engineering Structures

journal homepage: www.elsevier.com/locate/engstruct

Seismic performances and behaviour factor of wide-beam and deep-beam RC frames

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ARTICLE INFO

Article history:
Received 24 June 2015
Revised 17 May 2016
Accepted 22 June 2016

Keywords:
Wide beams
Deep beams
Seismic codes
Behaviour factor
Chord rotation
Ductility
Effective period
Collapse mechanism

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 European 5-storey RC housing unit is designed as WBF and DBF 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).

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 [1–12]. 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 in 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 [1–7,9,16–19]. 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. In [1], planar frames are

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http://dx.doi.org/10.1016/j.engstruct.2016.06.034
0141-0296/

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	cross-sectional 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 DBF with respect to WBF
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 resistance at beam end
M_{Rc}	moment 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 until 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
$S_{ae}\ (T)$	design elastic spectral acceleration
$S_{ae}\ (T)'$	design equivalent elastic spectral acceleration after corrections
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	portion of the beam width passing outside 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}	chord rotation capacity corresponding to the attainment of significant damage limit state
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

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 [20–23]. 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 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 [8] 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.