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A model for beam–column corner joints of existing RC frame subjected to cyclic loading

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ABSTRACT

Beam-to-column joints are commonly considered critic regions for RC frames subjected to earthquake actions. When designed for gravity loads only, beam-to-column corner joints strongly affect the global structural behaviour of a frame, and they can be cause of its collapse, as shown by recent earthquakes in Europe. In the paper, a component-based f.e. model for external beam-to-column joints is presented to simulate the seismic behaviour of r.c. existing structures designed without any capacity design criteria (smooth bars with hooked-end anchorages and with no transverse reinforcements in the joint). The joint deformation is modelled by means of two separate contributions, the shear deformation of the panel zone, and the rotation at the interface sections between the joint and the structural members, due to the reinforcing bars' slip within the joint core. The work focuses on the evaluation of the joint strength and stiffness, and it points out the importance of modelling the bar bond slip within the panel zone to describe the actual frame response. The component-based f.e. model is validated by experimental results of tests on beam-to-column corner joints realized according to the construction practice of the 1960s–1970s in Italy, thus confirming the effectiveness of the presented model for the assessment of existing structures.

1. Introduction

According to current seismic codes [1,2], new construction design is based on capacity design principles, ensuring a ductile structural behaviour. However, in Italy a large number of existing RC buildings have been designed for gravity loads only using plain bars, with joints characterized by inadequate reinforcement anchorage and no transverse reinforcement to confine the concrete core. As a result, past earthquakes have shown that, often, brittle mechanisms (thus more dangerous) lead to severe damage or even to the building collapse. The beam-to-column joint failure, due to the joint shear failure or to the bar slippage, is comprised among these mechanisms, which were often observed during the 2009 Italy earthquake in L'Aquila (Fig. 1) [3].

In the literature, several experimental research works devoted to studying the seismic performance of r.c. frames built before the 1970 and designed for gravity loads only may be found. Generally, tests were carried out on sub-assemblies with interior or exterior beam–column joints characterized by substandard reinforcing details. A critical review was carried out by Bonacci and Pantazopoulou [4,5] on the main parameter (axial load, joint confinement, concrete strength, bond behaviour, transverse beams) affecting the behaviour of interior joints [4] or exterior joints [5], based on the analysis of a test result database available at that time. Further critical discussion on the interpretation of the old joint behaviour is also presented in *fib* Bulletin n.24 [6]. However, most studies considered deformed bars bent in the joint [4,5,7–12] while few tests focused on joints with hooked-end plain bars and only some of them showed a joint shear failure [13–16].

The brittleness of this kind of beam-to-column joint was shown by tests carried out on a 2:3 scaled r.c. frame [13], designed with typical details of the Italian construction practice in the 1960s–1970s. The structures showed a significant damage in the exterior joints between the first and second floor, and the development of plastic hinges at the base of the columns at the ground floor. The development of a failure mechanism markedly different from that observed in the case of a rigid joint behaviour, for which a soft floor mechanism would be expected, was evident. This behaviour has been also confirmed during the recent L'Aquila earthquake of 2009 [3] (Fig. 1).

Despite the experimental evidence, the deformability of beam–column joints is commonly neglected by the designers even in

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Notation

a_c, a'_c	the strut depth in the column or in the join panel
A_g	area of the column section
A_s	area of longitudinal reinforcements
b_j	joint width
b_b	beam width
c	concrete cover
d	effective section depth
d_j	diagonal strut length
$d_{r,u}$	ultimate drift of sub-assembly
E_s	elastic modulus of steel
E_h	post-elastic tangent steel elastic modulus
f_a	mean compressive stress on the column section
f'_c	cylindrical compressive strength of concrete
f_t	average tensile ultimate rebar steel strength
f_y	average tensile yield rebar steel strength
h_b	beam depth
h_c	column depth
h_j	joint depth
h_s	the minimum value between the distance of the first stirrup outside the joint panel region to the column-joint interface section and a quarter of the column depth
k_1	constant relating the tensile principal stress to the compressive strength
k_h	stiffness of the spring associated to the hook slippage
k, k_B	stiffness of bar stress–slip relationship
K_φ	stiffness of the rotational spring associated to the bar slip
K'	elastic secant stiffness at peak strength of a tested sub-assembly
L_c	inter-storey height
L_b	half the distance between the column axes
L_{bn}	half of the free span of the beam
$M_{b,y}$	beam moment resistance at the bar yielding
$M_{c,y}$	column moment resistance at the bar yielding
M_{jh}	bending moment in the shear spring
N	axial force in the column
p_t	tensile principal stress strength
s_B	total slip of the rebars at joint-member interface
s_s	slip of straight rebars
s_h	slip of hook
T	bar tensile force
v_{jh}	shear stress resistance in the panel zone
V_{jh}	shear joint resistance
V_c	column shear action
$V_{jh,y}$	shear value which generates in the beam the bending moment $M_{b,y}$
w_u	the sum of the measured crack widths at incipient collapse
x	distance between the neutral axis and the outer compressive fiber
z	lever arm
γ_H, γ_V	horizontal and vertical rotations of the joint panel sides
γ_{jh}	panel shear distortion
$\gamma_{jh,y}$	joint panel distortion at the peak of the joint shear strength
$\gamma_{jh,u}$	ultimate shear distortion of the joint core at incipient collapse
ε_c, $\bar{\varepsilon}_c$	concrete strain and softened concrete strain in compression
ε_t	concrete strain in tension
ε_d, ε_r	strains along the principal directions
ε_v, ε_h	strains along the vertical and the horizontal direction
ε_s	steel bar strain
ε_{sy}, ε_{su}	bar steel strain at yielding and at failure
θ_j	the inclination of the diagonal strut
λ	normalized bar stress
σ_s	bar stress
σ_d	the average principal stress of concrete along the diagonal direction
ζ	softening coefficient of concrete
τ_E	bond stress related to plain bar adhesion in elastic bar stage
τ_y	bond stress related to plain bar adhesion beyond yield
ψ	$V_{jh}/V_{jh,y}$
φ_B	section rotation at joint-member interface due to bar slip within the joint
$\varphi_{B,y}$	section rotation at joint-member interface due to bar slip at bar yielding
φ_{jh}	rotation of the spring associated to the shear distortion
ϕ	bar diameter
Ω_y	tangential stress reduction factor related to plain bar adhesion beyond yield
a, b	constant values related to Ω_y

non-linear static and dynamic analyses: the nodal panel is assumed infinitely rigid and a verification of the strength of the joint is made only a posteriori. Moreover, the modelling of exterior RC joints as rigid leads to a non-conservative and unsafe response of the structure [17]. In the last two decades, different models have been proposed in order to evaluate the behaviour of beam–column joints subjected to cyclic loads [17–30]. Bonacci and Pantazopoulou [18] proposed a procedure based on the modified compression field theory (MCFT) [19] for calculating the complete response of the joint, in terms of shear stress against deformation, up to the failure. The formulation is based on equilibrium of stress resultant, compatibility of deformations within the joint, by taking into account the non-linear behaviour of materials. A similar approach was adopted also in the numerical studies of Lowes and Altoontash [24] who modelled the joint panel behaviour by means of a single finite element. However, these models were specifically developed for interior joints with transverse reinforcements and calibrated on the basis of experimental results on this type of joints. Furthermore, the MCFT may not entirely be suitable to predict joint shear behaviour if joint core without hoop reinforcements is considered. Other models predict the joint shear behaviour by assuming that the shear action is transferred by a confined concrete strut [29,30]. Nevertheless, due to their complexity these models have not provided a relatively simple tool for the seismic assessment of existing buildings, thus limiting their application to few case studies within research works. In a recent work of Sharma et al. [26], a straightforward model was proposed where two shear springs in the column portion of the joint and a rotational spring in the beam region relate the panel joint deformation to the principal stress on which the joint failure criteria is based. However, this model is developed for bent-in deformed bars and it does not separate the shear joint deformation from the bond–slip effect, which is fundamental to assess the non linear behaviour of r.c. structures subjected to seismic load [27,28].

This work aims at proposing a simple f.e. model for the nodal region of external joints in concrete frames designed for gravity loads only, focusing on poor details of the Italian construction practice in the 1960s–1970s (hooked-end smooth bars, and no stirrups in the joint region).

The proposed component-based joint model allows to evaluate separately the shear deformation of the panel zone and the added rotation at the interface sections between the joint and the