Comparison of Recent Code Provisions for Punching Shear Capacity of R/C Slabs Without Shear Reinforcement

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Abstract. In the last years the knowledge of the punching failure in R/C slabs increased thanks to several scientific studies. The progress obtained in this field is considerable, nevertheless achieved results are only taken into consideration by few Codes. The most updated code is the Model Code 2010, which adopted the Critical Shear Crack Theory (CSCT) for the punching shear capacity of R/C slab-column connections. At the same time, the EC2 formulation for punching is under revision, but the new formulation will not be available before three-four years. In this paper, the authors discuss main code provisions (ACI, current EC2, two proposals for revision of EC2, MC 2010, old Italian Recommendations) for punching shear capacity of R/C flat slabs without shear reinforcement. Through a parametric analysis, the authors investigate how each code takes into account the influence of main variables, which come into play in the punching phenomenon, on the evaluation of the punching capacity. Finally, results of each code formulation are compared with different literature experimental data.

Keywords: Punching failure · Flat-slab · CSCT · EC2 revision

1 Introduction

A flat-slab is a two-way structure that bears and transfers vertical loads to columns. This constructive system is often employed for multi-storey structures, used as offices and carparks, because it allows to increase the span between columns reducing the floor thickness. In other words it offers a greater flexibility in the choice of the internal layout allowing to reduce the building height.

Starting from fifties, for constructive reasons, the flat-slab deck are usually built without capitals. In this way the punching failure becomes predominant with respect to the flexural failure. The punching failure is due to a shear-stress concentration along the column's perimeter and it is characterized by a collapse surface with a truncated cone shape. This type of failure is rather brittle and it occurs without any warning sign. It is a local mechanism but it could bring to a progressive collapse of the entire building. For these reasons the punching issue is primary in the design of R/C flat-slab building. In the last years the knowledge of this failure mechanism increased thanks to several scientific studies. However, not all scientific results are adopted by international codes.

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The most updated code is the Model Code 2010 (fib 2010), which is grounded on the Critical Shear Crack Theory (CSCT) (Muttoni 2008). Furthermore, the EC2 formulation is under revision, but new provisions will not be available before three-four years.

2 Code Provisions

In this section the authors discuss main code provisions for the determination of the punching strength in R/C flat-slabs without shear reinforcement. In particular a parametric analysis of main variables that come into play in the punching failure, is presented. Models can be divided into two categories: empirical or mechanical. With regards to the first category, ACI 318 (ACI Committee 318 2014) and current version of EC2 (CEN 2004) are dealt with, while for the second category the Model Code 2010 and two proposals for revision of EC2 are dealt with. The first proposal is empirical and it has been developed at the Institute of Structural Concrete of RWTH Aachen University in Germany (Hegger et al. 2016). The second proposal is grounded on the CSCT (Critical Shear Crack Theory) and it has been developed at the EPFL in Switzerland (Muttoni et al. 2016). Furthermore, old Italian Recommendations (DM96 1996) are also analysed, as they could turn out to be a useful tool for preliminary design, although they can no longer be utilized for design purposes.

2.1 ACI 318 - 2014

ACI 318 formulation for punching of flat slabs is strictly empirical and its application is very easy. For slabs without shear reinforcement, the punching strength is the smallest of the three following values:

$$V_{ACI,a} = \frac{1}{6} \cdot \left(1 + \frac{2}{\beta}\right) \cdot \sqrt{f_{ck}} \cdot b_{0,ACI} \cdot d \tag{1}$$

$$V_{ACI,b} = \frac{1}{12} \cdot \left(\frac{\alpha_S \cdot d}{b_{0,ACI}} + 2\right) \cdot \sqrt{f_{ck}} \cdot b_{0,ACI} \cdot d \tag{2}$$

$$V_{ACI,c} = \frac{1}{3} \cdot \sqrt{f_{ck}} \cdot b_{0,ACI} \cdot d \tag{3}$$

where:

β is the ratio between long side and short side of the column;
f_{ck} is the characteristic compressive strength of concrete in MPa;
$b_{0,\,ACI}$ is the control perimeter set at *d/2* from the border of the support region in mm;
d is the effective depth of the slab in mm;
α_s holds 40 for inner column, 30 for edge column and 20 for corner column;

Formulas (1) and (2) were developed to account for non-square columns and different positions of the column (inner, edge or corner), respectively.

2.2 EC2-2004

EC2 formulation is also strictly empiric, but unlike ACI 318 it takes into account the flexural reinforcement ratio and size effects. Thus, the punching strength of flat slabs without shear reinforcement is given as:

$$V_{EC2} = \max\left(V'_{EC2}; V''_{EC2}\right) \tag{4}$$

where:

$$V'_{EC2} = C_{Rd,c} \cdot \frac{b_{0,EC2}}{\beta} \cdot d \cdot k \cdot (100 \cdot \rho_l \cdot f_{ck})^{\frac{1}{3}} \tag{5}$$

$$V''_{EC2} = v_{\min} \cdot b_{0,EC2} \cdot d \tag{6}$$

$$C_{Rd,c} = \frac{0.18}{\gamma_c} \tag{7}$$

$b_{0,EC2}$ is the control perimeter set at *2d* from the border of the support with circular corners;

β is a coefficient that takes into account the eccentricity of the shear reaction; for structures where the lateral stability does not depend on the frame action between slabs and columns, and adjacent spans do not differ in length by more than 25%, following approximate values for β can be used:

- $\beta = 1.15$ for inner columns
- $\beta = 1.4$ for edge columns
- $\beta = 1.5$ for corner columns

d is the effective depth of the slab in mm;
k is a factor accounting for the size effect:

$$k = 1 + \sqrt{\frac{200}{d}} \leq 2 \tag{8}$$

ρ_l is the flexural reinforcement ratio; if ρ_l is greater than 2%, ρ_l is assumed equal to 0.02:

$$\rho_l = \sqrt{\rho_x \cdot \rho_y} \leq 0.02 \tag{9}$$

(ρ_x, ρ_y: reinforcement ratio in *x* and *y* direction)

f_{ck} is the characteristic compressive strength of concrete in MPa;
v_{min} is the minimum punching shear strength:

$$v_{\min} = 0.035 \cdot k^{3/2} \cdot {f_{ck}}^{1/2} \tag{10}$$

2.3 MC 2010 (MC)

Model Code 2010, like SIA 262–2003 (Swiss Society of Engineers and Architects 2003), is grounded on the Critical Shear Crack Theory (CSCT). The punching failure depends on the slab rotation. For slabs without shear reinforcement, the punching strength is defined as:

$$V_{MC} = k_{\psi} \cdot \frac{\sqrt{f_{ck}}}{\gamma_c} \cdot k_e \cdot b_{0,MC} \cdot d_v \tag{11}$$

where:

k_{ψ} depends on the slab rotation:

$$k_{\psi} = \frac{1}{1.5 + 0.9 \cdot \psi \cdot d \cdot k_{dg}} \leq 0.6 \tag{12}$$

Ψ is the slab rotation, defined in the following, depending on the approximation level;
k_{dg} is the factor accounting for the influence of aggregate size, defined as:

$$k_{dg} = \frac{32}{16 + d_g} \geq 0.75 \tag{13}$$

d_g is the maximum aggregate size in mm;
γ_c is the partial safety factor for concrete material properties
k_e is a coefficient that takes into account the concentration of shear forces due to moment transfer between the slab and supported area. In cases where the lateral stability does not depend on frame action of slabs and columns, and adjacent spans do not differ in length by more than 25%, following approximated values may be adopted:

- $k_e = 0.9$ for inner columns
- $k_e = 0.7$ for edge columns
- $k_e = 0.65$ for corner columns
- $k_e = 0.75$ for corners of walls

$b_{0,MC}$ is the control perimeter set at a distance of d_v from the border of the support region with circular corners in mm;
d_v is the effective depth of the slab accounting for the effective level of the support region ($d_v \leq d$).

In this provision there are different levels of approximation. For each level a different expression of the slab rotation is defined. The rotation has to be calculated along the two main directions of the reinforcement.

Level I of approximation (LoA,I): "Fast pre-dimensioning"
For a regular flat slab designed according to an elastic analysis without significant redistribution of internal forces, a safe estimate of the rotation failure is:

$$\psi = 1.5 \cdot \frac{r_s}{d} \cdot \frac{f_{yd}}{E_s} \tag{14}$$

where:

r_s denotes the distance between the point where the radial bending moment is zero, and the support axis. For regular flat slabs where the ratio of spans is between 0.5 and 2, r_s can be calculated as the maximum of following values:

$$r_{s,x} \cong 0.22 \cdot L_x \tag{15}$$

$$r_{s,y} \cong 0.22 \cdot L_y \tag{16}$$

d is the effective depth of the slab;
f_{yd} is the design yield stress of the flexural reinforcement;
E_s is the Young modulus of the flexural reinforcement.

Level II of approximation (LoA,II): "Typical design of new structures"
In case where significant bending moment redistribution is considered in the design, the slab rotation can be calculated as:

$$\psi = 1.5 \cdot \frac{r_s}{d} \cdot \frac{f_{yd}}{E_s} \cdot \left(\frac{m_{Ed}}{m_{Rd}}\right)^{1.5} \tag{17}$$

where:

m_{Rd} is the average flexural strength per unit length in the support strip (for the considered direction);
m_{Ed} is the average moment per unit length for calculation of the flexural reinforcement in the support strip (for the considered direction);

– for inner columns:

$$m_{Ed} = V_{Ed} \cdot \left(\frac{1}{8} + \frac{|e_{u,i}|}{2 \cdot b_s} \right) \tag{18}$$

– for edge columns:

when calculations are made considering the tension reinforcement parallel to the edge:

$$m_{Ed} = V_{Ed} \cdot \left(\frac{1}{8} + \frac{|e_{u,i}|}{2 \cdot b_s} \right) \geq \frac{V_{Ed}}{4} \tag{19}$$

or perpendicular to the edge:

$$m_{Ed} = V_{Ed} \cdot \left(\frac{1}{8} + \frac{|e_{u,i}|}{2 \cdot b_s} \right) \tag{20}$$

– for corner columns:

$$m_{Ed} = V_{Ed} \cdot \left(\frac{1}{8} + \frac{|e_{u,i}|}{2 \cdot b_s} \right) \geq \frac{V_{Ed}}{2} \tag{21}$$

where:

$e_{u,i}$ is the eccentricity of the shear force resultant;
b_s is the width of the support strip for calculating m_{Ed}, defined as: m_{Ed}, defined as:

$$b_s = 1.5 \cdot \sqrt{r_{s,x} \cdot r_{s,y}} \leq L_{\min} \tag{22}$$

Level III of approximation (LoA,III): "For special design cases or for analysis of existing structures"

This level of approximation is recommended for irregular slabs or for flat slabs where the ratio of span lengths is not included between 0.5 and 2.

The coefficient 1.5 used in previous equations can be replaced by 1.2 if:

- r_s is calculated using a linear elastic (un-cracked) model
- m_{Ed} is calculated from a linear elastic (un-cracked) model as the average value of the moment for design of flexural reinforcement over the width of the support strip b_s
- b_s can be calculated as in level II, taking $r_{s,x}$ and $r_{s,y}$ as the maximum value in the investigated direction.

Level IV of approximation (LoA,IV): "For special design cases or for more detailed assessment of existing structures"
The rotation ψ can be calculated on the basis of a non-linear analysis of the structure and accounting for cracking, tension-stiffening effects, yielding of the reinforcement and other non-linear effects relevant for providing an accurate assessment of the structure.

2.4 RWTH Proposal for Revision of EC2

In the proposal for revision of EC2 developed at RWTH Aachen (Hegger et al. 2016), the punching shear strength is calculated similarly to the current EC2 formulation. The only substantial difference is given by the presence of the coefficient k_λ accounting for the influence of column size and shear slenderness:

$$V_{Rd} = \max(V'_{RWTH}; V''_{RWTH}) \tag{23}$$

$$V'_{RWTH} = C_{Rd,c} \cdot \frac{b_{0,RWTH.}}{\beta} \cdot d_v \cdot k_d \cdot k_\lambda \cdot (100 \cdot \rho_l \cdot f_{ck})^{1/3} \tag{24}$$

$$V''_{RWTH} = v_{\min} \cdot b_{0,RWTH} \cdot d_v \tag{25}$$

where:

$$C_{Rd,c} = \frac{1.8}{\gamma_c} \tag{26}$$

k_d is a coefficient accounting for the influence of size effects:

$$k_d = \left(1 + \frac{d_v}{200}\right)^{-\frac{1}{2}} \tag{27}$$

k_λ is a coefficient accounting for the influence of column size and shear slenderness:

$$k_\lambda = \left(\frac{a_\lambda}{d_v} \cdot \frac{u_0}{d_v}\right)^{-\frac{1}{5}} \tag{28}$$

a_λ/d_v is the shear span-depth ratio;

a_λ is the distance between the edge of the loaded area and the line of contraflexure; for non-symmetric cases a_λ can be calculated as:

$$a_\lambda = \sqrt{a_{\lambda,y} \cdot a_{\lambda,z}} \tag{29}$$

u_0/d_v is the specific column perimeter;
ρ_l is the flexural reinforcement ratio:

$$\rho_l = \sqrt{\rho_y \cdot \rho_z} \leq \min\left(0.02;\ 0.5 \cdot \frac{f_{cd}}{f_{yd}}\right) \tag{30}$$

f_{ck} (f_{cd}) is the characteristic (design) compressive cylinder strength of concrete in MPa;
f_{yd} is the design yield stress of steel in MPa;
d_v is the shear-resisting effective depth of the control section in mm;
$b_{0,RWTH}$ is the control perimeter set at 0.5d from the border of the support region with circular corners.

As regards ν_{min} and β, they assume the same values as in EC2-2004.

2.5 EPFL Proposal for Revision of EC2

The proposal for revision of EC2 developed at EPFL in Switzerland (Muttoni et al. 2016) is a closed-form formulation based on the Critical Shear Crack Theory (Muttoni 2008).

The punching shear strength is calculated as:

$$V_{EPFL} = \frac{1}{\gamma_c} \cdot \frac{b_{0,EPFL}}{\beta} \cdot d_v \cdot k_u \cdot \left(100 \cdot \rho_l \cdot f_{ck} \cdot \frac{d_{dg}}{r_s}\right)^{1/3} \tag{31}$$

$$V_{EPFL} \leq \frac{0.55}{\gamma_c} \cdot \sqrt{f_{ck}} \cdot d_v \cdot b_{0,EPFL} \tag{32}$$

where:

$$k_u = 8 \cdot \sqrt{\beta \cdot \frac{d}{b_{0,EPFL}}} \geq 2.0 \tag{33}$$

β is a parameter accounting for concentrations of shear forces due to acting moment transfer between slab and supported area; in cases where the lateral stability does not depend on frame action of slabs and columns and where adjacent spans do not differ in length more than 25%, following approximated values may be adopted:

- $\beta = 1.15$ for inner columns
- $\beta = 1.4$ for edge columns
- $\beta = 1.5$ for corner columns
- $\beta = 1.35$ for corners of walls

f_{ck} is the characteristic compressive cylinder strength of concrete in MPa;
$b_{0,EPFL}$ is the control perimeter set at a distance of $d_v/2$ from the border of the support region with circular corners in mm;
d_v is the effective depth of the slab accounting for the effective level of the support region ($d_v \leq d$);
ρ_l is the flexural reinforcement ratio limited to the maximum of 4%:

$$\rho_l = \sqrt{\rho_{l,x} \cdot \rho_{l,y}} \leq 0.04 \tag{34}$$

$\rho_{l,x}$ and $\rho_{l,y}$ should be calculated as mean values over the width of the support strip b_s defined as:

$$b_s = 1.5 \cdot r_s \leq L_{\min}[= \min(L_x, L_y)] \tag{35}$$

r_s denotes the distance between the point, where the radial bending moment is zero, and the support axis. The value of r_s may be calculated using a linear elastic (un-cracked) model. Otherwise, for regular flat slabs where the lateral stability does not depend on frame action between the slabs and the columns, and where the adjacent spans do not differ in length by more than 25%, can be approximated to $0.22\ L_x$ or $0.22\ L_y$ for the x- and y- directions, respectively:

$$r_s = \sqrt{r_{s,x} \cdot r_{s,y}} \geq d \tag{36}$$

d_{dg} is a coefficient taking account of concrete type and its aggregate properties:

- $d_{dg} = 32$ for normal weight concrete
- $d_{dg} = 16$ for light weight concrete

2.6 Old Italian Recommendations

Old Italian Recommendations (DM96 1996) for punching of flat slabs refers to a very simple mechanical model, where the punching capacity only depends on the concrete tensile strength. For slabs without shear reinforcement, the punching strength is given as:

$$V_{DM96} = 0.5 \cdot f_{ctk} \cdot b_{0,DM96} \cdot h \tag{37}$$

where:

$b_{0,\ DM96}$ is the control perimeter set at $d/2$ from the border of the support region;
h is the slab thickness;
f_{ctk} is the characteristic tensile strength of concrete.

3 Parametric Analysis

In this section the authors perform a parametric analysis to investigate the influence of different parameters on the punching strength predicted using different codes. In particular, the variation of the specific punching strength v_R is calculated at varying one of following parameters:

f_c concrete compressive strength
f_y steel yield strength
ρ flexural reinforcement ratio
b_0/d ratio between control perimeter and effective depth of the slab
d slab's effective depth
r_s/d shear span-depth ratio

Data chosen for the parametric analysis are the same used in (Muttoni 2008), which, for each investigated parameter, refers to specimens with different geometry and/or mechanical data. Results of the parametric analysis are summed up in following diagrams (Figs. 1, 2, 3, 4, 5 and 6) in terms of the specific punching strength v_R:

$$v_R = \frac{V_R}{\sqrt{f_{ck}} \cdot d \cdot b_0} \tag{38}$$

where V_R is the punching strength measured in kN and v_R is in $\sqrt{}$MPa. The coefficient β is taken equal to one, as no eccentricity is considered (null bending moment); furthermore, γ_C = 1.0 and mean values of material strengths are used instead of characteristic values: $f_c = f_{cm}$ and $f_y = f_{ym}$.

The parametric analysis highlights the limited capacity of some formulations to predict the punching capacity of R/C slabs without shear reinforcement. From previous graphs it results that ACI, MC-LoA,I and DM96 (1996) do not take into account the influence of some parameters on the punching strength.

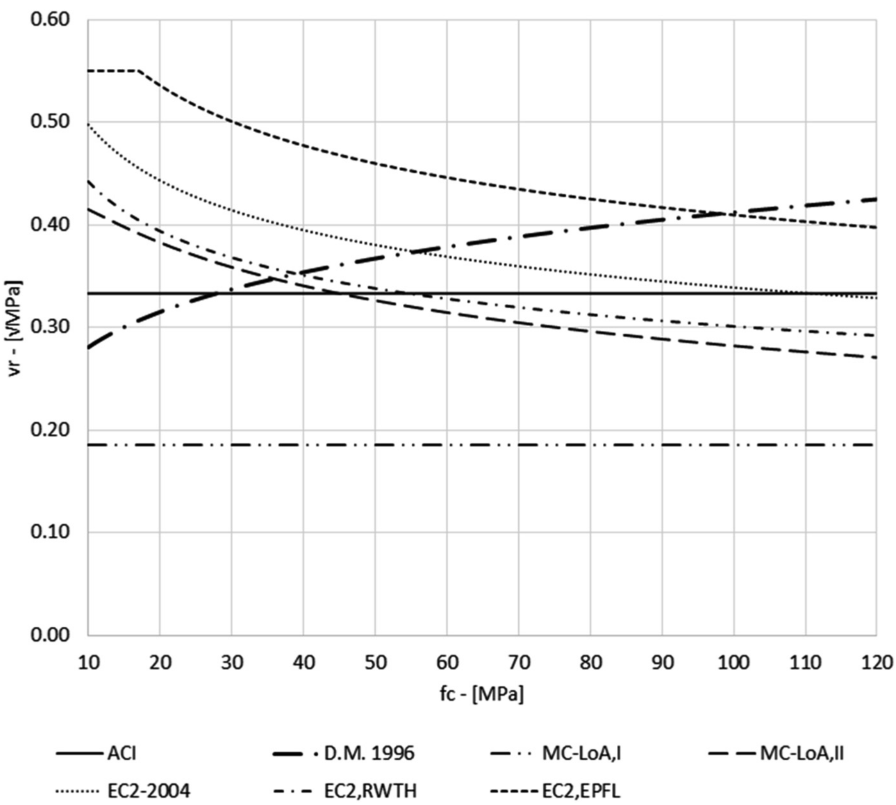

Fig. 1. Influence of concrete compressive strength on v_R (d = 98 mm, h = 125 mm, r_c = 75 mm, r_s = 850 mm, ρ = 0.8%, d_g = 10 mm, f_y = 550 MPa)

In particular, from all graphs it results that ACI provides a constant specific punching strength v_r equal to $0.33\ \sqrt{MPa}$, except for small values of f_y (Fig. 2) and ρ (Fig. 3) and high values of b_0/d (Fig. 4). ACI expression could give unsafe values of v_r for slender slabs and large columns ($r_s/d > 5$ and $b_0/d > 12$), in which cases other codes provide lower strength values. As regards the influence of concrete compressive strength, differently from other codes, DM96 (1996) gives increasing values of the specific punching strength at increasing the concrete strength (Fig. 1).

MC-LoA,I always provides a lower specific punching strength than other codes. This result is in agreement with the purpose of the level of approximation I, that is preliminary design based on safe hypotheses leading to quick and simple analyses. However it seems that MC-LoA,I is too much conservative.

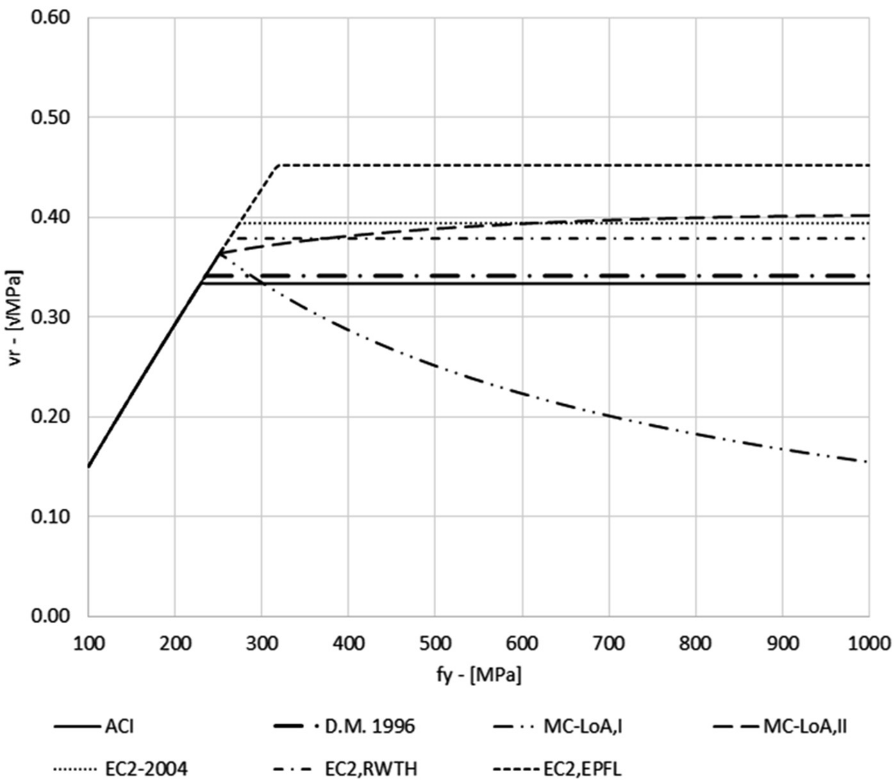

Fig. 2. Influence of steel yield strength on v_R (d = 114 mm, h = 152 mm, r_c = 162 mm, r_s = 982 mm, ρ = 1.15%, f_c = 24.6 MPa, d_g = 38.1 mm)

The current formulation of Eurocode 2 takes into consideration the influence of almost all variables on the punching strength, except for the slab slenderness (r_s/d). To overcome this lack, both proposals for revision of EC2 introduce the influence of the slab slenderness. In the RWTH proposal, the slenderness is taken into consideration through the coefficient k_λ, which includes the shear span-depth ratio a_λ/d_v ($a_\lambda \equiv r_s$), in the EPFL proposal it is considered including r_s in the expression of the punching strength.

Results given by these two proposals at varying the slab slenderness are very similar (Fig. 6). Results given by the RWTH proposal and MC-LoA,II are also very similar, but v_R values are lower than the EPFL proposal, in particular at varying the concrete strength and the steel yield strength (Figs. 1 and 2). Nevertheless, the assessment of the reliability of different formulations requires the comparison with experimental data, which is presented in following Sect. 4.

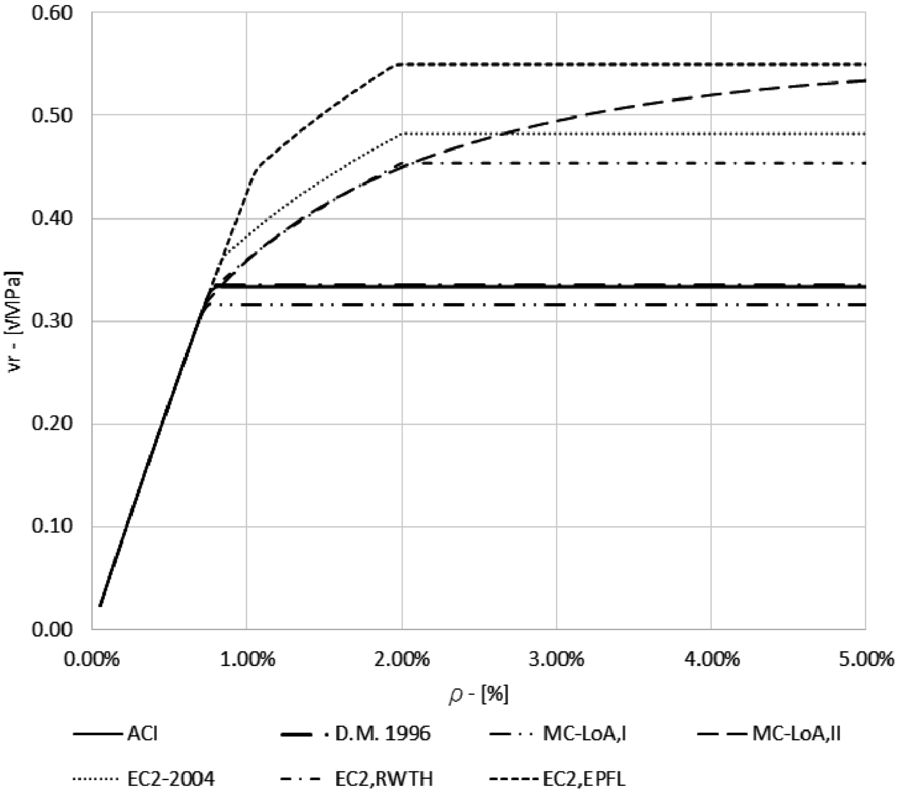

Fig. 3. Influence of flexural reinforcement ratio on v_R (d = 114 mm, h = 152 mm, r_c = 162 mm, r_s = 982 mm f_c = 22 MPa, d_g = 25.4 mm, f_y = 325 MPa)

4 Comparison with Experimental Data

In this section, values of the punching strength predicted using different code provisions are compared with literature experimental data. Following codes are taken into consideration for the comparison: EC2-2004, RWTH proposal and EPFL proposal for revision of EC2, and old Italian Recommendations (DM96 1996).

Several experimental campaigns are considered for a total of 173 slab specimens. Tests have been taken from a wider database, choosing those tests performed on specimens with similar geometry and reinforcement layout. All tests concern square isolated slabs equipped with uniformly distributed flexural reinforcement oriented along main axes *x/y*. The load is transmitted through line or points which react to the column load, along a circular or rectangular arrangement. Columns have square or circular cross-sections.

Results of the comparison are expressed in terms of the ratio V_{test}/V_{th} between the experimental (V_{test}) and the predicted value (V_{th}) of the punching strength, adopting all safety coefficients equal to one and using mean values of material strengths. The average

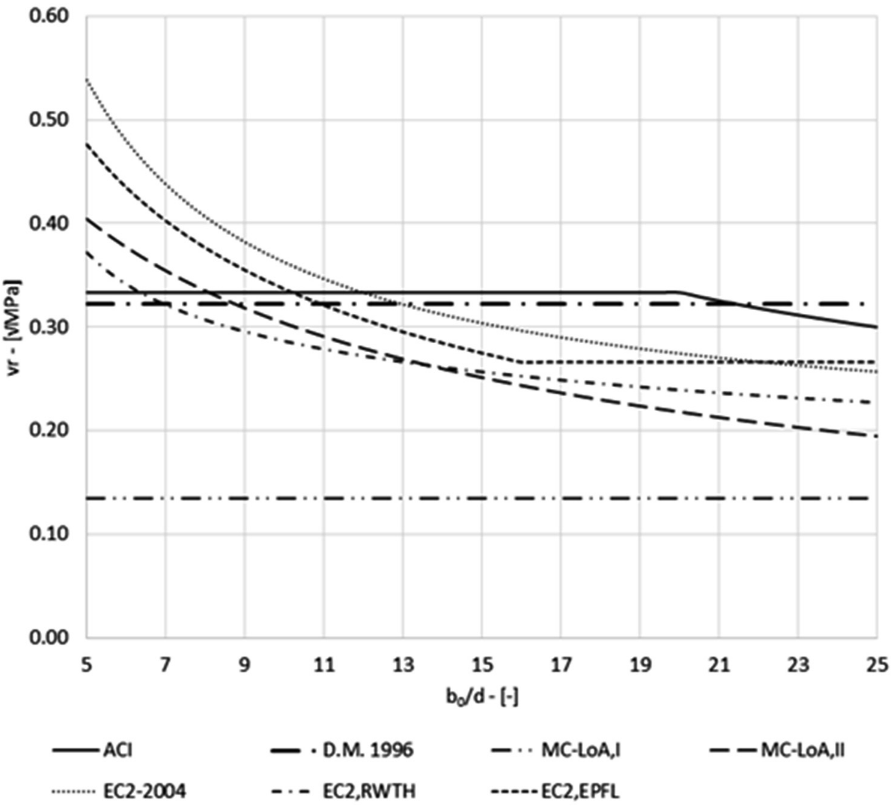

Fig. 4. Influence of punching shear control perimeter on v_R (d = 200 mm, h = 240 mm, r_s = 1270 mm, ρ = 0.8%, f_c = 33 MPa, d_g = 18 mm, f_y = 493 MPa)

value μ, the coefficient of variation *CV* and the 5%-quantile of the ratio V_{test}/V_{th} are listed in Table 1.

The current EC2 formulation gives an average value of the ratio V_{test}/V_{th} equal to 1.27, a *CV* equal to 0.33, and a 5%-quantile of 0.85. The unitary value of the ratio V_{test}/V_{th} corresponds to the 23%-quantile, meaning that in almost a quarter of all analysed cases the current EC2 formulation overestimates the experimental punching strength.

As regards the RWTH proposal for revision of EC2, the average of the ratio V_{test}/V_{th} is equal to 1.27 like EC2-2004, while the *CV* (=0.21) is the lowest among all formulations, and the 5%-quantile attains one of the highest values (0.93). As the unitary value of the ratio V_{test}/V_{th} corresponds to the 13%-quantile, in only 13% of analysed cases the RWTH proposal overestimates the experimental punching strength.

In summary, the RWTH proposal provides the same average strength of the current EC2 code (V_{test}/V_{th} = 1.27), but it gives a much lower *CV* value. Therefore, this proposal appears to be an improvement of the current EC2 code because, as results are less scattered.

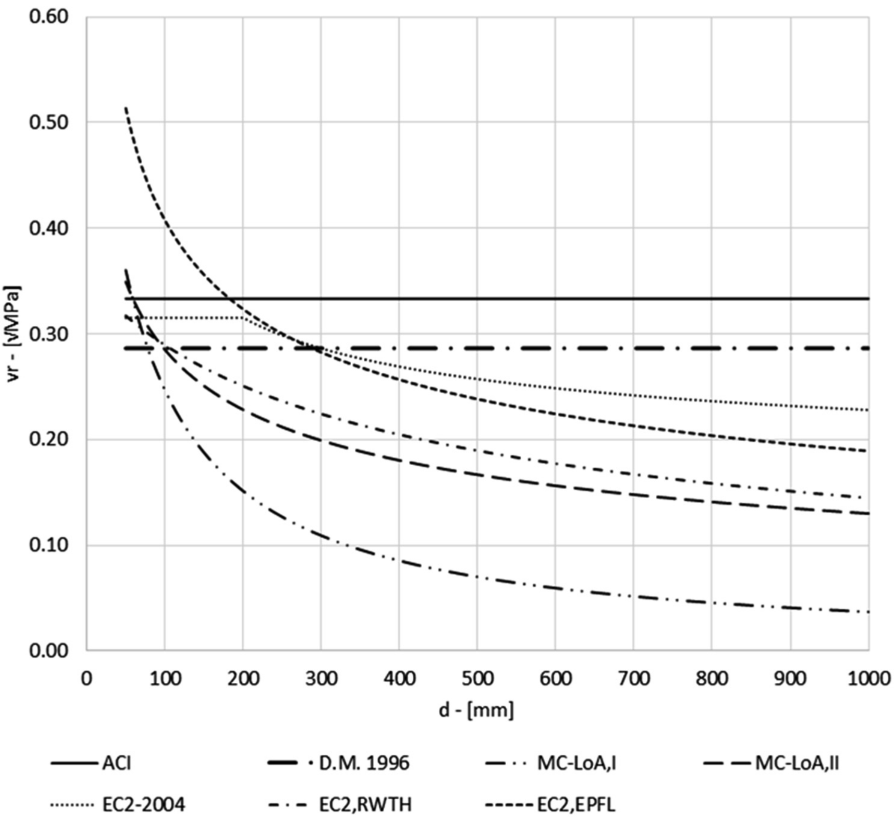

Fig. 5. Influence of effective depth on v_R ($h = 1.08d$, $r_c = 0.71d$, $r_s = 6.9d$, $\rho = 0.33\%$, $f_c = 30.5$ MPa, $d_g = 16$ mm, $f_y = 548$ MPa)

The EPFL proposal for revision of EC2 provides the best estimate of the average ratio V_{test}/V_{th}, equal to ≅1.00, and the *CV* value (=0.27) lies between RWTH proposal and current EC2 values. The 5%-quantile is quite low (0.65), leading to a higher probability of overestimation of the experimental strength than other codes. In fact, the unitary value of V_{test}/V_{th} corresponds to 56%-quantile of all 173 considered cases.

Finally, DM96 (1996) gives the worst results in terms of average value and *CV* of the ratio V_{test}/V_{th}, equal to 1.60 and 0.36, respectively, and the 5%-quantile is the highest (0.94). Strength predictions provided by this code seem too conservative, as the unitary ratio V_{test}/V_{th} corresponds to 9.8%-quantile, meaning that only in less than 10% of all studied cases DM96 (1996) underestimates the experimental punching strength. For these reasons, after comparisons with further experimental results, it could still be adopted in preliminary design since it is conservative and very easy to use, although it can no longer be used for accurate design of new structures or assessing existing ones.

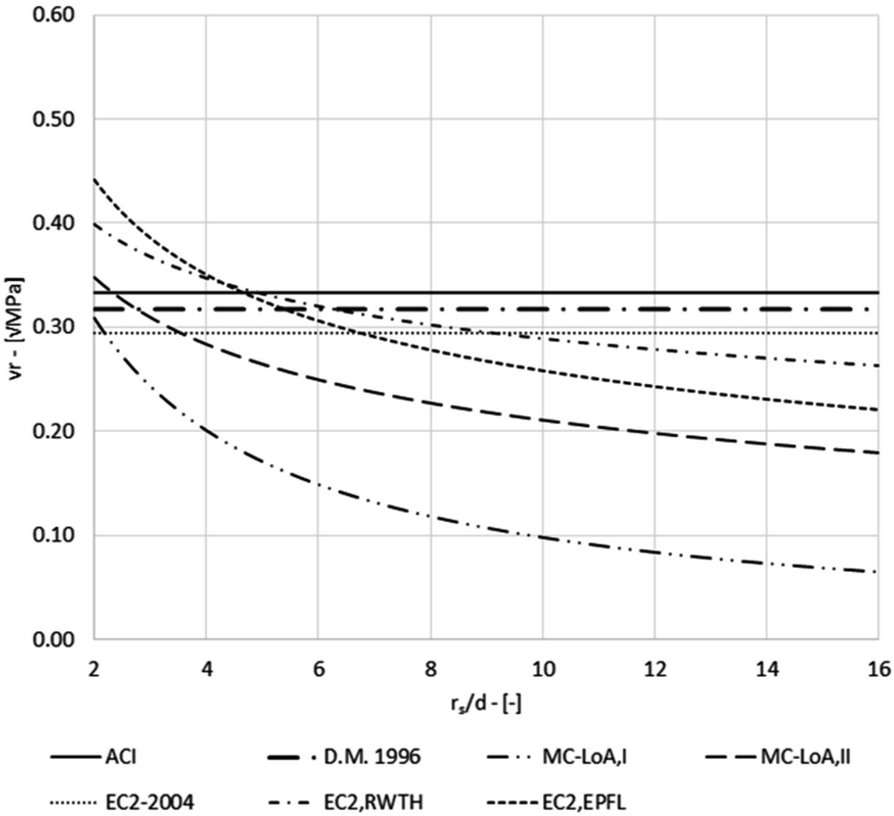

Fig. 6. Influence of slenderness on v_R (d = 300 mm, h = 360 mm, r_c = 300 mm, ρ = 0.5%, f_c = 30 MPa, d_g = 25 mm, f_y = 550 MPa)

Table 1. Comparison of literature test results with strengths predicted using code provisions: average μ, coefficient of variation *CV* and 5%-quantile of the ratio V_{test}/V_{th}.

Reference	Year	No. tests	EC2 2004 Current code			RWTH proposal for revision of EC2			EPFL proposal for revision of EC2			Old Italian Recomm. (DM96 1996)		
			µ	CV	5%-q	µ	CV	5%-q	µ	CV	5%-q	µ	CV	5%-q
Banthia et al.	1995	1	1.23	-	-	1.10	-	-	0.63	-	-	0.92	-	-
Broms	1990	1	1.00	-	-	1.12	-	-	0.89	-	-	1.34	-	-
Criswell	1974	4	1.02	0.10	0.93	1.08	0.13	0.96	0.72	0.24	0.57	1.12	0.25	0.86
Elstner & Hognestad	1956	22	1.04	0.11	0.94	1.08	0.11	0.94	0.94	0.15	0.75	1.50	0.20	1.01
Etter et al.	2009	1	1.02	-	-	1.30	-	-	1.03	-	-	1.38	-	-
Forssell & Holmberg	1946	7	1.32	0.06	1.22	1.39	0.06	1.30	1.14	0.06	1.05	2.38	0.06	2.22
Ghannoum	1998	3	1.03	0.10	0.96	1.13	0.10	1.05	0.92	0.10	0.86	0.93	0.05	0.88

(*continued*)

Table 1. (*continued*)

Reference	Year	No. tests	EC2 2004 Current code			RWTH proposal for revision of EC2			EPFL proposal for revision of EC2			Old Italian Recomm. (DM96 1996)		
			μ	CV	5%-q	μ	CV	5%-q	μ	CV	5%-q	μ	CV	5%-q
Graf	1938	2	0.97	-	-	1.03	-	-	0.82	-	-	1.77	-	-
Guandalini	2005	7	1.03	0.12	0.92	1.23	0.07	1.16	0.96	0.05	0.92	1.21	0.29	0.84
Lee et al.	2009	1	1.15	-	-	1.20	-	-	0.92	-	-	1.25	-	-
Li	2000	6	1.07	0.19	0.82	1.12	0.14	0.91	0.77	0.18	0.58	1.28	0.14	1.05
Lips	2012	5	0.98	0.09	0.88	1.18	0.04	1.13	0.94	0.08	0.86	1.43	0.10	1.24
Long & Masterson	1974	1	1.14	-	-	1.00	-	-	0.89	-	-	1.48	-	-
Manterola	1966	9	0.91	0.11	0.79	1.11	0.14	0.91	0.84	0.24	0.51	1.22	0.29	0.86
Marzouk & Jiang	1997	1	1.13	-	-	1.19	-	-	0.79	-	-	1.00	-	-
Marzouk & Hussein	1991	5	1.29	0.07	1.22	1.38	0.05	1.32	1.06	0.08	0.98	1.47	0.09	1.33
Matthys & Taerwe	2000	4	1.66	0.16	1.64	1.61	0.13	1.54	1.28	0.17	1.11	1.83	0.24	1.45
McHarg et al.	2000	1	1.07	-	-	1.17	-	-	0.96	-	-	1.13	-	-
Moe	1961	7	1.28	0.08	1.14	1.35	0.07	1.24	1.08	0.07	0.99	1.56	0.08	1.39
Mokhtar et al.	1985	1	1.07	-	-	1.13	-	-	0.78	-	-	1.22	-	-
Oliveira et al.	2000	2	1.18	-	-	1.30	-	-	0.94	-	-	1.27	-	-
Ospina et al	2003	1	1.06	-	-	1.12	-	-	0.90	-	-	1.03	-	-
Pilakoutas et al.	2003	1	1.34	-	-	1.47	-	-	1.11	-	-	1.60	-	-
Rankin & Long	1987	23	1.51	0.09	1.32	1.36	0.09	1.20	0.99	0.23	0.68	1.64	0.25	1.13
Regan	1984	29	1.73	0.44	1.09	1.54	0.27	1.13	1.27	0.33	0.90	2.07	0.47	1.33
Sistonen et al.	1997	10	1.28	0.06	1.18	1.39	0.07	1.28	0.85	0.08	0.78	1.59	0.08	1.46
Swamy & Ali	1983	2	1.13	-	-	1.23	-	-	0.93	-	-	1.07	-	-
Taylor and Hayes	1965	8	0.98	0.10	0.85	0.96	0.08	0.88	1.00	0.15	0.81	1.81	0.15	1.51
Timm	2003	3	1.05	0.01	0.97	1.03	0.09	0.95	0.99	0.11	0.892	1.72	0.09	1.57
Urban	1994	2	1.19	-	-	1.22	-	-	0.89	-	-	1.25	-	-
Widianto et al.	2010	1	0.84	-	-	0.90	-	-	0.81	-	-	0.68	-	-
Yamada et al.	1992	2	0.94	-	-	0.98	-	-	0.80	-	-	1.48	-	-
All tests		173	1.27	0.33	0.85	1.27	0.21	0.93	1.00	0.26	0.65	1.60	0.36	0.94

5 Conclusions

In this paper the authors presented and discussed formulations of main international codes (current EC2 version, ACI, MC2010, two proposals for revision of EC2), together with old Italian Recommendations, for the evaluation of the punching capacity of R/C slabs without transverse reinforcement. Firstly, formulations of different codes have been compared among them to evaluate if and how each of them takes into account the influence of different geometrical and mechanical parameters on the punching strength. Successively, values of the punching strength predicted using different codes have been compared with results of more than 170 literature experimental tests on specimens with similar geometry, reinforcement layout and load spatial distribution.

Results of the parametric analysis highlight that more recent formulations, that is to say Model Code 2010 and proposals for revision of EC2 are able to take into consideration all geometrical and mechanical variables which control the punching failure.

The current EC2 formulation does not take into account the influence of the slab's slenderness (r_s/d) on the punching capacity, giving unsafe results for high values of it ($r_s/d > 8$). For this reason, both proposals for revision of EC2 bridge this gap by introducing explicitly the dependence of the punching capacity from the parameter r_s/d.

Results of the comparison among different code formulations and literature experimental data allow for estimating the capability of each code in predicting the experimental strength, although with reference to a moderate number of cases. The current EC2 version overestimates the experimental punching capacity of less than 30%, with a significant data scattering around the average value. The RWTH proposal for revision of EC2 gives the same mean values of current EC2, nevertheless the data scattering is clearly lower and also the probability to overestimate the punching capacity. The EPFL proposal gives better results in terms of the mean value ($V_{test}/V_{th} \cong 1.00$), but, although the scattering is similar to the RWTH proposal, the probability of overestimating the punching capacity is evidently much higher. Nevertheless, to obtain results similar to the RWTH proposal, it would be enough to introduce a reductive coefficient of the theoretical strength.

The work presented in this paper is part of a wider and deeper study, which is currently in progress inside the task group CEN 250/SC 2/TG 4, in charge for the revision of EC2 sections referring to shear, torsion and punching of R/C structures. Results have preliminary nature, and they could be useful for the improvement of the two EC2 proposals.

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