PUNCHING SHEAR STRENGTH OF REINFORCED RECYCLED CONCRETE SLABS

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ABSTRACT

This paper reports on the experimental assessment of the punching shear behaviour of reinforced recycled concrete slabs characterized by fine natural aggregates and coarse recycled aggregates. In particular, the latter were obtained only from demolished concrete. The experimental campaign has been carried out on 12 specimens. Moreover, three reinforced natural aggregate concrete slabs have been casted and tested as benchmarks. Four replacement percentages (30, 50, 80 and 100%) of coarse recycled aggregates in place of coarse natural aggregates have been considered. The punching shear behaviour of simply supported reinforced recycled concrete slabs under a central patch load has been investigated by means of failure patterns, ultimate loads and deflection–load curves.

Moreover, comparisons and a review of international code models for slabs under punching shear have been developed. The results show a reduction in recycled concrete mechanical performance with increasing replacement percentage of natural aggregate with coarse recycled aggregates. However, the reduced recycled concrete performance does not translate directly to the punching shear strength of reinforced recycled concrete slabs; indeed, the punching forces of all recycled concrete slabs tested are very similar to those of slabs realized with ordinary reinforced concrete. Actually, although the

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theoretical models on the punching shear are based on the characteristics of the concrete, this study indicates that the reinforcement role is of paramount relevance.

Keywords: Recycled concrete aggregate, Recycled concrete, Critical shear crack theory, Punching shear strength.

Abbreviations:

NA	Natural Aggregates
RA	Recycled Aggregates
CDW	Construction Demolition Waste
RC	Recycled Concrete
RAc	Recycled Aggregates from CDW waste of concrete only
CRAc	Coarse Recycled Aggregate of concrete only
NC	Normal Concrete
CNA	Coarse Natural Aggregate
FNA	Fine Natural Aggregate

1. Introduction

Concrete is the world's most commonly used construction material, but the high use of Natural Aggregates (NA) for its production represents a significant problem regarding the preservation of natural resources [1-3]. In addition, the construction industry produces a large amount of waste every year, resulting from demolitions of constructions. Often, an important part of these wastes is composed of demolished concrete. European Policies & Strategies [4] encourage the use of recycled materials for new engineering products, so many researchers have focused their studies on the use of Recycled Aggregates (RA) from Construction Demolition Waste (CDW) in the production of new Recycled Concrete (RC) [5–16].

The benefits of using RA from recycled CDW in new concrete are known. The use of natural aggregates can be reduced, and the storage of CDW products in the landfill site could be significantly decreased, with considerable advantages to the environment. In recent years, increasing studies on the properties of RA, particularly on the properties of those from CDW waste of concrete only (RAc), have been undertaken.

Many studies have proved that the properties of these aggregates differ from those of natural ones [17–37]. In general, RAc are characterized by very high water absorption, lower particle density and higher Los Angeles values than NA. The main physical difference between RAc and NA is represented by the presence of the old adherent cement mortar in the surface of original NA, which is the major cause of the different properties between RAc and NA according to many researchers [17, 30, 32, 35]. Furthermore, the sources of RAc can be very different. For the sake of synthesis, it is possible to distinguish three main categories of sources: concrete casted for that very purpose, prefabricated concrete structure production waste, and CDW waste of concrete. The characteristics of these materials cannot be easily assessed, so neither can the corresponding ones for RAc.

Many countries have established standards or recommendations regarding the properties of RA and RAc [38–42].

The structural behaviour of RC element was also investigated. Many papers have been published concerning the performance of beams and columns realized with RC [11, 43–46], and studies on the seismic performance of RC frame structure – e.g., [47] – have been undertaken.

Structural systems with reinforced concrete slabs are a common structural solution. Their structural behaviour is not straightforward and has been analysed for many years, but even currently it is under investigation, particularly considering its environmental impact; see [48–52].

The slabs present several advantages such as reduced and simpler formwork, versatility and easier space partitioning, making flat slabs an economical and efficient structural system. Although simple

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in appearance, they present complex structural behaviour. Often, particularly for slender slabs, the critical structural assessment concerns the punching shear strength.

Actually, the punching failure mechanism is very dangerous because of its brittle nature and because it can be the origin of a progressive collapse. The first punching shear mechanical models were very complex [53–54], and the relative design formulas are very inconvenient for practical use. Many researchers have provided physical models and innovative theories [55–63] that led to simple design expressions in agreement with the most important international design code models [53, 64-67]. These expressions, rationally derived based on the physical models supporting the previous theories, include some parameters obtained by a regression analysis of experimental results. A few works concerning the punching shear strength of reinforced recycled concrete slabs can be found in the literature. Sudarsana Rao et al [68] investigated the punching shear behaviour of reinforced recycled aggregate concrete slabs. The recycled concrete was made with fine natural aggregates and coarse natural and/or recycled aggregates. RCA was obtained from the waste concrete from the runway of an Airport in Kadapa, Andhra Pradesh, India. Their results show that all slabs behaved in a similar way concerning the punching shear failure, regardless of the Coarse Recycled Aggregate of concrete only (CRAc) replacement percentage. Slabs made with RC present lower first crack load and ultimate load of slabs compared with Normal Concrete (NC). This trend was evident for RC slabs with replacement percentages greater than 40%.

Nuno Reis et al [69] presented an experimental, numerical and analytical investigation on the effects of CRAc substitution on the punching behaviour of reinforced concrete slabs. The original concrete used to produce the recycled aggregates had the same constituents (cement, aggregates) used in the different concrete mixes tested in this study. It presented a maximum aggregate size of 22.4 mm and an average cubic compressive strength of 42.8 ± 1.3 at 28 days. The authors showed that the punching strength of the NC slabs was similar to that of the RC slabs; for 100% replacement of coarse NA by

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CRAc, the strength reduction was only 2%. Regarding the analytical formulae, that study showed conservative estimates of the punching strength of RC slabs for each code examined (MC 2010 [64–65], ACI 318 [66], EC 2 [67]). The most accurate predictions were obtained using MC 2010 considering levels of approximation II, III and IV.

To improve the knowledge on punching shear failure of reinforced RC slabs, this paper presents new experimental data and the corresponding analytical assessments based on international design codes [64-67]. In particular, this work analyses the feasibility of using coarse recycled aggregates obtained by concrete waste with unknown mechanical properties to realize structural elements. Indeed, the coarse recycled aggregates have been produced by crushing concrete CDW. The strength and preservation status of these concretes are unknown.

A total of fifteen slabs with different mixtures have been casted. The mixtures have been divided into five groups: 0%, 30%, 50%, 80% and 100% replacement percentage of Coarse Natural Aggregate (CNA) with CRAc. The experimental results of a punching shear test of simply supported reinforced RC slabs are reported. Failure patterns, ultimate loads and deflection–load curves of slabs under punching shear have been evaluated.

The experimental framework and the geometric and mechanical data are reported in Section 2. The experimental results of the punching shear test are reported in Section 3. Failure patterns, ultimate loads and deflection–load curves of slabs under punching shear have been evaluated.

Section 4 presents a review of the slab punching models present in the international design codes with a comparison between the predictions obtained with these models and the field data. Finally, in Section 5, conclusions are given along with several expected developments.

2. Materials and Methods

2.1 Materials

Ordinary Portland cement CEM II/A-LL 42.5 R [70], locally available limestone sand as Fine Natural Aggregate (FNA), locally available CNA (limestone) and CRAc, with diameter between 4 and 12 mm, have been used.

CRAc was randomly taken from three different authorized storage sites located in south Sardinia. Thus, the strength and preservation status of the concretes used as aggregates sources is unknown.

Table 1 shows the physical properties of the FNA, CNA and CRAc (bulk density ρ_a, saturated surface dry density ρ_{ssd}, and water absorption WA_{24}). B450A steel welded mesh layers (wire diameter of 5 mm and mesh with aperture of 100 mm) have been used as slab reinforcement. The mechanical characteristics of the steel are as follows: yielding strength f_{yk} = 450 MPa, ultimate strength f_{uk} = 540 MPa, modulus of elasticity E_s = 200 GPa.

Table 1 Natural and recycled aggregate properties

Aggregates	Grading (mm)	ρ (kg/m^3)	ρ_{ssd} (kg/m^3)	WA_{24} (%)
FNA	0–4	2707	2630	2.00
CNA	4–12	2691	2600	1.40
CRAc	4–12	2630	2360	7.54

2.2 Mix of concrete

The concrete mixes are reported in Table 2. These mixtures have been divided into five groups: 0, 30, 50, 80 and 100% replacement percentages (Rep %) of CNA with CRAc.

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Table 2 Mix designs of concretes

Mix	Rep%	Cem (kg)	FNA (0–4) (kg)	CNA (4–16) (kg)	CRAc (4–16) (kg)	Water (kg)	Super plasticizer (kg)
NC0	0	420	827	897	0	175	4.87
RC1	30	420	827	628	229	175	4.90
RC2	50	420	827	449	381	175	5.54
RC3	80	420	827	179	610	175	4.44
RC4	100	420	827	0	763	175	4.96

2.3 Test samples

Three concrete cubic specimens (side length 150 mm) for each replacement percentage have been tested to determine the splitting tensile strength $f_{sp,cube}$ [71]. In the same way, three concrete cylindrical specimens (diameter 150 mm, height 300 mm) have been used to determine the compressive strength $f_{c,\,cyl}$ [72] and the stabilized secant modulus of elasticity E_c [73] of each concrete mix.

A total of fifteen reinforced concrete slab specimens (three specimens for each mix) have been tested. All specimens have the same dimensions: 1100 · 1100 · 50 mm.

The flexural reinforcement of specimens has been realized by using two 100 · 100 mm welded mesh layers (wire diameter 5 mm) placed in the top and bottom layers with a cover of 10 mm. In this way, the slabs have a low reinforcement ratio ρ (equal to 0.56%).

The investigated slabs present a very small thickness that can be interesting for the analysis of the punching shear behaviour. Indeed, the effective depth of the slab is an important parameter for many punching models and in particular for ACI 318, EC 2 and MC 2010. In [63] is reported that when two slabs with similar geometric proportions are investigated the thicker slab has a lower rotation capacity and fails in a rather brittle manner whereas the thinner slab exhibits a more ductile behaviour. So, in

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case of punching shear analysis size effect phenomena must be considered and investigated and the need for more experimental data is patent.

For these reasons it is interesting to explore what is the behaviour of very thin slab in case of punching shear failure and check if the current standard models are capable of modelling also these cases. But it is clear that in order to have a complete view of the problems further tests are needed with different slab depth and different slenderness values.

2.4 Experimental tests

Each slab was simply supported by HEA 100 steel beams along the four edges (Figs. 1 and 2). The load was applied on the geometric centre of the specimen through a steel plate with dimensions of 200 · 200 · 5 mm. The punching cone was detected in this critical region, and deflection was measured by means of an extensometer wire placed on the bottom of the specimen in the same geometric centre. Fig. 1 and Fig. 2 present the test setup details.

The vertical load was applied by 500 kN hydraulic jacks.

Continuous readings were provided by a high-quality compact data acquisition unit for applied load and deflection. Load was monotonically increased until punching failure occurred in the specimen. The punching failure corresponds to the significant degradation of the specimen characterized by a penetration of the steel loading plate into the top side of the slab (Fig. 3).

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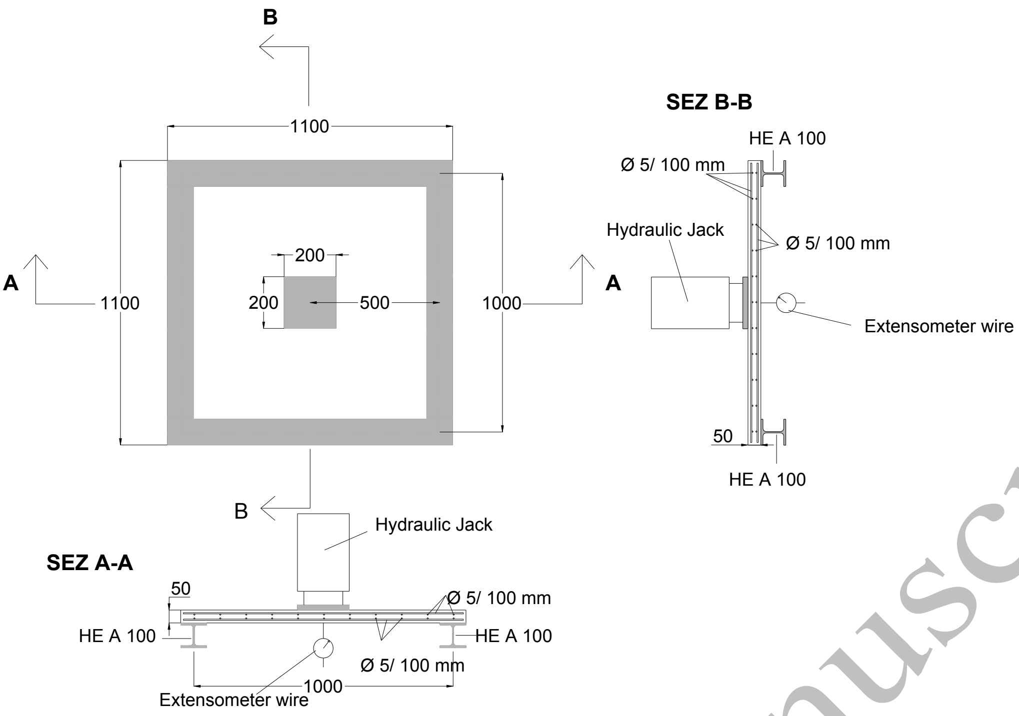

Fig. 1 Geometry of the slab specimen and test setup.

Fig. 2 Test setup.

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a) b)

Fig. 3 Typical degradation on bottom (a) and top (b) surfaces of slab after punching shear failure.

3. Results and Discussion

3.1 Concrete tests

The average values of the following concrete characteristics of density, cylindrical strength $f_{c,\ cyl}$, splitting tensile strength $f_{sp,\ cube}$, and modulus of elasticity E_c are shown in Table 3 for each replacement percentage (Rep%). The average has been evaluated considering three samples for each characteristic.

The use of CRAc reduces the mechanical performance of recycled concrete in agreement with [7, 12, 17, 19–21, 25, 29–32]. Indeed, when the replacement percentage of CNA with CRAc increases, the density, compressive strength, splitting tensile strength and modulus of elasticity decrease; e.g., the density, compressive strength, splitting tensile strength and modulus of elasticity of RC with 100% replacement percentage are reduced to 6%, 29%, 13%, and 26%, respectively, of the corresponding values of NC.

Actually, RC and NC have different failure mechanisms that probably cause performance reductions of RC with respect to NC. Their structures are very different, and the failure mechanism of the recycled concrete is fairly complex. Rupture in RC may be influenced by many factors including mortar strength, aggregate strength, mortar–aggregate bond properties, surface texture and shape of coarse recycled aggregate, and interface surface properties. In addition, some recycled aggregates can be formed by mortar only [33, 74–75]. In NC, there is only one type of interfacial transition zone between the original NA and the cement paste. In recycled concrete, there are two interface areas: one between the original NA and the old cement mortar and the other between the old cement mortar and the new cement mortar [31–33, 75].

Table 3 Average mechanical properties of concrete

Mix	Rep%	Density (kg/m^3)	$f_{c, cyl}$ (N/m^2)	$f_{sp, cube}$ (N/m^2)	E_c (N/m^2)
NC0	0	2403	71.1	4.20	42581
RC1	30	2343	63.6	4.40	40381
RC2	50	2329	62.0	3.94	37980
RC3	80	2260	56.3	3.83	28818
RC4	100	2257	50.8	3.65	31390

An analysis of the fracture surfaces after splitting tensile strength tests shows a lower original aggregate strength and higher bond strength between old and new cement mortar. It can be observed that the failure occurs in the original aggregate [75]. This failure behaviour is typical of high-strength NC [76].

3.2 Slab test

Details of cracking and punching load (V_{cr} and V_{exp}) of the simple supported slab and its corresponding deflection (δ_{cr} and δ_u) are given in Table 4. Deflection–load curves for the different cases are shown in Figs. 4 (a, b, c, d, e). The first crack condition is identified by the black rhombus. The results reported in Table 4 highlight that the first crack and ultimate loads of NC and RC are very similar. Indeed, the first crack loads of slabs RC1 and RC2 increase (approximately 7%) with respect to NC, and there is a reduction of 12% for RC3. In the RC4 case, the first crack load is practically equal to that of the NC slab.

Table 4 Slab test results.

Sample	V_{cr} (kN)	$V_{cr, average}$ (kN)	δ_{cr} (mm)	$\delta_{cr, max}$ (mm)	V_{exp} (kN)	$V_{epx, average}$ (kN)	δ_u (mm)	$\delta_{u, max}$ (mm)
NC0-1	17.00		2.27		72.5		40.79	
NC0-2	21.53	18.80	1.76	2.27	68.7	68.70	44.78	44.78
NC0-3	17.87		0.58		64.9		40.59	
RC1-1	17.87		1.29		64.9		43.88	
RC1-2	23.50	19.74	2.30	2.35	72.5	69.97	45.57	45.57
RC1-3	17.87		2.35		72.5		35.56	
RC2-1	17.87		1.54		64.9		50.28	
RC2-2	21.53	20.31	2.49	2.49	68.7	66.17	52.50	52.50
RC2-3	21.53		1.82		64.9		47.00	
RC3-1	14.23		2.09		68.7		50.01	
RC3-2	17.87	16.66	2.77	2.77	64.9	68.70	43.11	52.04
RC3-3	17.87		2.33		72.5		52.04	
RC4-1	17.87		3.16		68.7		47.72	
RC4-2	21.53	19.09	2.83	3.16	68.7	69.97	57.03	57.03
RC4-3	17.87		0.14		72.5		45.64	

Considering that the ultimate punching load insignificant differences between RC slabs and NC slabs have been found. This fact highlights that the presence of CRAc (even with a high replacement percentage) does not influence the first crack load or the ultimate load (in agreement with [68–69]). They probably depend on various factors i.e., physical–mechanical characteristics of aggregates, construction technology of slabs, and particularly the steel–concrete bond (first crack load) and ultimate reinforcement strength (first crack and ultimate load). Indeed, in the failure condition, the

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influence of the concrete mechanical characteristics is very low compared with the effect of the steel reinforcement, which has paramount relevance. The failure mechanism is mainly based on the residual strength provided by the steel reinforcement, and this fact can explain that the structural performance of RC slabs is very similar to that realized with NC.

Different considerations can be stated based on the displacement results. Actually, for each group of slabs (characterized by the same concrete), deflections at the first crack and ultimate load are significantly variable, but the general trend is that with increasing replacement percentage, displacements tend to increase. Table 4 also reports the maximum deflection at first crack and ultimate load for each group of slabs, confirming the abovementioned trend. The differences between the deflections of RC and NC slabs at first crack load reach 10% for replacement percentages under 50%. For replacement percentages of 80 and 100%, these differences reach 39% and 27%, respectively. Actually, deflection results show that for replacement percentages greater than 50%, the presence of recycled aggregates plays an important role. Fig. 4 (a, b, c, d, e) shows that the deflection–load curves have a typical linear trend until cracking load, followed by a plastic behaviour with reduced stiffness. It is quite easy to note that the area under the abovementioned curve (which is a rough measure of the strain energy supplied to the slab) tends to increase with increasing replacement percentage. In the authors' view, this is very important because it proves that RC slabs can absorb more energy than NC slabs at the ultimate limit state (even if the ultimate load is almost the same). This characteristic can be of paramount relevance in a modern capability design approach.

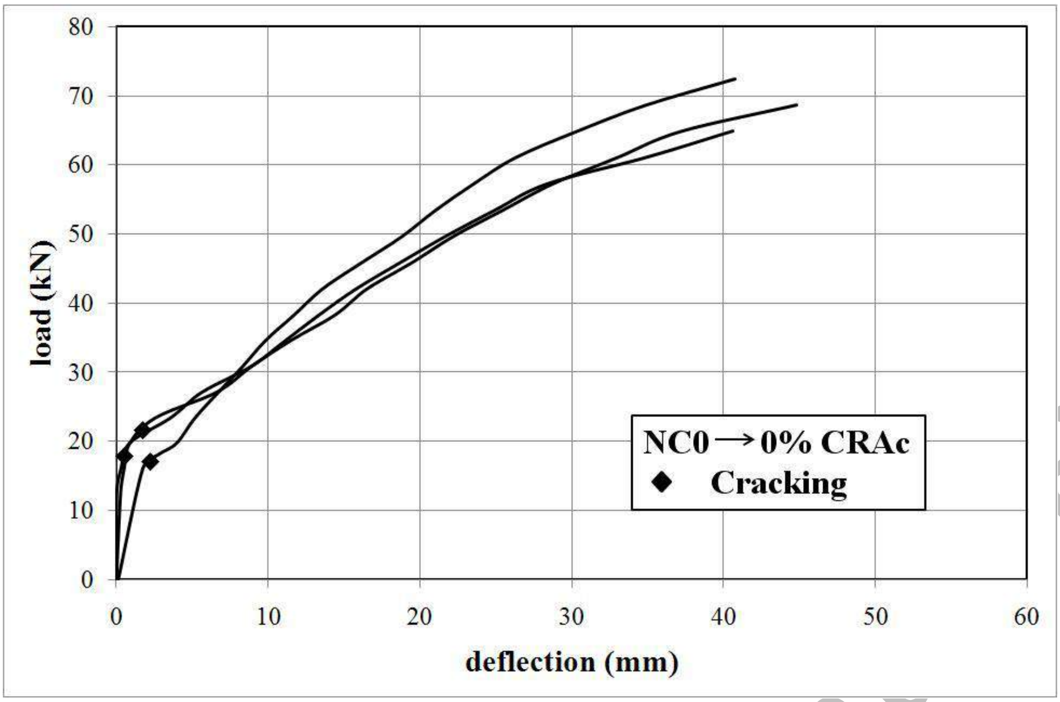

Fig. 4a Load–deflection curves of NC0 slabs.

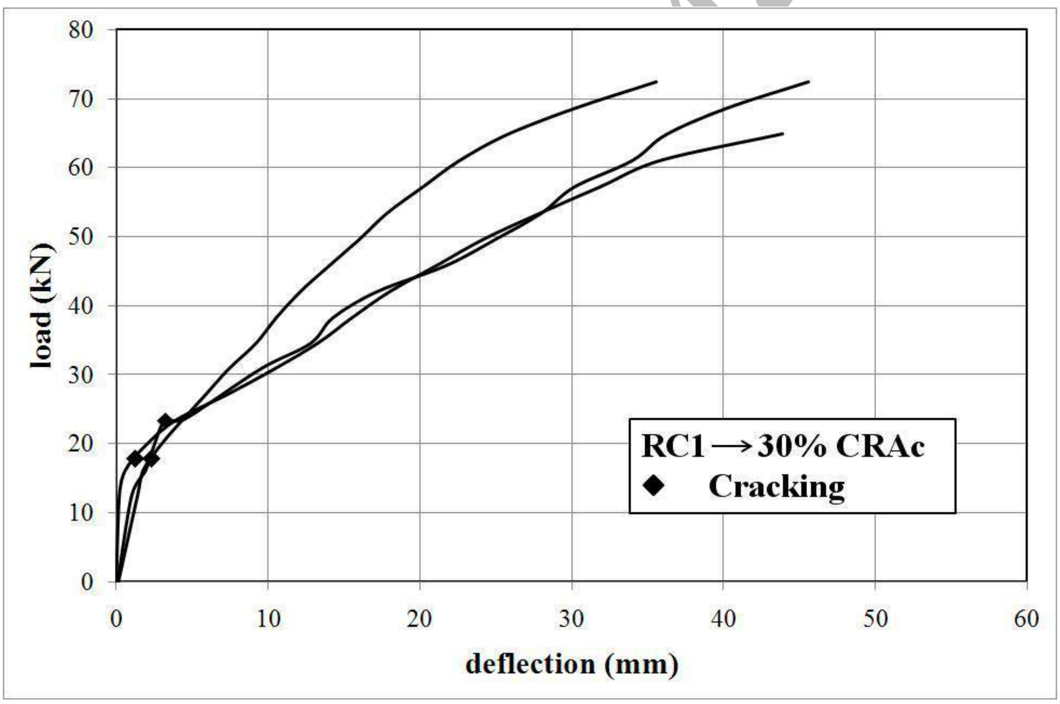

Fig. 4b Load–deflection curves of RC1 slabs.

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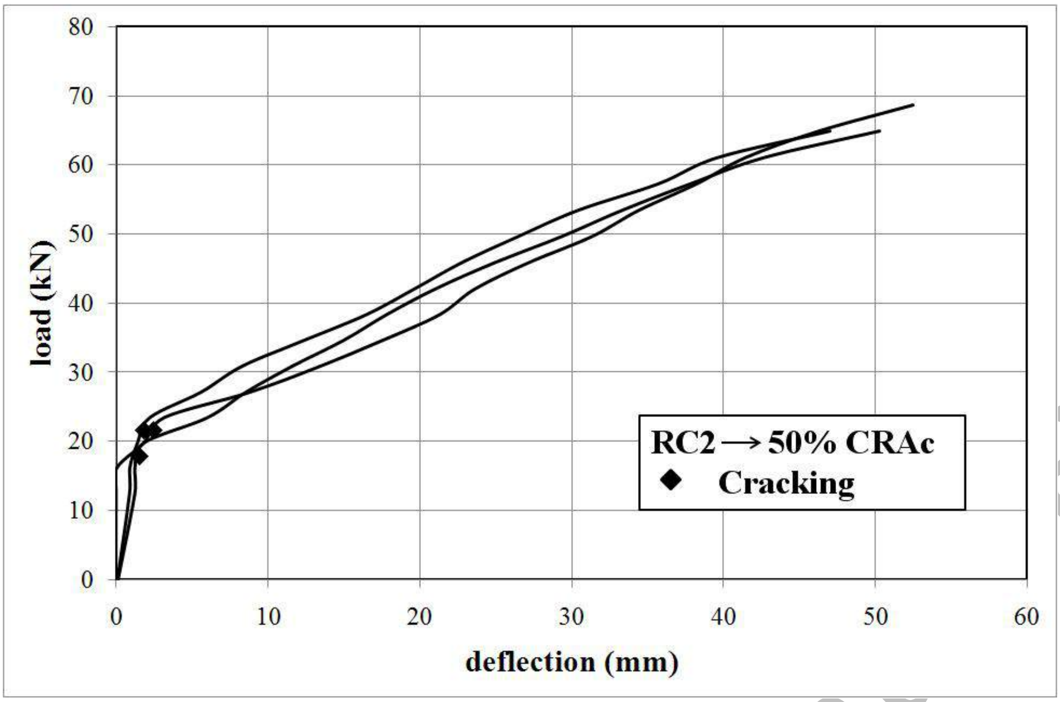

Fig. 4c Load–deflection curves of RC2 slabs.

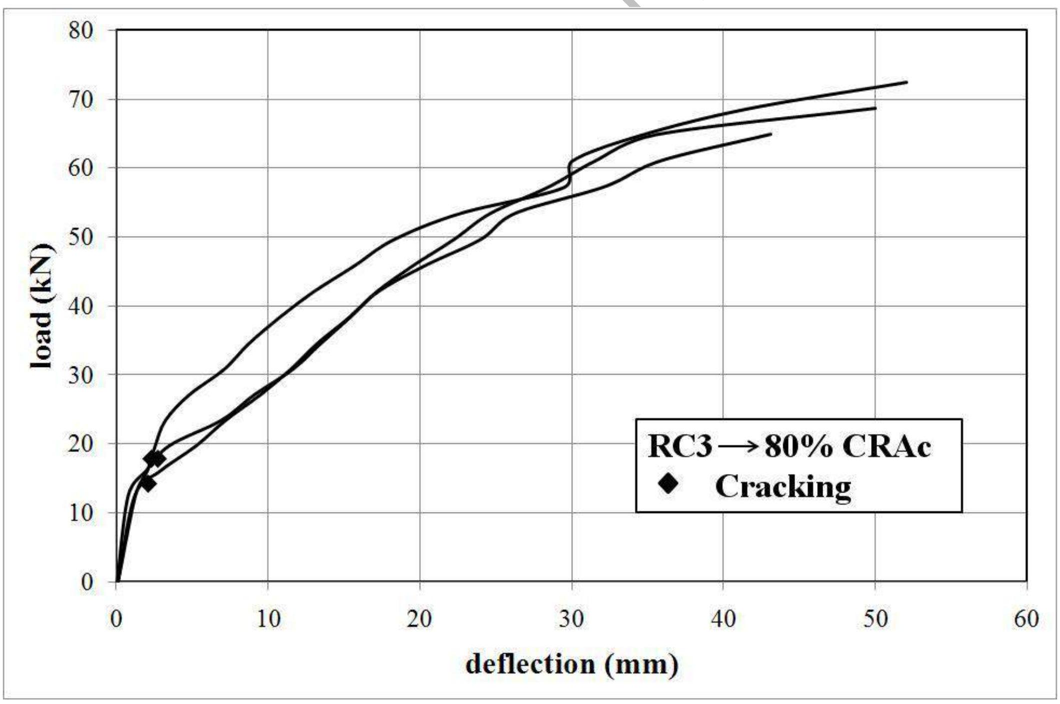

Fig. 4d Load–deflection curves of RC3 slabs.

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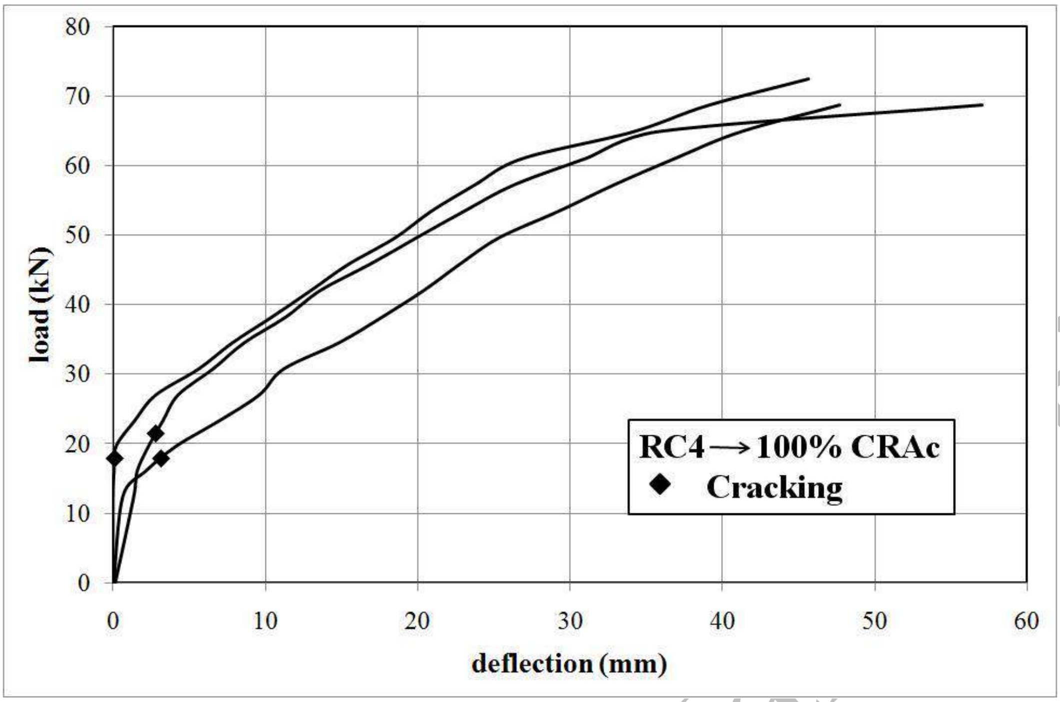

Fig. 4e Load–deflection curves of RC4 slabs.

Punching shear failure has been detected for all tested slabs. Its typical crack distribution was clearly visible on the top and bottom faces (see Fig. 5).

In all cases, the cracks first developed at the corners with an inclination of 45° with respect to the slab edges (see Fig. 6 thick line). This fact changes the constraint condition; indeed, the corners lift, and the slab is supported on shorter edge segments (approximately 500 mm long; see Fig. 7). At the end, punching shear failure occurs.

The cracking pattern on the top face of the slab is quite irregular, for each specimen the punching cone radius *r* has been measured every β=30°, see Fig. 8, the detailed results are reported in Appendix I while the maximum, minimum and average radius is shown in Table 5. These values do not highlight any evident relationship between the punching cone radius and the replacement percentage of CRAc.

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It was not possible to develop the same measurements on the bottom face because, in this case, the cracking pattern is strongly related to the reinforcement tensile strain. Indeed, its distribution is equal in all the slabs. It corresponds to a quite uniform orthogonal grid, with dimensions related to those of the reinforcements mesh, a typical pattern is shown in Fig. 5 (right side) and in Fig. 6 (thin line).

Table 5 Maximum, minimum and average punching cone radius on the slab top surface.

Sample group	**r_{max} (mm)**	**r_{min} (mm)**	**$r_{average}$ (mm)**
NC0	207	67	114
RC1	261	88	143
RC2	174	70	102
RC3	192	80	133
RC4	169	69	123

Fig. 5 Cracking patterns at failure of slabs after punching test for each replacement percentage (0%, 30%, 50%, 80%, 100%): on the left is the top face, on the right is the bottom face.

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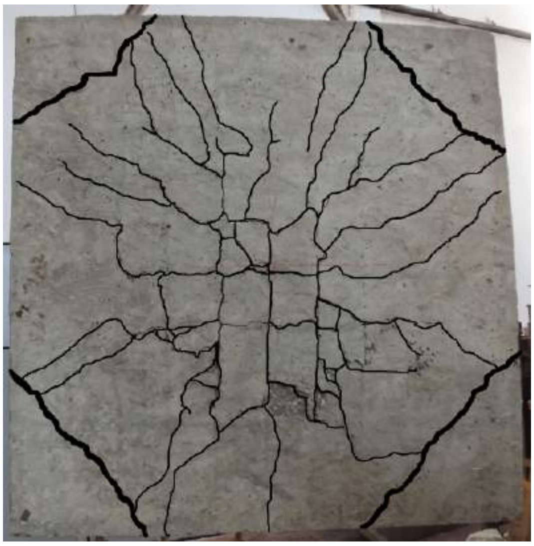

Fig. 6 Cracking pattern in punching shear failure of the bottom face.

Fig. 7. Constraint condition: initial on the left and final on the right.

A significant synthesis of the experimental tests is reported in Fig. 9. It shows the variation of concrete mechanical characteristics and slab structural performance as a function of replacement percentage of CRAc. This picture shows that the reduction of RC strength, compared with NC, does not determine the same differences in the performance of RC slabs, as noted previously. This can be explained by considering that the steel reinforcement plays a fundamental role in the ultimate limit state.

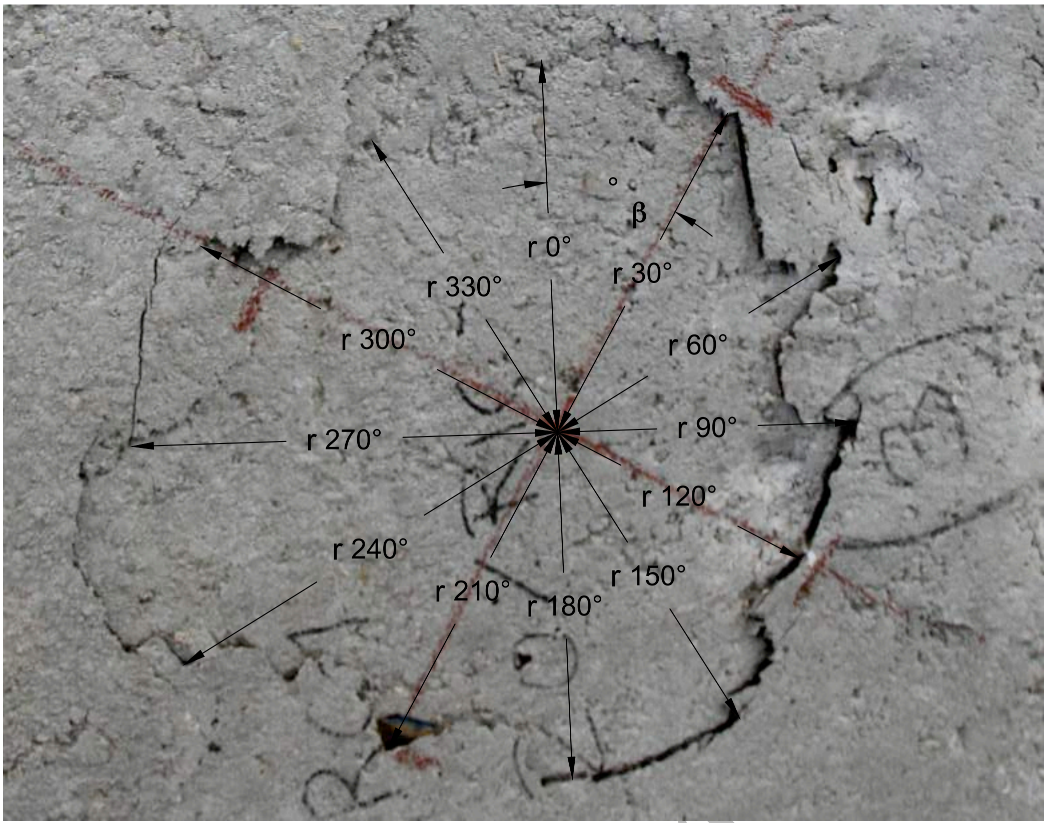

Fig. 8. Top face cracking pattern and radius of the punching cone measured every 30°.

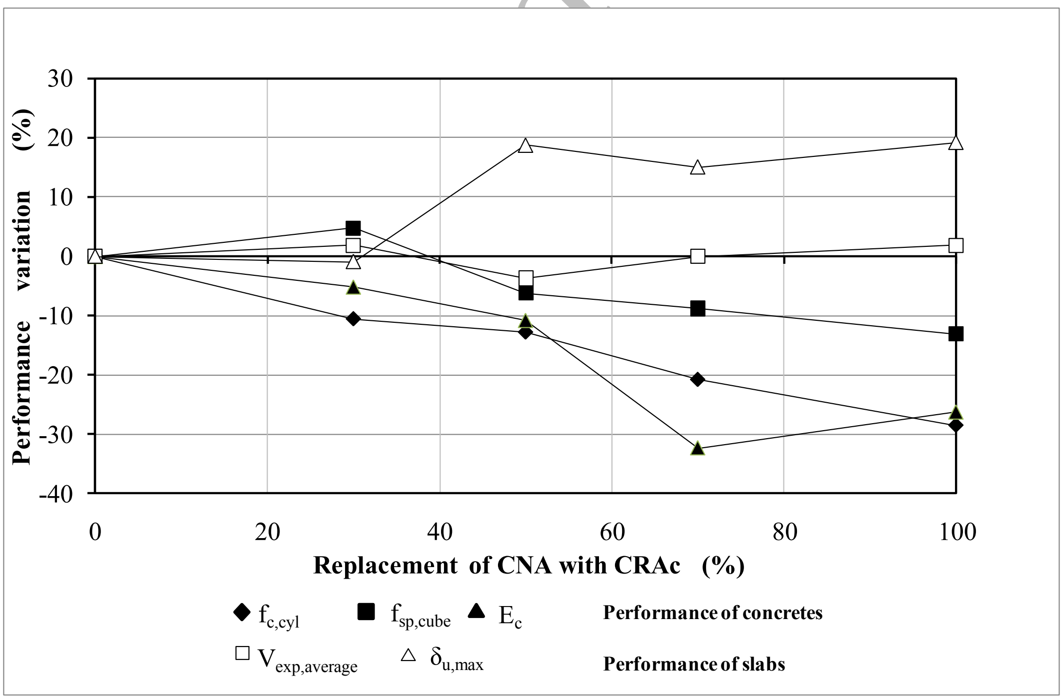

Fig. 9 Performance variation of RC and RC slabs with respect to NC slabs.

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4. Concrete Design Code Model for Punching

In this section, the punching shear strength of concrete slabs is determined according to three international design codes: MC 2010 [64-65], ACI 318 [66], and EC 2 [67]. To assess their accuracy, the experimental results presented in the previous sections are compared with these theoretical formulations. The predicted punching shear strength is computed considering the mean values for the material properties without considering the partial safety coefficients.

The abovementioned codes are devoted to normal reinforced concrete structures and do not consider the presence of CRAc. Thus, a sensitivity analysis on some parameters capable of accounting for the characteristics of RC is developed. This approach yields some important information about the design methods for RC slabs.

The tested slab can represent part of the largest flat slab supported by a central column. Its dimensions can be related to the distance (r_s) from the column axis to the points at which the radial bending moment is zero; see [65]. In the case of a regular flat slab in which the ratio of the spans (L_x and L_y) is between 0.5 and 2,

$$r_s \cong 0.22 \cdot L_x \text{ or } 0.22 \cdot L_y \quad (1)$$

where L_x and L_y are the centre-to-centre spans of the columns in the x and y directions, respectively.

Most international codes are based on the punching assessment for a critical cross-section of the structural element. The punching shear strength of slabs without shear reinforcements is defined as a function of the concrete compressive strength. Actually, some codes also consider other parameters:

- the reinforcement ratio (EC 2 and MC 2010 levels of approximation II to IV),
- the size effect (EC 2),
- the maximum aggregate size (MC 2010 with levels of approximation I to IV),
- the rotation of slab (MC 2010 with levels of approximation I to IV).

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The abovementioned critical cross-sectional area can be evaluated as the product of b_0 (the basic control perimeter) and d (the effective depth of the slab). It is located at a distance equal to d/2 from the column side for both models reported in ACI 318 and in MC 2010. For EC 2, this distance is equal to 2d. The length of the control perimeter is limited by several factors: *e.g.,* slab edges, presence of openings, pipes, and inserts.

The abovementioned models, capable of assessing the punching shear strength ($V_{Rd,c}$) of a reinforced concrete slab, are shown in the following. To simulate the real behaviour of the tested slab, prestressing action and shear reinforcement are not considered.

In addition, each code imposes design requirements (maximum aggregate size, concrete cover depth, slenderness) to ensure durability and good condition of the cast and to verify the deflection without analytical evaluation. In this paper, the main aim is to assess the punching shear model reliability at the ultimate limit state, so these requirements have not been assumed.

4.1 ACI 318

According to ACI 318, the punching shear strength ($V_{Rd,c}$) of a reinforced concrete slab is the smaller value of following relations:

$$V_{Rd,c} \leq \begin{cases} 0.17 \cdot \left(1 + \dfrac{2}{\beta}\right) \cdot \lambda \cdot \sqrt{f_c} \cdot b_0 \cdot d \\ 0.083 \cdot \left(\dfrac{\alpha_s \cdot d}{b_0} + 2\right) \cdot \lambda \cdot \sqrt{f_c} \cdot b_0 \cdot d \\ 0.33 \cdot \lambda \cdot \sqrt{f_c} \cdot b_0 \cdot d \end{cases} \tag{2}$$

where λ is a factor that reflects the different mechanical characteristics of lightweight concrete. It is equal to 1.0 for normal weight concrete and 0.75 for lightweight concrete. Otherwise, λ shall be determined considering volumetric proportions of lightweight and normal weight aggregates, but in any case, a conservative value is 0.85 (e.g., [77]);

β is the ratio of the long side to the short side of the column, concentrated load or reaction area;

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α_s is equal to 40 for interior columns;

b_0 is a perimeter of the critical section located at a distance d/2 from the side of the load area;

d is the distance between the extreme compression fibre and the centroid of the longitudinal tensile reinforcement;

f_c is the concrete compressive strength, based on a cylinder test.

This theoretical model (Eq. (2)) is employed to predict the punching shear strength of the tested slabs. The following results are obtained considering the average cylinder compressive strength of the concretes presented in Table 3. As reported in Section 3, the load was applied to the central zone of the slab on a square area ($200 \cdot 200$ mm^2), so $\beta = 1$. This experiment simulates the real behaviour of an interior column with a $200 \cdot 200$ mm^2 cross section ($\alpha_s = 40$). The side of the load area is a = 200 mm, the effective depth is d = 35 mm, and the perimeter of the critical section is $b_0 = (4 \cdot a + \pi \cdot d) =$ 910 mm. To evaluate the influence of recycled aggregates, two λ values have been considered: $\lambda = 1$ for normal weight aggregates and $\lambda = 0.85$ for lightweight aggregates. Indeed, the CRAc has lower density (see saturated surface dry density ρ_{ssd} in Table 1) than coarse natural aggregates. As a consequence, the influence of λ can be very important.

Table 6 reports the comparison between the theoretical and experimental punching shear strength, and Fig. 10 shows their ratio ($V_{exp}/V_{Rd,c}$) related to the replacement percentage of CRAc. It is evident that $\lambda = 0.85$ always yields theoretical results on the safety side compared with experimental data. Instead, $\lambda = 1.00$ can produce an overestimation of the slab punching shear strength when the replacement percentage is lower than 80%. Overall, the ACI 318 model is quite reliable in assessing the punching shear strength in all tested slabs.

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Table 6 Theoretical ($V_{Rd,c}$) and experimental (V_{exp}) punching shear strength of tested slabs.

Samples	**V_{exp} (kN)**	**$V_{exp, average}$ (kN)**	**$V_{Rd, c}$ ACI 318 (kN)**	
			λ = 1	**λ = 0.85**
NC0-1	72.5			
NC0-2	68.7	68.70	78.9	67.1
NC0-3	64.9			
RC1-1	64.9			
RC1-2	72.5	69.97	74.6	63.4
RC1-3	72.5			
RC2-1	64.9			
RC2-2	68.7	66.17	73.7	62.6
RC2-3	64.9			
RC3-1	68.7			
RC3-2	64.9	68.70	70.2	59.7
RC3-3	72.5			
RC4-1	68.7			
RC4-2	68.7	69.97	66.7	56.7
RC4-3	72.5			

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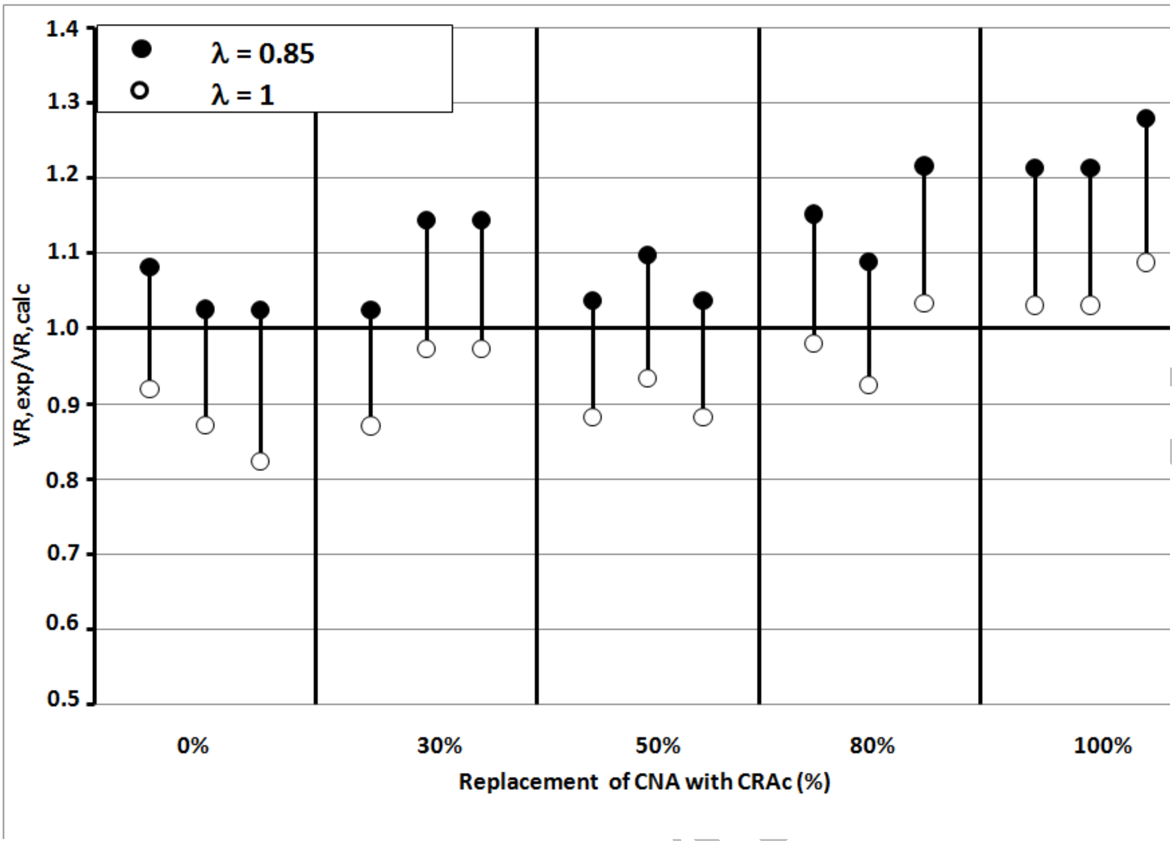

Fig. 10 Ratio between experimental V_{exp} and theoretical ACI 318 $V_{Rd,c}$ punching shear strength considering different replacement ratios and two extreme values of λ.

4.2 EUROCODE 2

According to EC 2, the design punching shear strength of a reinforced concrete slab may be represented by:

$$V_{Rd,c} = C_{Rd,c} \cdot k \cdot (100 \cdot \rho_l \cdot f_{ck})^{1/3} \cdot u_1 \cdot d_{eff} \geq \nu_{min} \cdot u_1 \cdot d_{eff}, \quad (3)$$

where:

$$C_{Rd,c} = 0.18, \quad (4)$$

The size parameter k is defined as follows:

$$k = 1 + \sqrt{\frac{200}{d_{eff}}} \leq 2, \quad (5)$$

where the effective depth d_{eff} of the slab may be expressed by:

$$d_{eff} = \frac{d_y + d_z}{2} \ , \qquad (6)$$

where d_y and d_z are the distances from the extreme compression fibre to the centroid of the longitudinal tensile reinforcement in the plane orthogonal directions y and z.

If a_y represents the length of the column cross section side in the y direction and a_z is the corresponding length in the z direction, the dimensions of the critical section perimeter are $b_y=a_y+6\ d_{eff}$ and $b_z=a_z+6\ d_{eff}$ (see Fig. 11), and the reinforcement ratio ρ_l is expressed by:

$$\rho_l = \sqrt{\rho_{ly} \cdot \rho_{lz}} \le 0.02 , \qquad (7)$$

where $\rho_{ly} = A_{sy}/(b_y \cdot d_{eff})$ and $\rho_{lz} = A_{sz}/(b_z \cdot d_{eff})$.

Considering the abovementioned parameter, the critical perimeter is expressed by:

$$u_1 = 2 \cdot (a_y + a_z) + 4 \cdot \pi \cdot d_{eff} , \qquad (8)$$

The lower limit value of punching shear stress v_{min} presented in Eq. 3 is defined by the following:

$$v_{min} = 0.035 \cdot k^{3/2} \cdot f_{ck}^{1/2} . \qquad (9)$$

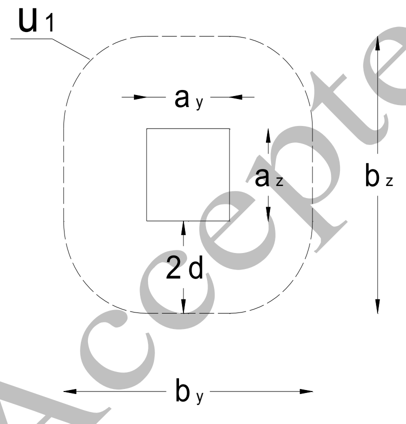

Fig. 11: Punching critical section perimeter for a rectangular column.

The EC 2 theoretical model (Eq. (3)) has been employed to predict the punching shear strength of the tested slabs. The following results are obtained considering the average cylinder compressive strengths of concrete presented in Table 3. As reported in Section 3, the load was applied to the central

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zone of the slab on a square area ($a_y \cdot a_z$), where $a_y = a_z = 200$ mm; $d_{eff} = 35$ mm; and $b_y = b_z = 410$ mm, so the critical section perimeter is $u_1 = (4 \cdot a + \pi \cdot d) = 1240$ mm.

As reported in Section 2, the slab reinforcements are two $100 \cdot 100$ mm^2 welded mesh layers (5 mm diameter), so $\rho_{ly} = \rho_{lz} = 0.0056 \leq 0.02$.

Table 7 Theoretical ($V_{Rd,c}$) and experimental (V_{exp}) punching shear force of tested slabs.

Samples	**V_{exp} (kN)**	**$V_{exp, average}$ (kN)**	**$V_{Rd,c}$ EC 2 (kN)**	
			k = 3.39	**k = 2**
NC0-1	72.5			
NC0-2	68.7	68.70	89.74	52.94
NC0-3	64.9			
RC1-1	64.9			
RC1-2	72.5	69.97	86.47	51.01
RC1-3	72.5			
RC2-1	64.9			
RC2-2	68.7	66.17	85.74	50.58
RC2-3	64.9			
RC3-1	68.7			
RC3-2	64.9	68.70	83.02	48.98
RC3-3	72.5			
RC4-1	68.7			
RC4-2	68.7	69.97	80.23	47.33
RC4-3	72.5			

Given the quite slender tested slabs, the influence of the size parameter k, defined in Eq. 5, on the punching shear strength has been investigated. Indeed, the theoretical results have been obtained considering its limit value k = 2 and the actual value for the present case k = 3.39. As shown in Table 7 and Fig. 12, which report the comparison between theoretical and experimental results, k = 2 yields conservative theoretical values, whereas k = 3.39 produces an overestimation of the punching shear strength of the slabs. This issue is very important and must be carefully considered for the design of slender slabs; indeed, the two choices of k yield very different punching shear strengths.

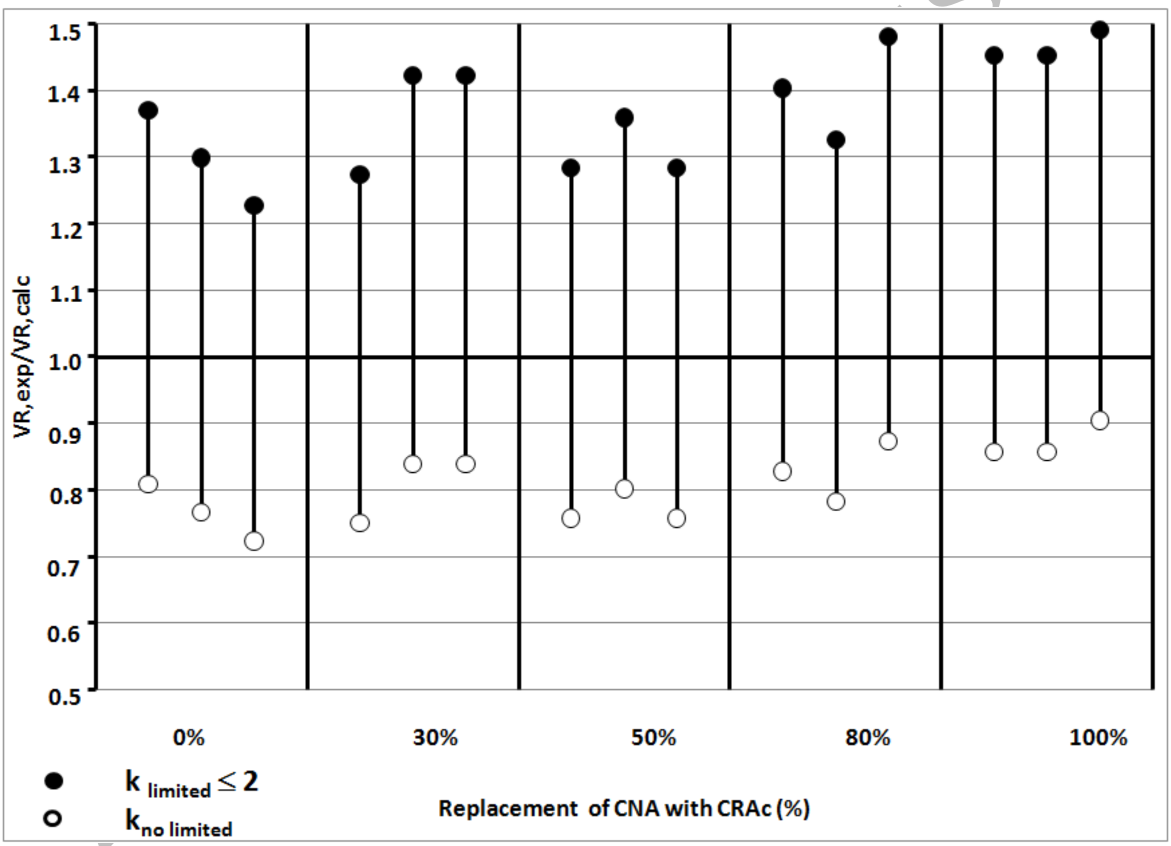

Fig. 12 Ratio between experimental V_{exp} and the theoretical EC 2:2005 $V_{Rd,c}$ punching shear strength considering different replacement ratios and two values of size parameter k.

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4.3 Model Code 2010

In Model Code 2010, the punching shear strength assessment is based on the critical shear crack theory; see [62] for further details. Thus, in the case of concrete without shear reinforcements, the punching shear strength can be evaluated by the following expression:

$$V_{Rdc} = k_{\psi} \cdot \frac{\sqrt{f_c}}{\gamma_c} \cdot b_1 \cdot d \,, \tag{10}$$

where the effective depth of the slab (d) is the distance from the centroid of the tensile reinforcement layers to the extreme compression fibre; b_1 is the basic control perimeter that may be assumed to be at a distance 0.5d from the column cross section sides or, generally speaking, from the support region (see Fig. 13). The parameter k_ψ depends on the rotations of the slab, and its definition is expressed as follows:

$$k_{\psi} = \frac{1}{1.5 + 0.9 \cdot k_{dg} \cdot \psi \cdot d} \leq 0.6 \tag{11}$$

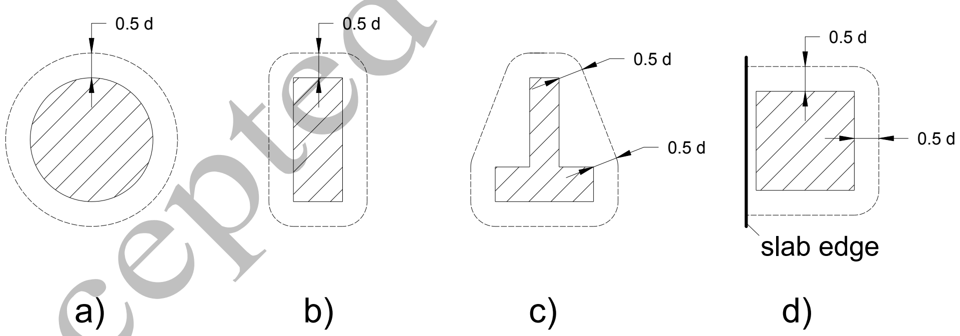

Fig. 13 Basic control perimeter for slab supported by columns with different cross sections.

The evidence that the punching shear strength is influenced by the maximum size of the aggregate d_g can be found in the literature [55, 57].

If the maximum aggregate size is smaller than d_g = 16 mm, the value of parameter k_{dg} is:

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$$k_{dg} = \frac{32}{16 + d_g} \geq 0.75. \tag{12}$$

Instead, if the size of the maximum aggregate d_g is larger than 16 mm, k_{dg} can be assumed to be equal to 1.0. For high-strength or lightweight concrete, the aggregate particles may break, resulting in a reduced aggregate interlock contribution as reported in MC 2010. In that case, $d_g = 0$ and $k_{dg} = 2.0$ (e.g., [78]). This failure behaviour has been highlighted by analysis of the fracture surfaces after splitting tests of RC (see section 3.1: Concrete tests). The parameter ψ refers to the rotation of the slab around the supported area; its value can be calculated by formulas characterized by various levels of approximation, which will be presented in the following. In general, when the approximation level is higher, the computational cost is higher, but the economic cost of the structural solution is lower.

4.3.1 Level of Approximation I

The level I method represents the simplest and straightforward approach, which is valid for standard cases. For regular flat slabs with elastic behaviour and without significant redistribution of internal forces, a conservative estimate of the rotation at failure is:

$$\psi = 1.5 \cdot \frac{r_s}{d} \cdot \frac{f_{yd}}{E_s}, \tag{13}$$

where r_s denotes the position at which the radial bending moment is zero with respect to the supporting column axis. The value of r_s can be approximated as 0.22 L_z or 0.22 L_y for the *z* and *y* directions, respectively, for regular flat slabs in which the ratio of the spans (L_z/L_y) is between 0.5 and 2.0.

4.3.2 Level of Approximation II

In cases where significant bending moment redistribution is considered in the design, the slab rotation can be expressed as:

$$\psi = 1.5 \cdot \frac{r_s}{d} \cdot \frac{f_{yd}}{E_s} \cdot \left(\frac{m_{Ed}}{m_{Rd}}\right)^{1.5}, \tag{14}$$

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where m_{Ed} is the average bending moment per unit length in the support strip, and m_{Rd} is the corresponding design average resistant bending moment. The rotation is calculated along the two principal directions of the slab. To evaluate the punching shear strength, it is necessary to consider the larger of the two values. The width of the support strip is expressed by:

$$b_s = 1.5 \cdot \sqrt{r_{sx} \cdot r_{sy}} \,. \qquad (15)$$

4.3.3 Level of Approximation III

A thorough estimation of the slab rotation can be assessed at this level of approximation by means of a linear elastic model. The slab rotation is expressed by the following expression:

$$\psi = 1.2 \cdot \frac{r_s}{d} \cdot \frac{f_{yd}}{E_s} \cdot \left(\frac{m_{Ed}}{m_{Rd}}\right)^{1.5}, \qquad (16)$$

where m_{Ed} is the average bending moment in the support strip b_s, which is calculated from a linear elastic (uncracked) model. In this case an accurate evaluation of r_s is required using the abovementioned model. The width of the support strip b_s is expressed by Eq. (15).

Model Code 2010 also considers a further level of approximation, level IV, for the estimation of the slab rotation ψ. In this latter case, it can be calculated by means of a nonlinear analysis of the structure, which usually has a quite important computational cost, but this is beyond the aims of this paper.

4.3.4 Comparison between MC 2010 theoretical results and experimental findings

The theoretical results are based on the concrete mechanical properties presented in Table 3. The slab rotation has been assessed considering the abovementioned three levels of approximation defined by MC 2010. To obtain the estimation of ψ at level III, a linear elastic Finite Element (FE) model of each slab has been developed using commercial code. This model represents the structural behaviour of a large, flat slab supported by a net of columns. The depth of the slab is 0.050 m, and the distance between the axes of the columns is 2.27 m. The tested slabs represent the central region of this whole structure around an interior column. It is limited by the distance between the points in which the bending moment is equal to zero and the column axis. The model considers 61182 degrees of freedom with 10400 4-noded plates and 25 2-noded beams.

Table 8 Theoretical and experimental punching shear force of tested slabs

$V_{Rd,c}$ Model Code 2010 (kN)								
Level			**I**		**II**		**III**	
k_{dg}			1.14	2	1.14	2	1.14	2
dg(mm)			12	0	12	0	12	0
Samples	**V_{exp} (kN)**	**$V_{exp, average}$ (kN)**						
NC0-1	72.5							
NC0-2	68.7	68.70	83.00	59.18	90.06	65.60	96.27	71.49
NC0-3	64.9							
RC1-1	64.9							
RC1-2	72.5	69.97	78.50	56.00	85.18	62.05	90.41	66.99
RC1-3	72.5							
RC2-1	64.9							
RC2-2	68.7	66.17	77.50	55.27	84.10	61.26	89.26	66.14
RC2-3	64.9							
RC3-1	68.7							
RC3-2	64.9	68.70	73.85	52.67	80.14	51.38	85.06	63.04
RC3-3	72.5							
RC4-1	68.7							
RC4-2	68.7	69.97	70.15	50.03	76.12	55.45	80.80	59.87
RC4-3	72.5							

Particular attention has been devoted to the value of the parameter k_{dg} defined in Eq. 12 and presented above. To investigate its influence on the punching shear strength estimation, two extreme cases have

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been considered: k_{dg} = 2.00 considers the reduced aggregate interlock contribution in the case of aggregate break, and k_{dg} = 1.14 is related to the maximum size aggregate equal to 12 mm.

Table 8 reports the comparison between the theoretical punching shear strength $V_{Rd,c}$ obtained by means of the MC 2010 model using the three different levels of approximation and the experimental value V_{exp}. The corresponding Figs. 14, 15 and 16 present the ratio between the theoretical and experimental values for each approximation level.

As expected, when the approximation level is higher, the punching shear strength estimation is better, so for the same slab, the punching resistance obtained with level III is higher than the corresponding one with level I. This trend is the same for both NC and RC slabs.

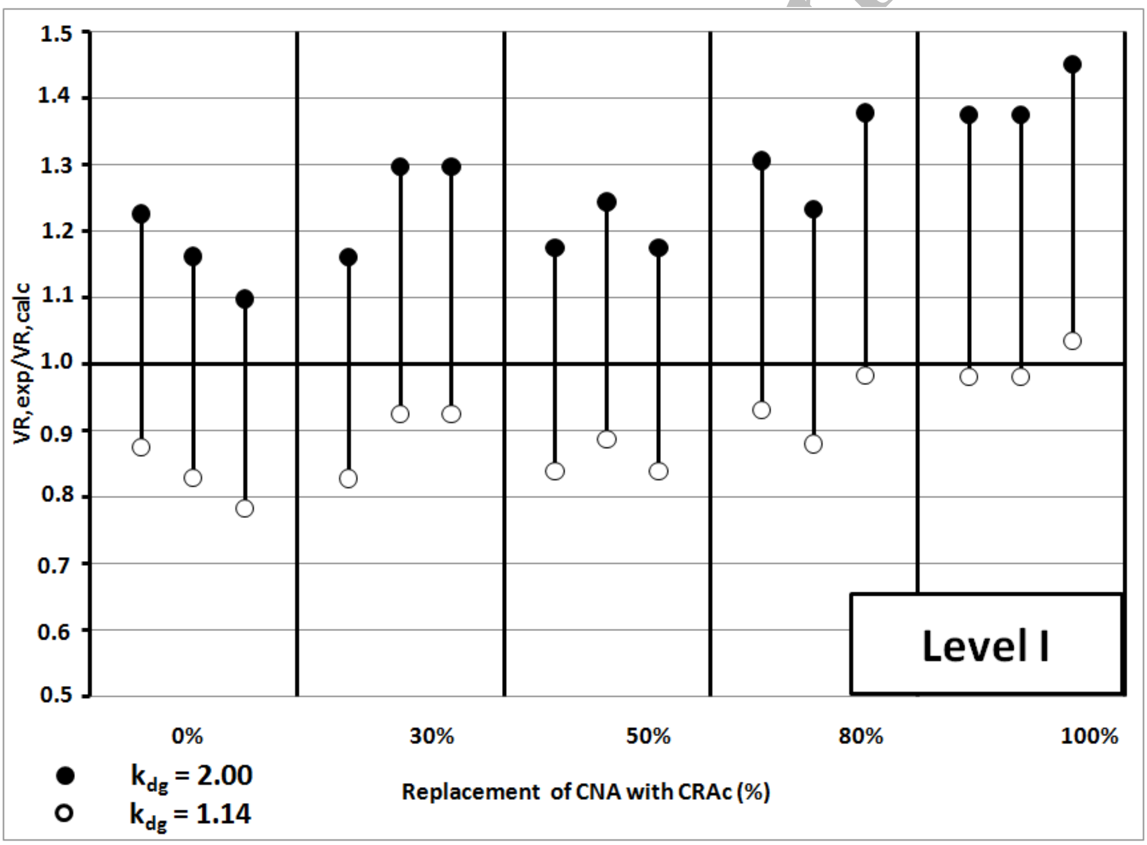

Fig. 14 Ratio between experimental V_{exp} and the theoretical level I MC 2010 $V_{Rd,c}$ punching shear strength considering different replacement ratios and two extreme values of k_{dg}.

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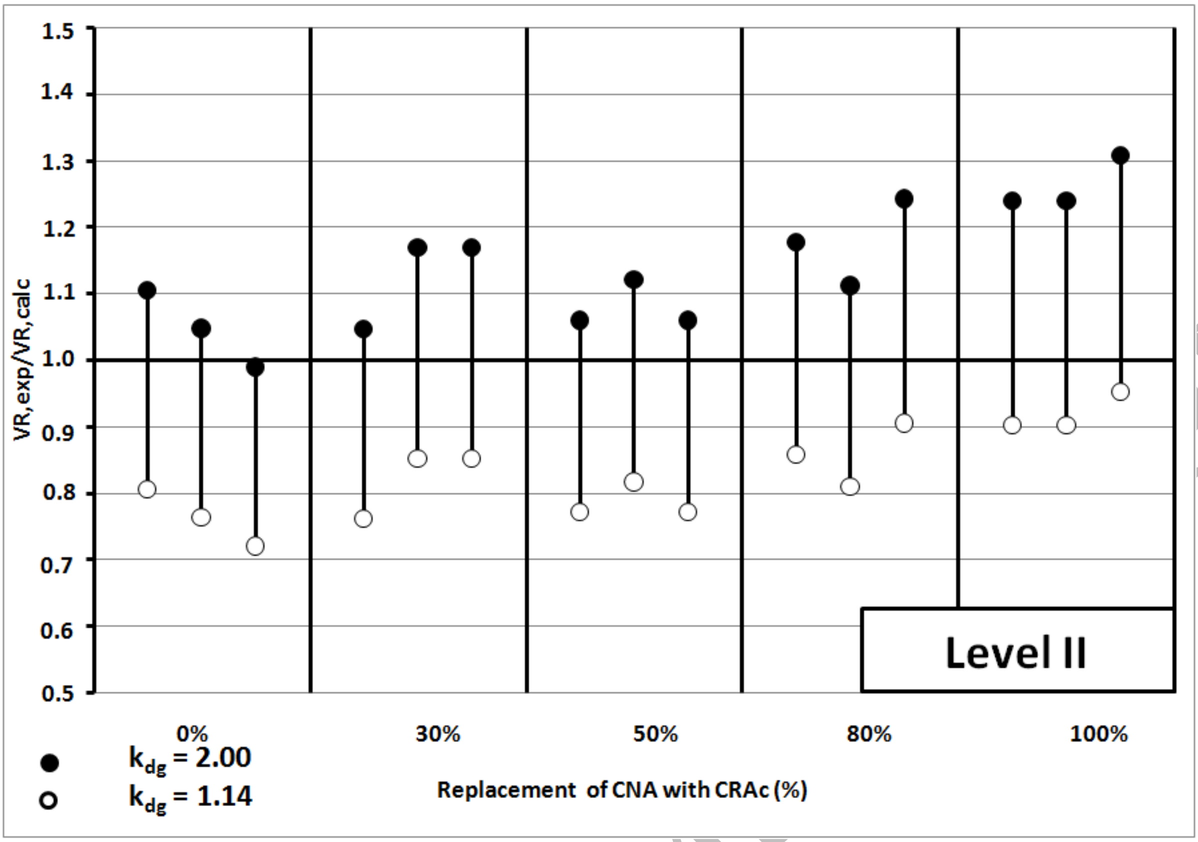

Fig. 15 Ratio between experimental V_{exp} and the theoretical level II MC 2010 $V_{Rd,c}$ punching shear strength considering different replacement ratios and two extreme values of k_{dg}.

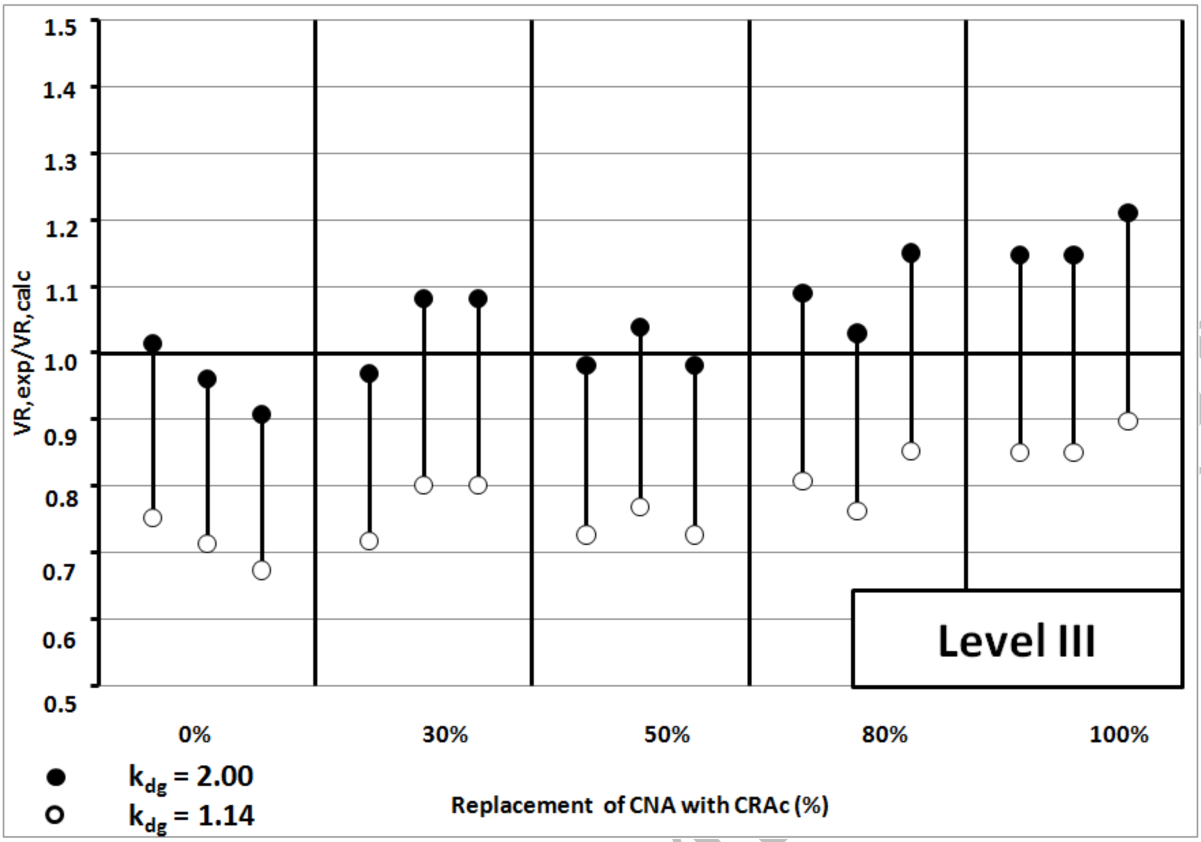

Fig. 16 Ratio between experimental V_{exp} and the theoretical level III MC 2010 $V_{Rd,c}$ punching shear strength considering different replacement ratios and two extreme values of k_{dg}.

Table 8 shows that values of k_{dg}, depending on the maximum aggregate size d_g, see equation (12), have a strong influence on the final result; indeed, k_{dg} = 2 produces a conservative result for each case, whereas k_{dg} = 1.14 yields non-conservative results. For this reason, in the case of RC slabs, a thorough analysis should be performed for its estimation and some further considerations are presented in the following.

The compressive strength f_c and the maximum aggregate size d_g are key parameters in MC 2010 punching shear model. As already pointed out the replacement percentage strongly influenced the mechanical characteristic of concrete (in particular f_c) and it has been proved that the bigger the replacement percentage is the lower the mechanical characteristics are. Indeed, the experimental average cylindrical compressive strengths presented in Table 3[1], highlight that RC with replacement percentage below 80% can be considered high strength concretes even if they are produced with RA. As mentioned in Section 3 it has been observed, in fracture surfaces after splitting tensile test, that often the failure occurs in the original aggregate. This failure behaviour is also characteristic of high strength concrete and, in this case, the MC 2010 considers d_g=0.

Table 9 Tuned values of d_g that minimize the difference between the MC 2010 model and the experimental results.

Sample	Rep%	$f_{c,\,cyl}$ (N/m^2)	d_g (mm) Level I	Level II	Level III
NC0-1	0	71.1	6.05	2.83	0.38
NC0-2			4.17	1.23	0.00
NC0-3			2.42	0.00	0.00
RC1-1	30	63.6	4.14	1.19	0.00
RC1-2			8.26	4.71	2.30
RC1-3			8.26	4.71	2.30
RC2-1	50	62.0	4.56	1.55	0.00
RC2-2			6.60	3.30	1.05
RC2-3			4.56	1.55	0.00
RC3-1	80	56.3	8.56	4.97	2.53
RC3-2			6.27	3.02	0.80
RC3-3			11.06	7.10	4.41
RC4-1	100	50.8	10.94	7.00	4.32
RC4-2			10.94	7.00	4.32
RC4-3			12.00	9.44	6.48

[1] According to MC 2010 concrete that present characteristic compressive strength f_{ck} higher than 50 MPa is considered high strength concrete. Please consider that Table 3 reports experimental average value f_c and that f_{ck}=f_c-8 MPa, see equation 5.1-1 MC 2010.

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However, the actual aggregate interlock contribution can not be assessed with high accuracy for each case and it is interesting to study the variation of d_g to the replacement percentage. Thus, if d_g is considered unknown and it is tuned to minimize the difference between the model and the experimental results using a least squared approach, a quite clear trend is obtained. Table 9 reports the tuned value of d_g for each slab while its average for each slab group is presented in Figure 17. As the replacement percentage increases the value of d_g increases. This trend can confirm the MC 2010 approach that considers d_g=0 for high strength concrete and the actual value of the maximum aggregate size for normal concrete. Indeed, the value of the actual maximum aggregate size is 12 mm, see Table 1, and it represents a superior limit of the d_g average trends reported in Figure 17.

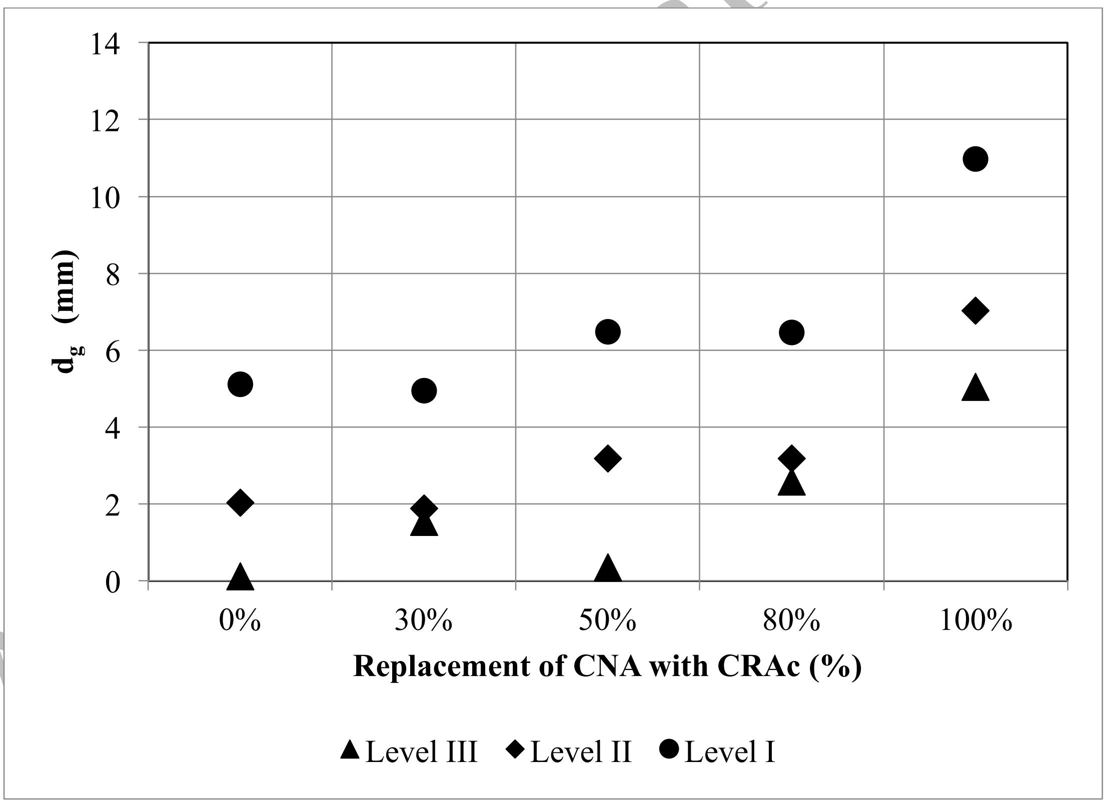

Fig. 17 Variation of the average tuned values of d_g that minimize the difference between the MC 2010 model with the three approximation levels and the experimental results.

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5 Conclusions

In this paper, the experimental results of the punching shear behaviour of simply supported NC and RC slabs are reported. Furthermore, a comparison between experimental results and those obtained by using the models of three international design codes (ACI 318, EC 2 and MC 2010) is discussed.

The use of CRAc produces a reduction in RC performance. Compressive strength, splitting tensile strength and modulus of elasticity decrease with increasing replacement percentage of CNA with CRAc.

In all slabs, punching shear failure was detected regardless of the presence of recycled aggregates.

The reduction in RC strength does not translate directly to the performance of RC slabs; indeed, their punching shear strength is very similar to that of NC slabs. This can be explained considering that the reinforcements, equal for all tested slabs, give an important contribution at the ultimate limit state regardless of the concrete mechanical characteristics.

There is no evident reduction in the first crack load with increasing replacement percentage of CNA with CRAc. The first crack load of RC slabs for replacement percentages of 30% and 50% is increased by 15% with respect to NC slabs; for an 80% replacement percentage, there is a reduction of 11%, and for higher replacement percentages, the first crack load is very similar to that of NC. As mentioned before, significant differences in the ultimate load between RC slabs and NC slabs have not been found.

Deflections at first crack and ultimate loads are significantly variable. The maxima deflections for each group of slabs increase with increasing replacement percentage. The differences between RC and NC slabs in deflection at first crack load reach 10% for replacement percentages until 50%.

The theoretical models of international design codes (ACI 318, EC 2, MC 2010), oriented to determine the NC slab punching shear strength, also present quite good performances in the case of

RC slabs. Indeed, the average differences between these models and the experimental results are approximately ±18%.

In the case of ACI 318, particular attention must be devoted to the estimation of the parameter λ that considers the density of concrete or the density of aggregates. It can strongly modify the punching shear strength.

In the case of EC 2, the parameter k seems to be very important in the punching assessment for the slabs analysed in this paper, probably because of their slenderness. Finally, by using MC 2010, the k_{dg} parameter, and consequently the maximum aggregate size d_g, play a role of paramount relevance for punching assessment at all levels of approximation and it is necessary to carefully evaluate it. The relationship between d_g and the punching shear strength for RC slabs is promising and the needs for further experimental tests is assessed.

In conclusion, this paper proves that the reinforced recycled concrete slab performance is similar to those realized with reinforced ordinary concrete, and the use of CRAc in reinforced slabs should be encouraged.

Furthermore, for the first time in the literature, this paper demonstrates the feasibility of using coarse recycled aggregates obtained by concrete waste with unknown mechanical properties to realize excellent reinforced RC slabs. Moreover, although the theoretical models on the punching shear are based on the characteristics of the concrete, this study suggests that the reinforcement plays a role of paramount relevance, and it would be interesting to propose a simplified analytical model for punching resistance that also considers this aspect. This idea is currently under investigation and is part of the further developments of this research, which include a nonlinear numeric model and a new experimental campaign with different geometrical dimensions and reinforcement ratios in order to investigate respectively the scale effect and the reinforcements contribution.

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Appendix I

In this appendix, the punching cone radius r recorded every β = 30°, see Fig. 8, are presented for each specimen (see Tables 10–14).

Table 10 Punching cone dimensions for NC slabs.

Sample	Direct. β°	r_i (mm)	Sample	Direct. β°	r_i (mm)	Sample	Direct. β°	r_i (mm)
NC0-1	0	112	NC0-2	0	125	NC0-3	0	79
	30	100		30	179		30	116
	60	98		60	126		60	130
	90	105		90	136		90	109
	120	103		120	102		120	97
	150	120		150	100		150	109
	180	124		180	183		180	98
	210	113		210	207		210	144
	240	100		240	159		240	119
	270	103		270	86		270	89
	300	99		300	67		300	94
	330	102		330	73		330	110

Table 11 Punching cone dimensions for RC1 slabs.

Sample	Direct. β°	r_i (mm)	Sample	Direct. β°	r_i (mm)	Sample	Direct. β°	r_i (mm)
RC1-1	0	138	RC1-2	0	153	RC1-3	0	102
	30	145		30	188		30	91
	60	121		60	141		60	96
	90	128		90	191		90	88
	120	128		120	131		120	108
	150	132		150	213		150	94
	180	167		180	214		180	92
	210	172		210	261		210	119
	240	153		240	183		240	138
	270	163		270	165		270	141
	300	136		300	130		300	112
	330	150		330	160		330	100

Table 12 Punching cone dimensions for RC2 slabs.

Sample	Direct. β°	r_i (mm)	Sample	Direct. β°	r_i (mm)	Sample	Direct. β°	r_i (mm)
RC2-1	0	85	RC2-2	0	82	RC2-3	0	93
	30	131		30	85		30	102
	60	114		60	85		60	101
	90	101		90	111		90	103
	120	91		120	88		120	98
	150	119		150	78		150	97
	180	174		180	107		180	110
	210	170		210	124		210	115
	240	125		240	122		240	125
	270	84		270	92		270	119
	300	108		300	70		300	100
	330	102		330	82		330	99

Table 13 Punching cone dimensions for RC3 slabs.

Sample	Direct. β°	r_i (mm)	Sample	Direct. β°	r_i (mm)	Sample	Direct. β°	r_i (mm)
RC3-1	0	169	RC3-2	0	183	RC3-3	0	84
	30	130		30	108		30	122
	60	182		60	170		60	121
	90	161		90	183		90	111
	120	144		120	183		120	116
	150	125		150	192		150	98
	180	135		180	191		180	111
	210	148		210	146		210	170
	240	147		240	117		240	133
	270	106		270	100		270	85
	300	133		300	133		300	81
	330	119		330	138		330	80

Table 14 Punching cone dimensions for RC4 slabs.

Sample	Direct. β°	r_i (mm)	Sample	Direct. β°	r_i (mm)	Sample	Direct. β°	r_i (mm)
RC4-1	0	83	RC4-2	0	99	RC4-3	0	87
	30	127		30	98		30	82
	60	131		60	151		60	121
	90	112		90	128		90	143
	120	69		120	126		120	143
	150	98		150	121		150	152
	180	104		180	166		180	135
	210	142		210	154		210	134
	240	150		240	148		240	133
	270	113		270	137		270	104
	300	138		300	133		300	109
	330	169		330	97		330	105