Destructive and non-destructive tests on columns and cube specimens made with the same concrete mix

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A R T I C L E I N F O

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A B S T R A C T

This research work presents the results of an experimental campaign performed on column specimens to investigate the capacity of destructive coring test and non-destructive tests to predict the in-place concrete strength. The considered non-destructive tests are Surface Hardness (SH) and Ultrasonic Pulse Velocity (UPV) ones. Using these tests' results also SonReb method is applied. The peculiarity of this research is that the results of all the performed test on the columns are compared to results obtained from cube specimens made with the same concrete of the columns. The non-destructive predictive methods are performed on both column and cube specimens during the curing of the concrete at different ages, while the destructive coring test on columns specimens at 28th day after casting. A discussion on the comparison between drilled cores' strength, determined using provisions of different building codes, and cube specimens' strength is provided. In particular, the code providing the safest concrete strength predictions is identified.

Moreover, the influence of different core diameters on the evaluation of in-place concrete strength is analyzed. The variation of core specimens' strength, depending on the position of test execution along the columns' height, is also investigated. To assess the in-place concrete strength by non-destructive tests, correlation formulae are calibrated on the bases of cores strength determined according to *ACI* Code. The prediction ability of these formulae is assessed. Results obtained from SH and UPV tests at different concrete ages are compared to the cube specimen strength at the same ages, and the influence on the measured parameters of different moisture conditions during curing is analyzed. The minimum number of cores to adequately predict the in-place concrete strength by means of SH and UPV methods is also determined.

Best predicting SonReb formulations are identified.

1. Introduction

In countries prone to seismic activity the existing building heritage is subjected to seismic risk, because many old buildings were designed without using principles of good seismic behavior.

In Italy more than 60% of the existing buildings are exposed to seismic risk, having been designed without a seismic code. Among the residential buildings, which constitute the majority of existing buildings (about 84% in Italy), 30% have a reinforced concrete load bearing structure [1]. Seismic events of recent decades have revealed that even RC buildings, if designed for gravity loads only, are prone to severe risk; in fact, they can develop severe damage or even collapse, as highlighted in [2].

Since the 2000s the Italian Government has encouraged seismic improvement interventions for existing structures, to secure the housing stock and the people who occupy it. To design these types of interventions, it is necessary to know the characteristics of the materials which constitute the buildings. For RC structures it means knowing the characteristics of the concrete and reinforcement [3]. As regards concrete, a comprehensive knowledge of its mechanical properties allows the investigator to predict, from the analysis, the deformation behavior of the building close to the real one. However, the estimation of in-place concrete properties is challenging. For this reason, the most advanced building codes [4–7] provide that knowledge of these properties shall be attained by carrying out destructive compressive tests, as well as by obtaining data through non-destructive methods.

The destructive test consists of drilling of core specimens from the structure and in subjecting them to compression tests [8]. Technical Codes [9,10,11] provide procedures to perform the drilling and formulations to obtain the concrete strength on the basis of some

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Table 1
Concrete mix design for 1 m^3 and slump test results.

Concrete type		C25/30	C28/35	C32/40
Aggregates weight	[kg]	1950	1970	1930
Cement weight	[kg]	280	310	350
Water	[l]	160	160	160
Water/cement ratio		0.57	0.52	0.46
Slump test lowering	[mm]	210	210	160

influencing parameters such as core diameter, aspect ratio, moisture condition, drilling direction, presence of steel reinforcement bars, and type and size of aggregate. Although destructive methods are expensive, time-consuming operations, their results are reliable and useful [12–14].

The effect of cutting damage on cores reduction strength increases with the decrease of core diameter [15]. For this reason building codes provide correction factor of cores strength accounting of core diameter effect. In this research the effect on core strength of accurately performed drilling, carried out paying attention in avoiding damages to specimens, is investigated.

Another important factor affecting cores strength is the position of drilling. In particular, Qasrawi showed that concrete casting and compaction methods may lead to strength variations along columns height [16], evidencing the presence of weaker zones at the bottom and at the top of the column. One of the aims of this research is to investigate if accurate concrete casting and compaction avoids strength variations along the structural element.

As regards non-destructive methods, the most widespread ones are the Schmidt surface hardness [17], the ultrasonic pulse velocity [18] and the Sonic Rebound (SonReb) [19], which is a combination of the Schmidt SH and the UPV methods.

The Schmidt sclerometer is the instrument used to perform the SH test [20]. It consists of a steel beating mass, driven by a mechanical spring, which, after release, projects a percussion rod on the test surface. The measured rebound height of the mass after the contact with the concrete surface is related to the surface hardness, which is an index of concrete compressive strength. At each measurement the sclerometer provides the Rebound Index (RI). The manufacturer of the sclerometer provides a strength estimation curve, which allows the user to determine a theoretical concrete strength from the measured values of RI. However, the validity of the relationship is restricted to specific testing conditions and an extension of that validity to different types of concrete or testing circumstances is impossible [21–23]. A reliable relationship between concrete strength and RI can be obtained only if experimental results obtained from destructive tests are available and the relationship is calibrated based on these results. Actually, surface hardness and compressive strength of concrete depend on several parameters, which are related to concrete admixture quality and to concrete degradation state. Some of these parameters have opposite effects on surface hardness and concrete compressive strength. It is the case of carbonation [24], which may lead to a significant increase of surface hardness and, at the same time, to a reduction of concrete compressive strength. Another kind of degradation affecting concrete mechanical properties is steel bars corrosion [25].

The UPV method is based on the phenomenon of propagation of ultrasonic waves with a variable frequency in the range of 20–120 Hz. The test device is made of two probes: a mechanical pulse emitter and a corresponding receiving device in transparency. The time between the emission of the signal and its reception is measured and, being the distance between the probes known, the wave velocity is obtained. Since the velocity is correlated to the concrete stiffness, this method allows the user to obtain the concrete modulus of elasticity, the Poisson's ratio and also the concrete strength. As in the case of the SH method, a reliable relationship between the waves propagation velocity and the concrete strength can be obtained only if it is calibrated based on results obtained from destructive tests performed on the same concrete.

Having RI and UPV measurements allows the combined SonReb method to be applied to estimate the concrete strength. The advantage of using this method is that the variation of some concrete parameters produces opposite effects on the results of the two component tests, thereby reducing the error in the concrete strength prediction, as outlined in [26]. In particular, the SonReb method reduces the effects connected to the aggregate size, cement type and content, water to cement ratio and moisture content. Also in this case a reliable relationship for the concrete strength determination should be obtained by taking account of destructive test results. However, several relationships are available in the literature to determine concrete strength with the SonReb method, as shown in [27]. Hence, without calibrating an *ad-hoc* relationship on destructive test results, it is also possible to use the available relationship which best predicts these results.

In this research, the capacity of predicting concrete strength based on core compressive strength tests and non-destructive surface hardness, ultrasonic pulse velocity and SonReb methods is investigated. In particular, the minimum number of core specimens necessary to obtain adequate predictions from correlation curves is determined.

2. Research goals

Regarding destructive tests on core specimens, a first goal of the research is to compare cores strength, from which in-place concrete strength is derived, with the strength of cube specimens made with the same concrete mix. The cores strength is determined by using provisions of different building codes and different cores diameters. The comparison allows to determine both what code and what diameter provide the safest concrete strength predictions with respect to the cube strength.

Another goal is to determine how much the core strength can vary depending on the drilling position.

Regarding non-destructive tests, both for SH and UPV tests, a first goal is to investigate if there are differences between these tests' results obtained from the column specimens and the corresponding cube specimens, for different days of curing. This comparison allows to understand how different curing conditions, between real structures and cube specimens, affect the measured parameters, which are used to derive the concrete strength.

Another goal is to determine how many core specimens are sufficient to calibrate formulas which adequately predict in-place concrete strength from the parameters measured in SH and UPV tests [28].

Finally, regarding the SonReb method, the goal of this research is to evaluate the in-place concrete strength predicting capacity of formulations available in the literature and to identify the ones providing the best predictions.

3. Experimental investigation

The experimental investigation has been carried out in order to meet the research goals previously specified. To simulate the execution of destructive and non-destructive tests on existing RC structural members, three short columns were prepared. The column specimens were made with three different concrete mixes, having different grades. After having subjected the column specimens to non-destructive tests, concrete cylinders were extracted by coring from the columns at the 28th day of curing.

To assess the capacity of destructive and non-destructive tests in predicting in place-concrete strength, cube specimens were prepared with the same concrete mixes used for casting the column specimens. The specimens characteristics, the test program and the methods of testing and data analysis are described in the following subsections.

3.1. Specimens characteristics

3.1.1. Concrete properties

Three different concrete mixes, with different grades, were used to

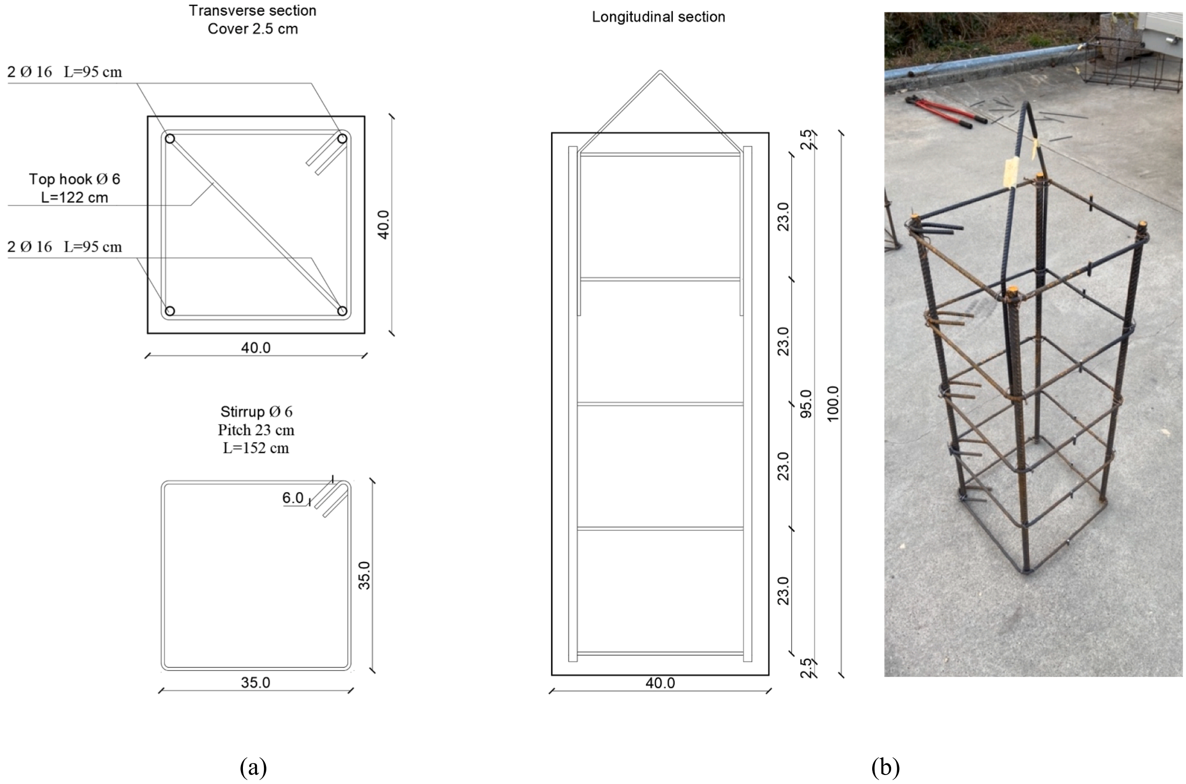

Fig. 1. Column specimen: (a) cross- and longitudinal sections; (b) photograph of the reinforcement cage.

Fig. 2. Cube specimens inside the curing tank.

produce the specimens described in the following. Their nominal cube strengths, R_{ck}, were 30 MPa, 35 MPa and 40 MPa. Hence they were identified with the strength class labels C25/30 (normal grade concrete), C28/35 (common concrete for domestic and commercial applications) and C32/40 (strong commercial grade concrete), respectively.

The cement used was a hydraulic binder obtained from the grinding of Portland clinker, natural limestone, and gypsum. The aggregates were composed of a mix of washed sand (diameter of 0–4 mm) and gravel (diameter of 8–16 mm). The mix design used to produce 1 m^3 of concrete is reported in Table 1, together with the results of the slump test.

(a) (b)

Fig. 3. Cylindrical specimen: (a) coring; (b) rectification.

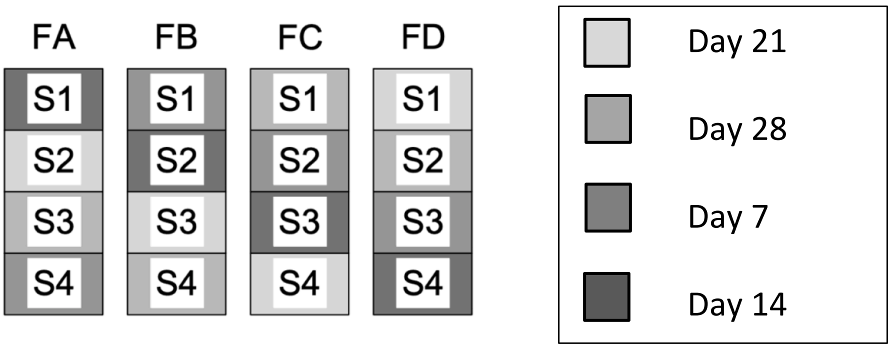

Fig. 4. Representation of the four groups of test sub-faces, on the faces of a single column specimen.

3.1.2. Column specimens

Each of the three columns was vertically casted with a different concrete mix (Fig. 1): one each with C25/30, C28/35, and C32/40. Each column had a square base with dimensions 400x400 mm^2 and height equal to 1000 mm.

Steel reinforcements in the columns were designed to be like those which can be found in existing RC buildings designed without reference to seismic codes. Ribbed bars of steel grade B450C [1] were used. Diameter and geometry of the reinforcements are depicted in Fig. 1, where it can be seen that there are 4 longitudinal bars with 16 mm diameter and stirrups with 6 mm diameter spaced of 230 mm apart.

The stirrups' spacing was considered sufficient for the execution of all the foreseen tests, in particular for the extraction of the concrete cores. Also a triangular hook, visible in the upper part of the reinforcement cage, was inserted to facilitate the handling of the specimens.

In order to avoid concrete strength inhomogeneities along the column height due to concrete segregation, each column was casted in two layers, with the same height, each of them singly vibrated.

All non-destructive tests on the column specimens were completed prior to the drilling of cores to be used for the destructive compression test.

3.1.3. Cube specimens

After the columns were cast, the remaining concrete was used to prepare 14 cube specimens for each type of concrete mix, as those used in [29]. These specimens were prepared according to UNI EN 12390–1 [30] and 12390–2 [31], hence their dimensions were 150x150x150 mm^3.

The specimens were casted in two phases, since, according to [31] they must be compacted in at least two layers. Then they were left inside the cube molds for 16–24 h, to protect them from shocks, vibrations and dehydration that could have damaged them. After this period of time, they were removed from the formworks and placed inside a curing tank (Fig. 2) ensuring a temperature of 20 $\pm$ 2 °C throughout the curing process.

All non-destructive tests on the cube specimens were completed prior to their subjection to destructive compression tests.

3.1.4. Core specimens

For each column, 8 cylindrical specimens with a height/diameter ratio equal to 1 were extracted (Fig. 3 (a)), 4 of which with a diameter of 104 mm and 4 with a diameter of 94 mm. Two different diameters were adopted in order to have an estimate of the size effect of specimen size on the evaluation of the concrete compressive strength.

After their circular bases had been rectified (Fig. 3 (b)), the specimens were subjected to destructive compression tests.

3.2. Test program

The testing program was organized as follows.

The SH and UPV tests were performed on the cube and column specimens at 7, 14, 21 and 28 days after casting. Moreover, on each of

(a) (b) (c)

Fig. 5. Phases of the SH test: (a) smoothing of the surface; (b) drawing of the test points; (c) execution of the test.

those days, two cube specimens per concrete mix typology were subjected to a compression test.

Two cube specimens were also subjected to a compression test on the 3rd day from casting, to have the complete curing curve of the three different concrete mixes.

On the 28th day, after having performed that day's SH and UPV tests, concrete cores were extracted from the columns. Then, the cores and the remaining cube specimens (6 per concrete mix typology) were subjected to the compression test.

To reduce the influence of possible inhomogeneities existing along the height and on the four vertical faces of the columns on tests' results, it was decided to carry out the investigations at different heights on these faces. In particular, with reference to Fig. 4, the lightest gray color indicates the heights at which the tests were performed on the different faces of the column on the 21st day. The other grays, progressively darker, indicate the heights at which the tests were performed on the 28th, 7th and 14th day, respectively. The abbreviations S1, S2, S3 and S4 simply indicate the heights of the tests area along the specimen, and FA, FB, FC and FD indicate the position of the test area with respect to the four vertical faces of each column. On a given day, each column would therefore have tests performed on four distinct sub-faces, e.g., FA-S1, FB-S2, FC-S3, and FD-S4 on the 14th day.

3.3. *Methods of testing and data analysis*

3.3.1. *Surface hardness test on column specimens*

Before performing the SH test, the test area was smoothed by means of the abrasive stone supplied with the sclerometer, as shown in Fig. 5 (a).

In order to comply with UNI EN 12504–2:2021 for the determination of RI [17], a template, visible in Fig. 5 (b), was applied to the test area. The template's holes, spaced 4.5 cm apart, were used to draw the hammer impact points on all 16 sub-faces of each column. Then the test was performed as shown in Fig. 5 (c), and the RI read at each test point was recorded.

For each day of investigation, 7th, 14th, 21st and 28th, the test was performed at multiple points on each column, in four distinct sub-faces, as depicted in Fig. 4.

The median of the recorded RIs was then determined for each sub-face. The overall column RI on a given day was taken to be the average of the four median RIs determined for the sub-faces of that column on that day.

3.3.2. *Surface hardness test on cube specimens*

Cube specimens were prepared for the SH test in a similar manner to the columns.

In order to prevent the specimen from moving under the hammer impact, it was placed between the plates of a press, as shown in Fig. 6(a), under a stress approximately equal to 1 MPa. These specimens were subjected to the test from the 14th day onwards.

For each cube, the median of the recorded RIs was determined for each day of testing. Then, the average of the median RIs determined for the cube specimens at a day of testing was determined.

3.3.3. *Ultrasonic test on column specimens*

The UPV test was performed on the aforementioned sub-faces used previously for the SH test, in order to subsequently use the obtained data for the application of the SonReb method.

To have knowledge of the exact distance between the pulse emitter and the receiving device, care was taken that the probes were placed at the same height on opposite faces of the column, as shown in Fig. 6(b). Then the UPV was recorded four times, moving the pair of probes around the column in all the possible combinations.

The average of the four measured propagation velocities obtained for each specimen was considered.

3.3.4. *Ultrasonic test on cube specimens*

The UPV test was performed on the cube specimens in a similar

(a) (b) (c)

Fig. 6. Execution of: (a) SH test on cube specimen, (b) UPV test on column, and (c) UPV test on cube specimen.

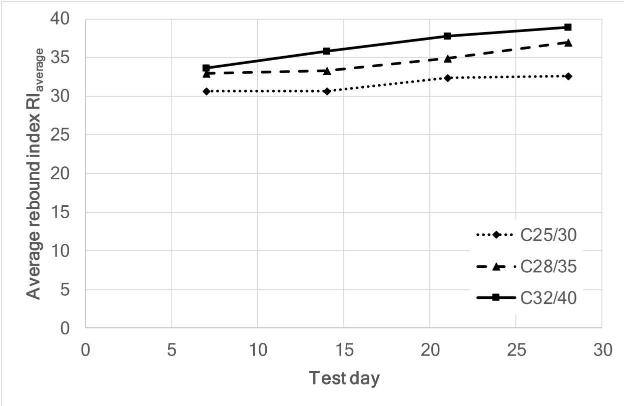

Fig. 7. Average RI values versus concrete age for each group of column specimens made with concrete of the same grade.

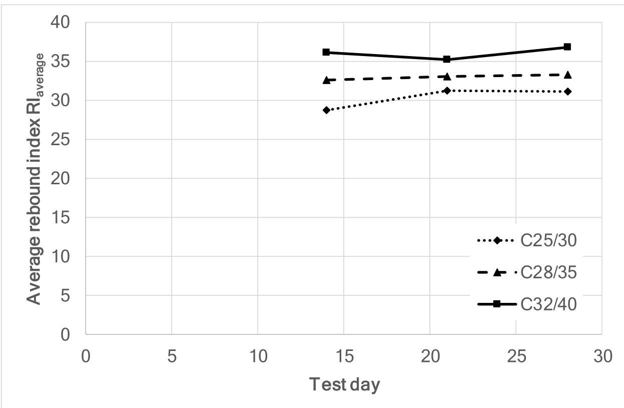

Fig. 8. Results of SH test on cube specimens.

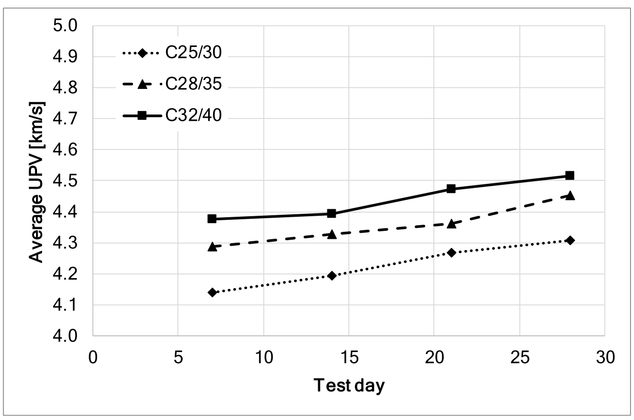

Fig. 9. Results of UPV test on column specimens.

manner to column specimens, identifying the centers of two opposite faces for the application of the probes Fig. 6(c).

For each concrete mix, the average UPV values of the 14 cube specimens obtained for each day of testing are considered.

3.3.5. Compression test on cube specimens

Compression tests were performed on cube specimens on the 3rd, 7th, 14th, 21st and 28th day from casting, to have the complete curing curve for the three used concrete mixes.

For each concrete mix, the average compressive strength values of

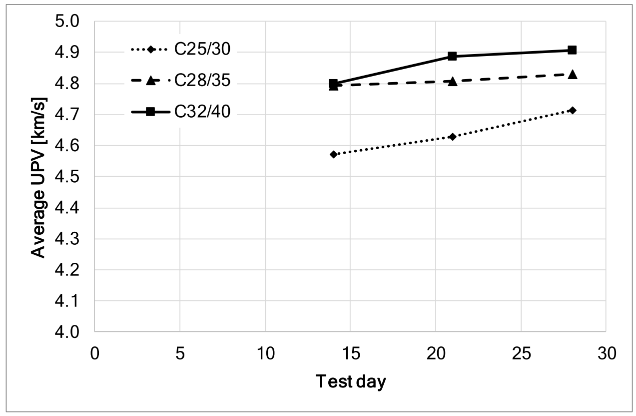

Fig. 10. Results of UPV test on cube specimens.

the cube specimens obtained for each day of testing are considered.

3.3.6. Compression test on core specimens

Compression tests were performed on the cylindrical specimens (cores extracted from the columns) at the 28th day, after having executed the non-destructive tests on the columns.

The average strengths of core specimens extracted from a column with the same diameter are considered.

4. Experimental results

4.1. Surface hardness test results

4.1.1. Column specimens

The individual measurements are reported in appendix A (from Table A.1 to Table A.3).

The average RI values for the individual column specimens are plotted versus concrete age in Fig. 7. As it can be seen from this figure, the average RI values increase almost linearly with the increase of the concrete age. Hence, in cases where it is useful/necessary to monitor the increase over time of fresh concrete strength, as those considered in [33], this can be done by monitoring the RI values over time.

Furthermore, specimens characterized by stronger, commercial concrete show consistently higher average RI values.

4.1.2. Cube specimens

The average RI values for the individual cube specimens are plotted versus concrete age in Fig. 8. Similar to the column specimens, for the cube specimens, the average RI values are greater for specimens prepared with higher-grade concrete.

4.2. Ultrasonic test results

4.2.1. Column specimens

The individual measurements are reported in Appendix B (from Table B.1 to Table B.3). The average values are shown in Fig. 9, plotted versus concrete age. As it can be seen from this figure, the average UPV values increase almost linearly with increase in concrete age. Hence, in cases where it is useful/necessary to monitor the evolution over time of concrete curing, as those considered in [33], this can be done by monitoring UPV values over time.

Furthermore, specimens prepared with higher-grade concrete show higher average UPV values.

4.2.2. Cube specimens

The propagation velocities determined for each cube specimen are shown in Fig. 10.

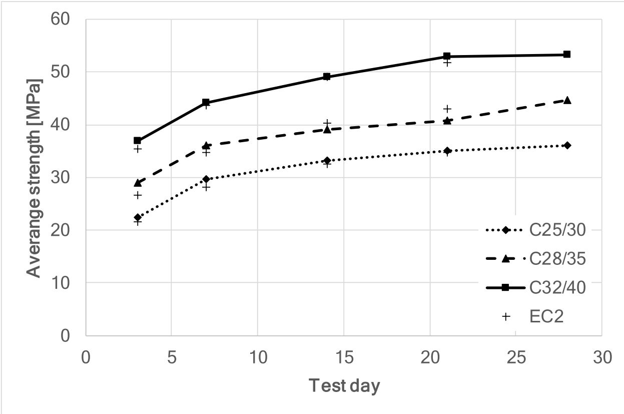

Fig. 11. Compression test results of cube specimens, and strengths predicted by Eurocode 2 [32].

4.3. *Compression test results*

4.3.1. *Cube specimens*

All the results of the tests are reported in Appendix C (from Table C.1 to Table C.3) and the average strength values are shown in Fig. 11 and reported in Table 2 at column (2).

As expected, concrete strength increases with the age and tends to stabilize from the 21st day of curing onwards.

It is interesting to compare the obtained concrete strengths with the strengths predicted by Eurocode 2 at age variation [32]. According to EC2, the compressive strength of concrete at an age *t* depends on the type of cement, temperature and curing conditions. For a mean temperature of 20 °C, and curing in accordance with EN 12,390 [31], as done in the present research work, the concrete compressive strength at time *t*, $R_c(t)$, may be estimated by.

$$R_c(t) = \beta_{cc}(t) \bullet R_{c28} \tag{1}$$

where R_{c28} is the cube strength at the 28th day of curing and $\beta_{cc}(t)$ is a coefficient that depends on the age of the concrete.

$$\beta_{cc}(t) = e^{\left\{ s\left[1-\left(\frac{28}{t}\right)^{\frac{1}{2}}\right]\right\}} \tag{2}$$

with s equal to 0.25 for C25/30 and C28/35 and s equal to 0.2 for C32/40 in this specific case.

The values of strength given by Eq. (1) for ages from the 3rd day to the 21st day are reported in Fig. 10, where it can be seen that EC2 formula appears to be accurate in its prediction, with the values calculated with Eq. (1), being very close to the measured ones in this case. The empirical points in Fig. 11 support the premise of Eq. (1); namely, that the increase of concrete strength with the age is governed by a nonlinear behavior, unlike the trends of RI values (Fig. 7) and UPV (Fig. 9) which increase almost linearly with curing age from day 7 to day 28.

It can also be observed that, for all the concrete mixes tested, the attained strength at the 28th day is greater than the nominal value. The percentage difference between the attained strength and the nominal one is equal to 20%, 27% and 33%, for C25/30, C28/35 and C32/40, respectively. Hence this difference increases with the increasing of the concrete strength.

4.3.2. *Core specimens*

The results of the tests are reported in Appendix D (from Table D.1 to Table D.3) and the average strength values are summarized in Table 2 at column (4). From the reported results it can be observed that, for all three concrete mixes, higher strengths are obtained for the smaller sized specimens. Generally, for drilled cores, the contrary is expected, i.e. the strength is expected to decrease with the diameter, because the ratio of cut surface area to volume increases and, hence, the possibility of strength reduction due to cutting damage increases [12]. For molded concrete, however, specimen strength is expected to decrease with increase in specimen size, due to the size effect [15]. According to this theory, the larger the specimens volume is, the more probable it is that they contain a defect and, thus, they fail at a lower stress.

In the specific case of tests described herein, it can be said that the results are more influenced by the size effect than by the effect of damage due to drilling. This is probably because great attention was paid to the drilling phase, to avoid, as far as possible, damages to the specimens.

5. Discussion of test results

A peculiarity of this research is that, in addition to the results of non-destructive and destructive tests on the columns, also the results of the same tests on cube specimens made with the same concrete as the columns and tested on the same days are available.

For the assessment of in-place concrete properties by means of non-destructive tests, building codes [9], [10] and [11] recommend calibrating the correlations between the non-destructive testing results and concrete strength using the results obtained from destructive tests carried out on the same concrete specimens. Since non-destructive tests were performed at the 7th, 14th, 21st and 28th day of curing while the cores were drilled only at the 28th day, cores strength may be used to derive only correlation laws for the prediction of concrete strength at the 28th day of curing. In order to obtain, for each non-destructive test carried out, correlation laws able to predict either the cube or the cylindrical concrete strength, the relationship between the strengths obtained from the extracted cores and the cube specimens is firstly determined.

Table 2
Average strength obtained from compression test and predictions of codes.

(1)	(2)	(3)	(4)	(5)	(6)	(7)
Concrete type	**Average strength** [MPa] **Cube specimens** (28 days)	**Cylindrical specimens** (28 days) *D** [mm]		**Italian Guidelines** [9]	***ACI* Code** [10]	**British Standard** [11]
C25/30	36.10	94	38.98	39.30	41.61	38.98
		104	32.10	33.76	33.97	32.10
C28/35	44.60	94	46.86	46.86	50.03	46.86
		104	39.19	39.44	41.47	39.19
C32/40	53.27	94	54.54	54.54	58.23	54.54
		104	48.89	48.89	51.74	48.89

**d* = diameter of cylindrical specimens.

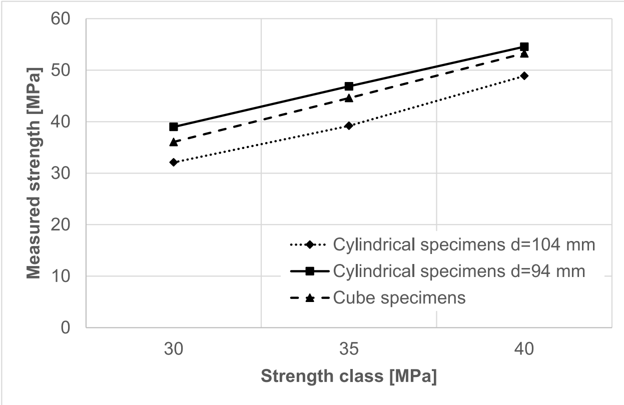

Fig. 12. Comparison among the strengths of cores with 94 mm and 104 mm diameters and the cube specimens.

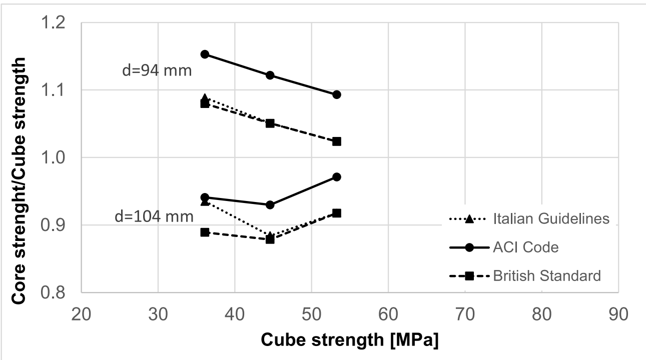

Fig. 13. Ratios of code-predicted in-place concrete strengths based on core compression tests (core strengths) to corresponding cube strengths of the three grades of concrete tested.

5.1. Compression test on extracted cores versus compression test on cube specimens

Fig. 12 shows the comparison between the 28-day strength obtained from the compression tests on the cylindrical specimens extracted by coring and that of the corresponding cube specimens. From this figure it can be observed that the cylindrical specimen results nearer to the cube compressive strengths are those of the cores having 94 mm diameter, which have slightly greater strengths than those of the cube specimens. This confirms that, for the cores with the smaller diameter (94 mm), the size effect prevails. Cores with diameter 104 mm have, instead, strengths that are lower than the strengths of the cube specimens, with a percentage difference ranging from 8% to 12%, as it can be derived from the values in Table 2 at columns (2) and (4).

The cores' strengths reported in Fig. 11 are those obtained directly from the tests on the extracted cylinders. However, to determine the in-place concrete strength it has to be taken into account that the core compressive strength is affected by a number of factors, such as core diameter, aspect ratio, moisture condition, drilling direction, presence of steel reinforcement bars, type and size of aggregate, and even strength class of the concrete. These are all the possible factors involved in the interpretation of core compressive strength test results, but the number and type of factors considered in building code provisions for this aim are different. In this research work the formulations of Italian Guidelines [6], *ACI* Code [7] and British Standard [11] are considered.

As regards Italian Guidelines [9], the in-place concrete strength is calculated by multiplying the measured strength of the core by a corrective factor, called the disturbance factor, whose value depends on the core compressive strength. In particular, the disturbance factor is equal to 1.06, 1.04 and 1.00 for core strengths equal to 30 MPa, 35 MPa and greater than or equal to 40 MPa, respectively. For intermediate strength values a linear interpolation can be made. The concrete strength calculated according to Italian Guidelines are reported in Table 2 at column (5).

According to *ACI* PRC-214.4–21 [10], the in-place concrete strength is computed using the following equation.

$$f_c = F_{l/d} \bullet F_{dia} \bullet F_{mc} \bullet F_D \bullet f_{core} \tag{3}$$

where f_c is the equivalent in-place concrete cylindrical strength and f_{core} is the concrete core strength. The other parameters have the following meanings and values:

- $F_{l/d}$ is the strength correction factor for aspect ratio, l/d (with l height of the core sample and d diameter), given by the following equation for the case of cores tested as received without being subjected to soaking or air drying

$$F_{l/d} = 1 - \left(0.13 - 4.3 \bullet 10^{-3} \bullet f_{core}\right) \bullet \left(2 - \frac{l}{d}\right)^2 \tag{4}$$

This correction factor allows to determine the concrete cylindrical strength f_c. Clearly, it assumes a value equal to 1 for $\frac{l}{d} = 2$ and lower values for $\frac{l}{d} < 2$.

In the present case, the cube strength has to be obtained, to have strength values homogeneous with those obtained from the cube specimens, since the strength is obtained from cores with $\frac{l}{d} = 1$. Therefore, the correction factor is calculated as the ratio between the factor relative to the aspect ratio of the core (given by Eq. (4)) and the correction factor relative to $\frac{l}{d} = 1$, given by $\left[1 - \left(0.13 - 4.3 \bullet 10^{-3} \bullet f_{core}\right)\right]$;

- F_{dia} is the strength correction factor for diameter, equal to 1.06, 1.00 and 0.98 for core diameters of 50 mm, 100 mm and 150 mm, respectively. For intermediate strength values a linear interpolation is made;
- F_{mc} is the strength correction factor for moisture condition of the core sample, equal to 1.00 for the case of cores tested as received without being subjected to soaking or air drying;
- F_D is the strength correction factor that accounts for the effect of damage due to core drilling, equal to 1.06.

The concrete strengths calculated according to *ACI* Code are reported in Table 2 at column (6).

According to BS 1881:part120 [11], the in-place concrete strength is computed using the equation.

$$f_c = C_{l/d} \bullet C_a \bullet f_{core} \tag{5}$$

where $C_{l/d}$ is the strength correction factor for aspect ratio, given by the following equation for core axis orthogonal to the cast direction, as in the present case.

$$C_{l/d} = \frac{2.5}{1.5 + \frac{d}{l}} \tag{6}$$

and C_a is the correction factor for the presence of reinforcing steel, equal to 1.00 for no bars, as in the present case.

The concrete strengths calculated according to British Standard are reported in Table 2 at column (7).

Fig. 13 shows the ratios between the cores compressive strength given by the aforementioned codes and the strength of the corresponding cube specimen, versus the cube strength. From this figure, it can be observed that, in general, *ACI* Code gives the highest strength predictions, while British Standard the lowest ones. Italian Guidelines gives predictions, which, except in one case, are very close to British Standard ones. On the whole, the codes mentioned above provide predictions that range from values 11 % lower to 15 % higher than the cube

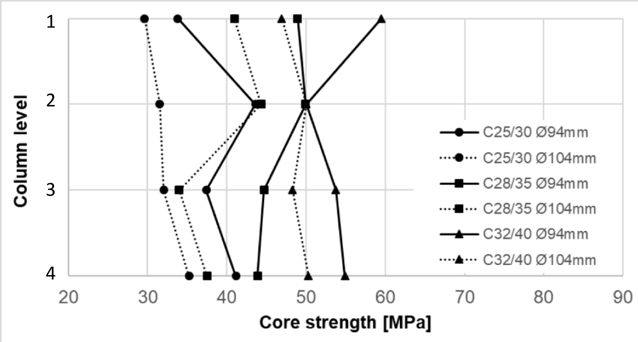

Fig. 14. Core strength along the column height (level 1 is the highest).

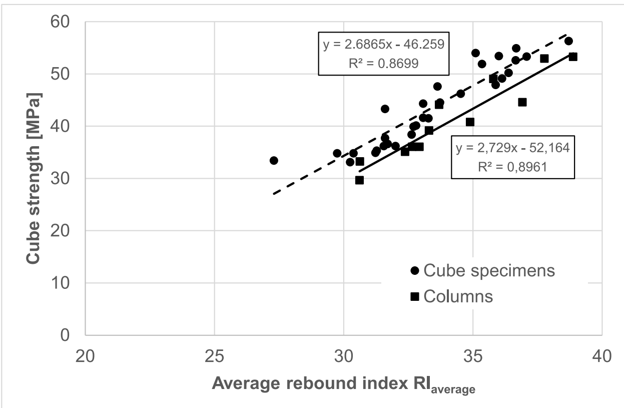

Fig. 15. Empirically based linear relationships between compressive strength of cube specimens and average rebound index obtained from tests on the columns and on the cube specimens.

strength.

While the contrary was to be expected for cores with 94 mm diameter, the predictions are greater than the corresponding cube strength, because, as it has already been observed, the cores of this specific research are affected more by size effect than drilling damage effect, as obtained in [34]. *ACI* Code and Italian Guidelines take account of the damage due to drilling by using correction factors that further increase the measured core compressive strength values. Being well known that in-place concrete strength is lower than the strength of the corresponding cube specimen, it can be said that the more realistic predictions are those based on test results obtained from cores with 104 mm diameter. This is also in agreement with guidelines provided in *ACI* Code and British Standard, i.e. that it is preferable to obtain core samples with diameters of 100 mm to 150 mm. For core samples with 104 mm diameter, the average and the coefficient of variation of the ratios reported in Fig. 13 are: AVG = 0.912 and COV = 0.028 for Italian Guidelines, AVG = 0.947 and COV = 0.023 for *ACI* Code, and AVG = 0.895 and COV = 0.023 for British Standard. Based on the AVG values it can be said that Italian Guidelines and British Standard are more conservative than *ACI* Code. Also Khoury et al. in [12] observed that *ACI* Code seems to provide the closest results to experimental data. Since the COV values are very low, it can be said that all the codes provide uniform predictions.

Also for core specimens with 94 mm diameter the AVG and COV of the ratios reported in Fig. 13 are calculated, resulting: AVG = 1.054 and COV = 0.031 for Italian Guidelines, AVG = 1.122 and COV = 0.027 for *ACI* Code, and AVG = 1.052 and COV = 0.027 for British Standard. Comparing the COV of cores with 104 mm and 94 mm diameter obtained from each of the considered building codes, it can be observed that cores with 104 mm diameter provide more uniform predictions than 94 mm diameter cores. From this result it can be drawn that cores with 104 mm diameter lead to in-place concrete strength predictions characterized by a smaller scatter than that of predictions obtained by 94 mm diameter cores. This result is in accordance with those present in the literature, as it can be seen in [12], [15] and [35].

Values given by *ACI* Code will be used in the following as reference to calibrate the non-destructive tests prediction formulae for using non-destructive testing to predict concrete compressive strength.

In Tables from D.1 to D.3, all the measured core compressive strength test results are reported, and the last number of the specimen label represents the level of the column height (Fig. 4) at which the core was extracted, from 1 at the column top to 4 at the bottom. Hence it is possible to evaluate the variation of concrete strength along the heights of the columns. By plotting column level versus the core strength in Fig. 14, it is observed that the variation along the height is quite random. Thus, a weaker zone in a fixed position along the columns height cannot be identified. This is due to the fact that, to avoid the formation of weaker zones due to concrete segregation, the column specimens were casted in two layers and each layer was accurately vibrated.

Moreover, from Fig. 14 it can be seen that 5 out of 6 curves show that, at the highest level, the cores strengths are lower than those of cores extracted at the level just below (level 2). This strength reduction is probably due to bleeding. The same phenomenon was observed in studies reported in [12] and [16].

The maximum percentage difference between the strengths at two heights within the same column, 25 cm apart from each other, is 30%, surveyed for specimen C28/35. The average on the 3 columns of the percentage differences between the lowest and highest strength along a column is about 13%. These observations put in evidence the effect of core position on in-place concrete strength in relation with the position where the coring is performed. In fact, even for concrete elements whose casting and vibration are performed in a laboratory setting, as done for specimens tested in this research, the percentage difference in concrete strength can be as much as 30% also for a just a short distance between the extraction points.

From Fig. 14 it can be seen that cores with 94 mm diameter are more sensitive to spatial variability of in-place concrete strength than cores with 104 mm diameter, in accordance with [15].

5.2. *Surface hardness test versus compression test*

5.2.1. *Comparison between column and cube specimen results*

In order to make a side-by-side comparison of the SH test results obtained from all column faces and from cube specimens, a linear equation was developed to describe the relationship between average RI of columns and the strength values obtained from compression tests on cube specimens taken from the same concrete mix. An analogous equation was developed for the relationship between the values of average RI obtained from the cube specimens and their compressive strength values (Fig. 15). From Fig. 15 it can be observed that the correlation relationship relating to the cube specimens is shifted to the left with respect to that relating to the columns. This indicates that RI values are greater for the columns, meaning that their surface strength is greater. The average percentage difference is equal to about 5 %. This result is in contrast with what is generally expected; i.e. the strength of cube specimens is generally greater than that of in-place concrete. A possible explanation is that the cube specimens retained a certain degree of moisture despite having been left to rest for about six hours after being taken from the curing tank. This residual moisture can have led to a reduction in RI, which is a measure of surface hardness, as the hardness is notoriously lower in the presence of moisture [36]. The columns, on the other hand, had been left in the air from the moment of the formwork removal, and the first SH test was performed at least 7 days after casting. By that time the columns had presumably lost their surface moisture.

The lower surface hardness of cube specimens, with respect to

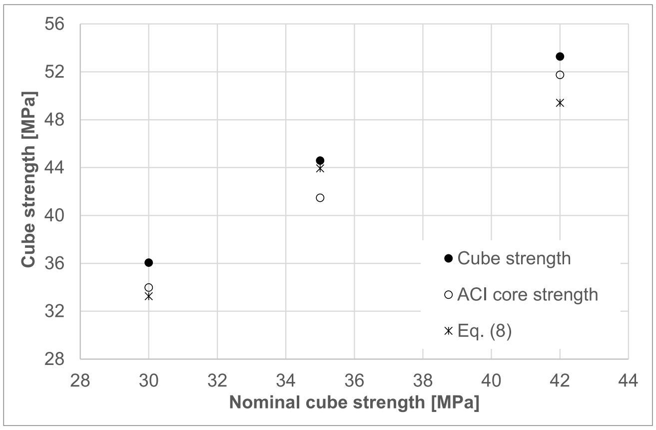

Fig. 16. Comparison between concrete strength values obtained from compression tests on cube specimens, and predictions obtained from Eq. (8), based on average RI.

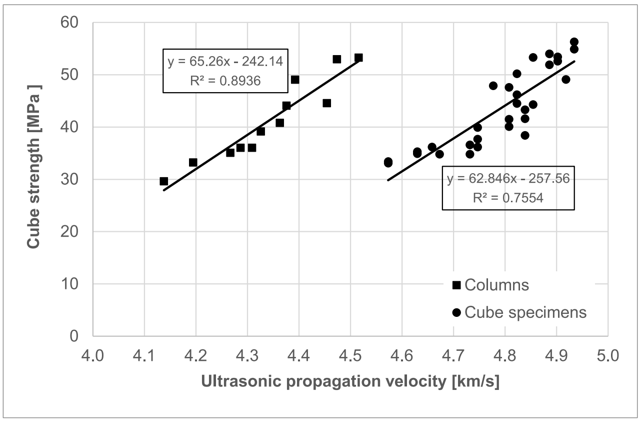

Fig. 17. Empirically based linear relationships between the compressive strength of cube specimens and the ultrasonic pulse velocity obtained from tests on the columns and on the cube specimens.

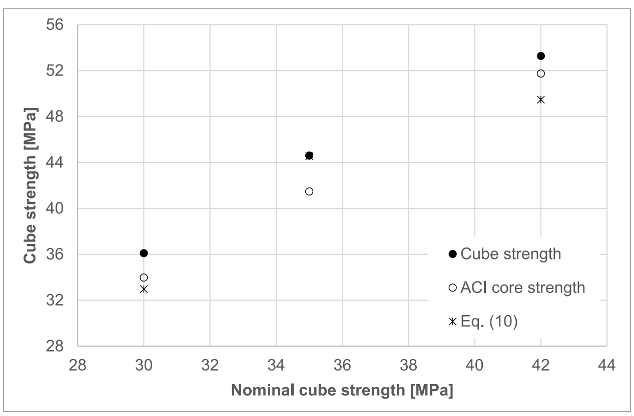

Fig. 18. Comparison between concrete strength values obtained by compression tests on cube specimens, and results obtained from Eq. (10), based on UPV.

column specimens, was also evidenced by more pitting and deeper imprints that were visible in these specimens after the SH test.

As noted in Fig. 14, when all the RI values are put in correlation with the corresponding cube strengths for all three mixes, the coefficient of determination for the relationship between column test results and cube strength is $R^2 = 0.896$, and the relationship between overall cube

Table 3
Formulations used for the SonReb method.

Authors	Year	Formulation	Units
Gasparik [40]	1992	$R_C = 0.0286 \bullet RI^{1.246} \bullet v^{1.85}$	MPa, km/s
Di Leo and Pascale [41]	1994	$R_C = 1.2 \bullet 10^{-9} \bullet RI^{1.058} \bullet v^{2.446}$	MPa, m/s
Arioglu and Koyluoglu [42]	1996	$R_C = 0.00153 \bullet (RI^3 \bullet v^4)^{0.611}$	MPa, km/s
Caiaro et al. [43]	2003	$R_C = 1.74 \bullet 10^{-7} \bullet RI^{-0.0674} \bullet v^{2.36}$	MPa, m/s
Del Monte et al. [44]	2004	$R_C = 4.4 \bullet 10^{-7} \bullet (RI^2 \bullet v^3)^{0.5634}$	MPa, m/s
Cristofaro et al. [27]	2020	$R_C = 41.59 - 0.02181 \bullet v - 0.859 \bullet RI + 5.808 \bullet 10^{-6} \bullet v^2 + 0.01539 \bullet RI^2$	MPa, m/s

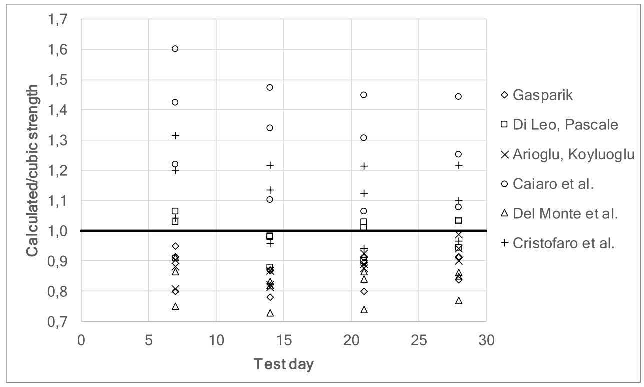

Fig. 19. Ratios between the concrete strength predicted by SonReb formulations (Table 3) and the corresponding cube strength obtained on each test day.

Table 4
Formulations used for the SonReb method.

Authors	% Difference between calculated and cube strengths			AVG and COV of calculated/cube strengths	
	C25/30	C28/35	C32/40	AVG	COV
Gasparik	−9.13	−10.41	−19.86	0.87	**0.06**
Di Leo and Pascale	**2.43**	**1.15**	−9.46	**0.98**	**0.06**
Arioglu and Koyluoglu	−13.18	−8.14	−14.06	0.88	**0.06**
Caiaro et al.	49.00	32.89	11.47	1.31	0.13
Del Monte et al.	−13.43	−15.96	−25.55	0.82	0.07
Cristofaro et al.	23.91	13.79	**−2.50**	1.12	0.11

$RI_{average}$ and cube strength results in $R^2 = 0.87$. Last result is very similar to that obtained in the study of Qasrawi [39], where a huge number of cube specimens made with various concrete mixes and also cubes of unknown history made under site conditions were subjected to SH and compression tests. As in the present case for the linear relationship used to correlate RI values and cube strengths, a R^2 value of 0.88 was observed in [39]. Hence, on the basis of this comparison, it can be said that the results presented herein are representative of real conditions.

To estimate the concrete strength from the measurements of RI, it is essential to calibrate the correlation curve between RI and concrete strength through the use of destructive tests results. The most commonly used correlation formula has usually the following form [21].

$$R_c = a \bullet RI^b \tag{7}$$

where R_c is the cube strength, RI is the rebound index and a and b are two coefficients, which are obtained by the least squares method.

By considering concrete strength predictions of the extracted cores with diameter 104 mm given by *ACI* Code formulation (Eq. (3)), the following calibrated formula is derived.

$$R_c = 0.0122 \bullet RI^{2.2693} \quad (8)$$

Fig. 16 shows cores strength predictions provided by Eq. (8) together with the concrete cube strengths measured at 28 days and the core compressive strengths, for each concrete mix. As it can be seen from this figure, the predictions given by Eq. (8) are all on the safe side with respect to the cube strengths. These strengths are expected to be effectively greater than the in-place strength, and the average percentage difference between them is equal to 5.5%. The coefficient of determination R^2 relative to Eq. (8) is equal to 0.93, indicating that Eq. (8) is a good fit for the observed data. The AVG and COV of the ratios between the strength predictions obtained from Eq. (8) and the cube specimens strength are equal to 0.945 and 0.037, respectively. It can be concluded that the correlation formula is able to reliably predict the strength of concretes of different grades based on RI values. The number of cores employed to obtain each of the three concrete strengths used to calibrate the formula is four. When conducting surveys on existing buildings, it is important to have such formulae to evaluate the concrete strength variation from one point in a building to another.

More conservative predictions can be obtained from the correlation relationships based on Italian Guidelines [9] or British Standard [11] formulations.

5.3. *Ultrasonic test versus compression test*

Taking into consideration the values of UPV obtained from the cube specimens and those obtained from the column specimens, it is possible to define two linear correlation relationships with the strength values obtained from the compression tests on the cube specimens (Fig. 17).

From Fig. 17 it can be observed that the representative points of the cube specimens are shifted to the right in the graph, with respect to the points relative to the columns. This means that the UPV of the former is greater than the UPV of the latter. The average percentage difference is equal to about 5%. This difference can be attributed to two reasons: the greater presence of voids inside the column specimens and the greater moisture content of cube specimens. The vibration of the columns turns out to be more complex than the vibration of the cube specimens, due to the greater dimensions and the presence of the steel reinforcement. Being easier to accomplish, vibration of cube specimens produces voids presumably smaller in number and size than those present in the columns. Larger voids or a greater number of them lead to a lower UPV, because ultrasounds, which do not propagate in a vacuum, must travel a greater length to circumvent the voids, therefore, the time they take to pass through the element is greater.

Moreover, it has already been demonstrated by other authors [37,38] that the UPV is higher in water-saturated concrete, as the cube specimens, than in dry concrete.

Finally, the coefficient of determination values relative to the linear interpolation relationships reported in Fig. 17, show that, in the case of the cube specimens ($R^2 = 0.75$), the linear law is not the best relationship between the UPV and cube strength. In the case of the column specimens, instead, the linear relationship gives a good correlation between the two parameters ($R^2 = 0.89$).

To estimate the concrete strength from the measurements of UPV, different correlation formulae are proposed in the literature. A linear correlation law is used herein, as done in [39].

$$f_c = A \bullet v - B \quad (9)$$

where *v* is the UPV expressed in km/s and *A* and *B* are two numerical parameters, which are obtained by the least squares method.

By considering, as previously done for the predictions based on rebound index value, concrete strength predictions of the extracted cores with diameter 104 mm given by *ACI* Code formulation (Eq. (3)), the following calibrated formula is derived.

$$f_c = 79.716 \bullet v - 310.44 \quad (10)$$

The predictions of this formula are shown in Fig. 18 together with the concrete cube strengths measured on the 28th day and the in-place concrete strengths calculated using the *ACI* Code formulation on the basis of the core compressive strengths. From this figure it can be observed that the predictions given by Eq. (10) are all on the safe side with respect to cube strengths, with an average percentage difference equal to 5.3% and a correlation coefficient R^2 equal to 0.9 indicating that Eq. (10) is a good fit for the observed data. The AVG and COV of the ratios between the strength predictions obtained from Eq. (10) and the cube specimens strength are equal to 0.947 and 0.048, respectively.

It can be concluded that the correlation formula is able to reliably predict the strength of concretes of different grades based on UPV values. The number of cores employed to obtain each of the three concrete strengths used to calibrate the formula is four. Similar to the correlation formula relative to RI, the correlation formula based on UPV is useful for evaluating the concrete strength variation from one point in a building to another. Hence, after having carried out some tests on extracted cores, and having determined the correlation formulae, it is possible to perform only non-destructive tests on other concrete elements, and then use the results of non-destructive testing to evaluate the strength of the in-place concrete.

Also in this case, more conservative predictions can be obtained from the correlation relationships based on Italian Guidelines [9] or British Standard [11] formulations.

5.4. *SonReb method applied to columns versus compression test*

Having the results of both the SH tests and the UPV tests performed on the columns, it is possible to use the SonReb method to determine the columns' concrete strength.

Recently, Cristofaro et al. [27] performed a study to identify new predicting models to evaluate concrete compressive strength using the SonReb method. Besides the proposed formulations, these authors checked the effectiveness of other numerous existing formulations in predicting a large number of data obtained from destructive and non-destructive methods. Among all the formulations considered, those that were most reliable in their predictions are reported in Table 3.

These formulations are used herein to assess which of them provide, based on input from the non-destructive tests' results, predictions close to the cube strength values obtained herein.

The predictions obtained from these formulations for all the measurements made on the columns at different curing days, divided by the corresponding cube strengths, are reported in Fig. 19. From this figure, it can be observed that, in general, the best predictions are obtained for the strengths at 28 day of curing, for which the lowest scatter occurs. This is because the formulations were derived from tests carried out on already cured concrete.

In Table 4 the percentage difference between the values calculated by the different formulations and the corresponding cube strength for each concrete mix are reported. Moreover, the average (AVG) and coefficient of variation (COV) values of the ratio between calculated strength and cube strength for all the measurements shown in Fig. 19 are also reported. From this table, it can be observed that the formulation of Di Leo and Pascale [41] provides the most accurate predictions. This is evidenced by the lowest percentage differences, aside from the case of concrete C32/40, and the fact that the AVG value for this formulation is very close to 1. Moreover, this formulation is also consistent, because the COV value is very low. Other formulations, which provide good predictions, all on the safe side, are those of Gasparik [40] and Arioglu and Koyluoglu [42].

Considering existing buildings, instead of cube strengths only the concrete strengths obtained from the extracted cores are available.

Using the in-place concrete strength values obtained from the *ACI* Code formulation as term of comparison with the values calculated by the formulations reported in Table 3, it results that the formulations of Gasparik [40], Di Leo and Pascale [41] and Arioglu and Koyluoglu [42] still provide the best predictions. In particular, the average percentage differences between the in-place strength and strength calculated with these three formulations are -6,44%, 5,74% and -0,74%. It can be concluded that using the concrete strength predicted by *ACI* Code, on the basis of the extracted cores strength, allows one to choose adequately reliable SonReb formulations based on rebound index and ultrasound pulse velocity.

In the analysis reported in [27] it is shown that the most accurate formulations are those of Caiaro and Cristofaro et al. In the present study it is obtained that the best predictions are given by the formula of Di Leo and Pascale. In light of this result, it is suggested to consider more than one predictive formula in order to choose the one providing the most accurate and uniform predictions in the considered case.

The AVG and COV of the ratios between the strength predictions obtained by the formula of Di Leo and Pascale and cube specimens strength are equal to 1.000 and 0.052, respectively. Comparing these values with those of the same parameters obtained for SH and UPV non-destructive tests, which are AVG = 1.059 and COV = 0.036 for SH tests and AVG = 1.058 and COV = 0.047 for UPV tests, it can be said that, in this case, the calibration of correlation curves leads to more uniform predictions than those provided by Sonreb method using formulations available in the literature.

6. Conclusions

The article addresses the problem of assessing concrete strength in existing RC buildings by means of destructive coring method and non-destructive surface hardness, ultrasonic pulse velocity and SonReb methods. Experimental tests on three column specimens made with three concrete mix typologies and corresponding cube specimens are carried out.

On the bases of the obtained experimental results, the following conclusions can be drawn:

1) As regards surface hardness test, for column specimens, the average rebound index values increase almost linearly with the increase of concrete age, while for cube specimens this trend is not so evident. Hence, in cases where it is useful/necessary to monitor the evolution over time of concrete curing, this can be done by monitoring the RI values over time. An approximately linear increase of these values indicates that curing proceeds correctly. Moreover, for both column specimens and cube specimens, the average rebound index values are greater for more resistant concrete.
2) As regards ultrasonic test, for both column specimens and cube specimens, the average propagation velocity increases almost linearly with the increase of the concrete age and is greater for more resistant concrete. Hence, in cases where it is useful/necessary to monitor the evolution over time of concrete curing, this can be done by monitoring UPV values over time. An approximately linear increase of these values indicates that curing proceeds correctly.
3) As regards compression test on extracted cores with diameter of 94 mm and 104 mm, greater strengths are obtained from the smaller sized specimens for all three concrete mixes used in this study. It is concluded that, if care is paid to the drilling operation, as done in this research, the results are affected more by the size effect than by the damage effect due to drilling.
4) Results of compression test on cores with 94 mm diameter are the closest to the corresponding cube compressive strengths, but are slightly greater, differently from what it is expected for in-place concrete. Cores with diameter 104 mm have, instead, strengths lower than the cube specimens' strengths, with a maximum percentage difference of 12%. Moreover, it is found that cores with 104 mm diameter provide more uniform predictions than 94 mm diameter cores. It is also observed that cores with 104 mm diameter are less sensitive to spatial variability of in-place concrete strength than 104 mm diameter cores. From the above, it is concluded that it is better to extract cores with diameters of 100 mm to 150 mm, according to the advices of the *ACI* Code and British Standard, because the results are on the safe side.
5) The location of the core influences prediction of in-place concrete strength. For concrete elements whose casting and vibration are performed in a laboratory setting, as done in this research, a weaker zone in a fixed position along the columns height is not identified. Only at the top of the column a slight strength reduction, probably due to bleeding, is observed. Moreover it is found that, also for accurately compacted columns the percentage difference in concrete strength can be as much as 30%, even with just a short distance between the extraction points.
6) By considering *ACI* Code, Italian Guidelines and British Standard for the prediction of in-place concrete strength on the basis of extracted core strengths, it is observed that, in comparison with corresponding cube specimens' strength, *ACI* Code gives the highest strength predictions, while British Standard the lowest ones. Italian Guidelines gives predictions, which are very slightly greater than British Standard ones. Overall, all the codes provide quite uniform predictions that range from values 11% lower to 15% higher than the measured cube compressive strength. In light of the above, it is suggested to adopt *ACI* Code formulation for core strength prediction, in order to reduce core strength underestimation.
7) Results of compression tests on four concrete cores for each concrete grade are sufficient to develop correlation formulae with the RI or the UPV, either of which are able to adequately predict strength of concretes of different grades. The predictions given by relationships built using the *ACI* Code formulation are all on the safe side with respect to the cube strengths, while even more conservative predictions can be obtained by using Italian Guidelines or British Standard formulations.
8) In the assessment of concrete strength in existing buildings, such correlation formulae allow for the evaluation of concrete strength variation from one point of the building to another by the exclusive use of non-destructive methods (surface hardness and ultrasonic velocity), which are easier to perform and cheaper than the core drilling destructive method.
9) Adequately reliable SonReb formulations can be identified by using the concrete strength predicted by the *ACI* Code formulation, based on the extracted cores strengths, as term of comparison. A subset of the most reliable existing formulations for the prediction of in-place concrete strength through the Sonreb method is selected. In this study it is observed that the best predictive formula is not the same giving the most accurate predictions in other works. In light of this result, it is suggested to consider more than one predictive formula in order to choose the one providing the most accurate and uniform predictions in the considered case.

CRediT authorship contribution statement

Giada Frappa: Formal analysis, Software, Validation, Writing – original draft, Writing – review & editing. **Massimiliano Miceli:** Data curation, Formal analysis, Investigation, Resources, Visualization. **Margherita Pauletta:** Conceptualization, Methodology, Supervision, Writing – review & editing.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Data availability

No data was used for the research described in the article.

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Appendix A. Results of SH test on the column specimens

Table A.1, Table A2, Table A.3

Table A1
Results of SH test on column made with C25/30.

Test day	7				14				21				28			
Position	**FA S4**	**FB S1**	**FC S2**	**FD S3**	**FA S1**	**FB S2**	**FC S3**	**FD S4**	**FA S2**	**FB S3**	**FC S4**	**FD S1**	**FA S3**	**FB S4**	**FC S1**	**FD S2**
Rebound Index (RI)	35.2	26.3	27.2	28.3	27.3	28.4	28.4	32.3	38.4	22.6	32.1	29.2	30.3	33.3	32.7	32.1
	30.3	27.4	26.5	27.3	30.4	30.6	29.4	28.1	34.3	30.4	34.1	29.2	30.4	36.3	25.1	31.1
	30.4	28.2	27.3	29.4	29.2	39.3	27.6	27.8	29.4	30.1	36.9	34.3	25.2	36.2	36	34.2
	36.3	29.2	29.2	33.4	31.2	28.2	30.2	31	35.4	30.3	33.8	31.3	34	28.2	30	36.1
	28.2	33.4	30.4	28.4	28.3	28.3	21.4	34.2	32.3	29.1	35.9	29.3	29.3	32.3	28	30.2
	30.2	32.2	27.3	27.4	28.3	32.4	30.3	32.2	29.4	32.2	33.9	32.3	37.4	34.2	32.9	32.2
	29.2	28.3	28.3	34.2	30.4	31.3	29.5	30.2	32.3	30.4	36	32.3	29.2	34.3	34	36.2
	32.6	29.4	29.3	32.3	32.2	32.2	29.5	32.4	29.3	28.3	40	32.2	21.3	35.4	32.4	37.1
	35.3	29.4	28.3	29.3	30.5	32.4	31.3	32.2	34.3	35.1	39.1	29.3	36.3	33.2	30.8	34.3
	31.2	29.9	30.4	29.2	32.1	30.5	29.3	35.3	37.5	29.3	36	33.4	35.2	36.2	30.9	31.2
	36.1	30.1	26.3	28.3	31.2	29.3	29.2	32.6	33.4	33.4	34.8	30.2	30.4	32.1	29.9	36.4
	34.3	29.2	29.4	32.4	30.3	29.3	33.2	36.9	33.5	31.5	31.8	29.3	28.7	34.3	30.8	32.3
	37.3	32.4	25.5	31.1	27.3	35.1	29.2	32.1	33.2	21.3	36.7	30.4		36.2	23.7	32.3
	36.2	34	23.3	33.1	30.1	28.2	31.8	32.3	29.4	21.4	34.8	30.2		38.2	30.8	34.2
	35	31.3	31.3	35.3	28.3	26.2	27.2	35.3	33.4	36.4		29.3		35.1	30.1	34.3
		22.3	34.2	33.2	32.3	29.3	29.5	39.3	30.3	29.5		33.5		36.2	30.3	34.3
		32.2	28.4	23.2	28	30.2	32		32.4	32.3		30.4		36.2	30.1	
		33.5	26.3	32.2		33.2	29.9		33.5	31.3		30.4		40.1		
				30.2		32.1	31.3		34.3	29.3						
				30.2						34.1						
										31.1						
										29.2						
RI_{median}	34.3	29.7	28.3	30.2	30.2	30.5	29.5	32.3	33.4	30.4	35.4	30.4	30.4	35.3	30.8	34.2
$RI_{average}$	30.6				30.6				32.4				32.7			

Table A2
Results of SH test on column made with C28/35.

Test day	7				14				21				28			
Position	**FA S4**	**FB S1**	**FC S2**	**FD S3**	**FA S1**	**FB S2**	**FC S3**	**FD S4**	**FA S2**	**FB S3**	**FC S4**	**FD S1**	**FA S3**	**FB S4**	**FC S1**	**FD S2**
Rebound Index (RI)	35.2	40.3	32.1	32.3	29.1	33.2	33.1	33.4	34.2	33.5	36.3	34.2	36.3	38.9	35.1	35.1
	31.2	30.1	31.1	33.4	33.9	36	35.2	33.2	32.1	34.4	34.3	30.2	34.4	35.8	34.9	35.1
	32.3	34.1	28.2	25.3	33.2	33.1	36.3	40.4	33.3	34.1	38.1	37.1	35.1	45.9	38.8	34.1
	32.2	29.1	33.2	32.4	34.9	35	32	36.1	33.3	33.4	35.1	33.2	35	43.8	38.9	33.5
	32.2	33.2	30.3	32.4	30.9	32.1	29	32.1	35.1	33	35.7	36.3	41.9	39.9	33.9	45.2
	43.6	33.1	23.2	32.3	27.4	34	34	33.1	35.3	34	36.1	34.2	40.2	43.2	38.9	31.8
	34.1	32.1	33	27.5	32.8	33.1	32.8	33.2	33.3	36.3	36.1	34.2	39.1	37.7	36.9	39.1
	33.1	29.2	31.1	30.2	31.9	38.1	33.1	36.2	33.1	32.3	28.1	31.2	35.1	39.9	36.9	36.1
	35.1	33.3	38	33.5	31.1	32.1	33.9	34.1	32.2	31.3	35	34.1	36	40.3	38.8	36.1
	31.4	31.3	34.1	29.3	31.9	31	33.1	33.1	35.1	37.2	33.1	35	37.1	41.9		35.1
	34	33.2	36.4	32.4	33	35	35.1	34.1	36.2	34.3	36	32.2	44.9			33.2
	35.1	36.2	33.1	32.4	33.2	29	33.1	36.1	40.4	37.1	40.3	36	35			35
	32.8	26.1	32.3		24.1	35.1	36.1	35	36.1	34.1	40.8	37.2	31			31.2
	32.1	33.2	36.1		33.1	33.1	42.1	32.1	32.1	34.2	34	36.2	45.2			28.1
	35.1	32.2	33.1		31.9	32.1	34.1	37.2	36.2	34.4	38	36.1	34.1			40.3
	39.2	33.3	32.3		30.9	33	33.9	33.2	36.4	38.2	42.8	26.2	33.9			
		31.3	36.1		34	32.1	31.6	33.3	34.2	37.2	32.9		33.1			
		34.1	28.1		36.1	32	35.1		35.2	36.1	35.9		43			
			33.1		34	32.1	32.1		36.4	40.9						
			32.1		35.2	35.1	36		35.4	34.2						
					31.9	36.1	34			36.2						
						32										
						34										
RI_{median}	33.6	33.2	32.7	32.4	32.8	33.1	33.9	33.4	35.1	34.3	36.0	34.2	35.6	40.1	36.9	35.1
$RI_{average}$	32.9				33.3				34.9				36.9			

Table A3
Results of SH test on column made with C32/40.

Test day	7				14				21				28			
Position	**FA S4**	**FB S1**	**FC S2**	**FD S3**	**FA S1**	**FB S2**	**FC S3**	**FD S4**	**FA S2**	**FB S3**	**FC S4**	**FD S1**	**FA S3**	**FB S4**	**FC S1**	**FD S2**
Rebound Index (RI)	33.1	32.9	32.3	33.4	37	33.1	40.3	32.4	37.2	35.2	39.2	37.2	41.5	37.1	39.4	35.1
	30.4	35	37.3	32.1	36.2	37.1	34.2	39.3	39.1	47.1	43.3	34.3	37.1	41.2	35.2	40.2
	32.1	33.9	44.3	31.3	37.2	32.9	39.2	34	43.1	35.1	39.2	40.1	37.2	51.1	37.1	38.1
	34.2	29.2	34.3	33.3	45.1	32.9	43.2	33.3	42.1	36	40.4	38.3	39.1	41	40.2	35.3
	29.9	35	36.5	33.2	38.1	34.9	45.1	44.4	37	34	39.3	38.2	40.1	42.2	31.2	36.1
	33.8	35.1	38.1	35.3	43.1	35.3	40.1	34.3	39.2	35.1	36.1	28.3	38.2	40.4	41.1	38.4
	31.2	35.1	44.2	32.3	35	35	42	36.4	33.2	45.1	35.3	35	39.1	40.1	35.3	40.1
	36.2	36.3	33.3	33.1	33.3	32.8	34.4	32.6	36.1	36.3	32.3	43.2	35.3	41	36.1	38.1
	34.2	39.1	34.2	33.2	37.2	36.3	37.1	34.3	35.1	36.1	40.4	34.1	37.1	45.2	39.1	37.4
	32.9	30.9	35.1	35.2	34.3	33.2	35.1	36.1	37	45.9	36.2	37.3	34.2	43.6	39.3	43
	33.9	34.4	34.3	33.2	37	32.9	35.2	34.2	37	35.2	36.3	36.2	38	42	36.4	42.3
	32.1	34.1	39.1	34.2	33.1	34	40.2	33.2	37.1	44.1	34.3	33.2		38	41.4	41.2
	37.9	30.1	39.4	33.4	35	38	37.1	38.2	43.2	38.2	43			39	40.2	38.2
	31.1	27.2	33.3	32.2	38	40	36.2	42.1	33.1	42.1	37.2			40.2	37.4	34.1
	31.7	32.8	33.1	32.1	39	34.1	39.3	35.2		44.9	45.1			37.1	36.1	41.3
	33.1	33.2	32.4	32.1	34	29.2	35.5			40.3	42.4			40.1	41.4	40.4
	26.1	32.1	35.3	33.4	31.9	36.1				40	41.2			40.2	41.2	39.2
	30.1	33.1	38.3	33.1	35					37.1	39.2			40	37.3	40.2
	36.9	29.1	34.1	33.3						43.2	37.3				40.3	44.1
	36.3		42.2	33						38.1					37.2	36
	33.8		35.2	32.5											38.1	
	32.9		42.3	37.1											42.3	
	33.1			31.1											38.4	
RI_{median}	33.1	33.2	35.3	33.2	36.6	34.1	38.2	34.3	37.0	38.2	39.2	36.7	38.0	40.3	38.4	38.8
$RI_{average}$	33.7				35.8								38.9			
									37.8							

Appendix B. Results of UPV test on the column specimens

Table B.1, Table B2, Table B.3

Table B1
Results of UPV test on column made with C25/30.

Test day	7				14				21				28			
Position	**FA S4**	**FB S1**	**FC S2**	**FD S3**	**FA S1**	**FB S2**	**FC S3**	**FD S4**	**FA S2**	**FB S3**	**FC S4**	**FD S1**	**FA S3**	**FB S4**	**FC S1**	**FD S2**
L [mm]	400	397	400	397	400	397	400	397	400	397	400	397	400	397	400	397
T [μs]	94.4	99.6	96.5	94.9	96.4	96.2	93.7	93.8	93	92.6	92.1	96	91.9	90.9	93.7	93.5
v [km/s]	4.24	3.99	4.15	4.18	4.15	4.13	4.27	4.23	4.30	4.29	4.34	4.14	4.35	4.37	4.27	4.25
$V_{average}$ [km/s]	4.138				4.194				4.267				4.309			

Table B2
Results of UPV test on column made with C28/35.

Test day	7				14				21				28			
Position	**FA S4**	**FB S1**	**FC S2**	**FD S3**	**FA S1**	**FB S2**	**FC S3**	**FD S4**	**FA S2**	**FB S3**	**FC S4**	**FD S1**	**FA S3**	**FB S4**	**FC S1**	**FD S2**
L [mm]	398	401	398	401	400	397	400	397	400	397	400	397	400	397	400	397
T [μs]	92.5	94.4	92.8	93.1	91.6	93.2	91.5	92.2	91.5	90.7	90.7	92.5	89.5	89.5	88.7	90.2
v [km/s]	4.30	4.25	4.29	4.31	4.37	4.26	4.37	4.31	4.37	4.38	4.41	4.29	4.47	4.44	4.51	4.40
$v_{average}$ [km/s]	4.287				4.326				4.363				4.454			

Table B3
Results of UPV test on column made with C32/40.

Test day	7				14				21				28			
Position	**FA S4**	**FB S1**	**FC S2**	**FD S3**	**FA S1**	**FB S2**	**FC S3**	**FD S4**	**FA S2**	**FB S3**	**FC S4**	**FD S1**	**FA S3**	**FB S4**	**FC S1**	**FD S2**
L [mm]	398	399	398	399	398	399	398	399	398	399	398	399	398	399	398	399
T [μs]	90.6	91.4	91.2	91.1	90.8	91.1	90.4	90.6	89	89.3	88.7	89.3	87.7	87.9	88.2	89.2
v [km/s]	4.39	4.37	4.36	4.38	4.38	4.38	4.40	4.40	4.47	4.47	4.49	4.47	4.54	4.54	4.51	4.47
$v_{average}$ [km/s]	4.376				4.392				4.474				4.516			

Appendix C. Results of compression test on cube specimens

Table C.1, Table C2, Table C.3

Table C1
Results of compression test on cube specimens made with C25/30.

Specimen label	Test day	Failure load [kN]	Strength [MPa]	Average strength [MPa]
1.7	3	509.6	22.6	22.35
1.8	3	501.5	22.1	
1.9	7	637.5	28.1	29.65
1.10	7	698.4	31.2	
1.11	14	757.1	33.4	33.25
1.12	14	744.5	33.1	
1.13	21	793.6	35.3	35.1
1.14	21	786	34.9	
1.1	28	813.4	36.2	36.1
1.2	28	782.5	34.8	
1.3	28	854.1	37.7	
1.4	28	787.6	34.8	
1.5	28	822.8	36.6	
1.6	28	813.6	36.2	

Table C2
Results of compression test on cube specimens made with C28/35.

Specimen label	Test day	Failure load [kN]	Strength [MPa]	Average strength [MPa]
2.7	3	625.1	27.8	28.95
2.8	3	676.4	30.1	
2.9	7	798.2	35.5	36.05
2.10	7	827.9	36.6	
2.11	14	896.7	39.9	39.15
2.12	14	869.2	38.4	
2.13	21	908.4	40.1	40.8
2.14	21	933.6	41.5	
2.1	28	1004.3	44.3	44.6
2.2	28	1007.7	44.5	
2.3	28	1045.8	46.2	
2.4	28	942.1	41.6	
2.5	28	981.1	43.3	
2.6	28	1079.2	47.6	

Table C3
Results of compression test on cube specimens made with C32/40.

Specimen label	Test day	Failure load [kN]	Strength [MPa]	Average strength [MPa]
3.7	3	847.7	37.4	36.8
3.8	3	814.5	36.2	
3.9	7	1010.8	44.9	44.1
3.10	7	980.7	43.3	
3.11	14	1137.8	50.2	49.05
3.12	14	1091.4	47.9	
3.13	21	1174.7	51.9	52.95
3.14	21	1224.2	54	
3.1	28	1250.6	54.9	53.27
3.2	28	1198.5	52.6	
3.3	28	1209.3	53.4	
3.4	28	1208.3	53.3	
3.5	28	1284.3	56.3	
3.6	28	1113.0	49.1	

Appendix D. Results of compression test on cube specimens

Table D.1, Table D2, Table D.3

Table D1
Results of compression test on core specimens made with C25/30.

Specimen label	Diameter [mm]	Height [mm]	Area [mm^2]	Weight [kg]	Mass [kg/m^3]	Failure load [kN]	Strength [MPa]	Average strength [MPa]
R30-104–1	104	105	8495	2.216	2370	267.7	31.51	32.10
R30-104–2	104	106	8495	2.156	2390	299.4	35.24	
R30-104–3	104	106	8495	2.130	2370	271.9	32.01	
R30-104–4	104	103	8495	2.299	2390	251.6	29.62	
R30-94–1	94	93	6940	1.541	2390	234.5	33.79	38.98
R30-94–2	94	92	6940	1.525	2390	302.5	43.59	
R30-94–3	94	93	6940	1.535	2380	259.7	37.42	
R30-94–4	94	96	6940	1.591	2390	285.4	41.13	

Table D2
Results of compression test on core specimens made with C28/35.

Specimen label	Diameter [mm]	Height [mm]	Area [mm^2]	Weight [kg]	Mass [kg/m^3]	Failure load [kN]	Strength [MPa]	Average strength [MPa]
R35-104–1	104	104	8495	2.186	2400	347.9	40.95	39.19
R35-104–2	104	105	8495	2.198	2420	376.4	44.31	
R35-104–3	104	105	8495	2.178	2400	288.9	34.01	
R35-104–4	104	107	8495	2.278	2390	318.4	37.48	
R35-94–1	94	92	6940	1.515	2370	339.7	48.95	46.86
R35-94–2	94	96	6940	1.614	2400	346.6	49.94	
R35-94–3	94	94	6940	1.550	2380	310.2	44.70	
R35-94–4	94	95	6940	1.618	2380	304.4	43.86	

Table D3
Results of compression test on core specimens made with C32/40.

Specimen label	Diameter [mm]	Height [mm]	Area [mm^2]	Weight [kg]	Mass [kg/m^3]	Failure load [kN]	Strength [MPa]	Average strength [MPa]
R40-104–1	104	115	8495	2.358	2410	398.6	46.92	48.89
R40-104–2	104	111	8495	2.282	2420	424.8	50.01	
R40-104–3	104	107	8495	2.202	2420	410.3	48.30	
R40-104–4	104	103	8495	2.119	2420	427.4	50.31	
R40-94–1	94	93	6940	1.545	2390	412.8	59.48	54.54
R40-94–2	94	96	6940	1.597	2400	347.0	50.00	
R40-94–3	94	95	6940	1.584	2400	373.1	53.76	
R40-94–4	94	91	6940	1.519	2410	381.0	54.90	

References

[1] M. Miani, C. Di Marco, G. Frappa, M. Pauletta, Effects of dissipative systems on the seismic behavior of irregular buildings—two case studies, Build. 10(11)202 (2020) 1–27.
[2] R.M. Thejaswini, L. Govindaraju, V. Devaraj, Experimental and numerical studies on setback buildings considering the ssi effect under seismic response, Civil Engineering Journal 7 (3) (2021) 431–448.
[3] S. Cattaneo, G. Zorzato, A. Bonati, Assessing method of shear strength between old to new concrete interface under cycling loading, Constr. and Build. Mater. 309 (2021), 125160.
[4] Ministero delle Infrastrutture e dei Trasporti. DM 17 gennaio 2018. Aggiornamento delle "Norme tecniche per le costruzioni". Gazzetta Ufficiale, 20 February 2018. n. 42. (In Italian).
[5] En, Design of structures for earthquake resistance, Part 3: Assessment and retrofitting of buildings Comitè Europeen 1998–3, 2005. de Normalisation (CEN.
[6] Fema 356, Pre-standard and Commentary for the Seismic Rehabilitation of Buildings, Applied Technology Council (ATC) 2000 Washington DC.
[7] Asce, sei, 41/06, Seismic Rehabilitation of Existing Buildings 2007 Reston, VA.
[8] M. Pauletta, C. Di Marco, G. Frappa, M. Miani, G. Campione, G. Russo, Seismic behavior of exterior RC beam-column joints without code-specified ties in the joint core, Eng. Struct. 228 (2021), 111542.
[9] Consiglio Superiore dei Lavori Pubblici, Servizio Tecnico Centrale. Linee guida per la valutazione delle caratteristiche del calcestruzzo in opera, 2017. (In Italian).
[10] ACI PRC-214.4-21: Obtaining Cores and Interpreting Core Compressive Strength Results.
[11] Bs,, Part 120:1983, Method for determination of the compressive strength of concrete cores, London, 1881, p. 6.
[12] S. Khoury, A.-H. Aliabdo, A. Ghazy, Reliability of core test – Critical assessment and proposed new approach, Alexandria Eng. J. 53 (1) (2014) 169–184.
[13] M. Tuncan, O. Arioz, K. Ramyar, B. Karasu, Assessing concrete strength by means of small diameter cores, Constr. Build. Mater. 22 (5) (2008) 981–988.
[14] I.M. Nikbin, M. Eslami, D.S.M. Rezvani, An experimental comparative survey on the interpretation of concrete core strength results, Eur. J. Sci. Res. 37 (3) (2009) 445–456.
[15] Barlett FM, MacGregor JG, Effect of Core Diameter on Concrete Core Strengths, ACI Materials Journal V. 91, No. 5, Sept.-Oct. 1994, pp 460-470.
[16] H. Qasrawi, Effect of the position of core on the strength of concrete of columns in existing structures, Journal of Building Engineering 25 (2019), 100812.
[17] UNI EN 12504-2:2021. Testing concrete in structures - Part 2: Non-destructive testing - Determination of rebound number.
[18] UNI EN 12504-4:2021. Testing concrete in structures - Part 4: Determination of ultrasonic pulse velocity.
[19] RILEM NDT 4, Recommendation for in Situ Concrete Strength Determination by Combined Non-destructive Methods, Compendium of RILEM Technical Recommendations, E&FN Spon, U.K., London, 1993.
[20] E. Schmidt, Rebound hammer for concrete testing, Schweiz Bauztg 68 (28) (1950) 378–379 [in German].

[21] K. Szilágyi, A. Borosnyói, I. Zsigovics, Rebound surface hardness of concrete: Introduction of an empirical constitutive model, Constr. Build. Mater. 25 (5) (2011) 2480–2487, https://doi.org/10.1016/j.conbuildmat.2010.11.070.
[22] G. Frappa, M. Pauletta, C. Di Marco, G. Russo, Experimental tests for the assessment of residual strength of r.c. structures after fire – Case study, Eng. Struct. 252 (2022), 113681.
[23] M. Jedidi Evaluation of the Quality of Concrete Structures by the Rebound Hammer Method Cur Trends Civil & Struct Eng 5 5 2020 CTCSE.MS.ID.000621 10.33552/CTCSE.2020.05.000621.
[24] A.V. Saetta, R.V. Vitaliani, Experimental investigation and numerical modeling of carbonation process in reinforced concrete structures: Part II, Practical applications, Cement and Concrete Research 35 (5) (2005) 958–967.
[25] G. Campione, F. Cannella, L. Cavaleri, Shear and flexural strength prediction of corroded R.C. beams, Constr. Build. Mater. 49 (2017) 395–405.
[26] H. Al-Mattarneh, M. Dahim, Comparison of nondestructive testing method for strength prediction of asphalt concrete material, Civil Engineering Journal 7 (1) (2021) 165–178.
[27] M.T. Cristofaro, S. Viti, M. Tanganelli, New predictive models to evaluate concrete compressive strength using the SonReb method, J. Build. Eng. 27 (2020), 100962.
[28] D. Breysse, J.P. Balayssac, Non-Destructive In Situ Strength Assessment of Concrete - Practical Application of the RILEM TC 249-ISC Recommendations, Springer, RILEM State-of-the-Art Reports, 2021.
[29] S.J.A. Al-Hasan, R. Balamuralikrishnan, M. Altarawneh, Eco-friendly asphalt approach for the development of sustainable roads, J. Human, Earth, and Future 1 (3) (2020) 97–111.
[30] UNI EN 12390-1:2021. Testing hardened concrete - Part 1: Shape, dimensions and other requirements for specimens and moulds.
[31] UNI EN 12390-2:2019. Testing hardened concrete - Part 2: Making and curing specimens for strength tests.
[32] EN 1992-1-1:1992. Design of concrete structures - Part 1-1: General rules and rules for buildings.
[33] G. Somma, M. Vit, G. Frappa, M. Pauletta, I. Pitacco, G. Russo, A new cracking model for concrete ties reinforced with bars having different diameters and bond laws, Eng. Struct. 235 (2021), 112026.
[34] J. Haavisto, A. Husso, A. Laaksonen, Compressive strength of core specimens drilled from concrete test cylinders, Structural Concrete. 22 (Suppl. 1) (2021) E683–E695.
[35] Carroll AC., Grubbs AR., Schindler AK., Barnes RW., Effect of core geometry and size on concrete compressive strength, Research Report No. 1 for ALDOT Project 930-828, Highway Research Center, Auburn, Alabama, July 2016.
[36] D'Amato, M., Laterza, M., Gigliotti, R., Indagini distruttive e non distruttive su edifici in ca esistenti secondo l'OPCM 3274/3431: evidenze sperimentali e significatività dei risultati. 12th Italian Conference on Earthquake Engineering, 2007, DOI: 10.13140/2.1.1524.4489 (in Italian).
[37] Lencis, U., Udris, A., Korjakins, A., Moisture Effect on the Ultrasonic Pulse Velocity in Concrete Cured under Normal Conditions and at Elevated Temperature, Constr. Sci. 10.2478/cons-2013-0011:71-78.
[38] H. Güneyli, S. Karahan, A. Güneyli, N. Yapıcı, Water content and temperature effect on ultrasonic pulse velocity of concrete, Russ. J. Nondestruct. Test. 53 (2) (2017) 159–166.
[39] H.Y. Qasrawi, Concrete strength by combined nondestructive methods simply and reliably predicted, Cement Concrete Res. 30 (2000) 739–746.
[40] J. Gasparik, Prove non distruttive nell'edilizia, Quaderno didattico AIPn.D, Brescia, 1992 in Italian.
[41] Di Leo, A, Pascale, G. Prove non distruttive sulle costruzioni in cemento armato, Convegno sistema qualità e prove non distruttive per l'affidabilità e la sicurezza delle strutture civili, 1994. Bologna SAIE (in Italian).
[42] E. Arioglu, O. Koyluoglu, Discussion of prediction of concrete strength by destructive and non-destructive methods by Ramyar and Kol, Cem. Concr. World 3 (1996) 33–34, in Turkish.
[43] R. Caiaro S. De Paola G. Porco Indagini non distruttive per il controllo dei calcestruzzi di media ed alta resistenza, Atti del 10° Congresso Nazionale dell'AIPnD 2003 Ravenna 360 371 in Italian.
[44] E. Del Monte, G. Lavacchini, A. Vignoli, Modelli previsionali della resistenza a compressione del calcestruzzo in opera, Ing. Sismica-Ital. 3 (2004) 30–40, in Italian.