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IN SITU TESTS FOR SEISMIC ASSESSMENT OF R.C. STRUCTURES

S. Biondi[1], E. Candigliota[2]

[1] *Associate Professor, Department of Design, Rehabilitation and Control of Architectural Structures, Pricos, University "G. D'Annunzio", Chieti-Pescara, Italy*

[2] *Ph.D. in Structural Engineering, Department of Design, Rehabilitation and Control of Architectural Structures, Pricos, University "G. D'Annunzio", Chieti-Pescara, Italy*

Email: s.biondi@pricos.unich.it, ecandigliota@unich.it, http://www.pricos.it

ABSTRACT:

In Italy a great part of the territory wasn't classified as seismic up to 2003, so that lower intensity earthquakes caused important structural damages and loss of human life (as for Umbria-Marche Region Earthquake 1997 and Molise Earthquake 2002). Particularly relevant buildings (Monumental Buildings as San Francesco Basilica in Assisi in 1997, Public Buildings as San Giuliano Primary School in Molise in 2002) suffered an extreme earthquake fragility. For this reason new Codes were issued by National Government; in all these Codes seismic zoning was improved and existing buildings diagnosis is considered as a relevant topic. At same time a wide program of structural analysis and seismic assessment was financed by Local and National Governments.
On the basis of these new seismic Codes, in the Abruzzo Region, in Eastern Central part of Italy, four recently built sport domes (Chieti, Lanciano, Ortona, Vasto) were analyzed in order to evaluate their seismic behavior. All of these buildings have been designed without any seismic provision. A wide program of in situ and laboratory tests was carried out regarding RC elements of these buildings. In particular the actual concrete strength is detected by means of combined non-destructive methods (Rebound index, ultrasonic velocity). Some concrete cores were drilled out for destructive laboratory tests in order to determine both compressive characteristics (strength and elastic modulus) and to validate in-situ non destructive tests. Some literature proposals are discussed for test data evaluation. Proposals for structural engineer involved in such activities are carried out.

KEYWORDS: RC existing frames, in-situ non destructive tests, SonReb method, existing frames seismic assessment

1. INTRODUCTION

From a geotectonic point of view, Italy is clearly a seismic prone Nation and a lot of earthquakes have been suffered in many parts of the Country. Some of these earthquakes were very destructive (such as Messina Earthquake in 1908, Friuli Earthquake in 1976, Irpinia Earthquake in 1980). Naturally the greater part of earthquakes was low and medium intensity earthquakes; in spite of that situation, a great part of National territory wasn't classified as seismic (as Molise region involved in 2002 earthquake, Mola et al. 2003) and these lower intensity earthquakes caused unexpected failure of relevant buildings (as Umbria-Marche Region Earthquake, De Sortis et al. 2000).
After the Molise Region Earthquake, the seismic zoning was upgraded and all national territory has been classified as seismic with 4 seismic input levels (OPCM (2003)). This Code wasn't a full Performance Based Code but it provided that an extensive program of seismic analysis could be carried in order to define a global strategy for seismic risk reduction. In particular strategic existing buildings, as schools, hospitals, fire-stations, had to be analyzed in order to obtain not only a Life Safety Performance Level but also an Operational or Immediate Occupancy Level. More recently another Code (D.M. 14.01.2008) confirmed these objectives and redefined seismic zoning introducing a grid of 10751 nodes of seismic spectral parameters for all national territory. In each node a different value for peak ground acceleration (a_g), local amplification factor (F_o) and control period (T_C, upper limit of the period of the constant spectral acceleration branch) is defined.
On the basis of these Codes four sport buildings are analyzed in order to evaluate their seismic behavior; this analysis was promoted by Chieti Provincial Government for 2007 Women European Basketball Championships.

The buildings are recently built with RC structure without seismic codes and preliminarily to a modeling activity (that was carried out by another Chieti-Pescara University research team), a wide program of in situ and laboratory tests was carried out regarding RC elements. This in-situ campaign consisted of determination of concrete carbonation degree, reinforcement disposal by means of cover meter instrumentation and concrete strength determination by combined non-destructive methods (rebound index and ultrasonic pulse velocity). Some cylindrical concrete specimens were drilled out in order to carrying out compressive laboratory tests for in situ data validation. Some results were discussed in Biondi et al. (2008). In the present paper the complete test program will be presented and some operative conclusions will be pointed out, useful for similar structures.

2. SEISMIC ZONING AND LOCALIZATION OF TEST SITES

In Figure 1 Italian seismic zoning evolution is pointed out, the 1984 Code map was in force at the time of Molise Earthquake because the 1998 National Commission Proposal wasn't issued as Technical Code. According to this 1984 zoning all four sites were seismically unclassified (grey color in map).In 2007 seismic zone in force was that of OPCM 3274 (2003): Chieti was 2nd category seismic zone, Lanciano, Ortona and Vasto were 3rd category seismic zones.
In Table 1 seismic parameters for different seismic categories according to OPCM 3274 (2003) are shown for C ground type. Today the latter Code is in force (D.M. 14.01.2008), in Table 1 spectral parameters of this Code are summarized considering two returns periods that quit represents respectively Damage Limit State ($T_R = 140$ years) and Ultimate Limit State ($T_R = 2475$ years) for class III buildings. The OPCM 3274 (2003) considers an importance factor $\gamma_I = 1.20$ for this class of buildings: so seismic parameters of two Codes are quite similar.

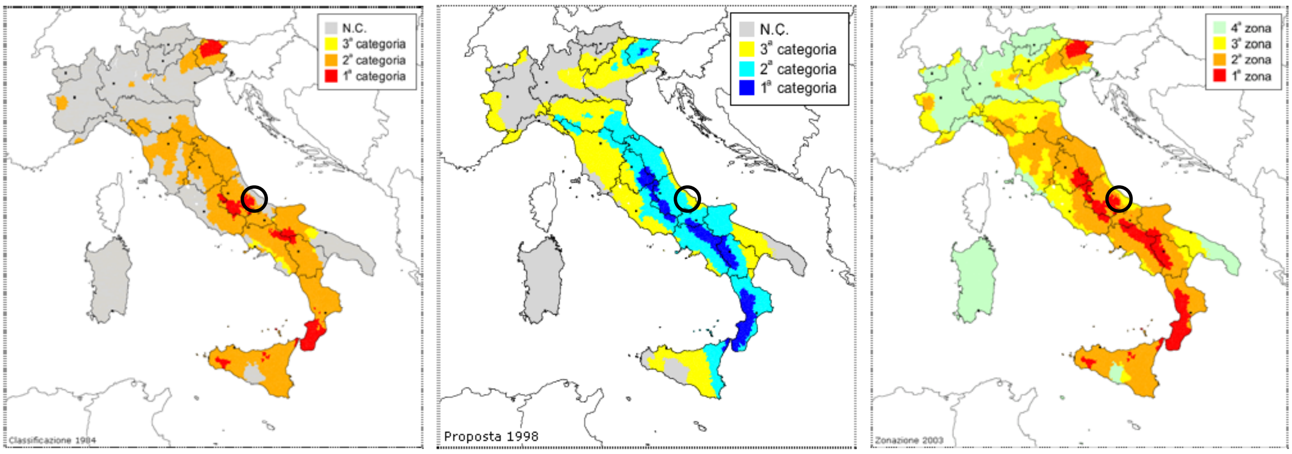

Figure 1. Italian seismic zoning (Chieti Province highlighted) after 1984 Code (left), 1998 National Commission Proposal (center), OPCM 3274 (2003) (right)

In Figure 2 photos of the Sport domes of this research are shown (Calzona (1982)); every structure is cast in situ reinforced concrete structure. Structural arrangement is generally characterized by a central hall with lateral (cast in sity or partially precast) tiers of seats; under these tiers generally are located dressing rooms, lavatories, technical rooms or, as in Lanciano dome, little boxing hall. As shown in Figure 2 RC frames are partially infilled and generally a soft storey portion is detected on the top of tiers; due to seismic fragility of this configuration, a great part of test in situ and core drilling was at this level.

Table 1. Seismic parameters for different seismic categories according to OPCM 3274 (2003)

City	2003 OPCM zoning				D.M. 14.01.2008 zoning					
					T_R = 140 years			T_R = 2475 years		
	Seismic Category	a_g/g [-]	S [-]	T_c [s]	a_g/g [-]	F_o [-]	T_c [s]	a_g/g [-]	F_o [-]	T_c [s]
Chieti	2	0.25	1.25	0.50	0.103	2.423	0.507	0.301	2.524	0.539
Lanciano	3	0.15	1.25	0.50	0.076	2.543	0.534	0.195	2.661	0.605
Ortona	3	0.15	1.25	0.50	0.068	2.618	0.535	0.160	2.764	0.641
Vasto	3	0.15	1.25	0.50	0.062	2.633	0.581	0.149	2.754	0.700

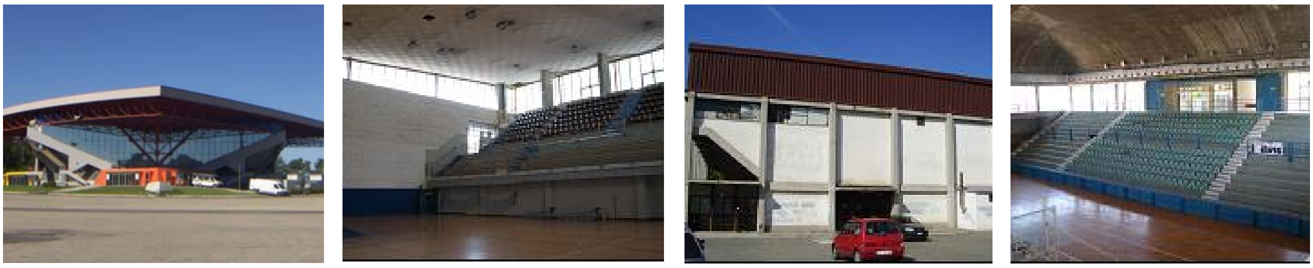

Figure 2. Sport domes involved in in-situ test campaign: Chieti, Lanciano, Ortona, Vasto

Due to direct contact of RC elements with atmosphere (often near the sea) and pollution, relevant carbonation effects on concrete were expected and controlled by means of carbonation degree measures.

3. NON DESTRUCTIVE METHODS FOR COMPRESSIVE STRENGTH EVALUATION

According to OPCM 3274 (2003) or to Eurocode 8 (CEN 2005), a knowledge level has to be preliminarily evaluated in order to define material properties for existing buildings and consequently to define a correct structural analysis and assessment design procedure.
This knowledge level considers three different evaluation steps: geometry, details and materials. Regarding materials evaluation OPCM 3274 (2003) stated that non-destructive test methods couldn't be used in place of destructive tests. This limit was removed by OPCM 3431 (2005), (Biondi (2008)). Basing on knowledge level definition goal some destructive tests have to be carried out: in the case of an RC structure both compressive tests on cores and tensile tests on rebars obtained by mean a concrete cover removal and a mechanical cut of longitudinal or transversal rebars. These procedures are too intrusive for RC structures, for this reason some local governments state different approaches: so Tuscany Region discourages the rebar extraction due to difficulty in a correct reconstruction (Ferrini Ed. 2004) while Basilicata Region, according to OPCM 3431, encourages to substitute destructive tests with non-destructive tests (Dolce et al. 2005) for a maximum of 50% of tests.
In the case this paper a similar procedure was adopted according with Public Owner Government: no tensile tests on existing rebars were carried out (considering the industrial origin of steel component) while an extensive series of concrete tests was defined. In particular 22 concrete cores were drilled out (some of them were subdivided in two parts for global 31 concrete cylindrical specimens), 68 rebound test and 78 ultrasonic test sites were defined (some of them were both direct and semi-direct or indirect measurements for a global number of 166 measurements). Before drilling out non destructive tests were carried out in the test site in the aim to correlate laboratory to in-situ results. As in practice compressive strength values is principal objective of this test series while a few tests for elastic modulus evaluation were carried out in laboratory.
In order to carry out each test phase, expert technical people was employed and standard provisions are taken into account: UNI EN 12504-1:2002 (core drilling and testing), UNI EN 12504-2:2001 (rebound number), UNI EN 12504-4:2005 (ultrasonic pulse velocity), UNI EN 13295:2005 (resistance to carbonation). Correlation formulas, basing on literature proposals or practice, will be discussed in the following paragraphs.

3.1. Rebound index

Four different Schmidt hammers were used in order to determine rebound index I; an horizontal position was assumed by the operator and equivalent cube compressive strength R_{cR} (in MPa) has be determined by using two different rebound index I correlation formulas:

$$R_{cR1} = \alpha I^2 + \beta I + \varphi \tag{3.1}$$

$$R_{cR2} = \left(9.167 \times 10^{-3}\right) I^{2.27} \tag{3.2}$$

Eqn. (3.1) parameters are defined on the basis of Schmidt hammer instructions while Eqn. (3.2) is a literature proposal; in Figure 3 the good approximation of this general correlation formula is pointed out with respect of different producer instructions in the range $30 \le I \le 60$. For this reason it is possible to note that the great difference between rebound index compressive strength and core compressive strength that will be pointed out in the next chapter, doesn't depend on correlation formulas but is deeply rooted in rebound index method itself.

	$R_{cR1} = \alpha I^2 + \beta I + \varphi$		
Instrument	α	β	φ
I	0.0430	-1.0465	10.126
II	0.0091	1.1028	-15.39
III	0.0158	0.5507	-6.846
IV	0.0190	0.5919	-11.32

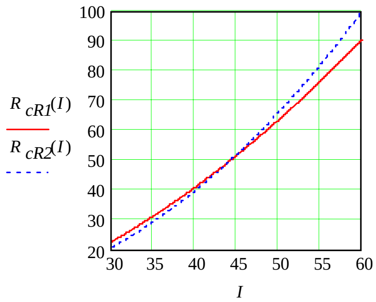

Figure 3. Comparison between average values of (3.1) [dot line] and (3.2) [solid line] equations for $30 \le I \le 60$

3.2. Ultrasonic velocity

Thus for Rebound index, correlations between the pulse velocity and strength of concrete are physically indirect and could to be established for the specific concrete mix (UNI EN 12504-4:2005). In case of existing buildings not only concrete mix is unknown but also reinforcement and random cracking distribution are unknown.

If an elastic isotropic medium is assumed, a compressive strength R_{cV1} (in MPa) can be determined by means of dynamic E_{cd}, static elastic modulus E_c, using ultrasonic pulse velocity V (m/s), Poisson ratio ν, concrete mass density γ (kg/m^3) as shown in Eqn. (3.3). In Eqn. (3.3) coefficients in functional relationship between dynamic and static elastic modulus are $\delta = 1.149 \times 10^{-6}$ and $\lambda = 1.5953$ for $V \ge 3600$ m/s. The compressive cubic strength, R_c, is obtained from cylindrical compressive strength, f_c, according to Code as $R_c = 0.83 f_c$:

$$E_{cd} = V^2 \gamma \frac{(1+\nu)(1-2\nu)}{1-\nu} = \frac{\gamma V^2}{\eta^2} \qquad E_c = 1000 f_{ck} = \delta V^{\lambda} E_{cd} \qquad R_{cV1} = \frac{f_{ck}}{0.83} = \frac{E_c}{830} \tag{3.3}$$

$$R_{cV2} = -5 + \frac{18000}{5000 - V} \qquad R_{cV3} = \left(9.90\eta^2 - 56\eta + 87.80\right) \times 10^{-7} \qquad R_{cV4} = 0.02073 e^{0.0016V} \tag{3.4}$$

In Eqn. (3.4) three other relationships are shown: the first, R_{cV2}, considers only the pulse velocity, the second, R_{cV3}, is an explicit relationship in term of elastic characteristics of medium and the third was proposed for low strength concrete. In Figure 4 a comparison for these four formulas is carried out in the range $2000 \le V \le 6000$ m/s. For R_{cV1} the conventional values $\gamma = 2350$ kg/m^3 and $\nu = 0.20$ are considered. It is possible to note that R_{cV2} shows an asymptotic value for $V = 5000$ m/s and is negative for higher pulse velocity; similar asymptotic behavior for R_{cV3}. As stated R_{cV4} is in the range of other relationships only for lower pulse velocity.

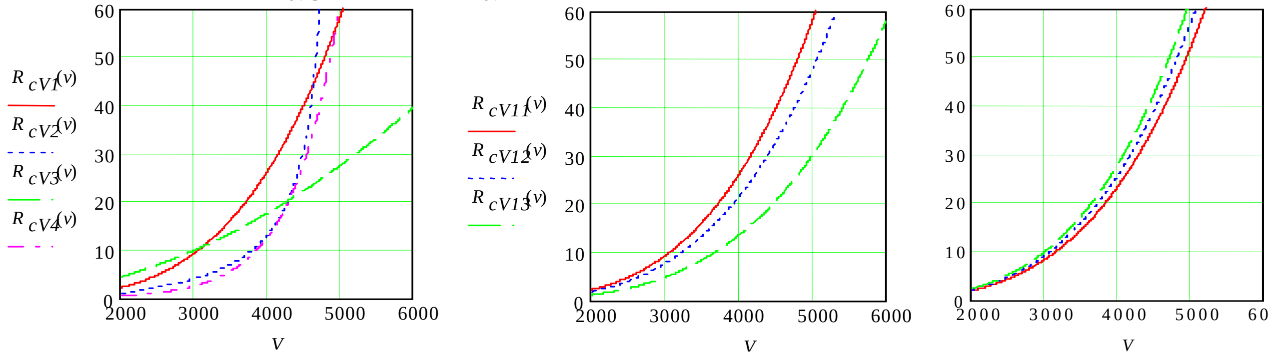

Figure 4. Comparison of different strength provisions (left). Variation of R_{cV1} value on Poisson's ratio (center, constant mass density $\gamma = 2350$ kg/m^3) and mass density (right, constant Poisson's ratio $\nu = 0.20$)

If Eqn. (3.3.) is assumed as unique stable relationships in concrete typical pulse velocity range, it could be interesting to evaluate the sensitivity of this relationship on Poisson's ratio and mass density. The first case is shown in Figure 4 (center) for $\nu = 0.20$ (solid line), $\nu = 0.30$ (dot line) and $\nu = 0.40$ (dashed line). The second one is shown in Figure 4 (right) for $\gamma = 2100$ kg/m^3 (solid line), $\gamma = 2300$ kg/m^3 (dot line) and $\gamma = 2500$ kg/m^3 (dashed line). It is possible to note that the first parameter is more relevant than second one.

Theoretically Eqn. (3.3)-(3.4) can be used for non direct tests: according to UNI EN 12504-4:2005 for semi-direct the path length is the distance measured from centre to centre of the transducer faces while for indirect transmission a series of measurements has to be made with the transducers at different distances. We will see that these provisions don't guarantee correct compressive strength evaluation.

3.3. SonReb method

The SonReb (Sonic & Rebound) method is a well known method that aims to increase the accuracy of single methods; two equations of similar mathematical structure are used, based on rebound index and pulse velocity,:

$$R_{cS1} = 7.695 \cdot 10^{-11} I^{1,40} V^{2,60} \tag{3.5}$$

$$R_{cS2} = 1.20 \cdot 10^{-9} I^{1.058} V^{2.460} \tag{3.6}$$

The first of these relationships is a Rilem Standard provision while the second is a literature proposal for low pulse velocity. In Figure 5 sensitivity of theses relationships on rebound index and pulse velocity are shown. The first case is shown in left side for different pulse velocity: $V = 3500$, $V = 4000$, $V = 4500$ m/s. The second one is shown in right side for $I = 35$, $I = 40$, $I = 45$. Eqn. (3.6) generally overestimates compressive strength in comparison with Eqn. (3.5). It is to note that both equations disregards theoretical dependence on Poisson's ratio and mass density of Eqn. (3.3).

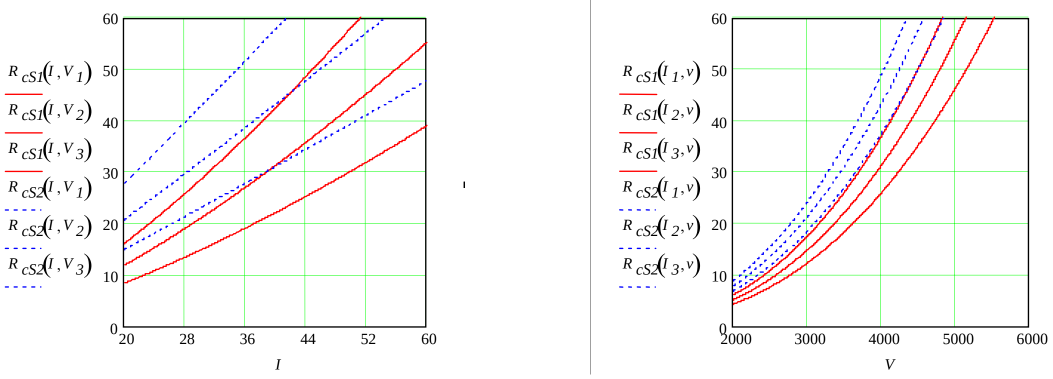

Figure 5. Comparison between R_{cS1} (solid line) and R_{cS2} (dot line) provisions for three different pulse velocity values (left) and three different rebound index (right)

4. TEST RESULTS ANALYSIS

A great number tests were carried out in situ (68 rebound and 78 ultrasonic tests, the latter for 166 different direct, semi-direct or indirect measures) and in laboratory (22 drilled cores were used to prepare 31 cylindrical concrete specimens. Each specimen was used in compressive and pulse velocity tests). In Figure 6 a preliminary evaluation of semi-direct and indirect pulse velocity measures is carried out.

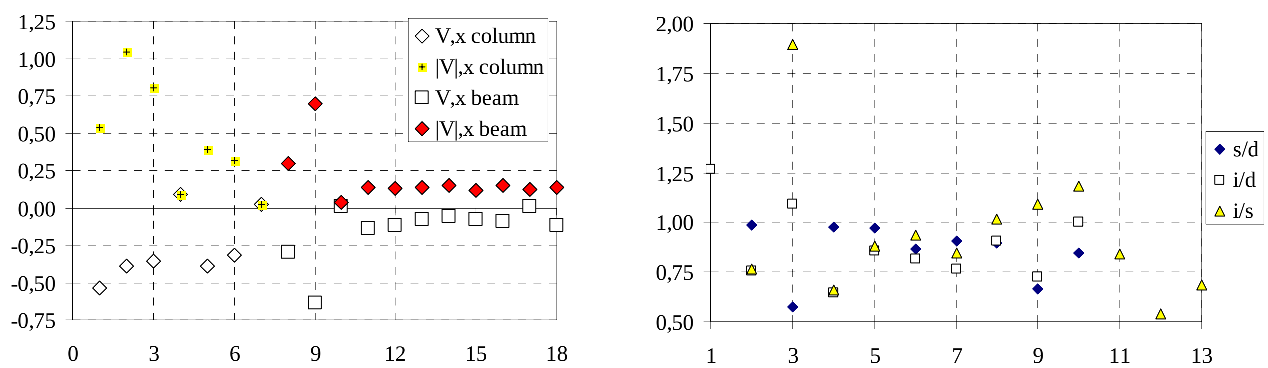

Figure 6. Pulse velocity slope in indirect tests for columns and beams (left), ratios between different pulse velocity procedure (*d* direct, *s* semi-direct, *i* indirect, right)

In the left side of Figure 6 average values of pulse velocity slope (adimensional variation for unity length) for 7 columns and 11 beams are shown both considering the sign, $\bar{V}_{,x}$, and in absolute value, $|\bar{V}_{,x}|$, where x_i and V_i are respectively position and pulse velocity at i^{th} point of measure of a series of n points of measure:

$$\bar{V}_{,x}=\frac{1}{n-1}\sum_{n-1}\frac{1}{\Delta x_i}\frac{\Delta V_i}{V_i}=\frac{1}{n-1}\sum_{n-1}\frac{V_{i+1}-V_i}{(x_{i+1}-x_i)V_i}\qquad |\bar{V}_{,x}|=\frac{1}{n-1}\sum_{n-1}\frac{1}{\Delta x_i}\frac{|\Delta V_i|}{V_i}=\frac{1}{n-1}\sum_{n-1}\frac{|V_{i+1}-V_i|}{(x_{i+1}-x_i)V_i} \tag{4.1}$$

It is possible to note a considerable variation of these parameters (≈46% and ≈19% in terms of absolute value respectively for columns and beams). This scattering is confirmed if the right side of Figure 6 is considered. In this diagram results in terms of semi-direct to direct (*s/d*), indirect to semi-direct (*i/s*), indirect to direct (*i/d*) pulse velocity ratios are shown for 13 measure points where at least two kinds of measure were carried out. A great difference can be observed. On the basis of these result the assumption of Toscana Region (Ferrini (Ed.) 2004) to avoid non direct ultrasonic measure has to be considered correct, above all for columns, and could be recommended to structural engineers when old existing buildings have to be investigated. On the contrary, if it should be impossible to make direct pulse velocity measures, a measure series has to be carried out for each test position and a local correlation function is strictly recommended.
In order to evaluate combined method provisions in Figure 7 a comparison between rebound strength R_{cR1}, pulse velocity strength R_{cV1}, Eqn. (3.5)-(3.6) SonReb strengths, R_{cS1} and R_{cS2}, is carried out. Data regard 39 test stations and consider only direct pulse velocity test, thereafter data are ordered for increasing values of R_{cS1}. It is possible to note a quite regular behavior: the R_{cS1} value is “average” of rebound and pulse velocity values. Thereafter for low pulse velocity results the combined method values are influenced by pulse velocity values in spite of rebound index values. These results could be considered as encouraging if it is possible to demonstrate the availability of R_{cS1} SonReb expression for this kind of concrete. As above said 31 cylindrical concrete specimens have employed for both compressive and direct pulse velocity test: results are discussed in Figure 8.

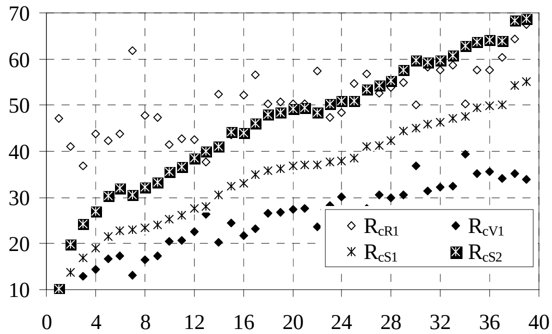

Figure 7. Comparison between rebound strength R_{cR1}, ultrasonic strength R_{cV1}, SonReb strengths, R_{cS1} - R_{cS2}

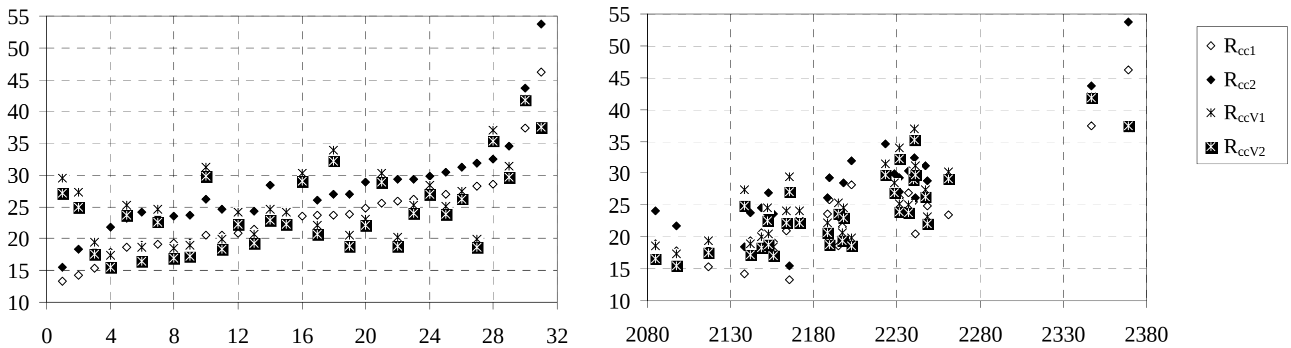

Figure 8. Comparison of R_{cc1} - R_{cc2} compressive strengths with core ultrasonic strengths, R_{ccV1} - R_{ccV2}. Data ordered for ascending core strength R_{cc1} (left side) and for ascending specimen mass density γ_i (right side).

Core specimens where $\varnothing = 75 \div 95mm$ diameter with various slenderness ratio $1.00 \le l/\varnothing \le 2.36$; these specimens are preliminarily tested using ultrasonic apparatus and then in compression; in this case two correlations between test cylindrical f_{cc} and equivalent cubic R_{cc} compressive strength were considered:

$$\left(\frac{f_{cc}}{R_{cc}}\right)_1 = 0.98\left(\frac{l}{\varnothing}\right)^{-0.21} \qquad \left(\frac{f_{cc}}{R_{cc}}\right)_2 = \frac{2 \cdot C_\varnothing C_r C_d}{1.50 + \varnothing/l} \tag{4.2}$$

The first (Biondi et alt. 2008) is valid in for $l/\varnothing \ge 1.00$, in the second $C_\varnothing$, C_r and C_d are correction coefficients depended on core diameter, rebar presence, damage due to drilling out (Dolce et al. 2006).
In Figure 8 together with R_{cc1} and R_{cc2} of Eqn. (4.1), Eqn. (3.3) is used in order to obtain both R_{ccV1} (with mass density $\gamma = 2350$ kg/m^3) and R_{ccV2} with actual mass density for each core ($2085 \le \gamma_i \le 2370$, average value $\gamma = 2200$ kg/m^3 with only two cores with $\gamma_i > 2260$ kg/m^3). In the right side of Figure 8 same data are shown in terms of specimen mass density γ_i. It is possible to note a clear dependence of compressive strength on core mass density (Biondi et al. 2008) and to point out that the use of actual mass density in Eqn. (3.3) reduces pulse velocity scattering. In Table 2 average values and standard deviation for each quantity are summarized (where $\rho_{ij} = \left|R_{ccVi} - R_{ccj}\right| / R_{ccj}$). As shown in Figure 4, probably a better fitting of experimental data could be obtained if actual Poisson's ratio is used in Eqn. (3.3). But unfortunately for structural engineer this value is quite difficult to determine than mass density.

Table 2. Average values and standard deviation in laboratory compressive and pulse velocity tests (31 cores)

	γ [kg/m3]	R_{cc1} [Mpa]	R_{cc2} [Mpa]	R_{ccV1} [Mpa]	R_{ccV2} [Mpa]	ρ_{c11} [-]	ρ_{c21} [-]	ρ_{c12} [-]	ρ_{c22} [-]
average	2200	23.72	26.15	25.76	24.25	0.21	0.20	0.19	0.20
standard deviation	63	6.54	7.26	6.20	6.48	0.27	0.22	0.17	0.18

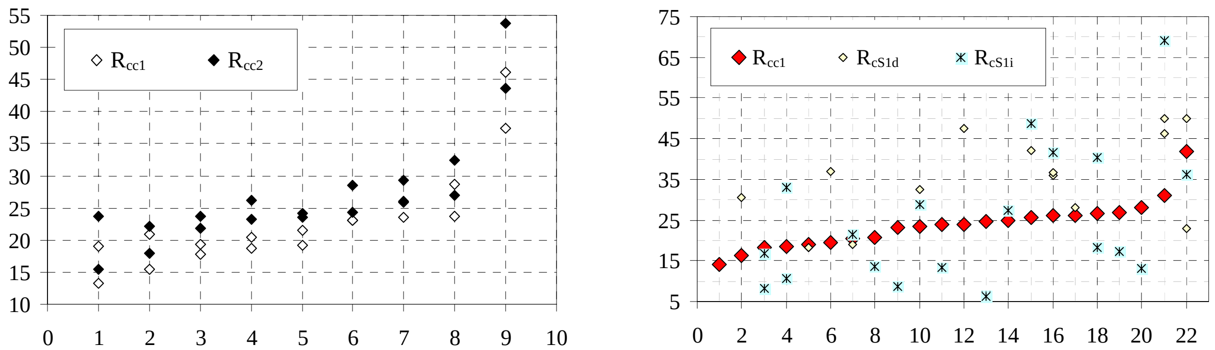

Figure 9. Core failure strengths, R_{cc1} and R_{cc2}, for 18 cores drilled out from 9 in-situ test positions (left side). Comparison of R_{cc1} (22 in-situ test, 9 twice cores) with in-situ SonReb results, R_{cS1d} direct, R_{cS1i} indirect

In Figure 9 a comparison that regards only those core specimens that can be correlated with in-situ tests is carried out. In the left side core failure strengths, R_{cc1} and R_{cc2}, for 9 double cores (i.e. for 9 drilled out cores, successively divided in two specimens) are shown. In spite of a great attention in laboratory cutting operation, a notable scattering of strength results is shown.
The average value of core compressive strength R_{cc1} ($R_{cc1} = \sum_2 R_{cc1i}$ if two cores are drilled out from the same in-situ test position) is shown in Table 2 ($R_{cc1m} = 23.72$ MPa) and it is quite similar to SonReb R_{cS1} values determined in in-situ test position ($R_{cS1m} = 28.71$ MPa). In the right side of Figure 9 a comparison of average values for 22 different drilling out sites (R_{cSm1}) distribution with in-situ SonReb results (Eqn. (3.5),

R_{cS1d} for direct pulse velocity tests, R_{cS1i} for indirect pulse velocity tests) is shown.
It is possible to note a strong scattering of in-situ provisions with direct SonReb tests that overestimate concrete strength (average value $R_{cS1dm} = 35.51$ MPa) and indirect SonReb tests that, on the contrary, show a quite similar average value of compressive ($R_{cS1im} = 23.95$ Mpa) with a discouraging dispersion.

5. CONCLUSIONS

In the first part of the paper, correlation formulas, basing on literature proposals or practice, to evaluate concrete compressive strength with non-destructive methods, have been discussed. It has shown the clear dependence of compressive strength on core mass density and the necessity to use the actual mass density in pulse velocity evaluation. In the second one, non-destructive and destructive methods to evaluated concrete failure strength, are been compared in order to outline some technical proposals.
In situ tests have shown a great, sometimes discouraging, dispersion of results, also for within the same building, due to several building phases, and the same element, due to environmental conditions and loading. This dispersion are due to cracking of structural elements and to poor mix design and a lack of concrete vibration during cast-in-situ too, both phenomena typical of old existing RC buildings. Particularly in the case of buildings with structural elements of wide dimensions or with large reinforcement ratio, the classification of concrete strength has shown notable levels of difficulty even if combined (SonReb) non-destructive method was used and an extreme difficulty to reach unitary conclusions can be detected (Biondi 2008).
Concrete strength determination by means of the combined non-destructive method has shown high levels of difficulty due to direct, indirect and semi-direct ultrasonic pulse velocity dispersion (Biondi et al. 2008).
On the basis of these results, in order to obtain correct compressive strength evaluation, it is strongly recommended the assumption of direct ultrasonic measure for combined method in particular to old existing buildings that have to be investigated, also for studies of local correlation function with destructive tests results. So the prudential statement of Tuscany Region (Ferrini (Ed.), 2004) to consider unacceptable indirect and semi-direct ultrasonic pulse velocity measurements could be considered as correct for this class of existing RC buildings.

REFERENCES

Biondi, S., Candigliota, E. (2008). Non destructive tests for existing R.C. structures assessment, *Proc. of fib 2008 Symposium: Tailor Made Concrete Structures: New Solutions for Our Society*, Amsterdam, May 19-22, 2008, Paper Italia-40
Biondi, S. (2008). The Knowledge Level in existing buildings assessment. *Proc of 14th World Conference on Earthquake Engineering*, Bejing, China, Paper No. 05-01-0462.
Calzona, R. (1982). Dome for the sports palace at Vasto (Chieti), *L'Industria Italiana del Cemento*, **1982:5**, 397-400, (in Italian)
CEN Comité Européen de Normalisation Ed. (2005). Eurocode 8: Design of structures for earthquake resistance - Part 3: Assessment and retrofitting of buildings. En 1998-3: 2005 E, Brussels
D.M. 14.01.2008 (2008). Norme tecniche per le costruzioni. *Gazzetta Ufficiale* **29: S.O. 30**, (in Italian)
De Sortis, A., Di Pasquale, G., Orsini, G., Sanò, T., Biondi, S., Nuti, C., Vanzi, I. (2000). Hospital behavior during the September 1997 earthquake in Umbria and Marche (Italy). *Proc of 12th World Conference on Earthquake Engineering*, Auckland, New Zealand, Paper No. 2514.
Dolce, M., Masi, A. (2005). Linee guida per la valutazione della vulnerabilità sismica degli edifice strategici e rilevanti. CRiS, Regional Seismic Department, Potenza, Italy, http://www.crisbasilicata.it, (in Italian)
Dolce, M., Masi, A., Ferrini, M. (2006). Estimation of the Actual In-Place Concrete Strength in Assessing Existing RC Structures, *Proc. of the 2nd International Fib Congress*. Naples, Italy. Paper ID 9-10.
Ferrini, M. (Ed.) (2004). Criteri per lo svolgimento di indagini diagnostiche finalizzate alla valutazione della qualità dei materiali in edifici esistenti in cemento armato. Tuscany, Regional Seismic Department, Florence, Italy, www.rete.toscana.it, (in Italian)
OPCM 3274 (2003). Primi elementi in material di criteri generali per la classificazione sismica del territorio nazionale e di normative tecniche per le costruzioni in zona sismica. *Gazzetta Ufficiale* **105: S.O. 72**, (in Italian)
OPCM 3431 (2005). Ulteriori modifiche ed integrazioni all'OPCM 3274, recante «Primi elementi in materia di criteri generali per la classificazione sismica del territorio nazionale e di normative tecniche per le costruzioni in zona sismica». *Gazzetta Ufficiale* **107: S.O. 85**, (in Italian)