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Structural safety assessment of existing structures: Italian Reinforced Concrete Buildings of first 20th century

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Department of Architecture
ARCHITECTURE: INNOVATION AND HERITAGE
Ph.D. Program
SSD: ICAR09 – STRUCTURAL ENGINEERING
Final Dissertation

Structural safety assessment of existing
structures: Italian Reinforced Concrete
Buildings of first 20th century

by
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Prof. Anna Bruna Menghini

Course n°31, 01/11/2015-31/10/2018



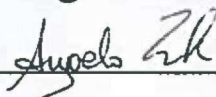
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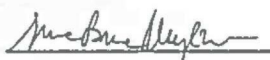
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Foreword

The structural safety evaluation of existing buildings is a topic of absolute importance at a global level and especially in Italy. During the World population increasing and economic development, occurred at the beginning of the twentieth century, the construction development reached its greatest diffusion. In this context, reinforced concrete structures represent a great part of construction heritage, both in Europe and in Italy. The "new technique", born in the late 1800s, reached its decisive impulse at the beginning of the twentieth century; the first applications were limited to the interstory slabs and large roofs, especially in industrial buildings. The most prestigious public buildings were built using reinforced concrete frames which gradually became competitive in residential construction field. In the latter case, the real development was after the Second World War, during the Italian economic upturn.

However, the catastrophic events that hit Italy in recent years, such as L'Aquila earthquake in 2009, Central Italy earthquake in 2016 and the most recent Morandi Bridge collapse in Genova (August 2018), have demonstrated the fragility of building heritage. In fact, most of reinforced concrete existing buildings, built more than 40 years ago, show advanced degradation conditions due to the inevitable material ageing as well as poor structural maintenance. Subsequently, about 70% of existing Italian buildings have been built without paying particular attention to seismic actions; because, the Italian territory was classified as almost entirely seismic only in 1974.

For these reasons, the topic of the structural safety of existing reinforced concrete structures represents an important issue, which should be addressed by the Engineering Community. However, the wide range of construction typologies makes the evaluation process particularly complex, especially considering the difficulties on standardizing methods related to verification and design of the interventions. This is the reason why modern scientific approaches devote particular attention to the knowledge of the construction, which is determined by the design indications at the time of construction, the construction details and the knowledge of material properties.

The aim of the research project is to deepen these aspects and to provide some solutions about analysis procedures used in the current Italian codes and those under development in Europe. These procedures, compared with advanced statistical approaches, are applied to

evaluate the structural capacity of reinforced concrete elements, extracted from an Italian school building built in 1940. The structural capacity estimations, evaluated according to different approaches and related to a variable number of material experimental data, are compared with the experimental tests actually performed on the structural elements.

In Chapter 1, the main sources of uncertainty related to the assessment of existing reinforced concrete structures are analysed, according to the evolution of Italian technical code; the main construction details of the time and the material mechanical properties, estimated by several experimental campaigns, are evaluated according to the main steps of the technical code.

In Chapter 2, considering the main approaches of structural safety assessment, the semi-probabilistic and the full-probabilistic approach, four of the most reliable methods in literature, are analysed with the aim to estimate the structural capacity of existing reinforced concrete elements. Starting from two distinct philosophies, the proposed methods, Design Value Method and Adjusted Partial Factor Method (semi-probabilistic approach) and Monte Carlo simulation and Sensitivity Analysis (full-probabilistic approach) are compared assuming the same probability of failure.

In Chapter 3, a complete and diversified experimental survey campaign is performed on structural elements (four column portions and a beam portion) extracted from an Italian school building, built in 1940 and recently demolished due to a negative results of the global structural capacity. Experimental techniques, destructive and non-destructive, are applied on materials; among the non-destructive tests, sonic tomography, performed on all the structural elements, allowed to estimate the quality of the concrete inside the structural elements, often heterogeneous and strongly cracked. Moreover, to estimate the concrete strength by indirect measurements, experimental correlation laws are developed for the case study and they are compared with those present in literature. Finally, capacity tests were carried out on the structural elements, whose parameters of interest are acquired through Digital Image Correlation analysis. The results of the experimental tests on the elements are also compared with F.E.M. analyses based on the indirect measure results.

In Chapter 4, semi-probabilistic methods (Design Value Method and Adjusted Partial Factor Method) and full-probabilistic (Monte Carlo simulation and Sensitivity Analysis) are applied for the evaluation of the structural capacity of existing reinforced concrete elements, according to the variable material test number and therefore different strength values. The application results are compared with the experimental tests actually carried out on the structural elements. Moreover, the Italian code approach, which considers a reduction factor

(e.g. Confidence Factor) for mean values of strengths, determined according to the degree of knowledge (Knowledge Level) of the structure/element is applied to the same structural capacities.

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Chapter 1

The reinforced concrete existing structures

1.1 Abstract

Reinforced concrete constructions represent a large part of the Italian building heritage, especially in the main cities. The technique of reinforced concrete applied to civil constructions was born in the early twentieth century. Thank to high mechanical performance of the materials and rapid construction execution, the reinforced concrete technique had a rapid development. In Italy, the diffusion of this new technique was a solution to the seismic problem: the monolithic nature of the reinforced concrete frame showed good behaviour during the application of horizontal actions, especially compared to traditional masonry constructions. However, the Italian code classified the national territory almost entirely as seismic only in 1974 and the appropriate construction details of the structural elements were prescribed. For this reason, approximately 70% of existing concrete constructions have been designed against vertical loads only without particular attention to construction details. As a consequence, for existing RC buildings, the knowledge of the construction chosen as case study represents the basis of the structural safety evaluation process. In fact, the current Italian code approach focuses on three main sources of uncertainty: the geometry of the structure, the construction details (especially the position of the steel reinforcement) and the material mechanical properties (relevant for concrete strength mainly). In this Chapter, the focus is on the importance of construction details and the high variability of the mechanical properties of the concrete in existing buildings through the examination of existing design and experimental data, with reference to the main steps of the Italian code evolution concerning these aspects.

Le costruzioni in calcestruzzo armato ricoprono una parte importante del patrimonio edilizio

italiano, soprattutto nei principali centri urbani. La tecnica del calcestruzzo armato, applicato alle costruzioni civili, nacque agli inizi del XX secolo, e grazie alle elevate prestazioni meccaniche dei materiali ed alla velocità di esecuzione del cantiere, ebbe un rapido sviluppo. In Italia, tuttavia, la svolta decisiva alla diffusione della nuova tecnica avvenne soprattutto come soluzione al problema sismico: la monoliticità del telaio in calcestruzzo armato presentava un comportamento migliore durante l'applicazione delle azioni orizzontali, soprattutto rispetto alle costruzioni tradizionali in muratura. Tuttavia, solo nel 1974 la normativa Italiana classificò quasi interamente il territorio come sismico, prescrivendo appositi dettagli costruttivi per gli elementi strutturali. Quindi, circa il 70% delle costruzioni in calcestruzzo esistenti è stato progettato per resistere ai soli carichi verticali e senza particolare attenzione ai dettagli costruttivi. Per questo motivo, alla base del processo di valutazione della sicurezza strutturale si pone il tema della conoscenza della costruzione. L'approccio normativo odierno, infatti, pone l'attenzione su tre principali fonti di incertezza: la geometria della struttura, i dettagli costruttivi (soprattutto la posizione delle barre di armatura) e le proprietà meccaniche dei materiali (la resistenza del calcestruzzo risulta spesso variabile). In questo capitolo, ripercorrendo le tappe principali dell'evoluzione normativa Italiana, attraverso lo studio di alcuni progetti e dati sperimentali relativi alle costruzioni del primo Novecento, è evidenziata l'importanza dei dettagli costruttivi e l'elevata variabilità delle proprietà meccaniche del calcestruzzo nelle costruzioni esistenti.

1.2 The origins of reinforced concrete technique

The Reinforced Cement Concrete (RCC) was born in France in the nineteenth century thanks to the theories and projects of Francois Hennebique, who was the first to patent the combination between steel and concrete finalized to beam elements. However, Monier began to make small objects of concrete (tanks, basins, tubes and garden tools) first, and he was able to largely reduce concrete thickness by adding the steel reinforcement. In 1867 his first patent inspected a simple garden vase made of wire mesh in which a few centimetres compound of concrete and sand was spread. Monier spent his life to test the technique of the new material even if his researches receive no attention.

The modern history of the reinforced concrete has begun by Hennebique and the development of the technique was given by solving the monolithic problem of structure: reinforced concrete elements assume unique identity according to longitudinal and transversal steel. This solution distributed tensile and compression stress in steel and concrete, respectively. In 1882 the patent of a slab with T-section truss has been realized: both longitudinal and vertical rebars were arranged (vertical rebars was placed in order to connect the lower rebars to slab steel reinforcement but mainly to resist the shear stress). The latter problem was dear to Francois Hennebique, who put vertical plate which connected the lower reinforcement with upper compressed concrete (see Figure 1.1); these elements became the

symbol of the new constructing system. Hennebique proposal laid the bases for the rigid

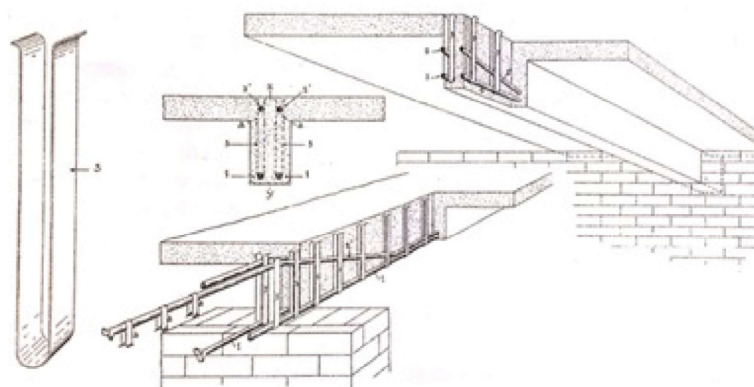


FIGURE 1.1: Explanatory drawing attached to Hennibique's patent on 18th December 1897 in France

frame structure composed by beams and columns; in addition, he showed entrepreneurial talent since the reinforced concrete structures became a valid solution in terms of speed and ease execution compared to the traditional techniques. In fact, different work phases were carried out at the same time, according to specific competences, without attend the typical phases of masonry structure (Muratore [73]). Moreover, Hennebique was able to support the new technique: in his location in Bruxelles and then in Paris (from 1897), he started to promote the reinforced concrete by sending brochures, private letters and invitations to the most influential technicians and architects of the period. He recruited engineers from the best institutes of France and gave them the possibility to write for the most important magazines.

The Hennebique system propagated in Italy as in Europe. Engineer Giovanni Narici, located in Naples, heard about it in Switzerland and he became the first Italian representative of the system. In 1894 he started to have contacts with entrepreneur Giovanni Antonio Porcheddu from Turin.

However, the linear elasticity theory, elaborated by Hooke, Navier and De-Saint Venant in the 1800s, couldn't be applied to a non-homogeneous, anisotropic and strangely elastic material as reinforced concrete (Iori [48]). The civil engineer Koenen (1849 – 1924) elaborated some simplified hypothesis about this problem. Considering a bending rectangular-section slab, he imagined that neutral axis was on barycentre (e.g. as happens with homogeneous material). Thus, in the upper side, compressed concrete behaved as a homogeneous and elastic material, while, in the lower side, only steel reinforcement resisted to external stress. This simplification ignored the concrete tensile strength, even though Koener knew about it. Moreover, he wrongly claimed that both materials could reach simultaneously resistance limit.

Nevertheless, in 1894, Edmond Coignet and Napoleon de Tedesco presented a paper to the

Civil Engineers Society of Paris and it was the first time that *ideal homogeneous section* was considered. Elasticity theory applied to reinforced concrete was not considered suitable to non-homogeneous material, so the French engineers claimed that increasing steel area, by setting an appropriate coefficient, could be sufficient to consider the entire concrete section. They demonstrated that coefficient was equal to material elastic modulus ratio by applying the flat section conservation and, for the first time, the congruence between steel and concrete deformation. Although the method was defined *without any scientific value*, it increasingly propagated in the university courses and practices, especially in Italy, because it was considered appropriate, safe, user-friendly and an apparently correct instrument.

In the construction sector, there were two companies that operated even before the introduction of reinforced concrete and they represented a notable example for experimenting the new system but with a different approach.

The first one was Odorico company, which was established in Milan in 1827 and take advantage of the prestigious European patents (i.e. especially using Monier patent); the second one was Carlo Gabellini and partners, which was the oldest construction company in Italy. It never implemented foreign patents and on September 1896, proposed its original system. This latter (e.g. *Gabellini small net concrete*) was made by modelled narrow diameter rebars and thin concrete wall; this technique was used mainly to make tubes.

In addition, there were other Italian companies which used European patents: Porcheddu company and Ferrobeton excelled. At the beginning Italian companies were supported by the foreign ones but they continued their studies and elaborated their own patents, consequently. This is the case of Porcheddu company which, in 1908, patented a special rebar for reinforced concrete constructions: steel reinforcement, along its longitudinal axis, changed transversal section (see Figure 1.2). This solution was very important because it increased the bond between concrete and steel; in fact, from this patent the ribbed bars would be elaborated.

The production of different types of concrete is another important aspect of this field, in particular Portland Concrete, by the name of the British island, was an excellent hydraulic binder. The production of this concrete quickly became part of an industrial process, therefore in Civitavecchia (Cerrano company) (see Figure 1.3) and in Venice the first two factories were established. In the early 1900s, reinforced concrete technique was limited to factories, warehouses and in general to slabs because the traditional construction technique was still widely used, especially in the local realities. However, the turning-point is represented by the seismic problem: the new reinforced concrete technique started to be frequently implemented when the earthquake in Messina and Reggio Calabria (1908) occurred. In 1920s, the anti-seismic frame (the first house was built by Porcheddu company in Genova in 1906, see Figure 1.4) spread in Italy: it radically changed how to use the reinforced concrete.

However, thanks to Arturo Danusso, who received the first reward of the international

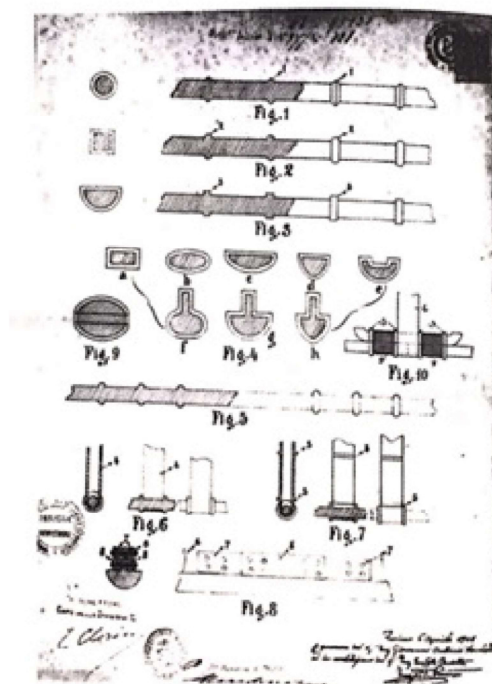


FIGURE 1.2: Patent no. 95235 (1908) Special refined rebars and their application to reinforced concrete constructions; Engineer Giovanni Antonio Porcheddu



FIGURE 1.3: Advertisement of Cerrano company (from *il Cemento*, 1911)

contest, there was a breakthrough; he had attended Prof. Guidi school and learnt practice from Porcheddu, therefore, in 1909, he discovered that the seismic problem had a solution which was light and ductile structure rather than a massive and extremely rigid one. The reinforced concrete could guarantee that the structure fluctuated without disconnect itself after a shock since it was considered to be monolithic.

1.3 The Italian heritage of existing RC structures

The catastrophic events that occurred in Italy in the recent years, which caused many loss of lives, highlighted the importance of structural safety evaluation. The last seismic events,



FIGURE 1.4: The first frame house made completely of reinforced concrete and realized by Porcheddu company in Genova in 1906

first *L'Aquila Earthquake* in 2009 and after *Central Italy Earthquake* in 2016, demonstrated the *high vulnerability* of the Italian structural heritage. Moreover, the latest disastrous event, that is the collapse of *Morandi Bridge* on 14 August 2018 in Genova, showed that several existing RC structures, built 50 years ago, needed to be monitoring or repaired. Nowadays, Italian buildings are more than 14 million distributed, almost uniformly throughout the territory (e.g. from ISTAT census 2011 [31], Italian buildings are divided in: 19% North-West, 17% North-East, 14% Centre, 21% South and 13% Islands). In this context, the 16% are public buildings and the 84% are residential buildings; the latter, according to their remarkable number (over than 12 million buildings), are analysed following different categories.

Most of residential buildings, almost 57%, are masonry constructions, whereas, only 29% are reinforced concrete structures and 13% related to other construction techniques. Obviously, the previous percentages consider the number of buildings, not their volume: it means that, each reinforced concrete building usually, develops a larger volume than masonry structure. Therefore, people living in each reinforced concrete building are more than those who live in masonry building, with important consequences in terms of safety.

Moreover, almost 60% of residential buildings were built before 1970 (see Figure 1.5); it means that these structures are over 50 years old. In addition, considering that the first modern seismic zone differentiation was made in 1974 [28], it is clear that many Italian structures need a careful evaluation, according to structural safety and seismic risk.

However, people are less sensitive to structural safety, in fact the results of ISTAT Census 2011 [31] about the conservation state of residential buildings are: 2% very bad, 15% lousy, 51% good and 32% excellent. Probably, no structural evaluation is made on these results.

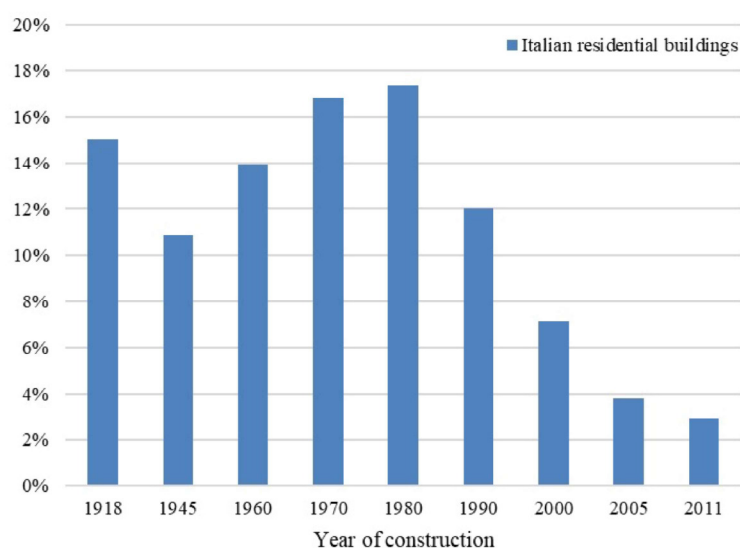


FIGURE 1.5: Italian residential buildings according to year of construction

Moreover, concerning the construction material, it is interesting to evaluate the evolution trend of residential buildings. Figure 1.6 represents the evolution of Italian residential buildings; it shows that the total trend had a slight decline during the World War II and another important one after 1980. In fact, the peak was reached about 1980, when re-

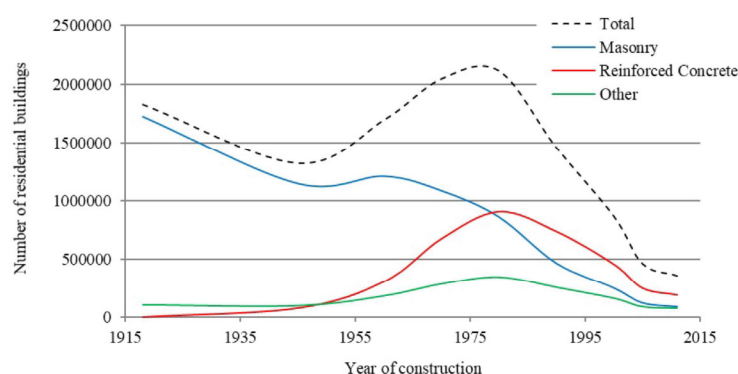


FIGURE 1.6: Evolution trend of Italian residential buildings according to construction materials

inforced concrete structures had maximum development. Actually, while the reinforced concrete building increased, masonry structures decreased. Furthermore, other types of constructions (i.e. steel or timber building) were constant over time.

In this context, in order to understand the structural safety state of existing constructions, it is very important to know the main steps of the Italian codes over time: particularly, the evaluation of construction details and material strength limits.

1.4 Italian code approach

Many Italian reinforcement concrete structures built before the World War II present architectural and historical values; whereas, the most of the RC constructions (almost the 35% of the total, see Figure 1.5) were designed during the economic development (about 1960s) and today they are over 50 years old.

Moreover, concerning that the seismic regulation at the time was inadequate, it is easy to understand how much the assessment of the existing RC structures is important. Usually, the existing structures have a lower structural safety level than the new structures, nevertheless, their use is permitted.

For this reason, *Italian Technical Codes NTC2008* [24], *Technical Specification 2009* [21] and its updating *NTC2018* [27] consider the knowledge of geometry, details and material properties as the main step of *structural assessment of existing structures*. However, it is not easy to standardize a single procedure to evaluate the structural safety of all the existing RC structures, because there are many different typologies with various constructive details.

The performance approach of Italian Technical Code considers several aspects related to the seismic design and the structural upgrading by different intervention levels. In this Section, only the aspects related to knowledge of geometry, details and material properties are evaluated; particularly, construction details are evaluated according to the Technical Code evolution and material properties (especially related to concrete) are examined in relation to several Italian existing RC structures.

In fact, the main uncertainties on structural safety assessment of existing RC structures are: the positions of steel reinforcement (usually, steel strength is not very variable) and the variability of concrete strength (usually, cross-section size is measurable).

However, in order to better understand how these uncertainties affect existing RC structures, it is suitable to go back over the main milestones of the Italian Technical code. According to the occurred seismic events, the Italian Technical code was developed as follows:

- *R.D.* 1909 [78]: after *Messina Earthquake* (1908). It was the first Technical code which considered seismic loads (proportional to gravity loads) and fixed the maximum stress for materials. It was also the first code which regulated reinforced concrete structures, especially for special public buildings;
- *R.D.* 1929 [80]: the first seismic zoning was introduced. It considered two categories (I and II) and imposed columns cross-section equal to 30 cm x 30 cm. In Table 1.1 the main characteristics between two categories are shown;

<i>Characteristics</i>	<i>Cat. I</i>	<i>Cat. II</i>
Maximum building height [m]	10	12
Maximum story number	2	3
Interstory [m]	5	5
Minimum masonry depth for I story buildings [cm]	60	50
Seismic vertical loads related to gravity loads	0.50	0.33
Horizontal loads related related to mass (ground floor)	1/8	1/8
Other stories	1/6	1/10

TABLE 1.1: The main characteristics of Technical Code *R.D.* 1929 [80] related to two seismic categories

- *R.D.* 1937 [79]: in this case seismic loads were reduced for each categories, furthermore, the dynamic nature of earthquake was lost. The latter concept was taken up again only in 1975;
- *Code n.1684*, 1962 [26]: after *Irpinia Earthquake* (1962). It introduced geological evaluation of site building, design and technical report, but reduced the prescriptions in order to promote new technologies and economic development;
- *Code n.64*, 1974 [28]: after *Tuscania Earthquake* (1971). It considered design and check of structural elements. In this code response spectrum, structural plastic behaviour and concept of safety life were introduced for the first time;
- *Code n.214*, 1981 [23]: after *Irpinia Earthquake* (1980). It introduced the III seismic zone (see Figure 1.7) and described repair and retrofitting interventions related to masonry and reinforced concrete buildings. Herein, semi-probabilistic approach and capacity design were introduced, besides prescriptions on construction details are described for the first time;
- *D.M.* 1996 [25]: it was written according to technology innovation and International Eurocodes. In this code, limit states were used for the design process and a response coefficient to define seismic loads (capacity design) was introduced;
- *OPCM* 2003 [22]: after *San Giuliano di Puglia Earthquake* (2002). It introduced the IV seismic zone and exclusively adopted the semi-probabilistic approach according to limit states, dynamic analyses with response spectrum. All type of constructions were considered and it was the first code with totally performance approach;
- *D.M.* 2008 [24]; after *L'Aquila Earthquake* (2009);
- *D.M.* 2018 [27]; after *Central Italy Earthquake* (2016).

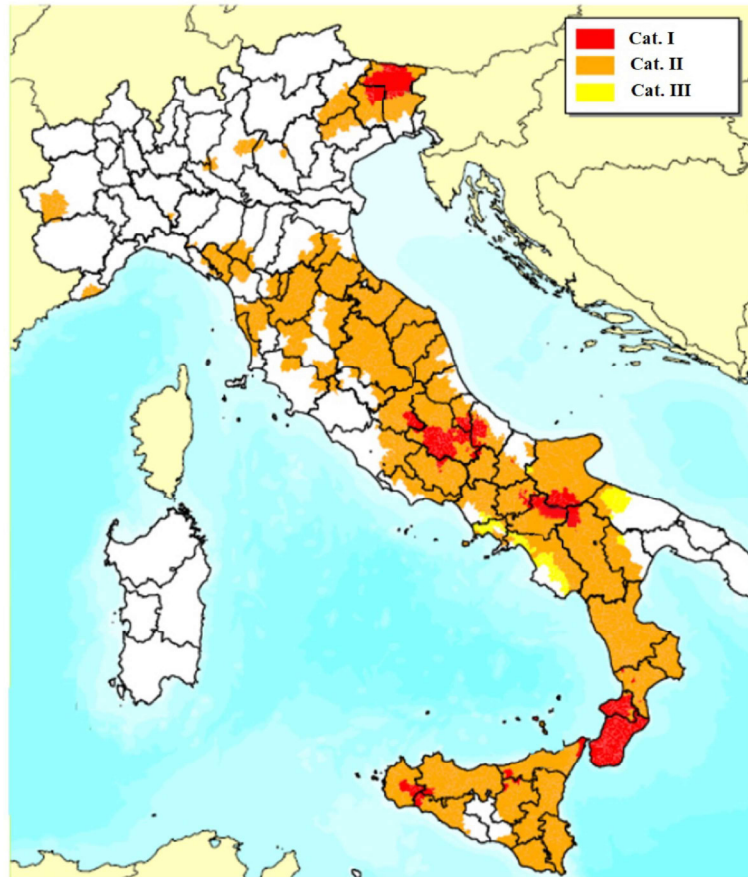


FIGURE 1.7: Seismic zoning after *Code n.214*, 1981 [23] according to Categories I, II and III

1.4.1 The importance of construction details

Previously as described, construction details represent one of the main uncertainties on the structural safety assessment, especially for existing structures. Usually, steel reinforcement strength is less variable than rebar positions; for this reason, the knowledge of constructive details is a fundamental step for the structural capacity evaluation.

As explained in Section 1.4, specific prescriptions related to construction details have been carried out only thanks to performance approach (e.g. after 1980). Moreover, at the beginning of 20th century only few design prescriptions were described and no construction details were regulated.

In this subsection, some reinforced concrete building designs, used as industrial and public building, are evaluated to know the usual constructive details at the time of construction. Some of the most interesting structures built before 1930 are shown in Santarella [91] and displayed below:

- *Catoni's cotton mill* (Milan, 1905 – 1919)

The structure was composed of three storeys, with $6 \times 8 = 48$ spans; these latter have a dimension 6.90m x 6.70 and the principal beams which belong to the structure

are oriented in the same direction, whereas the secondary beam spacing is equal to 1,20m. The slab thickness is equal to 7 cm. Plan view and transversal section of structures are shown in Figures 1.8 and 1.9. It is well known that structures with

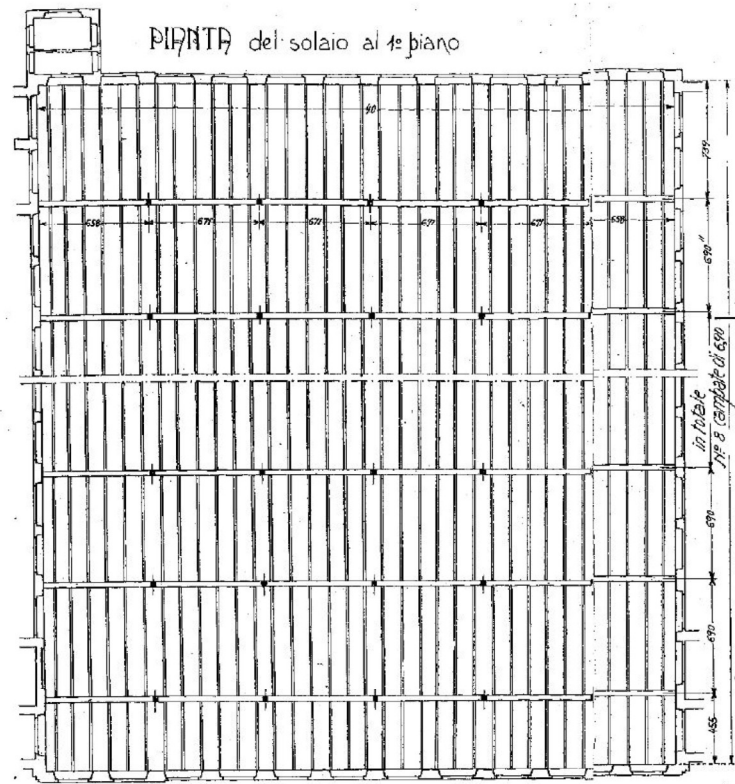


FIGURE 1.8: Catoni's cotton mill: plan view of first floor

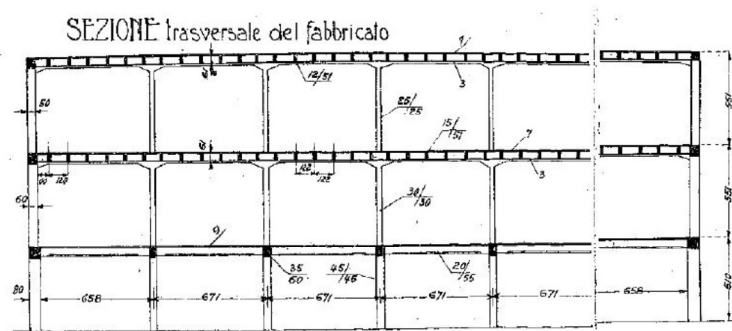


FIGURE 1.9: Catoni's cotton mill: transversal section

resistant frame on only one direction show several problems during seismic events. However, the peculiarity of this structure is the steel reinforcement in principal beams. Figure 1.10 shows a principal beam: steel reinforcement layout is composed by $4\phi34 + 4\phi25$ (lower reinforcement) and $4\phi25$ (upper reinforcement); in this case, steel reinforcement are anchored in beam support according to different lengths. In tensile zone (upper side of support) anchor length is about 60 cm (for diameter of 34mm, $L_{aT} = 24\phi$), whereas in compression zone (lower side of support) anchor

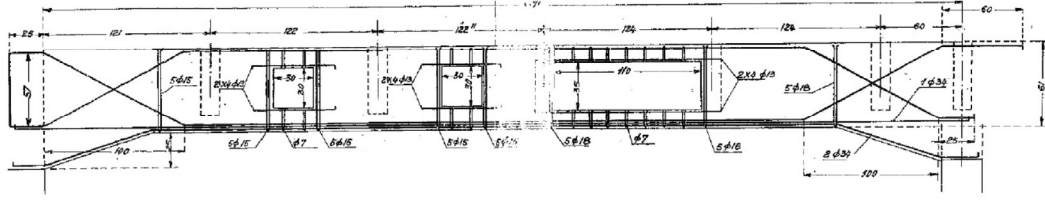


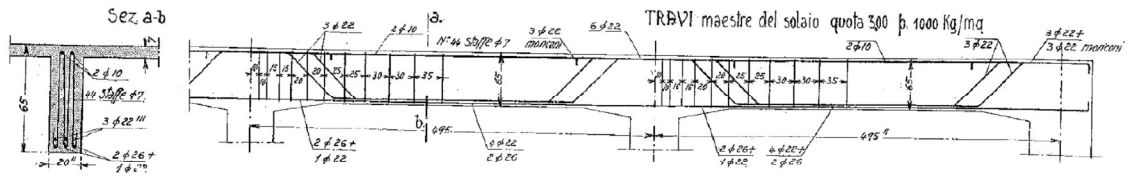
FIGURE 1.10: Catoni's cotton mill: principal beam

length is about 25 cm (for diameter of 25mm, $L_{aC} = 7.35\phi$). Nevertheless, in the modern design approach, steel anchor length is usually $L_a \approx 40\phi$ (with ribbed rebars).

- *Appula's tartaric acid factory* (Linate, 1924)

The factory was built by Odorico building company (the oldest building company in Italy) and it was composed of the main building (in plan 20m x 54m and with height 28.5m) and two additional building with only one story (in plan 20m x 30m and 43m x 30m). However, the structure covers 3600m² of surface.

The slabs and the beams were designed for loads from 1000kg/m² to 3000kg/m²; one of the principal beams is shown in Figure 1.11. According to the shear stress,

FIGURE 1.11: Appula's tartaric acid factory: principal beam designed for load 1000kg/m² and transversal section

the spacing of transversal steel reinforcement decreases near the support, from 35 cm to 10 cm; however, the transversal section *a-b*, shown in Figure 1.11 (left side), displays double U stirrups (they are not closed) which connect lower and upper steel reinforcement.

This solution was similar to Hennebique's system, patented in 1897 as described in Section 1.2, that connected longitudinal rebars without confine the cross-section.

However, principal beams were designed to preserve elastic behaviour of steel during the load application ($\sigma_{s,amm} = 115$ MPa), but was a failure in concrete side ($\sigma_{c,amm} = 4.75$ MPa). In that period, reinforced concrete plastic theory was developed by Danusso and Colonnetti, as described by Iori [49].

An other interesting structural element is the sloped beam that supports the steps (see Figure 1.12); in this case, according to torsional stress no lateral longitudinal rebars are designed, while triangular stirrups connected upper bar and lower reinforcement. Anchor length is not considered both in the beam and in the step.

- *Bianchi's car factory* (1920-21, Milan)

The structure was composed by two buildings with two spans and two stories (see Figure 1.18); however, considering also the internal buildings, with only one story, the total area is equal to $13500m^2$. All the spans are equal to 6.0 m x 6.65 m, they are

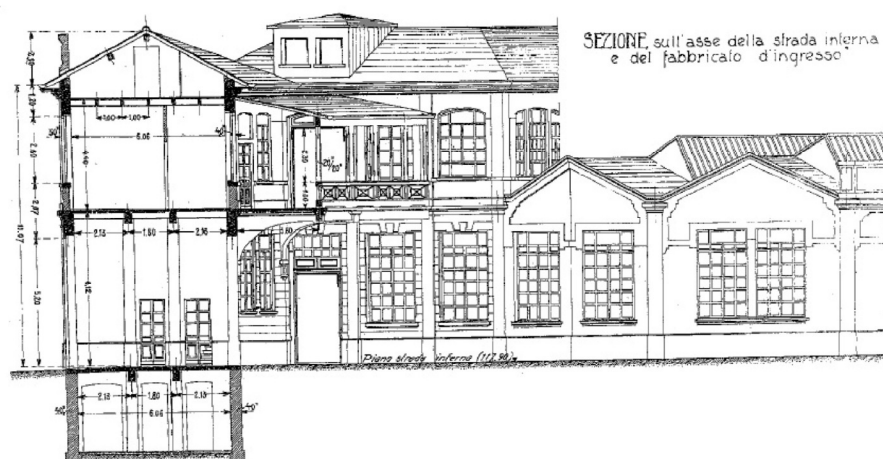


FIGURE 1.18: Bianchi's car factory: transversal section of internal building side

composed by principal beams (40 cm x 68 cm), secondary beams (20 cm x 43 cm) and slab (the thickness is 8 cm). The column cross-sections are equal to 28 cm x 48 cm on ground floor and 28cm x 28cm on first floor; Figure 1.19 shows the beam-column joint, where column cross-section changed.

It is noted that the beam-column joint is unconfined, there are not stirrups into the joint and anchor lengths of beam reinforcement are very limited. Moreover, the anchor lengths related to steel reinforcement in columns started/finished near the upper side of beam. Thus, this joint is very weak according to modern design standard.

The truss beams were made on the ground and then they were located on the roof; the upper side of column was modelled specifically.

- *Cotton factory* (1922, Savona)

The structure was extended on rectangular area (40 m x 120 m) of three stories (see Figure 1.20), the lateral span are 7.1 m long and the central one is equal to 10.8 m; in this latter span the railway lines were contained.

The stress on structural elements, beams and columns, were evaluated considering the entire resisting frame with elastic joint; however, the manual calculation of system stress led to simplified approach (e.g. graphical-numerical) which distributed the stress from the joint to the elements. According to this method, the stresses on elements were calculated considering fixed load and two variable loads (negative and positive); for this reason, the columns were verified according to axial force and

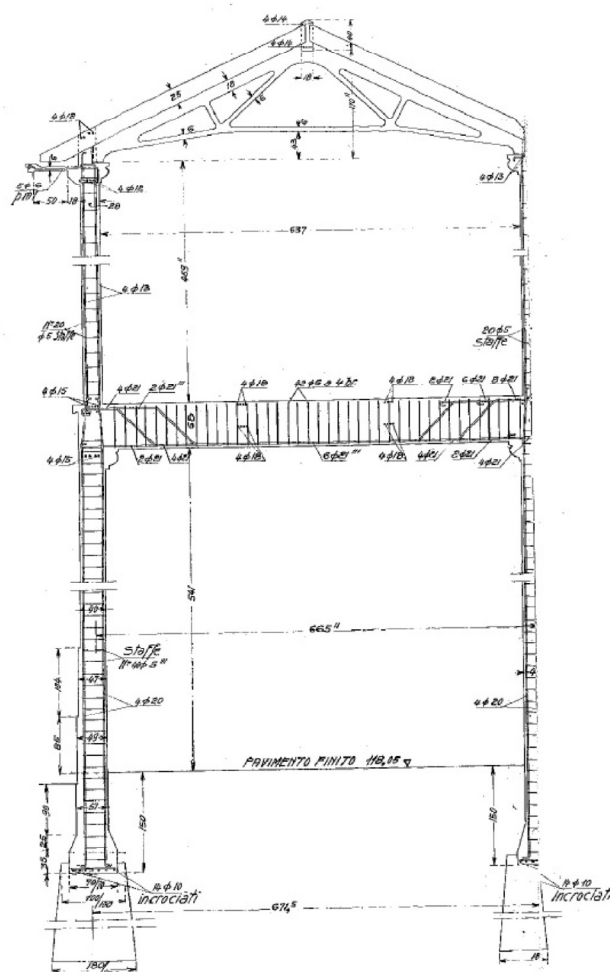


FIGURE 1.19: Bianchi's car factory: transversal section of external building side

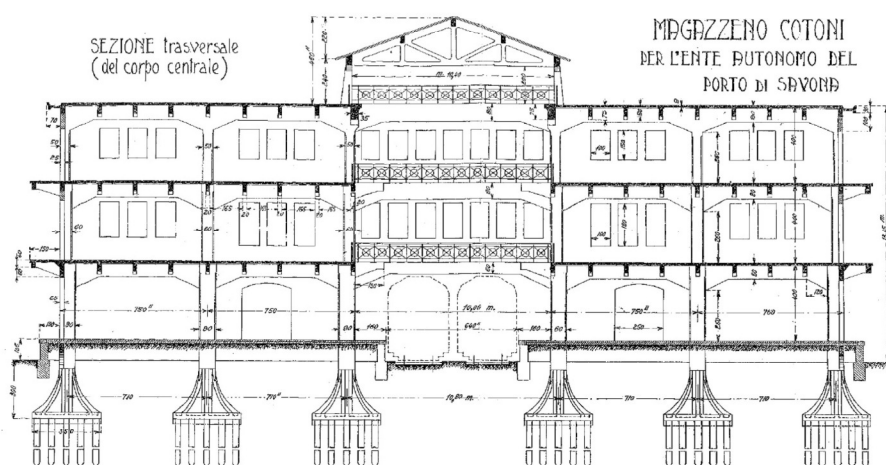


FIGURE 1.20: Cotton factory in Savona: transversal section

bending moment (see Figure 1.21).

The use of this method led to calculate more steel reinforcement on the structural element:

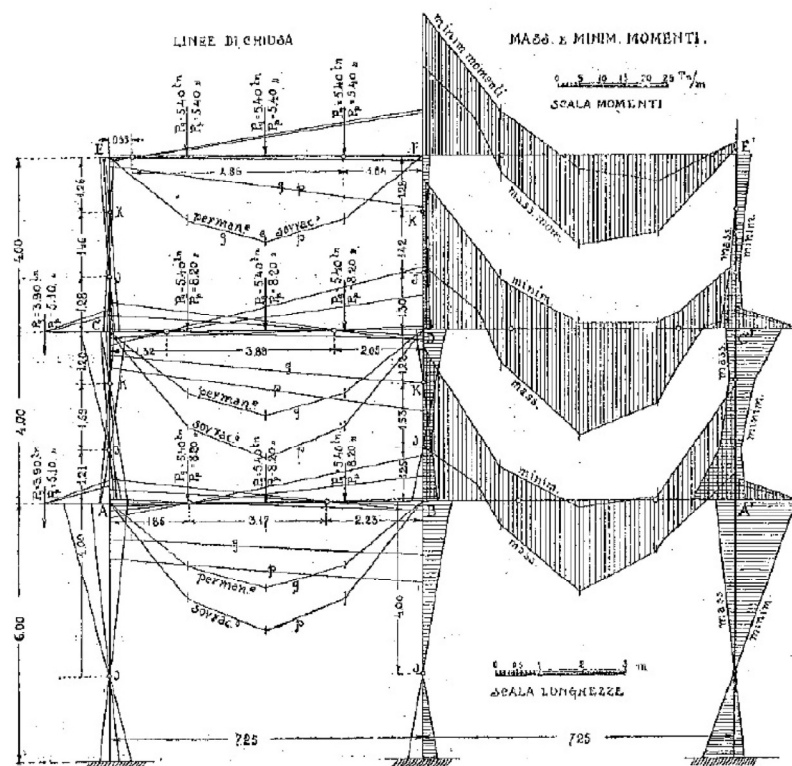


FIGURE 1.21: Cotton factory in Savona: bending moment stress on resisting frame

- More steel reinforcements into the joints and close them were designed, both for the beams and for the columns;
- Anchor length, related to steel reinforcement in column, was longer than usual one;
- The stirrups of columns continued also into the joints.

All these information are shown in Figure 1.22.

- *Bank of Italy* (1922-1923, Reggio Calabria)

The building was designed according to technical code for structures built in seismic zone, which considered seismic coefficients (μ) in order to calculate the horizontal loads (i.e. $\mu = 1/10$ for the basement, $\mu = 1/8$ for the ground floor and $\mu = 1/6$ for the first floor). Moreover, vertical component of seismic load was calculated both in increasing of 50% the gravity loads (seismic action downward direction) and decreasing of 25% that loads (seismic action upward direction).

However, the structure was composed by the main building, for office use and the posterior building, for residential use. The structural design was carried out considering elastic frame (see Figure 1.24), which was stressed by horizontal forces. These latter were applied in both directions and the stresses on joint were distributed to all structural elements (see Figure 1.26).

According to this method, structural design presented accurate construction details:

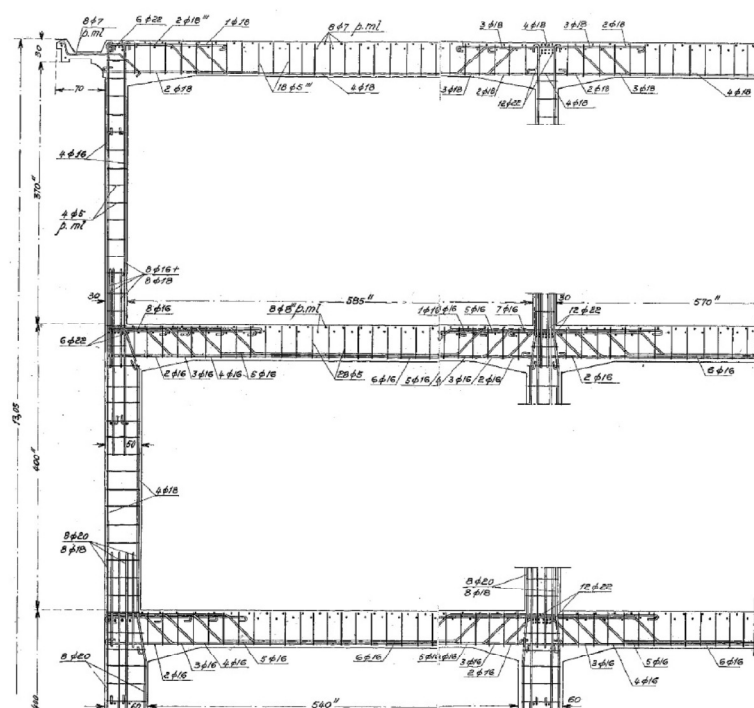


FIGURE 1.22: Cotton factory in Savona: steel reinforcement layout in resisting frame

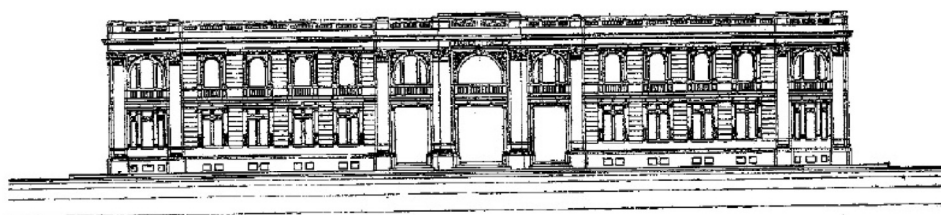


FIGURE 1.23: Bank of Italy in Reggio Calabria: front view

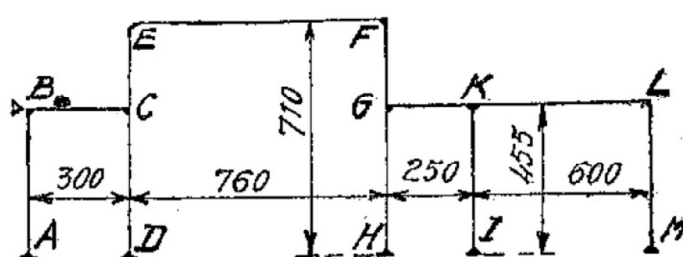


FIGURE 1.24: Bank of Italy in Reggio Calabria: resisting frame

- Longitudinal steel reinforcement in columns continued over the joint and the stirrup spacing was very small;
- According to seismic action upward direction, some beams presented upper longitudinal reinforcement in the span;
- The stirrup spacing was smaller near to the joint and stirrups of columns continued into the joint.

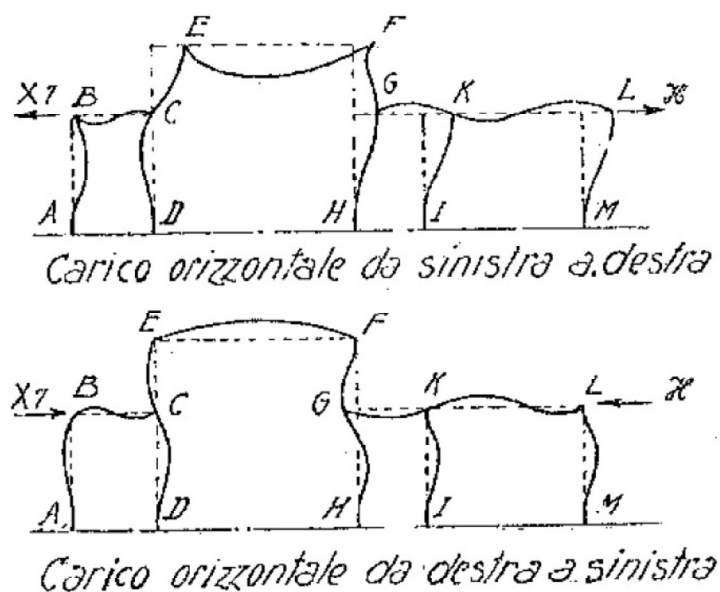


FIGURE 1.25: Bank of Italy in Reggio Calabria: application of horizontal forces to resisting frame

The Figure 1.26 shows steel reinforcement layout of designed resisting frame.

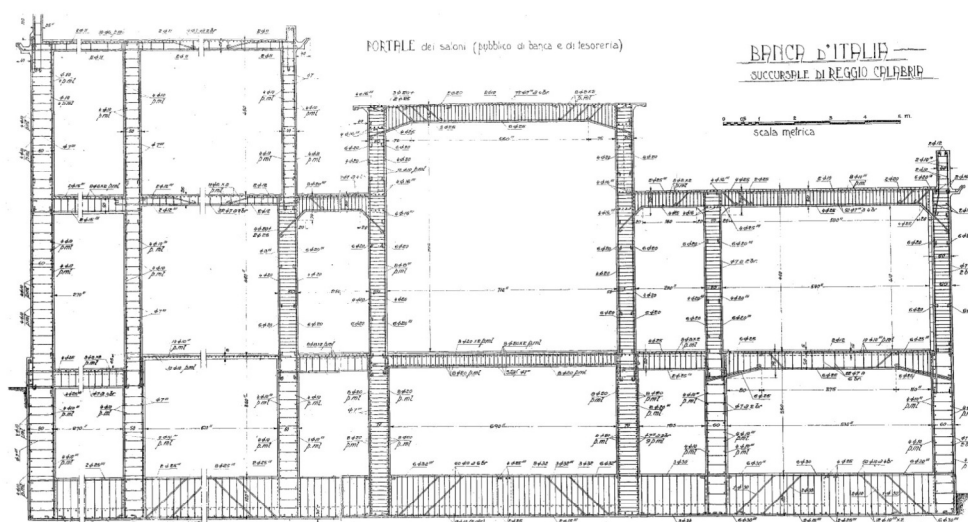


FIGURE 1.26: Bank of Italy in Reggio Calabria: steel reinforcement layout of resisting frame

1.4.2 The knowledge of material properties

Compressive concrete strength is one of the most important parameters on structural safety assessment of existing reinforced concrete buildings. Usually, existing structures show a great variability of concrete strength both on the same structure but also in the same structural element. For this reason, evaluation of concrete strength uncertainty is a key point of structural assessment.

In this subsection, several compressive concrete tests, performed by *Structural Laboratory of Roma Tre* (PRISMA), are described and analysed according to different evaluation parameters:

- Year of construction;
- Building function;
- Location;
- Number of samples extracted from each building.

The entire database concerns 40 reinforced concrete buildings built from 1929 to 1975 in Italy; totally, from these constructions were extracted over than 370 concrete samples which are tested in the Structural Laboratory of Roma Tre.

These structures are located mainly in Lazio (44%), Tuscany (41%), Calabria (13%) and other regions (3%). Moreover, evaluated structures are divided on the following functions:

- *School buildings* represent the 35% of the samples extracted and they were built from 1958 to 1975;
- *Public buildings* represent the 34% of the samples extracted and they were built from the entire database time (e.g. from 1929 to 1975);
- *Hospitals* represent the 15% of samples extracted and they were built about to 1975;
- *Industrial buildings*: represent the 12% of the samples extracted and they were built from 1929 to 1939;
- *Tall building*: represents the 5% of the samples extracted and it was built in 1960.

According to the entire database, it is possible to identify Italian structures with relevant architectural and historical values: Figure 1.27 shows *Customs of river port* (1929) and *Naval tank* (1929), *Ex slaughterhouse* (1932) and *Exam Palace* (1936) are shown in Figure 1.28; finally, *Farnesina Building* (1937-59) and *Termini Railway Station* (1939-47) are shown in Figure 1.29. Starting from compressive concrete tests, for each structure it is

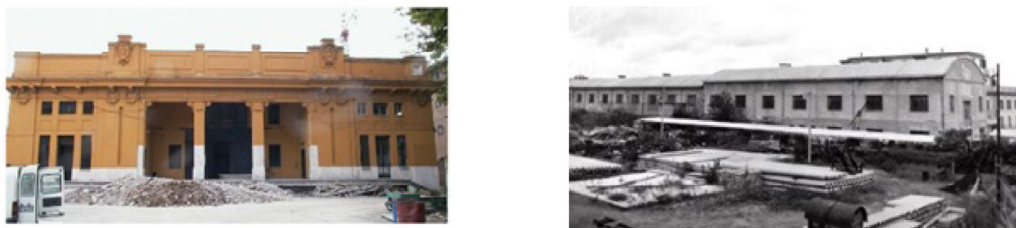


FIGURE 1.27: On left side, Customs of river port (1929); on right side, Naval tank (1929)

possible to evaluate the mean value and the relative coefficient of variation. In the Figure



FIGURE 1.28: On left side, Ex slaughterhouse (1932); on right side, Exam Palace (1936)



FIGURE 1.29: On left side, Farnesina Building (1937-59); on right side, Termini Railway Station (1939-47)

1.30, the mean values of compressive concrete strength are shown according to the year of construction. Previously as seen, during the Second World War very few structures were built; before that time mean values of compressive strength were very low (e.g. $\mu_{fc} \approx 13$ MPa). However, *R.D.* 1909 [78] (e.g. item 7) fixed the minimum value for concrete strength equal to 15 MPa; in that time only few industrial buildings were built on reinforced concrete. Obviously, the degradation state can be caused by a loss of strength. After 1960 the



FIGURE 1.30: For each building, the mean values of compressive concrete strength according to year of construction

concrete strengths increased, in this period many schools and public buildings were built. However, in order to understand the variability of concrete strength, coefficient of variations (related to concrete strength) are evaluated for each building. In Figure 1.31, coefficients of variation are shown according to the year of construction. It is cleared that only very few values lower than $COV = 0.15$ are present (usually, this value is used for new structures); moreover, the mean value of coefficients of variation is about 30%.

In the Figure 1.32, the coefficient of variation is related to the number of specimens ex-



FIGURE 1.31: For each building, the coefficient of variation of compressive concrete strength according to year of construction

tracted. Usually, less than 5 samples are extracted on an existing structure; moreover, very few cases in which more than 10 sample cores were extracted.

In this context, it is also more interesting to evaluate the concrete strengths according to

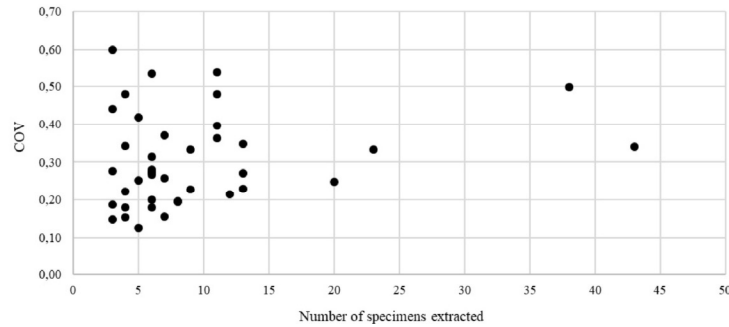


FIGURE 1.32: For each building, the coefficient of variation of compressive concrete strength according to the number of specimen extracted

the building function; Figure 1.33 shows the density probability function of concrete compressive strength according to the building functions. Here, industrial and school buildings show the lowest values: industrial buildings were built at the beginning of 20th century and they were also the first applications of reinforced concrete structures (mean value of concrete strength is almost 13 MPa and $COV \approx 0.45$); indeed, school buildings were built after 1960, but in that time Italian technical code reduced the prescriptions in order to promote economic development.

In the latter case the variability of concrete strength is very high ($COV = 0.57$) with the mean value equal to 15 MPa. Hospitals, public buildings and tall building present the same values of mean values and coefficients of variation; in the latter cases the mean value of concrete strength is almost 20 MPa (with $COV \approx 0.35$).

However, in order to match concrete strength values with technical code development, compressive test results are divided according to the main steps of the Italian technical code. In Figure 1.34, the concrete tests are evaluated according to Technical codes: *R.D.* 1929 [80], *R.D.* 1937 [79] and *Code n.1684*, 1962 [26]. In the first case, the minimum values of

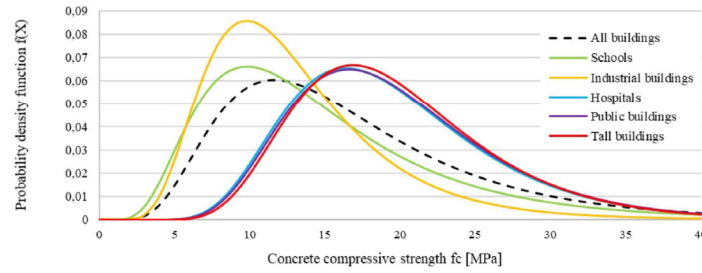


FIGURE 1.33: Probability density function of compressive concrete strength according to building functions

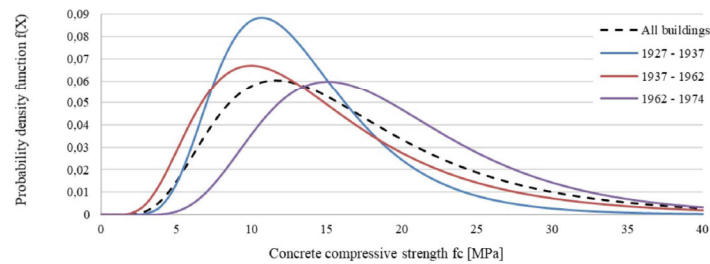


FIGURE 1.34: Probability density function of compressive concrete strength according to Technical code evolution

concrete strength is 15 MPa, but the concrete tests show that mean value is almost 13.5 MPa ($COV \approx 0.40$). After 1939 the minimum value of concrete strength was reduced (e.g. 12 MPa), for these reason many schools built before 1962 presented low values of concrete strength (in this case $COV > 0.52$). Finally, after 1962 concrete tests show that mean value of compressive strength is higher than 19 MPa with $COV = 0.41$.

1.4.3 NTC 2018: Knowledge levels and Confidence factors

The latest Italian Technical code (e.g. *NTC 2018* [27]) assigns more attention to the knowledge of structures: geometry, construction details and material properties are fundamental steps to evaluated the structural capacity of existing constructions.

It is fundamental to know the geometry: all structural and non-structural elements must be measured (i.e. column cross-section, beam depth, length and height of slab, etc.).

Indeed construction details (i.e. steel reinforcement position), may be impossible to know; for this reason, according to the structural safety level to reach, three *knowledge levels* can be identified:

- *Limited survey*: a limited survey is conducted, slab warping is known by design drawing or by simulate design (according to historical information of practice and design code at time);
- *Extended survey*: a extended survey is conducted when design drawings are not available or alternatively in order to perform simulated design;

- *Complete survey*: a complete survey is conducted according to the design drawings;

According to the survey level conducted, it is important to know the following elements: the steel reinforcement on beams, columns and walls; bent rebars on beams for shear stress; steel reinforcement on beam-column joint; upper reinforcement on slab; cross-section cover and anchor length of bars.

Previously as seen, the material property evaluation is another key point for the assessment of existing structures. Starting from the structural safety level to reach, the *knowledge level* can be achieved according to different material tests performed on the structure. Three material testing levels can be obtained:

- *Limited testing*: a limited number of material testing is performed to increase the material information known from the construction design, historical data (from literature) or material testing certificates;
- *Extended testing*: a higher number of material testing than previous category is performed to increase the material information known from the construction design, historical data and material testing certificates or in case *limited* test results are lower than those reported on the design;
- *Complete testing*: a higher number of material testing than previous category is performed to increase the material information known from the construction design, historical data and material testing certificates or when *limited* or *extended* test results are lower than those reported on the design.

In order to identify the material mechanical properties, the following characteristics are evaluated: for concrete, compressive strength f_c and elastic modulus E ; for steel reinforcement, yield and ultimate strength and the deformation.

The *Confidence Factors* are used to reduce the mechanical properties evaluated from the tests (destructive and non-destructive tests). They are applied as dividends to the mean values of the material test results. Numerically, the confidence factors represent the obtained knowledge level; the relationship between *Knowledge Levels* (LC) and *Confidence Factors* (FC) are described in details below:

- *LC1*: it can be obtained when the geometry is complete, limited survey are conducted on the constructive details and *limited testing* are performed on the material properties; in this case *Confidence Factor* is $FC = 1.35$;
- *LC2*: it can be obtained when the geometry is complete, extended survey are conducted on the constructive details and *extended testing* are performed on the material properties; in this case *Confidence Factor* is $FC = 1.20$;

- *LC3*: it can be obtained when the geometry is complete and accurate, complete survey are conducted on the constructive details and *complete testing* are performed on the material properties; in this case *Confidence Factor* is $FC = 1.00$.

On the capacity evaluation of the RC elements, the geometry uncertainties can be reduced through the relief, whereas it is impossible to reduce the material uncertainties (generally more consistent in relation to concrete than to steel). Finally, position, quantity and diameter of the reinforcements are the uncertainties related to the construction details. However, all these uncertainties are attributed only to the materials.

Chapter 2

Structural capacity evaluation of RC existing structures

2.1 Abstract

The structural safety assessment of existing reinforced concrete buildings is an important and actual topic, both in Europe and in Italy. In fact, many of the existing buildings present advanced degradation conditions, due to the unavoidable ageing of materials, especially in structures that were built over 40 years ago. In the modern approach, structural safety is usually defined by a performance (or limit state), beyond which the structure/element does not satisfies the design requirements; moreover, a probability of failure is often associated to this limit condition, which is applied to both actions and resistances. In the case of resistance estimation (e.g. structural capacity), it is necessary to evaluate the main sources of uncertainty: element/structure geometry, material mechanical properties and the structural model used. However, only few sample can be extracted in order to know the variability of material mechanical properties without compromising the element/structure integrity, alternatively non-destructive methods could be extended. For this reason, starting from the well-known approaches for structural safety evaluation (semi-probabilistic and full-probabilistic approach), in this Chapter, four approaches proposed in literature are analysed to estimate the structural capacity of existing reinforced concrete elements. The aim of the work is to compare the design values of the structural capacity, estimated considering the same probability of failure, taking into account the variability of the number of available tests and therefore of the material strengths.

La valutazione della sicurezza strutturale delle costruzioni esistenti è un tema di assoluta importanza e attualità, sia in Europa che in Italia. Infatti, molti degli edifici esistenti presentano una fase di degrado avanzata, dovuta all'inevitabile invecchiamento dei materiali, soprattutto in quelle costruzioni costruite da oltre 40 anni. Nell'approccio moderno,

la sicurezza strutturale è solitamente definita da una performance (o stato limite), oltre la quale la struttura o l'elemento non soddisfa più le esigenze per le quali è stata progettata/o; inoltre, a tale condizione limite viene associata una probabilità di superamento, la quale viene applicata sia alle azioni che alle resistenze. Nel caso della stima della resistenza, ovvero della capacità strutturale, è necessario valutare le principali fonti di incertezza che ne caratterizzano il risultato: la geometria dell'elemento o della struttura, le proprietà meccaniche dei materiali e il modello teorico utilizzato. Tuttavia, per conoscere la variabilità delle proprietà meccaniche dei materiali e non compromettere l'integrità dell'elemento o della struttura, solo pochi campioni possono essere prelevati (in alternativa si potrebbero estendere metodi di indagine non distruttivi, però non sempre affidabili). Per questa ragione, partendo dagli approcci più conosciuti per la valutazione della sicurezza strutturale (semi-probabilistico e puramente probabilistico), in questo Capitolo, quattro dei principali metodi presenti in letteratura sono analizzati per stimare la capacità strutturale di elementi in calcestruzzo armato esistenti. L'obiettivo di questo lavoro è confrontare il valore di progetto della capacità strutturale, stimato considerando la medesima probabilità di fallimento e analizzando la variabilità del numero delle prove a disposizione e quindi la relativa resistenza del materiale.

2.2 Introduction

In many European countries, particularly in Italy, most of reinforced concrete structures were built after the Second World War, during the economic development. In addition, at the beginning of 20th century, a number of reinforced concrete structures (especially industrial buildings, public office buildings and some bridges) were built as prototypes of reinforced concrete construction technique.

It is well known that 50 years after their realization, the concrete of these structures began to show significant deterioration phenomenons, often due to the poor quality of concrete caused by a limited knowledge of concrete mix design and a limited number of quality controls. At the same time, the concrete degradation leads to a greater exposure of steel reinforcement bars which can bring to corrosion. These phenomenons can reduce the structural capacity of the affected elements with resulting safety problems for owners and occupants. Moreover, in Italy the seismic regulations were inadequate at the time, because only a limited portion of territory was classified as seismic. Currently, almost the entire Italian territory is considered subjected to earthquakes and for this reason, most of modern and historical constructions have been designed based on a code which has taken into account only partially for the seismic actions and in some cases not at all. The effects of recent seismic events occurred in Italy (i.e. structural collapse evaluation based on field surveys are investigated by Fiorentino et al. [39] [40], and school building damage are studied by Abruzzese et al. [7], related to Central Italy Earthquake in 2016) are proofs that structural

safety of existing structures is a topic of absolute interest.

Structural safety, and particularly the assessment of real strength in existing structures, can't be solved by technical solutions only, because it affects aspects of different nature: economic, social and environmental. Furthermore, the importance of this topic will certainly increase in the future, given the ageing effect of materials.

Regarding the assessment of structural resistance of existing structures, particular attention must be paid to geometrical problems (e.g. steel reinforcement positions) and mechanical aspects (e.g. evaluation of mechanical properties of concrete). For poor quality and hand-made concrete, as often occurred in the past, this aspect is particularly critical due to the variability of strength. In fact, the main problem of the assessment of existing reinforced concrete structures is the uncertainty evaluation, especially related to material strength, geometry, but also applied to structural capacity.

Uncertainty related to structural capacity (i.e. shear resistance model and corrosion damage) are investigated by Sykora et al. [110], [111] for existing RC structures. For this reason, capacity models can be calibrated according to experimental testing, as evaluated by Monti et al. [70], [71]. On the other hand, the variability of material strength (especially for concrete) is considered fundamental for the assessment of existing buildings: the concrete strength depends on several factors that may not be properly controlled during the construction process. Direct compression tests on cores extracted from a structure provide the most reliable measurement of the strength. Unfortunately, the accepted number of cores is often limited because the method is expensive and invasive. Thus, in-situ concrete test results can be correlated taking into account uncertainty and specific confidence level, as described by Soutos et al. [95], or the concrete compressive strength is updated using prior information, as studied by Caspeele and Taerwe [17]. Otherwise, Giannini et al. [45] combined direct and NDT measurement using bayesian inference; however, concrete strength estimation by indirect measurements depends on the degree of correlation, as highlighted by Pucinotti and Tripodo [86].

The principles for assessment of existing structures are based on probability concepts and applied statistics described by Ang and Tang [9], Montgomery and Rudger [69], Lutes and Sarkani [61]. However, the limited development of European standards, highlighted by Holicky et al. [47], encouraged several authors, as Rucker et al. [90], to compose a guideline for the assessment of existing structures. For the purpose of estimating the uncertainties related to the assessment of existing structures, the most recurring approaches in literature are: semi-probabilistic (see Section 4.2) and full-probabilistic approach (see Section 4.3).

In the first case, an objective verification format for existing structures based on alternative partial factors presented by Caspeele et al. [16], compatible with the current Eurocodes [2], [3] for new structures design is applied. In this framework, alternative values for target reliability level related to remaining working life are proposed by Steenbergen et al. [107] for existing structure evaluation, according to first order reliability method (FORM) based

on reliability analysis. In fact, the direct application of usual partial safety factors for materials strength in such models was not appropriate in order to obtain design formulations coherent with some level of reliability. For this reason, Mancini et al. [62], according to empirical or semi-empirical formulations calibrated empirical coefficients adjusting them to fit a set of experimental data.

However, partial factor methods are more conservative than other approaches, for this reason many probabilistic procedures, in the reliability assessment of structural heritage, are applied by Sykora et al. [109] and Sykora and Holicky [108]. Pereira and Romao [84] used probabilistic approaches to assess material properties (i.e. concrete strength) in existing buildings. Advanced statistical approaches involved high computational effort, as described by Matos et al. [66], but they are more accurate than partial factor methods.

Differently from above, Eurocode 8 [4] approach devotes to assessment and retrofitting, taking into account the uncertainty on existing structures. According to the knowledge of existing structure, it defines an adjustment factor (called *Confidence Factor* (FC)), which depends on the *Level of Knowledge* (LC) of properties such as geometry, construction details and material properties. However, as described by Franchin et al. [42], this solution is feasible from a logical point of view but it cannot yet explained with theoretical and practical bases.

The aim of this work is to evaluate different approaches in literature and to compare them. Firstly, a suitable way to compare partial factor methods with statistical approaches has been studied (FORM-based method is applied in each approach); then, the results of structural capacities are compared to experimental tests. Destructive and non-destructive tests are considered to calculate structural capacity, according to the different number of test results (in real cases, very few destructive tests are performed on the same element).

In other cases, the estimation of the characteristic strength values from limited data, usual situation on existing structure assessment, is considered as a difficult but important task by Zupan et al. [114]. In previous works (e.g. Forte et al. [41] and Lavorato et al. [56]) characteristic values of structural capacity are evaluated using a specific tolerance factor, proposed by Monti and Petrone [72], which is used to attain any given fractile, as a function of the small number of tests, with any assigned confidence level; the latter solution is more flexible and accurate solution than well-known tables presented on *ISO Standard* [97].

Thus, starting from the same aim, that is to compare *Partial Factor Methods* and *Advanced Statistical Methods* on the structural safety, Caspeepele and Taerwe [18] estimated the safety level of existing elements from limited data, but without comparing the results with experimental tests.

In the following sections, structural safety overview is presented according to target failure probabilities. All basic variables of structural capacity are examined, considering material properties as dominant variable.

However, this work does not pretend to provide a solution to the problem of structural

safety assigning the best method, because further steps are necessary in this direction: material properties are only considered as basic variables on structural capacities, and geometry and model uncertainties could be also considered. Finally, these comparisons could be extended on entire case studies, for example existing reinforced concrete buildings or bridges.

2.3 Structural safety overview

Many sources of uncertainty are intrinsic in the structural design of new structures and the evaluation of existing ones. Despite what we often think, the parameters of the loading and the load-carrying capacities of structural members are not deterministic quantities, but they are random variables, and thus absolute safety (or zero probability of failure) cannot be reached.

However, society expects building and bridges to be designed or assessed with a reasonable safety level. In practice, these expectations are accomplished by following code requirements specifying design values for minimum strength, maximum load, and so on. Code requirements include design criteria which take into account some of the sources of uncertainty in design/assessment; such criteria are often referred to as *reliability-based design criteria*.

It is noted that the reliability of a structure is its ability to fulfill its design purpose for some specified design lifetime. Reliability is usually considered as the probability that a structure will not fail to perform its intended function. However, the term *failure* does not necessarily mean catastrophic failure but is used to indicate that the structure does not perform as desired.

Moreover, structural reliability concepts can be applied both to the design of new structures and the assessment of existing structures. Several design codes, European and American, is based on probabilistic models of loads and resistances, as described by Nowak and Collins [76]. The reliability of a structure can be considered as a rational evaluation criterion, because it provides a good basis for decisions about repair, rehabilitation or replacement. A structure can be condemned when the nominal value of load exceeds the nominal load-carrying capacity, but in most cases a structure is a system of components and failure of one component does not necessary mean failure of structural system. When a component reaches its ultimate capacity, it may continue to resist the load while loads are redistributed to other components. Therefore, system reliability gives a methodology to establish the relationship between the reliability of an element and the reliability of the system.

In this context, there are some questions which it is needed to answer:

- *How can we measure the safety of structures?* Safety can be measured in terms of reliability or the probability of uninterrupted operation; the complement to reliability is the probability of failure;
- *How safe is safe enough?* It is impossible to have an absolutely safe structure, because every structure has a certain non zero probability of failure. Conceptually, we can design/assess the structure to reduce the probability of failure, but increasing the safety (or reducing the probability of failure) beyond a certain optimum level is not always economical (the optimum safety level has to be determined);
- *How does a designer implement the optimum safety level?* Once the optimum safety level is determined, appropriate design provisions must be established so that structures will be designed/assessed accordingly.

In the structural reliability analysis, two terms are very important: *safety* and *reliability*. Underlining the concept of *limit state* which is the boundary between desired and undesired performance of structure is useful in order to define safety and reliability. This boundary is often represented mathematically by a *limit state function* or *performance function*. In the bridge structures, for example, failure could be defined as the inability to carry traffic. However, this undesired performance can occur in many modes of failure: cracking, corrosion, excessive deformations, exceeding load-carrying capacity for shear or bending moment, local or overall buckling.

As described by Nowak and Collins [76], in structural reliability analyses three types of limit states are considered:

- Ultimate limit state (ULS). Examples of modes of failure are: exceeding the moment carrying capacity, formation of a plastic hinge, crushing of concrete in compression, loss of the overall stability, etc.
- Serviceability limit state (SLS). Examples of modes of failure include: excess deflection, excess vibration, permanent deformations and cracking.
- Fatigue limit state (FLS).

A performance function or limit state function, can be defined analytically for this mode of failure as:

$$g = R - E \quad (2.1)$$

The limit state, corresponding to the boundary between desired and undesired performance is when $g = 0$. If $g \geq 0$, the structure is safe (desired performance); if $g \leq 0$, the structure is not safe (undesired performance). The probability of failure, P_f , is equal to the probability that the undesired performance will occur. Mathematically, this can be expressed in terms

of the performance function as:

$$P_f = P(R - E \geq 0) = P(g \geq 0) \quad (2.2)$$

If both R and E are continuous random variables, then each variable has a probability density function (PDF) such as shown in Figure 2.1. Furthermore, the quantity $R - E$ is also a random variable with its own PDF and the probability of failure corresponds to the shaded area in Figure 2.1.

If the state variables are the basic load and resistance parameters used to formulate the

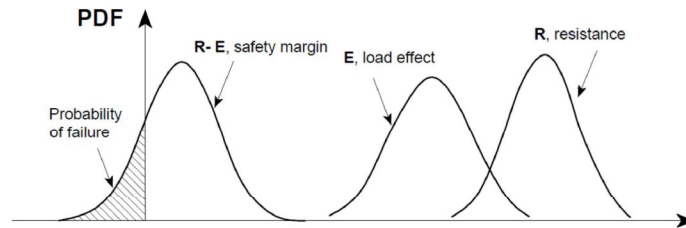


FIGURE 2.1: PDFs of load, resistance and safety margin

performance function, for n state variables, the limit state function is a function of a n parameters. In addition, if all loads are represented by the variable E and total resistance by R , then the space of state variables is a two-dimensional space as shown in Figure 2.2. Within this space, we can separate the *safe domain* from the *failure domain*; the boundary

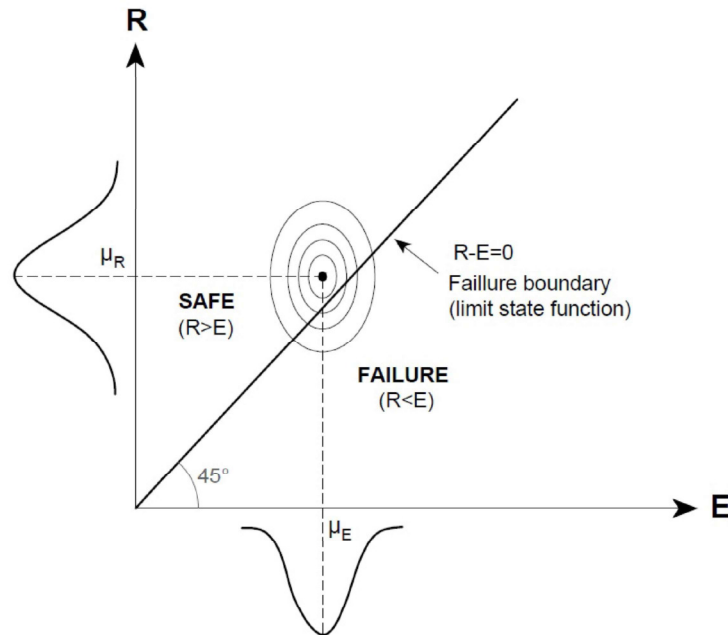


FIGURE 2.2: Safe domain and failure domain in a two-dimensional state space

between the two domains is described by the limit state function $g(R, E) = 0$. Since both

R and E are random variables, we can define a joint density function $f_{RE}(r, e)$ which is plotted in Figure 2.3. Also here, the limit state function separates the safe and failure

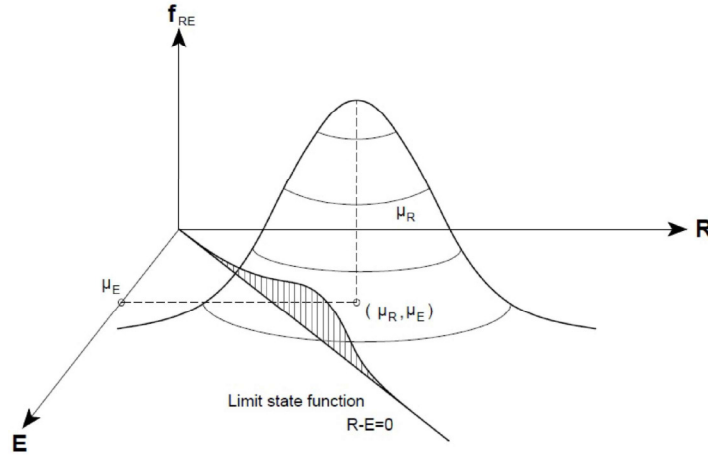


FIGURE 2.3: Three-dimensional representation of a possible joint density function f_{RE}

domains. Thus, the probability of failure is calculated by integration of the joint density function over the failure domain (i.e. the region on which $g(R, E) < 0$). However, this probability is often very difficult to evaluate, so the concept of a reliability index is used to quantify structural reliability.

Several methods were developed to evaluate the reliability index but all of these convert all random variables to their *standard form* which is a non-dimensional form of variables (these variables are sometimes called *reduced variables*). A version of the reliability index is the inverse of the coefficient of variation, as described in the Equation 2.7 and in this case is defined as the shortest distance from the origin of reduced variables to the limit state function g (as shown in Figure 2.4): using geometry, it can calculate the reliability index (*shortest distance*) from the following equation:

$$\beta = \frac{\mu_R - \mu_E}{\sqrt{\sigma_R^2 - \sigma_E^2}} = \frac{\mu_g}{\sigma_g} \quad (2.3)$$

Assuming that g is Normally distributed, it is possible to describe:

μ_g is the mean value of g ;

σ_g is the standard deviation of g .

Obviously, the probability of failure P_f can be expressed through a performance function g such that a structure is considered to survive if $g > 0$ and to fail if $g \leq 0$:

$$P_f = Prob(g \leq 0) \quad (2.4)$$

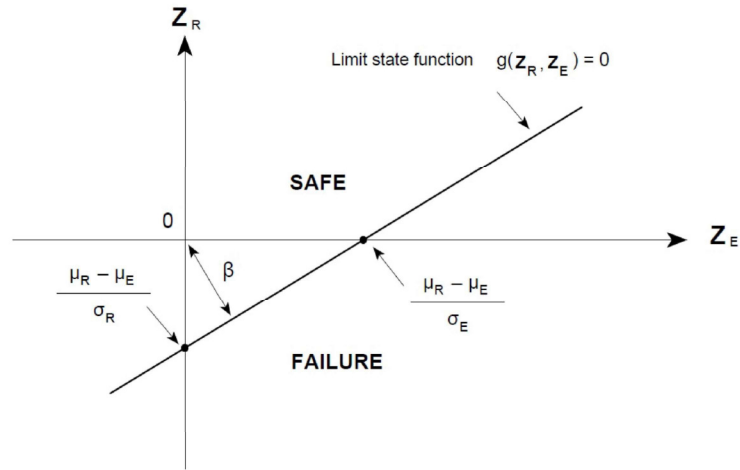


FIGURE 2.4: Reliability index defined as the shortest distance in the space of reduced variables

with R , E and g random variables. From 2.1 and 2.7:

$$\mu_g - \beta\sigma_g = 0 \quad (2.5)$$

and

$$P_f = \text{Prob}(g \leq 0) = \text{Prob}(\mu_g - \beta\sigma_g) \quad (2.6)$$

For other distribution of g , β is only a conventional measure of reliability $P_s = (1 - P_f)$. However, measure of reliability is conventionally defined by the reliability index β which is related to P_f by:

$$P_f = \phi(-\beta) \quad (2.7)$$

where:

ϕ is a cumulative distribution function of the standardised Normal distribution.

Moreover, the relation between β and P_f is given in Table 2.1.

Considering a linear limit state function, the reliability index β depends only on mean

P_f	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}
β	1.28	2.32	3.09	3.72	4.27	4.75	5.20

TABLE 2.1: Relation between β and P_f , according to EN1990:2002 [2]

and standard deviations of the variables. In this case, β is called a *second-moment* measure of structural safety because only first two moments (mean and variance) are required to calculate β . There is no relationship between β and the type of probability distributions

of random variables, but if the random variables are all normally distributed and uncorrelated, the formula is *exact* in the sense that β and P_f are related by the Equation 2.7. Otherwise, Equation 2.7 provides only an approximate mean of relating β to a probability of failure.

Considering the case of nonlinear limit state function, it is needed to approximate it by linearizing the nonlinear function using Taylor series expansion. One choice for this linearization point is the point corresponding to the mean values of the random variables.

In this latter case, the reliability index is called a *first-order second-moment mean value reliability index*:

- *First order* because it uses first-order terms in the Taylor series expansion;
- *Second moment* because only mean and variance are needed;
- *Mean value* because the Taylor series expansion is about the mean values.

However, the first-order second moment mean value method, based on approximating non-normal CDFs of the state variables by normal variables, has both advantages and disadvantages in structural reliability analysis. The main advantages are: it is easy to use and it does not require knowledge of the distributions of random variables. Moreover, among its disadvantages: results are inaccurate if the tails of the distribution functions cannot be approximated by a normal distribution, there is an invariance problem (e.g. the value of the reliability index depends on the specific form of the limit state function).

Several works in literature proposed a modified reliability index that did not exhibit the invariance problem related to evaluate the limit state function at a point known as the *design point* instead of the mean values (the design point is a point on the failure surface $g = 0$). Since this design point is generally not known a priori, an iteration technique must be used to solve for the reliability index.

According to different order of approximation, several methods in literature evaluate the design point:

- *First Order Reliability Method* (FORM)
- *Second Order Reliability Method* (SORM)

In this work, *First Order Reliability Method* (FORM) is presented in 2.3.1.1 and then it is applied to both Partial Factor Methods and Advanced Statistical Methods.

2.3.1 Target failure probabilities

The structural assessment/design process can be seen as an optimization problem about the expected value of benefits or minimization of costs subject to constraints. In the Figure 2.5, the Life Quality Index (LQI), with regard to by Nathwani et al. [74], starts from the

concept that the decisions of a society with regard to investments into health and life safety improvements can be described in terms of the life expectancy at birth, the Gross Domestic Product (GDP) per capita and the ratio between working time and leisure time.

The LQI facilitates the assessment of decisions with regard to their conformity with societal

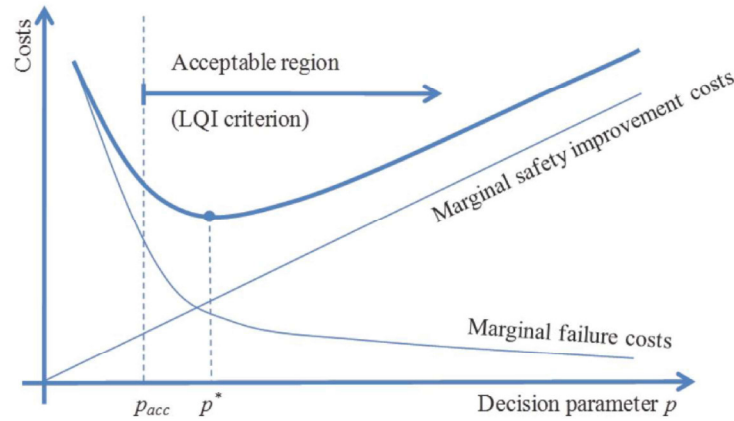


FIGURE 2.5: LQI acceptance criterion as a boundary condition for monetary optimization, according to ISO2394:2015 [99]

preferences in regard to life safety investments. Based on LQI, it is possible to derive the so-called limiting value of lifesaving costs, i.e. the necessary and affordable spending which should be invest into saving on additional life. In addition, the marginal lifesaving cost as assessed through the LQI depends on the economic capacity and decisions to invest into life safety for a given society, as well as on the effect of the life-saving measured over the population.

For structures which do not take risks to life, the decision basis for design and assessment can be taken from a perspective of pure economic optimization. In practice, such situations are very rare if at all nonexistent; however, all structures impose some life safety risks during some part of their service life.

However, the *Probabilistic Model Code* [81] provides tentative target reliabilities for different structural classes, based on economic optimization. The target reliabilities are given as a function of the costs of the risk reduction measure and the consequences in case of failure, both defined relative to the initial construction costs of the structure at hand. The target reliabilities, related to structural failure events, can be used for failure events ranging from component failures, over partial to full structural collapse by adjustment of the failure consequences. The optimal reliability of structural systems can be related to the safety costs and failure consequences generically as provided in Table 2.2 (in the following table, the main classes of consequences of failure are presented):

where:

Consequences of failure

Relative cost of safety measure	Class 2	Class 3	Class 4
Large	$\beta = 3.1(P_f \approx 10^{-3})$	$\beta = 3.3(P_f \approx 5 * 10^{-4})$	$\beta = 3.7(P_f \approx 10^{-4})$
Medium	$\beta = 3.7(P_f \approx 10^{-4})$	$\beta = 4.2(P_f \approx 10^{-5})$	$\beta = 4.4(P_f \approx 5 * 10^{-6})$
Small	$\beta = 4.2(P_f \approx 10^{-5})$	$\beta = 4.4(P_f \approx 5 * 10^{-6})$	$\beta = 4.7(P_f \approx 10^{-6})$

TABLE 2.2: Tentative target reliabilities related to ultimate limit states, based on monetary optimization and according to Model Code [81]

Class 2 indicates material damages and functionality losses of significance for owners and operators but with small or no societal impact. Expected number of fatalities fewer than 5 (e.g. smaller buildings and industrial facilities, minor bridges, etc.);

Class 3 indicates material losses and functionality losses of societal significance, causing regional disruptions and delays in important societal services over several weeks. Expected number of fatalities fewer than 50 (e.g. most residential buildings, typical bridges and tunnel);

Class 4 indicates disastrous events causing severe losses of societal services and disruptions and delays at national scale over periods in order of months. Expected number of facilities fewer than 500 (e.g. high-rise buildings, grandstands, major bridges and tunnels, etc.).

The target reliabilities given in Table 2.2 should be seen as indicative for the support of economic optimization and might not be acceptable for what concerns life safety risks, as described by ISO2394:2015 [99].

In order to complete the previous list, it is also important to describe the classes which represent less common structures:

Class 1 indicates predominantly insignificant material damages (e.g. low-rise buildings where only few people are present);

Class 5 indicates catastrophic events causing losses of societal services and disruptions and delays beyond national scale over periods in the order of years. Expected number of fatalities larger than 500 (e.g. buildings of national significance, major containments and storages of toxic materials, major dams, etc.).

On the structural capacity evaluation, generally, the structure should be adequately modelled and the limit state functions should be clearly defined in order to determine the reliability requirements. The target reliability level used for verification of an existing structure can be determined based on calibration with the current code, the concept of the minimum total expected cost and the comparison of other social risks.

In principle, the performance requirements of safety and serviceability for the assessment of existing structures are the same as for the design of new structures. However, ISO13822:2010 [98] indicates some fundamental differences between these two activities affecting the differentiation in structural reliability:

- Economic considerations: the incremental cost between acceptance and upgrading the existing structure can be very large, whereas the incremental cost of increasing the safety of structural design is generally very small, consequently conservative generic criteria are used in design standards;
- Social considerations: these include disruption or even displacement of occupants and activities, also heritage values, considerations that do not affect the structural design of new structures;
- Sustainability considerations: reduction of waste and recycling, considerations of less importance in the design of new structures.

As a consequence the goal of *minimum structural intervention*, which aims to re-use the existing materials in the structures as much as it is possible, applies for most existing structures of normal occupancy and use.

It is noted that the remaining working life determined at the assessment is considered as a reference period for an existing structure for serviceability while design working life is often considered as a reference period for a new structures.

For this reason, a framework is needed considering reliability differentiation for existing structures, as well as an associated target probabilities failure. However, the intended reference period (and also working life) of these different types of structures is different (see Figure 2.6). Caspee et al. [16] proposed a coherent design format, compatible with the current standards for new structures, and also applicable to existing and temporary structures.

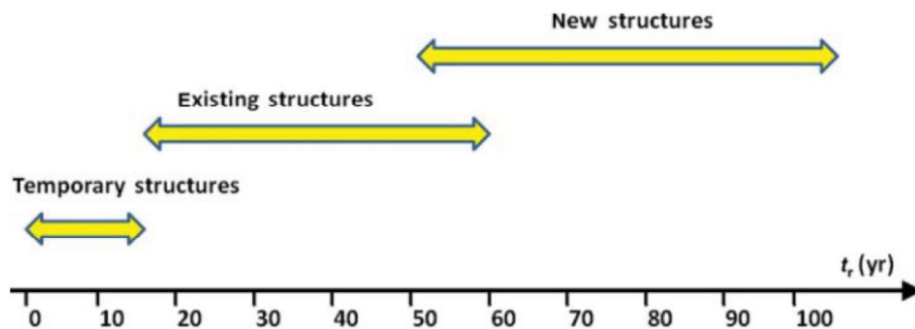


FIGURE 2.6: Shifted time frame for structural design, described by Caspee et al. [16]

2.3.1.1 FORM-based method

There are many methods to evaluate the design point on surface $g = 0$. In this work, First Order Reliability Method (FORM) is presented both to calibrate the partial factors and also to evaluate a design point of structural capacity using full probabilistic methods.

Therefore, in the design value method of reliability verification, design values need to be defined for all the basic variables. A design is considered to be sufficient if the limit states are not reached when the design values are introduced into the analysis models (in symbolic notation this is expressed as $E_d < R_d$).

The Figure 2.7 shows design values based on the values of the basic variables at the FORM design point, which can be defined as the closest point on the failure surface ($g = 0$) closest to the average point in the space of normalised variables.

The design values of load E_d and resistance R_d should be defined such that the probability

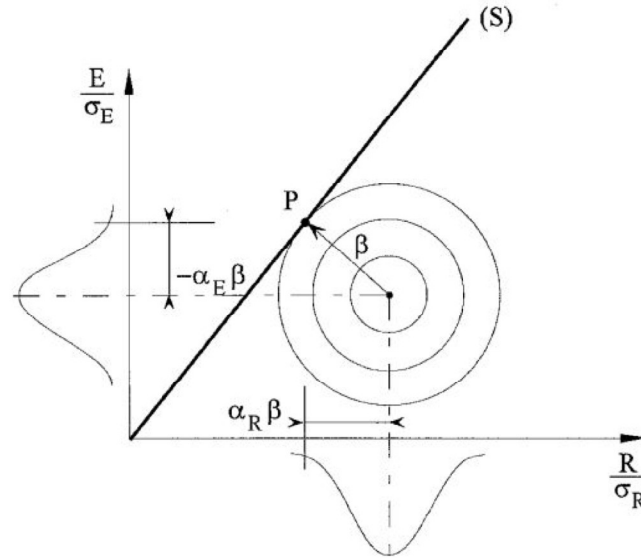


FIGURE 2.7: Design point and reliability index β according to the First Order Reliability Method (FORM) for Normally distributed variables, according to EN1990:2002 [2]

of having a more unfavourable value is as follows:

$$P(E > E_d) = \phi(+\alpha_E \beta) \quad (2.8)$$

$$P(R \leq R_d) = \phi(-\alpha_R \beta) \quad (2.9)$$

where:

β is the target reliability index;

α_E α_R are the values of the FORM sensitivity factors. The value of α is negative for unfavourable actions or loads, and positive for resistances.

In developing new codes or revising existing ones, calibration is generally required. Often, calibration refers to the determination of the values of all the parameters in a given code format and, in this section, the presented approach takes basis in the principles of reliability of structures.

It is important to note that strength or resistance variables are often modelled by Lognormal distributions in order to avoid the possibility of negative realizations. Moreover, the coefficient of variation varies with the material type considered, for example typical values are 5% for steel and reinforcement, 15% for concrete compression strength. Generally, the characteristic value is chosen as the 5% quantile.

In many cases, as described by ISO2394:2015 [99], model uncertainties are modelled by Lognormal distributions if they are introduced as multiplicative random variables, whereas they are modelled by Normal distributions if these are modelled by additive random variables. Typical values for the coefficient of variation varies from 3% to 15% but should be chosen very carefully.

In the last period, structural design codes are established for the purpose of providing a simple, safe and economically efficient basis for the design of ordinary structures under normal loading, operational and environmental conditions. In this context, code calibration can be seen as a decision optimization problem where the expected societal net benefits are maximized by optimizing the parameters in the design code.

Usually, FORM-based method is used to calibrate design values in design-value methods, which design value x_{id} of variable X_i depends on the parameters of the probability distribution of the variable X_i , the assumed type of distribution, the target reliability index β for the limit state and design situation of concern and a factor α_i describing the sensitivity to variations in X_i with regard to attaining the limit state, according to the definition given by FORM calculation.

For X_i with an arbitrary distribution $F(x_i)$, the design values are given by:

$$F(x_i) = \phi(-\alpha_i\beta) \quad (2.10)$$

If X_i is assumed to be normally distributed, then:

$$x_{id} = \mu_i(1 - \alpha_i\beta V_i) \quad (2.11)$$

where:

μ_i is the mean value of the random variable X_i ;

V_i is the coefficient of variation of the random variable X_i ;

Whereas, if X_i is assumed to be lognormally distributed:

$$x_{id} = \frac{\mu_i}{\sqrt{1 + V_i^2}} \exp\left(-\alpha_i \beta \sqrt{\ln(1 + V_i^2)}\right) \quad (2.12)$$

As proposed by ISO2394:2015 [99], for a small value of V_i (e.g. $V_i \leq 0.25$), the equation 2.12 can be approximated as following:

$$x_{id} = \mu_i \exp(-\alpha_i \beta V_i) \quad (2.13)$$

Moreover, if the random variables are independent, the factors α_i in a FORM analysis have the following properties:

$$-1 \leq \alpha_i \leq 1 \quad (2.14)$$

$$\sum \alpha_i^2 = 1 \quad (2.15)$$

The values of α_i should be found from a number of representative FORM calculations, through many iterative computations which are very inconvenient. However, based on experience, a set of standardized α_i values has been developed (see Table 2.3):

In order to use the Table limiting errors, it is usually required that $0.16 < \sigma_E/\sigma_R < 6.6$,

X_i	α_i
Dominating resistance parameter	0.8
Other resistance parameters	$0.4 * 0.8 = 0.32$
Dominating load parameter	-0.7
Other load parameters	$-0.4 * 0.7 = -0.28$

TABLE 2.3: Sensitivity factors, according to ISO2394:2015 [99]

where σ represents the standard deviation and E is the dominating load and R is the dominating resistance parameter. Applying the sensitivity factor in Table 2.3, one does not know in advance which variable should be regarded as *dominating*; however, the only way to find this out is by making all variables *dominating* one at the time and see which one governs the design.

Semi-probabilistic methods take place on the basis of a direct check of the relevant design situations and corresponding design equations using design values for the basic variable which are determined on the basis of reliability assessments.

2.4 Structural Capacity

The *Structural Capacity* is the analytic representation of a *structural performance* which is described through several parameters and it can be referred to different *Limit States*.

Usually, models, geometry and material properties are the parameters that describe the structural performance, and they are considered as sources of uncertainty.

For material models, it is important to distinguish linear and non-linear relations both in concrete behaviour and steel one. In linear cases, material models consisting of relations between stresses (or forces) and deformations and the variables in such relations are: the modulus of elasticity and maximum strength for concrete and the modulus of elasticity, the yield limit and the ultimate strength, for steel reinforcement. In non-linear models, the constitutive relationship may be a function of the combined or individual stress, strain or strain rate and may also be path dependent with regard to the load history. However, models should be set up to establish the relation between the various parameters and all of these can be considered as uncertain quantities.

The geometry is a fundamental information to evaluate the structural capacity of existing structure or element and it should be considered adequately; in order to draw attention to irregular design or different situations from design documents, an accurate inspection of all parts of structure has to be conducted. Generally, the geometrical quantities which are included in the model refer to nominal values, i.e. the values given in drawings, descriptions, etc. However, the geometrical quantities of a real structure (especially for existing structures) differ from their nominal values, i.e. the existing RC structure has geometrical imperfections both in concrete section size and reinforcement position. If the overall structural behaviour or element capacity is sensitive to such imperfections, these should be included in the model. For this reason also geometry is considered as variable on the structural capacity evaluation.

In addition, material properties should be defined as the properties of material specimens of defined size and conditioning (sampled according to given rules), subjected to an agreed testing procedure. Material properties vary randomly in space and time, thus structural materials should be modelled by functions and parameters (e.g. material properties) and corresponding uncertainties taken into account.

Therefore, all uncertainties described before establish the structural performance variables and they must be processed on structural capacity evaluation through probabilistic methods.

As described in 2.3, structural performance is referred to the limit states which concept is very helpful in assessment of the performance of structure, but in some cases, the desirable performance can depend on the stakeholders like founders, owners, residents, users, neighbours and contractors. Moreover, when public structure is considered, the stakeholders are the government and the society which interests are mainly safety and sustainability.

Thus, for each specific limit state model, which describes the behaviour/performance of a structure, a limit state function should be established and the relevant basic variables should be identified. The limit state function is denoted as $g(X)$, where $X = (X_1, X_2, \dots)$

are the random basic variables, as the following equation:

$$g(X) = 0 \quad (2.16)$$

Basic variables should be defined by all necessary random input information for the model such as: physical quantities (i.e. material and soil properties, geometrical dimensions, etc.), model parameters and input values that describe requirements related to the performance of structural system. However, basic variables can be represented by one or more sources of uncertainty, as for instance statistical uncertainties, measurement uncertainties, natural variability and model uncertainties. The geometry of the structure and the material properties are often referred to as aleatory uncertainties (i.e. random events), whereas uncertainties arising from insufficient information, for example, due to a small number of material tests or idealized models are often referred to as epistemic uncertainty.

All uncertainties that are considered to be of relevant importance for the reliability assessment of structural system should be considered in the analysis using the theory of probability. In the structural reliability analysis, no distinction should be made between the treatment of aleatory and epistemic uncertainties.

In order to assess the structural performance of a element or structure, the possible structural responses shall be assessed and divided into two domains consisting of safety and unsafety states. Thus, according to these basic principles, design equations should be formulated taking basis in the modelling of structural performance in terms of limit state functions and the probabilistic modelling of uncertainties. The principal form of design equations is given in 2.17, which is easy to apply for all relevant failure modes of the structures. In this case, the values of structural parameters, such cross-sectional properties z , can be determined uniquely; moreover, these latter should be conformed with given requirements to risk and reliability (in the equation 2.17, safety state is indicated):

$$G(z) = R_d(z) - E_d(z) > 0 \quad (2.17)$$

where:

z is a vector of design parameters;

$R_d(z)$ is the design value for the resistance;

$S_d(z)$ is the design value for the action;

The resistance is assumed to be obtained by the following general model (see 2.4.1):

$$R = \frac{\theta}{b} R(X, a) \quad (2.18)$$

where:

$R(X, a)$ is a resistance model as defined in relevant materials standard;

X are the material properties;

a are the geometrical parameters (the design parameters z are generally a subset of a);

θ is the model uncertainty related to resistance model (it can be evaluated using the method described in 2.4.1);

b is the bias in resistance model.

In the design equations, the values for the actions and materials characteristics should be determined evaluating the uncertainties associated with loads and resistances which are relevant for the given design situation.

In this work, the study is focused on resistance parameters which properties of materials are defined for some relevant volumes of material and they are represented by their characteristic values X_k . For the new structures, the characteristic values should in principle be presented as a priori specified quantile of the statistical distribution of material property (usually they are produced with relevant standard material). Instead, for existing structures, the estimated values of materials represents the actual part of existing structure and they can be considered in design. However, it must be considered that material properties used in nonlinear analyses can either be based on design values, characteristic values, or mean values. It provides a consistent safety concept is used that results in a design with the required target reliability (the latter is presented in 2.3.1).

In this context, in order to evaluated the design value of resistances, the partial factor format is utilized as basis for the definition of semi-probabilistic safety formats.

In the partial factor method, the design value of the resistance R_d can be determined by different cases:

- **Case 1**, where a design value of the resistance is determined using values of the material resistance parameters:

$$R_d = \frac{R(X_d, a_d)}{\gamma_{Rd}} \quad (2.19)$$

where:

a_d are design values for the geometry;

X_d are design values for resistance parameters;

γ_{Rd} is a partial factor related to the model uncertainty for the resistance model, geometrical uncertainties and bias in the resistance model.

Design values for material resistance parameters are determined as:

$$X_d = \eta \frac{X_k}{\gamma_m} \quad (2.20)$$

where:

η is the conversion factor that considers the uncertainty related to the transformation from laboratory to real structure (i.e. load duration effects, temperature, scale effects, ect.);

X_k is the characteristic value of the resistance parameter generally defined by the 5% quantile;

γ_m is the partial factor for material property.

If more than one resistance parameter is used in resistance model, then design values are applied for each resistance parameter in 2.19.

The partial factor γ_m depends only on the uncertainty of the material properties and γ_{Rd} depends on the uncertainty of the resistance model and geometrical uncertainties, including b :

$$\gamma_{Rd} = \gamma_{Rd1} * \gamma_{Rd2} \quad (2.21)$$

and

$$\gamma_{Rd1} = \frac{b}{\theta} \quad (2.22)$$

where:

γ_{Rd1} is a partial factor that considers the model uncertainty;

γ_{Rd2} is a partial factor that considers the geometrical uncertainty;

θ is a parameter depending on the model uncertainty (it will be described in the subsection 2.4.1).

- **Case 2**, where a characteristic value of the resistance is obtained using values of the material resistance parameters:

$$R_d = \frac{R(\eta X_k, a_k)}{\gamma_M} \quad (2.23)$$

where:

γ_M is a partial factor related to uncertainty of the resistance parameters X , model and geometrical uncertainties.

In the resistance equation, the uncertainty depends on the model uncertainty θ , the uncertainty related to the resistance parameters X and the uncertainties of geometry. For practical reason, all uncertainties can be considered in the material partial factor which are correspondingly obtained from:

$$\gamma_M = \gamma_m * \gamma_{Rd1} * \gamma_{Rd2} \quad (2.24)$$

where:

γ_m is the partial factor for material property;

γ_{Rd1} is a partial factor that considers the model uncertainty;

γ_{Rd2} is a partial factor that considers the geometrical uncertainty.

- **Case 3**, where a design value of the resistance is determined using a characteristic value of the estimated resistance based on tests:

$$R_d = \frac{R_k}{\gamma_M} \quad (2.25)$$

where:

R_k is the characteristic value for the estimated resistance based on tests (R_k is generally defined by the 5% quantile);

γ_M is the partial factor related to the uncertainty of the obtained resistance based on tests including statistical uncertainty.

The material partial factor γ_m , γ_{Rd1} and γ_{Rd2} should be calibrated such that failure probabilities for relevant failure modes are close to the *target reliability level* (described in 2.3.1.1).

Whereas the representative values in the design equations are determined in terms of quantile values which are selected according to convention, the partial factors values can be determined by means of calibration. The partial factors and the representative values should be determined such that they take into account both the aleatory and epistemic uncertainties of relevance for the considered design situation and failure modes. Typically, representative values for variables of importance for action effects are selected as upper quantile values whereas representative values of importance for resistances are selected as lower quantile values.

As presented above, the design value of resistance R_d can be determined in three different cases; particularly in the equations 2.23 and 2.25, the partial factor γ_M depending on material property, model and geometrical uncertainties. Thus, starting from 2.24, this approximation is always possible on linear analysis cases but for non-linear analysis it may lead to unsafe conditions. On the left side of the Figure 2.8, the under proportional behaviour could lead to overestimate the resistance with dangerous structural assessment; whereas on the right side the over proportional behaviour could lead to *capacity design* problems considering underestimated resistance (especially on the beams analysis model).

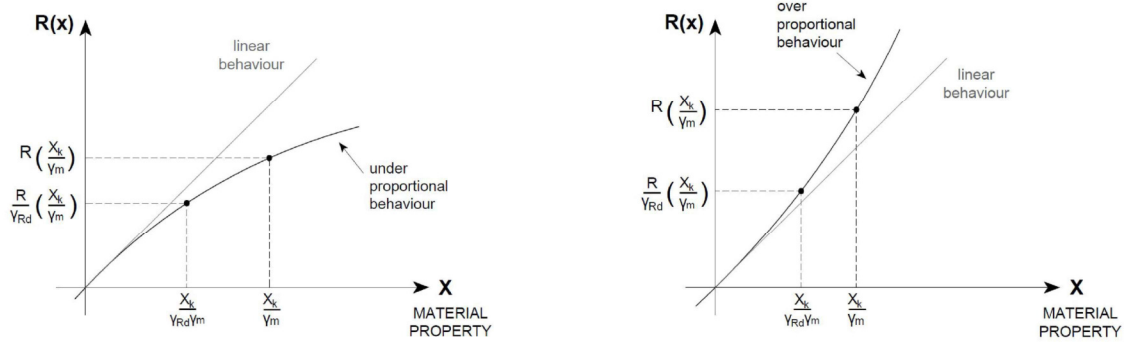


FIGURE 2.8: Non-linear Analysis: (a) Under-proportional behaviour, left side; (b) Over-proportional behaviour, right side

2.4.1 Model uncertainties

Structural capacity analysis depends on a non-negligible way to model uncertainties. In fact in semi-probabilistic safety formats, the partial factors take into account also the uncertainties that refer to the resisting model and, as described by fib Bulletin 80 [37], it is very important in the verification of existing structures. Usually, in the model analysis for the structural property, model is considered as complete except for an unknown coefficient θ , which is determined from the tests. This model can be written as:

$$Y = \theta g(X, W) \quad (2.26)$$

where:

X is the vector of random basic variables;

W is the set of measurable deterministic variables;

$g(X, W)$ is the model;

Y is the measurable output parameter of the model;

θ is the unknown coefficient, to be determined by the experiment.

The parameter θ is referred to the model uncertainty, and assuming that θ has a normal distribution and a series of experiment $I = 1, 2, \dots, n$ is carried out, where:

- values of W have been set to w_i , $i = 1, 2, \dots, n$;
- values of X have been measured as x_i , $i = 1, 2, \dots, n$;
- values of Y have been measured as y_i , $i = 1, 2, \dots, n$;

From these results, one can derive the following set of observations for the unknown coefficient θ :

$$\theta_i = \frac{y_i}{g(x_i, w_i)} \quad (2.27)$$

The mean and standard deviation of θ are computed as follows:

$$m[\theta] = \frac{1}{n} \sum_{i=1}^n \theta_i \quad (2.28)$$

$$s[\theta]^2 = \frac{1}{n-1} \sum_{i=1}^n (\theta_i - m[\theta])^2 \quad (2.29)$$

Assuming the FORM-based method, presented in 2.3.1.1, where $\beta_R = \alpha_R \beta$ with α_R and β equal to the design value for the FORM influence coefficient and the target reliability index, respectively; the design value of θ is:

$$\theta_d = m[\theta] - \alpha_R \beta V[\theta] \quad (2.30)$$

where:

$V[\theta]$ is the coefficient of variation of θ (i.e. $V[\theta] = m[\theta]/s[\theta]$).

From FORM-based method, one should use $\alpha_R = 0.8$ if uncertainty of R is dominating and $\alpha_R = 0.8 * 0.4 = 0.32$ otherwise; usually, material property uncertainties are dominating and model uncertainties are non-dominating, according to König and Hosser [53].

The design resistance R_d of the structural element designed by tests are calculated as follows:

$$R_d = \frac{1}{\gamma_{Rd}} \eta_d g(x_d, w) \quad (2.31)$$

where:

γ_{Rd} is described in 2.21 uncert mod1 and $\gamma_R = 1/\theta_d$ is the design value of the model uncertainty;

η_d is the the design value of conversion factor described in 2.20.

2.5 Semi-probabilistic and Full-probabilistic methods: structural capacity evaluation

The semi-probabilistic method, usually related to limit states, is based on the use of partial safety factors. This method is indicated as semi-probable, because it allows to carry out a verification which is related to probabilistic value but it is carried out following the methodology used in deterministic methods.

In this method, both E (loads) and R (resistance) have to be considered as random variables with an associated probability density. Among these distributions, the obtained characteristic values are replaced by the design values: the coefficients γ_E amplify E and the coefficients γ_M reduce R.

The coefficients γ_E and γ_M are called *partial safety factors* and take into account all the aleatory and uncertainties that are not taken from the characteristic values (e.g. model and geometry uncertainties), and they are calibrated by code according to the limit states and the type of used material.

Full-Probabilistic method is a discipline within engineering design and assessment. It deals primarily with the consideration of the effects of random variability upon the performance of an engineering system during the design phase and also during the assessment one. Typically, these effects are related to quality and reliability. Thus, full-probabilistic approach is a tool that is mostly used in areas that are concerned with quality and reliability. It differs from the classical approach to design/assessment by assuming a small probability of failure instead of using the safety factor.

2.5.1 Bayesian procedure

In structural reliability analysis, the Bayesian interpretation of probability should be considered as the most adequate basis for the consistent representation of uncertainties, independent of their sources. It facilitates the joint consideration of purely subjectively assessed uncertainties, analytically assessed uncertainties and evidence as obtained through observations.

Thus, in full-probabilistic processing, the first step is the establishment of a so-called prior distribution parameters of the resistance R . Such a distribution should reflect all the available prior information about these parameters. Given this prior distribution and given the statistical test data, a posterior distribution can be derived from:

$$f''(q) = C * L(\bar{x}|q) f'(q) \quad (2.32)$$

where:

$f''(q)$ is the posterior distribution of q ;

$f'(q)$ is the prior distribution of q ;

$L(\bar{x}|q)$ is the likelihood function for the parameters q given the observation $\bar{x}$;

q is the vector of distribution parameters (e.g. mean and standard deviation);

C is the normalizing constant.

Then, the updated distribution of R itself, given the prior information and the test data, is obtained by:

$$f''_R(r) = \int f_R(R|q) f''(q) dq \quad (2.33)$$

where:

$f_R(R|q)$ is the distribution of R for given values of q ;

$f_R''(r)$ is the updated distribution of R .

This distribution for R can directly be used in a probabilistic design procedure and it is also possible to derive design values on the basis of equation 2.33. The case that has to be considered further is when R has a normal distribution with mean μ and standard deviation σ . Thus, the prior distribution is given by:

$$f''(\mu, \sigma) = k\sigma^{-(\nu' + \delta(n') + 1)} \exp\left(-\frac{1}{2\sigma^2}(\nu'(s')^2 + n'(\mu' - m')^2)\right) \quad (2.34)$$

where:

$$\delta(n') = 0 \text{ for } n' = 0;$$

$$\delta(n') = 1 \text{ for } n' > 0.$$

The prior distribution, expressed in 2.34, contains the parameters: m' , n' , s' and ν' , whose meaning is explained below. It is important to note that the parameters s' and ν' characterize the prior information about the standard deviation, in fact the expectation and the coefficient of variation of the standard deviation σ can asymptotically (for large ν') be expressed as:

$$E(\sigma) = s' \quad (2.35)$$

$$V(\sigma) = \frac{a}{\sqrt{2\nu'}} \quad (2.36)$$

Whereas, the prior information about the mean is characterized by m' , n' and s' , the expectation and the coefficient of variation of the mean μ can asymptotically (for large values of ν') be expressed as:

$$E(\mu) = m' \quad (2.37)$$

$$V(\mu) = \frac{s'}{m'\sqrt{n'}} \quad (2.38)$$

Alternatively, it is possible to interpret the prior information as the result of hypothetical prior test series, one for the mean and one for the standard deviation. In that case, the information for the standard deviation are:

s' is the hypothetical sample value;

ν' is the hypothetical number of degrees of freedom for s' .

Whereas, it requires two additional parameters for the information about the mean:

m' is the hypothetical sample average;

n' is the hypothetical number of observations for m' .

Thus, m' and s' represent the best estimates for the mean and standard deviation and through the choice of n' and ν' , the uncertainty with respect to the estimates can be expressed. Moreover, it is noted that for a test, normally $\nu = n - 1$ instead of the prior parameters, n' and ν' can be chosen independently from each other (n' and ν' should be chosen equal to zero, if very little information are available).

In many cases, it could be reasonable to assume that there is very little or no prior information about the mean (so $n' = 0$), but is possible to obtain a fairly good estimate of σ' . As an example, let the coefficient of variation of σ be about 30%, which, according to the equation 2.36, corresponds to $\nu' = 5$. It is usual for concrete cubes that the result of many test samples shows considerable variability in the mean but significantly less variability in the standard deviation.

Using the equation 2.32, one can combine the prior information characterized by equation 2.34 and the test result of n observations with sample mean m and sample standard deviation s . The result is a posterior distribution for the unknown mean and standard deviation of R , which is again given by the equation 2.34, but its parameters are given by the following updating rules:

$$n'' = n' + n \quad (2.39)$$

$$\nu'' = \nu' + \nu + \delta(n') \quad (2.40)$$

$$m''n'' = n'm' + nm \quad (2.41)$$

$$(\nu''(s'')^2) + n''(m'')^2 = (\nu'(s')^2 + n'(m)^2) +^2 + nm^2 \quad (2.42)$$

where:

$$\nu = n - 1;$$

$$\delta(n') = 0 \text{ for } n' = 0;$$

$$\delta(n') = 1 \text{ for } n' > 0.$$

Using the equation 2.32, the predictive value of R can be found in:

$$R = m'' - t_{\nu''} s'' \sqrt{1 + \frac{1}{n''}} \quad (2.43)$$

In the equation 2.43, $t_{\nu''}$ has a central t-distribution whose values for given probabilities of exceeding the limits are given in Table 2.4 However, modifications for lognormal distributions of R are straightforward.

It is noted that if σ_R is known, $\nu = \infty$ should be used.

<i>Degrees of freedom (ν)</i>										
β_R	$\phi(-\beta_R)$	1	2	3	5	7	10	20	30	∞
1.28	0.10	3.08	1.89	1.64	1.48	1.42	1.37	1.33	1.31	1.28
1.65	0.05	6.31	2.92	2.35	2.02	1.89	1.81	1.72	1.70	1.64
2.33	0.01	31.8	6.97	4.54	3.37	3.00	2.76	2.53	2.46	2.33
2.58	0.005	63.7	9.93	5.84	4.03	3.50	3.17	2.84	2.75	2.58
3.08	0.001	318	22.33	10.21	5.89	4.78	4.14	3.55	3.38	3.09

TABLE 2.4: Values of $t_{\nu d}$, according to ISO2394:2015 [99]

2.6 Partial Factor Methods

In this section, reliability-based partial factor formats are provided for the assessment of existing structures within a Eurocode framework (the focus is related to common everyday structural design situations). In the partial factor formats, the assumption consists of model uncertainties that are the same for new and existing structures. This approach is valid under the assumption that the same models and the same methods for the structural assessment are used with the same epistemic uncertainties. Therefore, two different partial factor formats are described in this section: the design value method (DVM) and the adjusted partial factor method (APFM), making possible the incorporation of specific reliability-related aspects for existing structures. The DVM proposes a fundamental basis for evaluating partial factors, whereas the APFM provides adjustment factors to be applied on the partial factors for new structures in EN1990:2002 [2].

In order to extend the principles of limit state design of new structures to the reliability assessment of existing structures, it is necessary to focus the following matters:

- the definitions of the design values and partial factors;
- the derivation of these safety elements using a reliability-based framework, so as to include additional information from experimental tests and inspections, an alternative target reliability level or remaining working life.

In this work, only the resisting side of the limit state function was evaluated and elaborated.

2.6.1 Design Value Method

The partial factor method (recommended in the Eurocode for new structures) for the assessment of existing structure, refers to the design value approach conforming to ISO2394:2015 [99] and EN1990:2002 [2]. In this approach, the partial factors γ_X should be derived from the actual distribution of the variable X under consideration, which is based on prior information or results of tests or the combination of both of them.

The design value R_d of the resistance variable R is defined in the equation 2.23 and the

partial factor can be obtained from 2.44, as follows:

$$\gamma_M = \gamma_m * \gamma_{Rd} \quad (2.44)$$

where γ_m is the reliability-based partial factor accounting for variability of the material and γ_{Rd} denotes the partial factor accounting for model uncertainty and geometrical uncertainties.

Assuming a lognormal distribution for the material property, variation of the partial factor γ_m is obtained from the equation 2.45:

$$\gamma_m = \frac{X_k}{X^*} = \frac{\mu_X * \exp(-1.645V_X)}{\mu_X * \exp(-\alpha_R \beta V_X)} \quad (2.45)$$

where V_X is the coefficient of variation of the material property and the 5% fractile is considered for the characteristic values.

It is possible to graph (see Figure 2.9) the values of γ_m as function of the coefficient of variation V_X and target reliabilities $\beta = 2.3, 3.1, 3.8$ or 4.3 (very low, low, medium and high failure consequences in ULS, as described in ISO13822:2010 [98]) with $\alpha_R = 0.8$.

As shown in Figure 2.9, assuming $\beta = 2.3$ it is obtained always $\gamma_m < 1.15$, also with

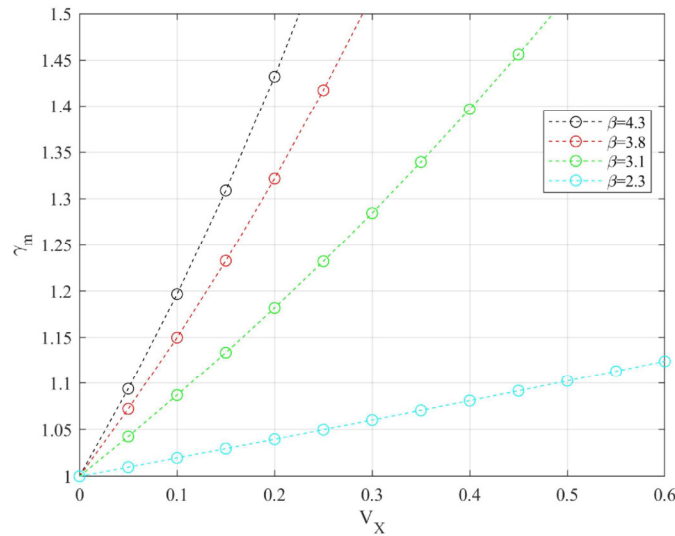


FIGURE 2.9: Variation of partial factor γ_m with the coefficient of variation V_X for $\alpha_R = 0.8$ and $\beta = 2.3, 3.1, 3.8$ or 4.3

a very high coefficient of variation (i.e. $V_X = 0.60$). Moreover, if $V_X > 0.25$ and the target reliability index is $\beta = 3.1$, the partial factor for material properties is higher than $\gamma_m > 1.23$ (this latter is an usual partial factor for concrete strength with $V_X = 0.15$ and $P_f = 0.001$).

In the Figure 2.10 is shown the variation of partial factor γ_m for selected coefficients of variation $V = 0.05, 0.10, 0.15, 0.30$ and 0.60 (these latter values are usual for concrete

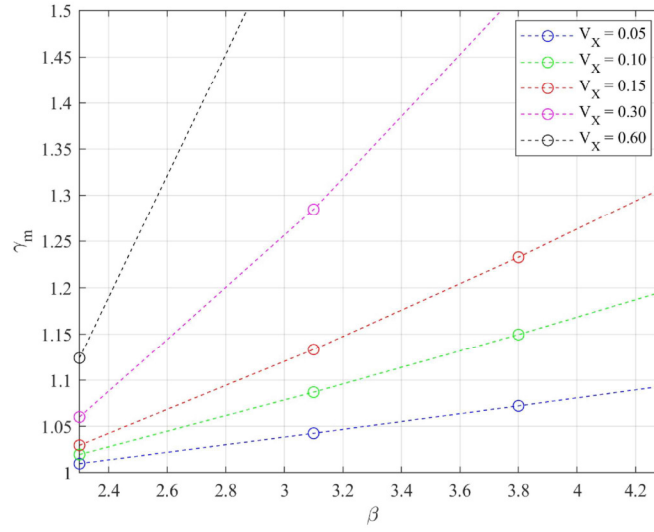


FIGURE 2.10: Variation of partial factor γ_m with reliability index β for selected coefficients of variation $V = 0.05, 0.10, 0.15, 0.30$ and 0.60

specimens extracted from Italian existing structures of first 20th century). It is possible to note that for high values of coefficient of variation ($V_X \geq 0.30$) with target reliability index $\beta = 3.8$, it is obtained the partial factor method for material property $\gamma_m > 1.5$ (e.g. partial factor values for concrete strength are always higher than $\gamma_C > 1.7$). Moreover, if the coefficient of variation for steel reinforcement is $V_s = 0.10$ (this value is more common in existing structures), it could be assumed $\beta = 2.9$ to obtain $\gamma_s < 1.07$.

The equation 2.45 derives from matching the two way to evaluate the design value for material property in the *Annex D.7* related to EN1990:2002 [2]: assessment via characteristic value and direct assessment of the design value for ULS verifications. Assuming the coefficient of variation of new structures (i.e. for concrete $V_c = 0.15$ and for reinforcement $V_s = 0.05$) and a probability of failure lower than 0.1% (i.e. $\alpha_R \beta = 0.8 * 3.8 = 3.04$), it is obtained as follows: $\gamma_c = 1.23$ for concrete and $\gamma_s = 1.08$ for steel reinforcement (the same values are proposed in MC2010 [81] for new structures).

As described in 2.4.1, when model uncertainties are also taken into account, the limit state function becomes:

$$g = \theta_R R - \theta_E E \quad (2.46)$$

where:

θ_R describes the uncertainties related to the resisting model;

θ_E takes into account the uncertainties related to the load effect model.

Assuming a Gaussian distribution for both model uncertainties, as proposed by König and Hosser [53], the partial factor γ_{Rd} and γ_{Ed} can be defined as following:

$$\gamma_{Rd} = \frac{\mu_{\theta R}}{\theta_{Rd}} = \frac{1}{1 - \alpha_R \beta V_{\theta R}} = \frac{1}{1 - 0.32 \beta V_{\theta R}} \quad (2.47)$$

$$\gamma_{Ed} = \frac{\theta_{Ed}}{\mu_{\theta E}} = 1 - \alpha_E \beta V_{\theta E} = 1 + 0.28 \beta V_{\theta E} \quad (2.48)$$

where:

$\mu_{\theta R}, V_{\theta R}$ are the mean values and coefficient of variation of the random variables θ_R ;

$\mu_{\theta E}, V_{\theta E}$ are the mean values and coefficient of variation of the random variables θ_E .

Note that in this work only the partial factor model of resistance is discussed.

However, assuming $\beta = 3.8$ (target reliability index for new structures) in the equation 2.47, the sensitivity factor equals to $\alpha_R = 0.32$ (non-dominant variable), $V_{\theta R} \approx 0.14$ for concrete and $V_{\theta R} \approx 0.06$ for steel reinforcement the same partial factor model, presented in *Model Code* [81], will be obtained: $\gamma_{Rd} = 1.21$ for concrete strength and $\gamma_{Rd} = 1.08$ for reinforcement. These latter values are usual when both model and geometrical uncertainties are taken into account (e.g. lack of design drawing and measurement, scattered data, ect.).

Instead, assuming $V_{\theta R} \approx 0.08$ for concrete and $V_{\theta R} \approx 0.02$ for steel reinforcement in the equation 2.47, with target reliability index $\beta = 3.8$ and sensitivity factor $\alpha_R = 0.32$, it is possible to obtain the partial factor model presented in *Model Code* [81] when relevant measurements of an existing structure indicate insignificant variability of geometrical properties ($\gamma_{Rd} = 1.10$ for concrete strength and $\gamma_{Rd} = 1.02$ for steel reinforcement).

However, larger model uncertainty may need to be considered, especially for shear when concrete crushing is prevalent.

Starting from the equation 2.47, variation of the partial factor γ_{Rd} with the target reliability β for $\alpha_R = 0.32$ is indicated in Figure 2.11 with $\mu_{\theta R} = 1$: Figure 2.11 shows that partial factor for model and geometrical uncertainties for concrete is very variable according to the target reliability index: $\gamma_{Rdc} \approx 1.21$ for $\beta = 3.8$ (e.g. probability of failure $P_f = 0.001$), whereas $\gamma_{Rdc} \approx 1.15$ for $\beta = 2.9$ (e.g. probability of failure $P_f = 0.01$). In this latter case the partial factor decrease is about 5%.

2.6.2 Adjusted Partial Factor Method

Adjusted Partial Factor Method (APFM) is a simplified approach, whose partial factors for resistance parameters in existing structures can be determined according to the following equation:

$$\gamma_{X,exist} = \omega_\gamma \gamma_{X,new} \quad (2.49)$$

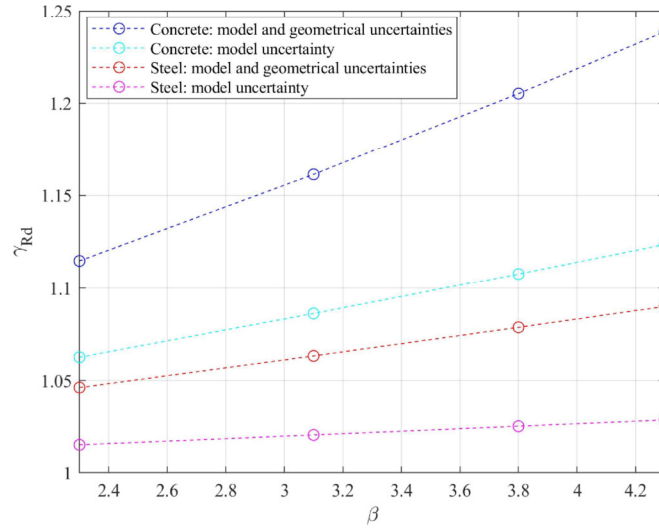


FIGURE 2.11: Variation of partial factor γ_{Rd} with the target reliability β for $\alpha_R = 0.32$

where:

ω_γ is adjustment factor;

$\gamma_{X,new}$ is the partial factor for new structures given in the Eurocodes (i.e. on resistance parameter $\gamma_C = 1.5$ for concrete strength and $\gamma_S = 1.15$ for steel reinforcement).

As described before, in this work the resistance partial factor is considered only, therefore the equation 2.49 becomes:

$$\gamma_{M,exist} = \omega_\gamma \gamma_{M,new} \quad (2.50)$$

Considering the equation 2.44, it is possible to evaluated the adjustment factor ω_γ through simple steps:

$$\gamma_{Rd,exist} * \gamma_{m,exist} = \omega_\gamma * \gamma_{Rd,new} * \gamma_{m,new} \quad (2.51)$$

thus:

$$\omega_\gamma = \frac{\gamma_{Rd,exist} * \gamma_{m,exist}}{\gamma_{Rd,new} * \gamma_{m,new}} \quad (2.52)$$

Taking into account that the partial factor for material property is evaluated in 2.45 (assuming material property Lognormal distributed) and given the exponential property ($\frac{e^a}{e^b} = e^{a-b}$), the adjustment factor is described as following:

$$\omega_\gamma = \frac{\gamma_{Rd,exist}}{\gamma_{Rd,new}} * \exp\left(\alpha_R \beta_{new} V_{X,new} \left(\frac{\beta_{X,exist} * V_{X,exist}}{\beta_{X,new} * V_{X,new}} - 1\right) - 1.645 V_{X,new} \left(\frac{V_{X,exist}}{V_{X,new}} - 1\right)\right) \quad (2.53)$$

where:

X is the material property (e.g. concrete strength and steel reinforcement);

$V_{X,exist}$ is the coefficient of variation of experimental test (for concrete or steel reinforcement);

$V_{X,new}$ is the coefficient of variation of material property for new structures (e.g. $V_C = 0.15$ for concrete strength and $V_S = 0.05$ for steel reinforcement);

α_R is the sensitivity factor for material property (for dominant variables $\alpha_R = 0.8$);

β is the target reliability index (usually it is assumed $\beta_{new} = 3.8$ and $\beta_{exist} < \beta_{new}$);

$\gamma_{Rd,new}$ is partial factor for model and geometrical uncertainties for new structures (e.g. with $\beta_{new} = 3.8$);

$\gamma_{Rd,exist}$ is partial factor for model and geometrical uncertainties for existing structures (e.g. with $\beta_{exist} < \beta_{new}$).

In the equation 2.49, the model uncertainty can be calculating according to equation 2.54 (see also equation 2.22):

$$\gamma_{Rd} = \gamma_{Rd1} * \gamma_{Rd2} \quad (2.54)$$

where:

$$\gamma_{Rd1} = \frac{1}{1 - 0.32\beta V_{\theta,Rd1}} \quad (2.55)$$

$$\gamma_{Rd2} = \frac{1}{1 - 0.32\beta V_{\theta,Rd2}} \quad (2.56)$$

Taking a reliability index of 3.8, the coefficient of variation associated with model uncertainties proposed for new structures (in common cases, $\gamma_{Rd1} = 1.10$ for concrete strength and $\gamma_{Rd1} = 1.02$ for reinforcement steel) can be calculated from 2.55 and yield $V_{\theta,Rd1,c} = 0.075$ and $V_{\theta,Rd1,s} = 0.02$, respectively. However, a larger model uncertainty may need to be considered, for instance in the cases of shear when concrete crushing is governing.

Partial factors that may be assumed to account for the geometrical uncertainties related to a concrete section size or reinforcement position are $\gamma_{Rd2} = 1.10$ and $\gamma_{Rd2} = 1.05$ respectively. Taking a reliability index of 3.8, the associated coefficient of variation of these geometrical uncertainties can be calculated from 2.56 and yield $V_{\theta,Rd2,c} = 0.075$ for concrete section size and $V_{\theta,Rd2,s} = 0.04$ for reinforcement position.

As described in fib Bulletin 80 [37], when relevant measurements on an existing structure indicate insignificant variability of geometrical properties, $\gamma_{Rd2} = 1.0$ may be used.

Considering different coefficients of variation of material property for existing structure and target reliability index $\beta_{exist} = 2.3, 3.1, 3.8$ and 4.3 , the adjustment factor ω_γ is shown in Figure 2.12 and 2.13, for concrete and steel reinforcement, respectively. It is assumed for new structure $\beta_{new} = 3.8$ and different values of the coefficient of variation for new structures (i.e. $V_{C,new} = 0.15$ for concrete strength and $V_{S,new} = 0.05$ for steel reinforcement). The Figure 2.12 shows that assuming a target reliability index $\beta_{exist} > 3.8$ (e.g.

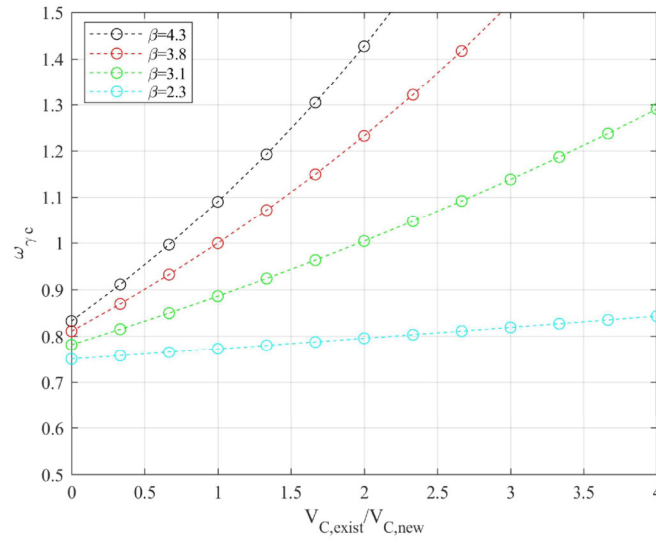


FIGURE 2.12: Influence of the ratio $V_{C,exist}/V_{C,new}$ on adjustment factor ω_{γ_c} ($\alpha_R = 0.8$, $\beta_{new} = 3.8$, $V_{C,new} = 0.15$)

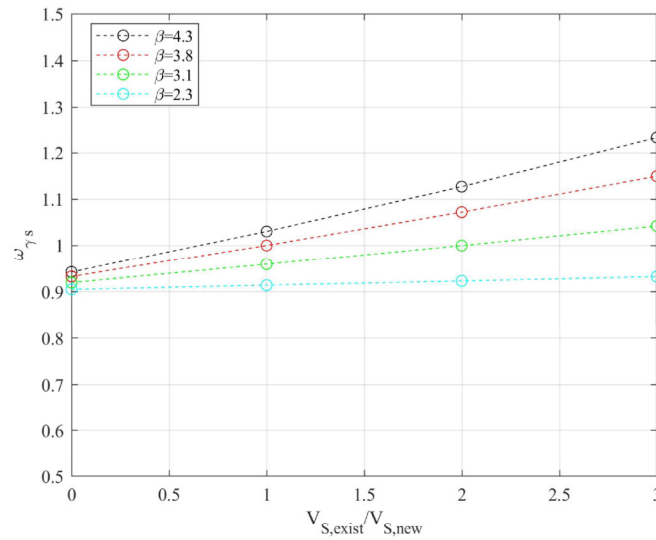


FIGURE 2.13: Influence of the ratio $V_{S,exist}/V_{S,new}$ on adjustment factor ω_{γ_s} ($\alpha_R = 0.8$, $\beta_{new} = 3.8$, $V_{S,new} = 0.05$)

with probability of failure $P_f < 0.001$) in the equation 2.53, it is obtained high adjustment factor values (e.g. $\omega_{\gamma_c} > 1$), especially for coefficient of variation $V_{C,exist} > 0.10$ (this is very common for RC existing structures, as seen in previous Chapter). Whereas, assuming $\beta_{exist} = 2.3$ (e.g. very low probability of failure, $P_f \approx 0.07$), the values of adjustment factor are always $\omega_{\gamma_c} < 1$, also with very great coefficient of variation $V_{C,exist} > 0.60$. In addition, $\omega_{\gamma_c} > 1$ with target reliability index $\beta_{exist} > 3.1$ are obtained, if the coefficient of variation of concrete experimental tests is higher than $V_{C,exist} > 0.30$.

The adjustment factor for steel reinforcement ω_{γ_s} is less variable than concrete one, as shown in Figure 2.13. In this case, assuming the target reliability index $\beta_{exist} = 2.3$, it

always leads to values of adjustment factor smaller than 1, also with very high coefficient of variation (i.e. $V_{S,exist} > 0.15$ it is very uncommon for steel reinforcement, confirmed by several experimental tests). Whereas, for coefficient of variation $V_{S,exist} > 0.03$, $\omega_{\gamma_s} > 1$ is obtained for target reliability index $\beta_{exist} = 4.3$. Finally, to obtain $\omega_{\gamma_s} < 1$, with target reliability index $\beta_{exist} = 3.1$, the coefficient of variation has to be $V_{S,exist} < 0.10$. The curves presented in Figures 2.12 and 2.13 could be shift upwards (e.g. obtaining higher adjustment factor) assuming a target reliability index for new structures $\beta_{new} < 3.8$.

2.6.2.1 Bayesian Updating with APFM

If no further information is available with respect to the coefficient of variation of material properties used for new structures, $V_{C,new} = 0.15$ can be assumed for concrete and $V_{S,new} = 0.05$ for reinforcement steel. These proposed coefficients of variation are consistent with commonly assumed values and are based on calibration, as described in EN1990:2002 [2].

However, if prior information with respect to the coefficient of variation of material property in the existing structure is available (i.e. data from literature or previous test investigations), it is possible to calibrate adjustment factor using *bayesian updating*. Thus, the equation 2.53 becomes:

$$\omega_{\gamma} = \frac{\gamma_{Rd,exist}}{\gamma_{Rd,new}} * \exp \left(\alpha_R \beta_{new} V_{X,prior} \left(\frac{\beta_{X,exist} * V_{X,test}}{\beta_{X,new} * V_{X,prior}} - 1 \right) - 1.645 V_{X,prior} \left(\frac{V_{X,test}}{V_{X,prior}} - 1 \right) \right) \quad (2.57)$$

where:

$V_{X,prior}$ is the coefficient of variation of material property for previous test investigations or data from literature (for concrete or steel reinforcement);

$V_{X,test}$ is the coefficient of variation of current experimental test (for concrete or steel reinforcement);

In the equation 2.57, it can assume the target reliability index $\beta_{new} = 3.8$, as for new structures and the other parameters are the same of equation 2.53. Obviously, assuming $\beta_{new} < 3.8$ for new structures (e.g. with higher probability of failure, $P_f > 0.001$) in equation 2.57, an higher adjustment values with the same ratio $V_{X,test}/V_{X,prior}$ are obtained.

2.7 Advanced Statistical Methods

Several techniques, full-probabilistic approach based, may be used to solve structural reliability problems. In this section, both a simulation technique and sensitivity analysis are

presented as one possible way to solve such problems.

In the first case, in Monte Carlo method, to numerically simulate some phenomenon and then observe the number of times some event of interest occurs; the basic concept behind simulation is relatively uncomplicated, but the procedure can become computationally intensive.

Sensitivity analysis is performed to identify the most affecting uncertainty parameters; in this work, material properties are considered as input uncertainties in order to evaluate the uncertainty of numerical or analytical model (i.e. structural capacity).

2.7.1 Monte Carlo simulation

In Monte Carlo method, it is assumed that we have information from N tests and supposed that we need a sample of n test results. Instead of doing n additional tests, we could randomly select n from the N test results.

This method is a special technique that we can use to generate some results numerically without actually doing any physical testing. We can use results of previous tests (or other information) to establish the probability distributions of the important parameters in our problem. Then we use this distribution information to generate samples of numerical data. In this work, Monte Carlo simulation starts from experimental test performed on samples extracted from RC existing structures. Considering as basic variables $X = [x_1, x_2, \dots, x_n]$ the material properties (e.g. concrete or steel reinforcement strength), Monte Carlo method is developed in the following steps:

1. Calculate mean value μ_x and standard deviation σ_x (thus also coefficient of variation V_x) from experimental test samples, considering all the sub-sample combinations evaluated. An example of mean values and standard deviations related to the yield strengths of 5 steel reinforcement extracted from an existing RC concrete school building built in Italy in 1957 are shown in Figures 2.14 and 2.15, respectively. For each sub-sample combination of test results (in this case steel reinforcement yield strength is evaluated), statistically represented by a mean value μ_x and standard deviation σ_x , the coefficients of variation V_x are calculated. For the same example and according to different sub-sample combinations the coefficient of variation is shown in Figure 2.16.
2. Select the maximum and minimum values of coefficient of variation ($V_{x,max}$, $V_{x,min}$) for each sub-sample combination (according to the n number of test results needed). Starting from the previous example, in the Figure 2.17 maximum and minimum coefficient of variation are shown for $n = 3$;

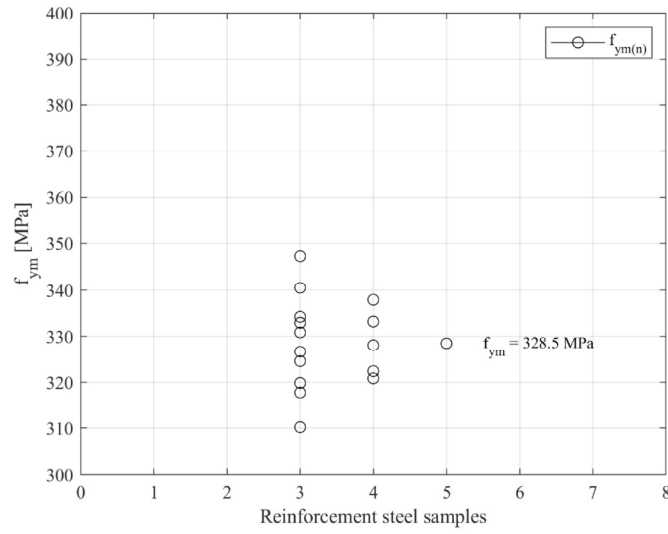


FIGURE 2.14: Mean values of yield strength related to 5 steel reinforcement extracted from an existing RC concrete school building built in Italy in 1957, according to different sub-sample combinations

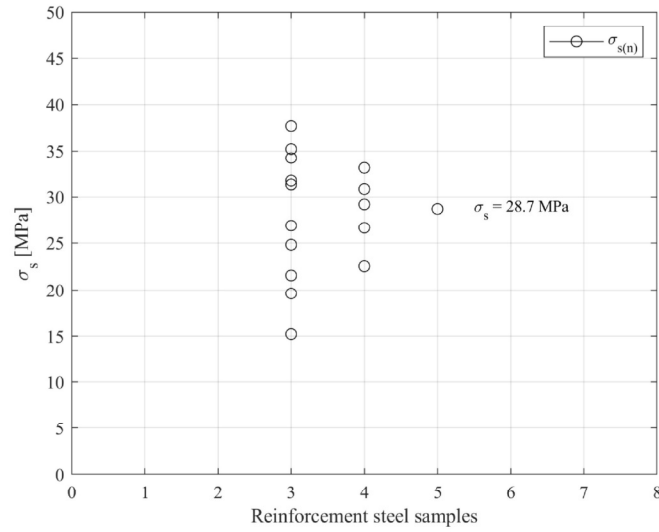


FIGURE 2.15: Standard deviations of yield strength related to 5 steel reinforcement extracted from an existing RC concrete school building built in Italy in 1957, according to different sub-sample combinations

3. Assuming a Lognormal distribution, a sample of N elements (i.e. 10000 test results) is generated for each $V_{x,max}$ and $V_{x,min}$ considered. In the Figure 2.18, two samples of 10000 elements (e.g. yield strength test results) are generated for the maximum and the minimum coefficient of variation calculated previously.

4. Calculate the *Capacity Model* $R(x_i)$ for each generated sample.

According to the two samples described in the previous step, the probability density functions of the resisting bending moment capacity evaluated for a beam, extracted

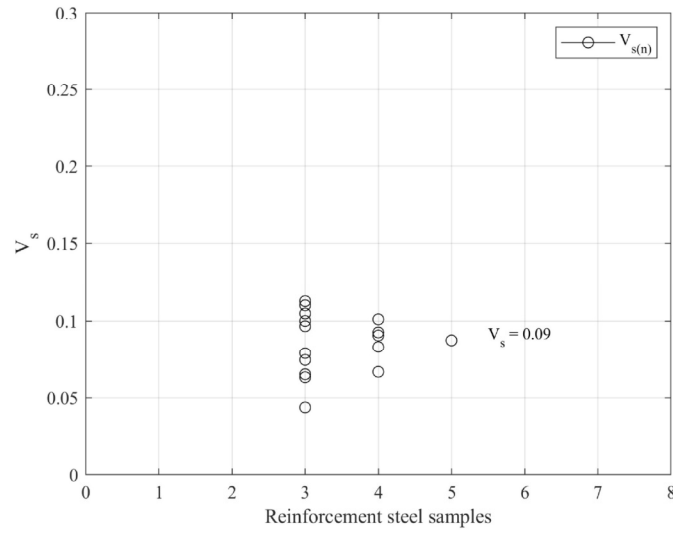


FIGURE 2.16: Coefficients of variation of yield strength related to 5 steel reinforcement extracted from an existing RC concrete school building built in Italy in 1957, according to different sub-sample combinations

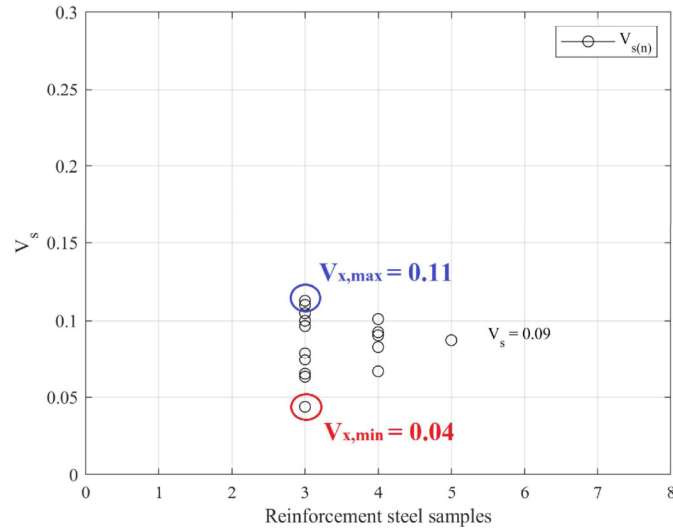


FIGURE 2.17: Maximum and minimum coefficient of variation of yield strength for the sub-sample with $n = 3$, related to 5 steel reinforcement extracted from an existing RC concrete school building built in Italy in 1957

by an existing RC school building built in Italy in 1957, are shown in Figures 2.19.

In the adopted capacity model ($R(x_i) = M_r = A_s * f_y(d - 0.5 \frac{A_s * f_y}{b * f_c})$), only one parameter (in this case: steel reinforcement yield strength f_y) is considered variable; the other parameters is evaluated as deterministic values. In the Figure 2.19, it is noted that the coefficient of variation of capacity model (e.g. resisting bending moment) is different from the coefficient of variation of basic variable (e.g. yield strength). Moreover, basic variables with high coefficient of variation lead to capacity

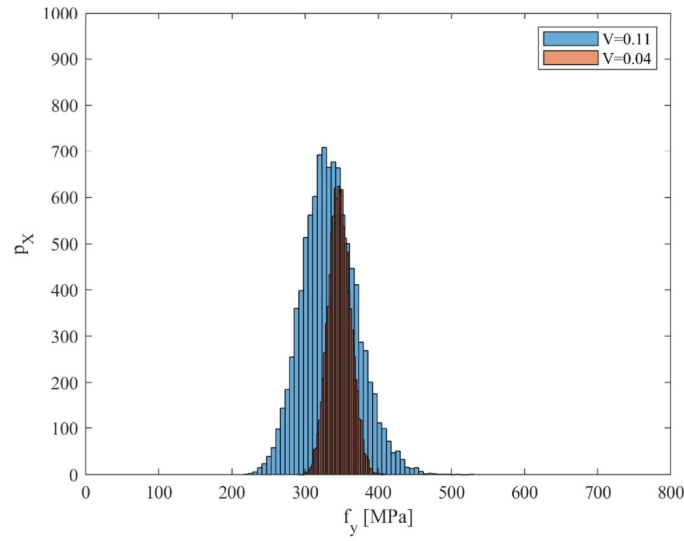


FIGURE 2.18: Probability density function of the two samples generated according to the maximum and the minimum coefficient of variation of steel reinforcement yield strength for the sub-sample with $n = 3$

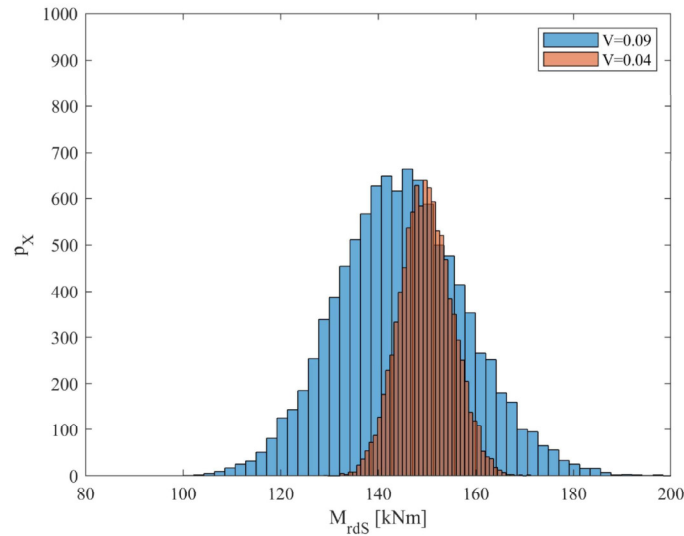


FIGURE 2.19: Probability density function of resisting bending moment capacity calculated from two samples generated according to the maximum and the minimum coefficient of variation of steel reinforcement yield strength for the sub-sample with $n = 3$

model with high coefficient of variation.

5. Choose a probability distribution by fitting the distribution of the capacity model calculated; For the previous capacity model, lognormal distribution fitting is examined and the respective results are shown in Figure 2.20. In this latter Figure, it is noted that lognormal distribution fits well to the adopted capacity model.
6. Estimate the *Design Value* of capacity model R_d , assuming a FORM-based method (the values of β_t are settled according to Partial Factor Methods). For example,

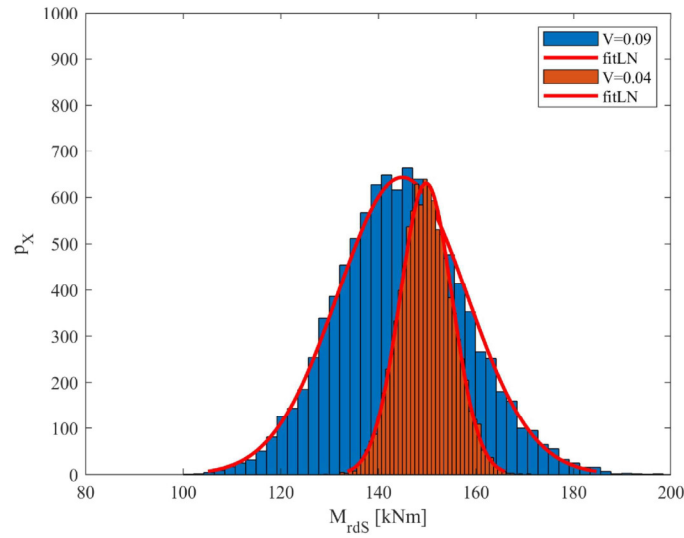


FIGURE 2.20: Lognormal distribution fitting of probability density functions of resisting bending moment capacity calculated from two samples generated according to the maximum and the minimum coefficient of variation of steel reinforcement yield strength for the sub-sample with $n = 3$

assuming a lognormal distribution, the design value of capacity model is:

$$R_d = \mu_R * \exp(\alpha_R \beta_t V_R) \quad (2.58)$$

where:

μ_R is the mean value of R ;

α_R is the sensitivity factor of resistance parameters;

β_t is the target reliability level;

$V_R = \frac{\sigma_R}{\mu_R}$ is the coefficient of variation of R ;

According to all sub-samples, design values of capacity model are calculated for the maximum and the minimum coefficient of variation: in this way all possible results are taken into account.

7. Make a graphic ($R_d - n$) for all the design value of capacity model calculated (according to the n number of test results needed). Figure 2.21 shows the design values of resisting bending moment evaluated for a beam extracted from an existing RC school building built in Italy in 1940. The design values, calculated by the equation 2.58, consider maximum and minimum coefficient of variation of capacity model and target reliability index $\beta = 3.8$ (usual value for new structures). The design values are also compared with the experimental test result.

As just shown, Monte Carlo simulation is applied to the material property only, particularly to the *dominating variable* of the structural capacity. In the previous example (the

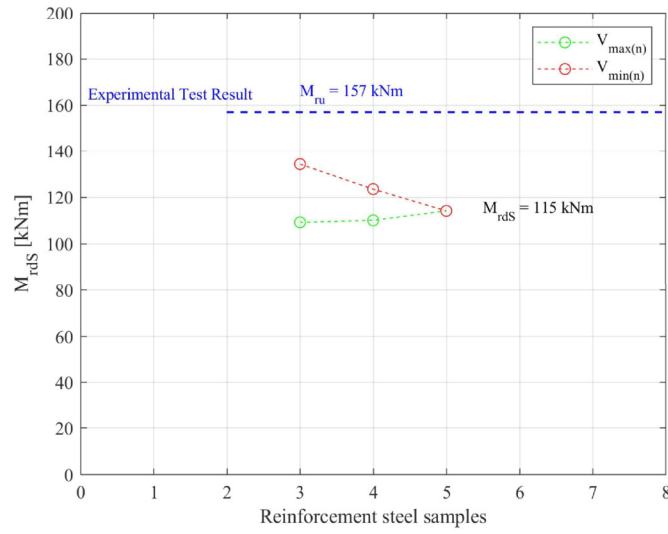


FIGURE 2.21: Design values of resisting bending moment evaluated for maximum and minimum coefficient of variation, according to the sub-sample number. Design value is compared with experimental test result

resisting bending moment), steel reinforcement strengths are generated, whereas concrete strength and geometry are fixed.

2.7.2 Sensitivity Analysis

Sensitivity analysis is the study of how the uncertainty in the output of a mathematical model or system can be assigned to different sources of uncertainty in its inputs.

Before describing the adopted method step by step, it is essential to provide some preliminary information.

Let be R a performance indicator through which an existing structure is assessed (structural capacity model):

$$R = R(X) \quad (2.59)$$

where:

$R(\bullet)$ is a analytical or numerical (e.g. finite element-based) model;

$X = [x_1, x_2, \dots, x_n]$ is a vector of random basic variables (e.g. materials, geometry, details);

Within the framework of the proposed methodology, it is required that $R(X) \in C^1$ (e.g. $\partial R(X)/\partial X_i$), exists and is continuous $\forall i$.

Without loss generality, it is assumed that the basic variables X_i for $i = 1, \dots, n$ are independent each other and the uncertainty X_i is described by the sample mean μ_{X_i} and simple variance $\sigma_{X_i}^2$, which are estimated from n samples extracted during experimental campaigns.

As presented for Monte Carlo method, also in this case Sensitivity Analysis starts from experimental test performed on existing RC structures. In this work, material properties (e.g. concrete and steel reinforcement strength) represent basic variables; the evaluated procedure is described in the following steps:

1. Calculate mean value μ_x , standard deviation σ_x and the related coefficient of variation V_x from experimental test samples, considering all the possible sub-sample combinations. This step is also described in 2.7.1 and a real example is shown in the Figures 2.14, 2.15 and 2.16.
2. Select the maximum and minimum values of coefficient of variation ($V_{x,max}, V_{x,min}$) for each sub-sample combination (according to the n number of needed test results, $2n - 5$ values of coefficient of variation is calculated). Also in this case, it is possible to evaluate an example shown in Figure 2.17.
3. Starting from all the couples of μ_x and σ_x , related to $V_{x,max}$ and $V_{x,min}$ for each sub-sample considered, a first-order second moment approximation is adopted to estimate the mean and the variance of the capacity model R . So doing, it is possible to approximate the mean value of the outcome R as follows:

$$\mu_R = R(\mu_x) \quad (2.60)$$

whereas the approximated variance of the outcome R is:

$$\sigma_R^2 = \sum_{i=1}^n \left(\frac{\partial R(X)}{\partial X_i} \Big|_{\mu_x} \right)^2 * \sigma_x^2 \quad (2.61)$$

4. Estimate the *Design Value* of capacity model R_d , assuming a FORM-based method. Assuming a Lognormal distribution for the capacity model (according to the step n.5 of 2.7.1), the design value of capacity model is:

$$R_d = \mu_R * \exp(\alpha_R \beta_t V_R) \quad (2.62)$$

where:

μ_R is the mean value of R ;

α_R is the sensitivity factor of resistance parameters;

β_t is the target reliability level;

$V_R = \frac{\sigma_R}{\mu_R}$ is the coefficient of variation of R ;

With the aim to compare this procedure to the *partial factor methods*, the values of β_t are assumed as the same. Also in this case, design values of capacity model are

calculated for the maximum and the minimum coefficient of variation, according to all sub-sample considered.

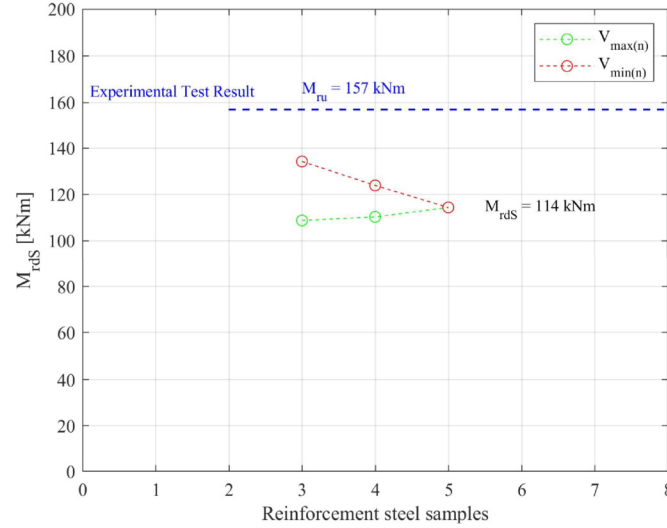


FIGURE 2.22: Design values of resisting bending moment evaluated for maximum and minimum coefficient of variation, according to the sub-sample number. Design value is compared with experimental test result

- According to the n number of test results considered, a graphic $(Rd - n)$ for all the design values of capacity model is calculated. To compared the same example presented in the step $n7$ of Section 2.7.1, Figure 2.22 shows the design values (according to target reliability index $\beta_t = 3.8$) of resisting bending moment evaluated for the beam extracted from an Italian existing RC school building built in 1940. The design values of capacity model, calculated by the equation 2.58 for each maximum and minimum coefficient of variation, are compared with the experimental test result.

Chapter 3

The knowledge about RC existing structures: the experimental activity

3.1 Abstract

In the structural safety assessment process of existing structures, the knowledge of mechanical material properties is a key point. In fact, different experimental activities carried out on materials extracted from existing reinforced concrete buildings show a high strength variability, especially for concrete. In the past, the lack of standardized material controls and manual techniques of workmanship, determined not always uniform and homogeneous flows of concrete, with consequent very different values of strength within the same structure, but also in the same structural element. The most accurate and reliable experimental techniques are direct tests on the material, but these are very expensive and invasive. Alternatively, indirect methods estimate the material strength by correlating very different quantities; however, the non-destructive methods have a lower cost and are easily executable, moreover, causing no damage to the structural elements. In this Chapter, a complete experimental activity, concerning destructive and non-destructive tests, was conducted on some elements (four portions of column and a beam portion) extracted from an Italian school building built in 1940. This building, demolished in 2003 following a negative evaluation of the structural capacity, represents an interesting case study, especially for the important functionality. Destructive and non-destructive methods have been applied on materials, from which appropriate correlation laws have been developed. Furthermore, cyclic axial compression tests were conducted on two column portions and a bending test on three supports was conducted on the beam portion. The results of the experimental tests on the elements were compared with a F.E.M. analysis based on the results of the indirect measurements.

Nel processo di valutazione della sicurezza strutturale delle costruzioni esistenti, la conoscenza delle proprietà meccaniche dei materiali rappresenta un punto di fondamentale importanza. Infatti, diverse attività sperimentali condotte sui materiali estratti da edifici esistenti in calcestruzzo armato dimostrano un'elevata variabilità della resistenza, soprattutto relativa al calcestruzzo. In passato, la mancanza di controlli standardizzati sul materiale e tecniche di lavorazioni di tipo manuale, hanno determinato getti non sempre uniformi e omogenei del calcestruzzo, con conseguenti valori molto diversi di resistenza all'interno della stessa struttura, ma anche nello stesso elemento strutturale. Le tecniche di indagine più accurate e affidabili sono le prove dirette sul materiale, queste però hanno lo svantaggio di essere molto costose ed invasive. In alternativa, i metodi indiretti stimano la resistenza del materiale correlando grandezze molto diverse tra loro; tuttavia, i metodi non-distruttivi risultano molto economici e facilmente eseguibili, inoltre, non arrecano danno agli elementi strutturali. In questo Capitolo, una completa campagna di indagini, distruttive e non distruttive, è stata condotta su alcuni elementi (quattro porzioni di pilastri e una porzione di trave) estratti da un edificio scolastico italiano costruito nel 1940. Questo edificio, demolito nel 2003 a seguito di una valutazione negativa della capacità strutturale, rappresenta un caso interessante, soprattutto per la sua funzionalità. Tecniche di indagine distruttive e non-distruttive sono state condotte sui materiali, da quali sono state costruite opportune leggi di correlazione. Inoltre, test ciclici di compressione assiale sono stati condotti su due porzioni di pilastro e una prova di flessione su tre appoggi è stata condotta sulla porzione di trave. I risultati dei test sperimentali sugli elementi sono stati confrontati con analisi di previsione agli elementi finiti basate sui risultati delle misure indirette.

3.2 Introduction

Before defining structural repair or retrofitting, it is fundamental *to know* degraded state of structural elements and its causes. Diagnostic phase is a crucial step on structural safety evaluation, especially in monitoring and maintenance processes.

For this reason, it is necessary to conduct experimental tests to evaluate chemical-physical characteristics of materials, mechanical properties and its degraded state, according to environmental conditions.

The knowledge of reinforced concrete existing structures is a methodological process that considers: (a) visual inspection, (b) historical information collection and (c) experimental test campaign.

The ageing of reinforced concrete existing structure heritage, highlighted by recent catastrophic events occurred in Europe, and especially in Italy, indicates to pay more attention to material property evaluation. In fact, as described by Giannini et al.[44], most of existing structures are built over than 50 years ago, which is the usual working lifetime for ordinary

structures.

However, the knowledge process about RC existing structures, is not only limited by technical reasons: social, economical and environmental aspects are not negligible. Usually, experimental tests must be conducted in such a way to guarantee structure functionalities and avoiding high structural damage.

Thus, early stage evaluation of reinforced concrete (RC) elements is an important step in establishing a detailed diagnosis analysis of existing structures. For this reason, assessing concrete properties using non-destructive testing (NDT) methods can play an important role in the process of RC structure management before any expensive maintenance is undertaken.

Therefore, due to practical and economic reasons, only a limited number of specimens can be extracted, and, as highlighted by Masi [63], non-destructive tests can be used together with core tests, thus reducing the amount of coring required to gain a sufficiently complete and reliable understanding of the concrete strength of the structure.

According to non-destructive methods, ultrasonic pulse velocity test and rebound hammer are most common; however, the use of rebound hammer technique only is not sufficient to estimate and predict concrete strength. As described by Qasrawi [87], high variability of rebound index is obtained, which makes engineering judgement quite difficult. Moreover, starting from the first applications of longitudinal pulse velocity for concrete strength estimation, Komlos et al. [52], Sbartaï et al. [92] implemented a methodology for non-destructive testing of concrete structures, considering: the sensitivity of the NDT techniques, the uncertainty of the NDT measurements and especially, the optimal combination of NDT techniques to enhance the evaluation of concrete properties.

For these reason, the development and validation of methodology that would lead to a reliable strength estimate, with an acceptable level of confidence, is the aim of many researches. One of the most important step is the measurement calibration; usually, this latter is specific to the structure and depends on relationships between indirect measurements and material strength. The difficulties encountered with calibration lead to the development of combined approaches, as evaluated by Breysse [14], since a second measurement can enable correction of the influence of some uncontrolled factor(s) on a first measurement.

Several approaches are given in literature, but the most used are Di Leo and Pascale [30], Gasparik [43] and RILEM [89]; these latter are applied by D'Amato et al. [19] on existing RC structures and by Faella et al. [34] to calibrate test results from Italian school buildings. However, calibration approaches can lead to different results: in some cases, concrete strength (evaluated by compressive strength) and ultrasonic pulse velocity show a good correlation; otherwise, both rebound index and ultrasonic velocity are not correlated with compressive strength results. Moreover, as shown by Viso et al. [29], compressive strength can be influenced by shape and size of specimens.

In alternative way, sonic tomography can represent a reliable tool to estimate degrade state

of reinforced concrete elements, especially, as studied by Liu and Yeh [59], to evaluate size and location of internal cracks. Moreover, Hoegh et al. [46] extended this technology also on infrastructural field. However, the evaluation of crack development in structural elements (i.e. beams, columns or walls) represents the main use, as described by Behnia et al. [10].

Starting from preliminary studies, conducted by Buyukozturk [15], demand for the development of non-destructive testing methods for concrete structures has increased according to the deterioration condition of World's infrastructures. For this reason, efficient and accurate imaging techniques are used by Lorenzi et al. [60] as reliable tools to evaluate structural safety of concrete structures. Nowadays, tomographic testing are used both on heritage buildings, as shown by Michaux and Grill [68], and on new structures to evaluate material conditions in the cross section and crack states; these latter aspects are investigated by Pahlavan et al. [82] and Lee et al. [58]. For all these reason, the development of a strategy for the management and maintenance of the built heritage is a key challenge for research.

In this work, several destructive and non-destructive tests are performed on structural elements extracted from an existing reinforced concrete school building. Four portions of column and a portion of beam are investigated to estimate the variability of mechanical material properties; in fact, different values of material strengths are extracted, comparing elements close to each other, but also evaluating the same structural element.

The case study is an Italian school building, located in Tuscany, and demolished in 2003 for structural safety problems. Several existing RC structures, especially school buildings, are needed to be repaired or retrofitted, because most of these were designed for gravity loads only according to inadequate seismic regulation. In addition, in Italy, the use of smooth bars was allowed until 1970; experimental results, conducted by Abbiati et al. [6] and Verderame et al. [113], have highlighted rotation capacity of structural elements with smooth bars is rather high than elements with ribbed bars, but in the first case bond strength is very low.

For these reasons, Masi et al. [64], [65] developed procedures and techniques to investigate material properties and constructive details.

Herein, destructive and non-destructive tests are described and performed on existing RC elements; results are used to carry out specific correlation, in order to compare rules in literature with the case study proposed in this work. Moreover, considering the importance of experimental capacity tests, highlighted by Braga et al. [13], Lavorato et al. [55] and Nuti et al. [77], cyclic axial compressive tests and bending moment capacity test are conducted on two portions of column and on a portion of beam, respectively.

Simultaneously to traditional measurement tools, a contactless optical method is performed to evaluate displacements and deformations during the tests. Digital Image Correlation (D.I.C.) is used in several fields, as described by Pan et al. [83] and Tekieli et al. [112];

however, in this work D.I.C. analysis is applied on bending moment test performed on existing beam.

Finally, tomography maps are used also to conduct forecast FEM analyses on evaluation of structural element capacity; according to inelastic concrete behaviour, studied by Kusinski and Mroz [54], section fibre are modelled according to sonic tomography results.

The research aim is to evaluate the variability of material properties on existing reinforced concrete structures using non-destructive tests to predict structural failure. The case study represents an usual school building, built in Italy, without construction seismic details. Thus, starting from few destructive tests, several non-destructive methods are performed and its results represent a reliable tool to evaluate material properties. However, only few elements, extracted from the same structure, were tested and thus this methodology could be extended for other cases, considering also different structural failures.

3.3 Experimental techniques for concrete

The assessment of existing reinforced concrete structure entails a series of steps, among which the evaluation of mechanical material properties can be considered a corner-stone. In fact, the experimental tests are used to estimate material strength which, combined with geometrical and model information, lead to evaluate structural capacity.

Usually, direct tests on samples, extracted from a structure, provide the most reliable measurement of material strength. However, the accepted number of samples is limited because these methods are expensive and invasive. For this reason, non-destructive methods are extensively used especially for concrete, and the results are usually calibrating using a limited number of destructive tests. In this way, it is possible to provide some preliminary information about the homogeneity of investigated concrete and the possibility to suggest the zones where to extract other cores.

Alternatively to experimental techniques that involve localized damage on material portion (*in situ*), it is more appropriate to refer to semi-destructive tests (SDT). In this case, the affected area is very limited and easy to repair.

In this Section, destructive, semi-destructive and non-destructive techniques are described, in order to explain advantages and disadvantages for each technique.

The concrete strength evaluation is a key point of the structural safety assessment; usually, element cross-section is known, but the material strength is very variable.

3.3.1 Destructive testing

The concrete sample extraction requires a special machine called *coring machine*, which must be used carefully by specialized employees (see Figure 3.1). During hole drilling, particular attention must be made to the execution speed (low drilling speed causes less



FIGURE 3.1: Concrete cores extraction

disturbance to the sample). However, the International Standards (i.e. in Italy UNI EN 12504-1:2009 [103]) regulate the modality and the quality control of the concrete strength evaluation.

In addition, particular attention must be paid on the sample extraction localization: it is preferred the areas where the concrete stresses are low and there are no steel reinforcement. Concrete strength results can be influenced by following factors:

- The sample position related to the structural element (i.e. top or bottom of column, the drilled direction related to the flowing);
- The disturbance during the sample extraction;
- The dimension of sample (i.e. micro-sample or ratio height/diameter < 2);
- The presence of steel reinforcement portion;

Immediately after the sample extraction, it is possible to perform the carbonation control using the so-called *colorimetric methods*. The colorimetric method is based on phenolphthalein hydroalcoholic solution. This latter assumes from pink to purple color according to the increasing alkalinity (see Figure 3.2); in other words, the increasing of pH value.

However, the compressive concrete test is performed by means of technical machine (i.e. MTS compression testing), according to the International Standard [104], [100]; in order



FIGURE 3.2: Phenolphthalein hydroalcoholic solution applied on concrete core

to evaluate elastic modulus, displacement transducers or strain-gauges can be applied on sample core during the test (see Figure 3.3).

The compressive concrete strength evaluation depends on several factors, which are con-

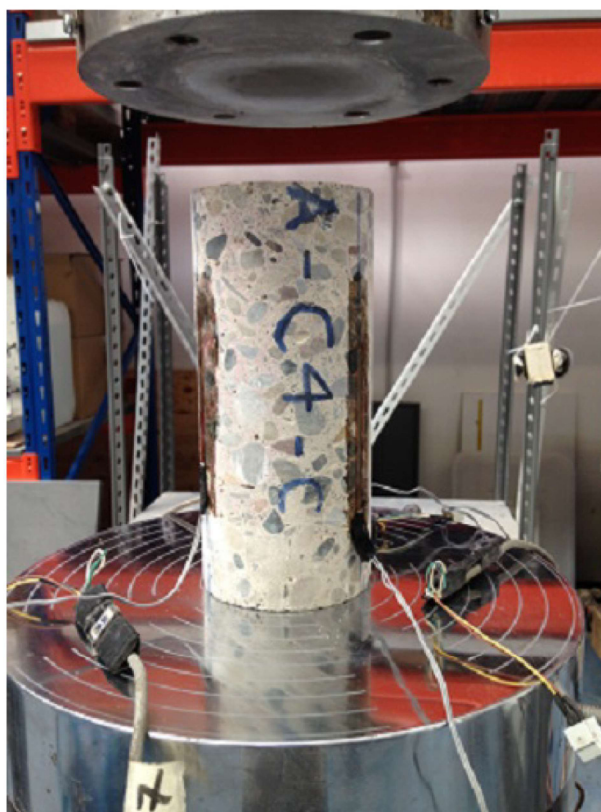


FIGURE 3.3: Compressive concrete testing

sidered on the concrete strength estimation. Several factors are taken into account in order to evaluate: the drilled direction, the sample diameter, the disturbance during sample extraction, the sample shape etc..

However, the factors which influence concrete strength are divide in two groups:

- Factors which depend on *concrete characteristics*: test humidity level, porosity, drilled direction related to flowing and flowing imperfections;
- Factors which depend on *extraction* and *test modality*: sample diameter, height/diameter ratio, compressed face flatness, face correction, drilling disturbance and presence of steel reinforcement portion;

Usually, cylindrical or cubical compressive strengths are the reference values for numerical analyses different; calibration factor proposals are presented in British Standard 1881 [1], Concrete Society [94] and American Concrete Institute [5]. However, Braga et al. [12], D'Ambrisi et al. [20], Faella et al. [33] and Ferrini et al. [36] propose correction factors in order to evaluate *in-situ* concrete strength. In some cases, these factors are evaluated considering destructive and non-destructive tests, as described by D'Amato et al. [19], Faella et al. [35] and Masi [63].

In this work, starting from the direct compressive test results, cylindrical compressive strength is evaluated according to the equation 3.1 (from Bartlett & Mac Gregor, 1994; it is presented also in FEMA 274 [8]).

$$f_{cyl,situ} = f_{test} * F_{H/\phi} * F_{\phi} * F_r * F_{mc} * F_d \quad (3.1)$$

where:

$F_{H/\phi}$ is the factor which considers the ratio height/diameter sample, described following:

$$F_{H/\phi} = 1 - [0.144 - 4.3 * 10^{-4} * f_{test}] * (2 - H/\phi)^2 \quad (3.2)$$

F_{ϕ} is the factor which considers the ratio diameter sample related to cylinder with diameter 100 mm:

$$F_{\phi} = 0.0008 * \phi^2 - 0.024 * \phi + 1,16 \quad (3.3)$$

F_r is the factor which considers the presence of steel reinforcement portion ($F_r = 1.08$ for one bar, $F_r = 1.13$ for two bars);

F_{mc} is the factor which considers the humidity ($F_{mc} = 0.96$ for dry sample, $F_{mc} = 1.09$ for wet sample);

F_d is the factor which considers drilling disturbance ($F_d = 1.06$);

However, it is possible to convert the cylindrical compressive strength to the cubical compressive strength, but it is not covered by this work.

3.3.2 Non-destructive testing

Previously as described, the destructive tests provide the most reliable estimation of the mechanical material properties. Especially for concrete, the direct compression tests are more expensive and very few samples can be extracted from the same structure. For this reason, non-destructive tests (NDT) are mostly used, whose results are usually calibrated using a limited number of destructive tests. However, a correlation formula between the in situ measurements and concrete strength is required. In many cases, such formula cannot be easily generalized and must be restricted only to the case study.

In order to estimate the mechanical properties of concrete, the most common non-destructive techniques are the rebound method, the ultrasonic pulse velocity method and the SONREB combined method. These methods are supported by a vast and consolidated literature, additionally their application is approved in the code framework of many Countries.

However, the strength quality estimation may be affected by some errors and uncertainties: firstly, factors may influence the NDT measurement and thus the NDT-strength relationship; secondly, the NDT measurement variability and its roots. Obviously, this variability can be analysed according to several scales, as described by Breysse [14].

In this Section, the main non-destructive methods for the concrete strength estimation are presented: the rebound test, the ultrasonic pulse velocity and the pull-out test (semi-destructive test). Each method are described following:

- **Rebound test:**

The rebound test is regulated by UNI EN 12504-2:2012 [105] and it is very common in current professional practice, although the lack of reliability on the concrete strength evaluation. The rebound machine is composed by a hinged steel mass, which is activated by a spring, and contrasts a percussion rod in contact with test surface (see Figure 3.4). The method measures the rebound height of the mass, which is directed



FIGURE 3.4: Rebound *Schmidt*

against the surface to be tested. According to the specific curves (see Figure 3.5), the rebound index (proportional to the rebound height) is compared to the compressive concrete strength. As described by UNI EN 12504-2:2012 [105], the rebound method can be used to evaluate the concrete homogeneity (*in situ*), the degraded or poor concrete areas and the variations of concrete properties in time: However, it can not replace the destructive methods for the strength evaluation; the latter can be carried out only through available experimental calibration curve.

According to the high dispersion results, more than 9 measurements (see Figure 3.6)

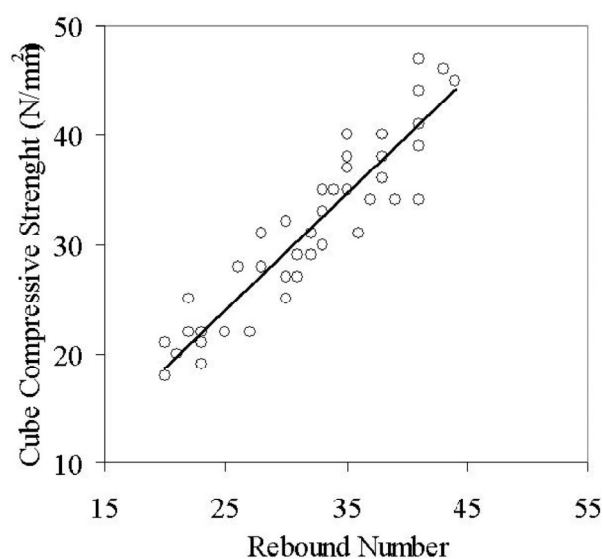


FIGURE 3.5: Rebound index-Cubical compressive concrete strength curve, [1]

are required for each test station. The measurements must to be performed on two opposite sides of the structural element, at the same height. The obtained mean

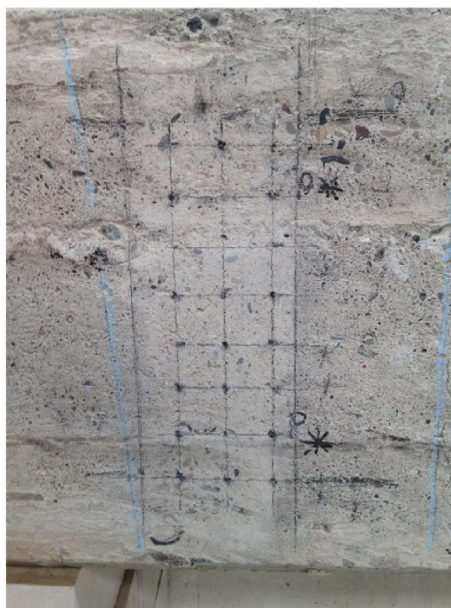


FIGURE 3.6: Example of rebound measurement station

value is called rebound index (I_R). The rebound test area must correspond to the area investigated by other non-destructive and destructive methods, moreover the stoning must be performed and reinforcement bars are avoided (these conditions lead to invalidated results).

The rebound test is simple and fast, but its results can be strongly influenced by many factors: the surface conditions, the carbonation, the concrete ageing, the temperature and the humidity.

However, the rebound test evaluates only the external state of the element, providing concrete uniformity indications.

- **Ultrasonic test:**

The ultrasonic test measures the time taken by sonic waves (usually adequate frequency is about 40 – 120 KHz) using two transducers placed in a given distance, related to the structural element. In this way, it is obtained the propagation speed. The ultrasonic velocity method is regulated by UNI EN 12504-4:2005 [102]. The ultrasonic pulse measurements can be carried out in different ways related to transducer positions (see Figure 3.7). However, transparency method is the most correct way to

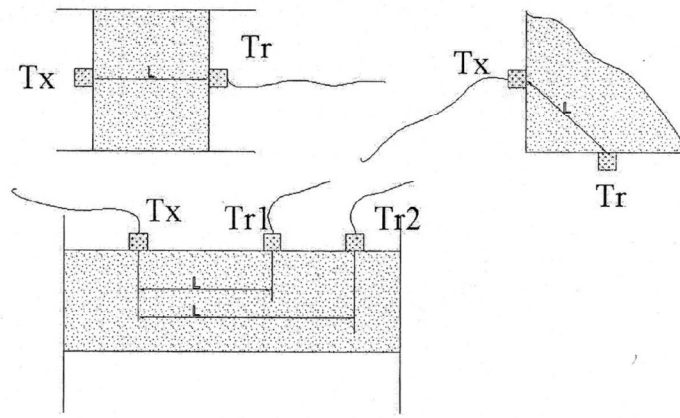


FIGURE 3.7: Ultrasonic measurements: transparency, corner, surface

carry out the measurements.

As seen for the rebound method, the ultrasonic pulse velocity (UPV) and concrete strength are correlated by specific curves. However, also in this case, the use of the direct correlations involves unreliable concrete strength estimation.

Therefore, a theoretical relationship that describes the longitudinal wave velocity for an infinite, isotropic, homogeneous and elastic material is presented following:

$$V = \sqrt{\frac{E_d(1 - \nu)}{\rho(1 + \nu)(1 - 2\nu)}} \quad (3.4)$$

where:

E_d is the dynamic elastic modulus [MPa];

ν is the dynamic Poisson modulus;

ρ is the density mass [kg/m³];

V is the velocity [km/s].

Thus, the ultrasonic pulse velocity is proportional to dynamic elastic modulus square root; from the equation 3.4, it is easy to calculate:

$$E_d = \rho * V^2 \frac{(1 + \nu)(1 - 2\nu)}{(1 - \nu)} \quad (3.5)$$

However, considering that concrete is not homogeneous, isotropic and elastic, the previous relationship must take into account the real physical-chemical material properties. The main factors that influence the ultrasonic pulse measurements are described following:

1. The concrete type;
2. The aggregate size: large aggregates increase the ultrasonic velocity even if the concrete strength remains constant;
3. The water/cement ratio and the cement dosage: reducing W/C ratio, the velocity remain almost constant while the concrete strength can considerably increase;
4. The conglomerate ageing: the velocity is inversely proportional to the curing age (it is opposite for the concrete strength); in fact, according to micro-cracks, the velocities decrease;
5. The moisture: increasing moisture content, the velocity increase (about 5%) and the failure load decreases;
6. The stress state: the ultrasonic pulse velocity is not influenced by stress state (especially, up to material stresses equal to 50% of the failure load); as described by Braga et al. [12] for higher stress levels, the ultrasonic velocity decrease according to micro-cracks;
7. The presence of steel reinforcement: the reinforcements should be avoided if it is possible, because the transmission speed in steel is about 40% higher than the transmission speed in concrete.

Usually, the correlations between the ultrasonic pulse velocity and the concrete strength don't lead to reliable results, especially for existing structures; however, it is possible to evaluate the homogeneity and the crack state of concrete.

• Pull-out test:

The pull-out method is non-destructive test (or semi-destructive test) and it is easy and quick to perform. This technique, regulated in the National Standard [101], is not more invasive because the structural disturbing is limited. The pull-out test measures the force necessary to extract a small conical concrete sample. The extraction is performed by means of a pre-incorporated insert or by a post-inserted device (see

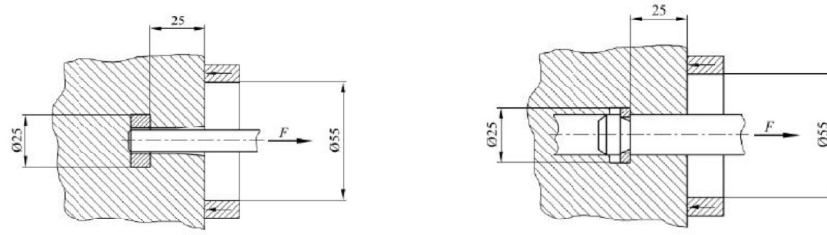


FIGURE 3.8: Pull-out insert pre-incorporated (left side), Pull-out insert post-inserted (right side)



FIGURE 3.9: Schematic section of extraction test

Figure 3.8). Figure 3.9 shows a schematic section of extraction test. Usually, the pull-out method is used for *in-situ* concrete strength verification on new structures (in these cases pre-incorporated plugs are used). Whereas, on the existing structures assessment, post-inserted devices are used. In this latter case, test positions must be spaced at least 200 mm from each other and 100 mm from concrete edge. Moreover, the inserts must be positioned in such a way that all the reinforcements are outside of the fracture area, at least a diameter from steel reinforcement or at distance equals to maximum size of the aggregate.

The minimum thickness of the concrete to be tested must be 100 mm. The number of tests needed to represent an area or a part of the structure depends on: the expected variability of the concrete, the test purpose and the accuracy required.

In order to convert the extraction force to the concrete strength, correlation law is needed, which is provided by the instrumental manufacturer.

3.3.3 Tomography on RC existing structures: only graphic representation?

The demand for the non-destructive testing (NDT) development for reinforced concrete (RC) structures is increased with the growing concern about the deteriorating condition of the World's infrastructure (particularly in Italy). Efficient and accurate imaging techniques

are needed for a reliable assessment of safety and serviceability of RC structures.

In recent years, elastic stress wave methods are attracting increasing attention from researchers and practitioners for real monitoring of structures as well as quantitative evaluation of damage. Behnia et al. [10] evaluated the fracture development in beams, according to parametric analysis and tomographic reconstruction. In other cases, vertical spectral tomogram is used to provide a direct image of the concrete cross-section. Thus, as described by Liu and Yeh [59], tomography test can be a useful tool in the non-destructive inspection of concrete structures.

Pahlavan et al. [82] used the ultrasonic waves to evaluate mechanical-load-induced cracks on reinforced concrete beams.

However, as highlighted by Buyukozturk [15], imaging of concrete structures presents interesting challenges due to the fact that the concrete is a non-homogeneous material. Variable grain size distribution and different properties of the constituent materials make it difficult to produce accurate images. In addition, the generally complex physical geometry of the structure, restricted accessibility, and the existence of reinforcement and prestressing tendons further complicate the problem.

The sonic tomographic method allows to obtain maps (or tomograms) of waves propagation velocity distribution along cross sections. These maps are elaborated through software, that discretized the propagation times of the longitudinal waves measurements through a series of paths. These latter, according to different inclinations, cross each other mutually. The transmission and reception signal points delimited the area geometry. The investigated cross-section is discretized through a grid (usually based on rectangular meshes), where the velocities are determined in the nodes. Moreover, the mesh size is chosen so that it is comparable to sonic signal wavelength.

In this work, an iterative inversion algorithm of S.I.R.T type (Simultaneous Iterative Reconstruction Technique) is used to obtain the velocities fields, given the path times and initial model. Starting from initial speed model, the algorithm reduces progressively the gap between the measured times along the different measurement paths and the calculated times based on the speed model determined by the previous iteration.

Figures 3.10 and 3.11 show two different ways to perform ultrasonic pulse velocity: in the first case, all element surfaces are used (especially for the column evaluation); in the second case, two opposite element surfaces are examined (it is more common for the beam evaluation).

Thus, ultrasonic tomography (UT) is a powerful non-destructive evaluation technology in order to examine the condition of concrete structures through focused images. However, UT's application has been limited to constant-velocity concrete structures; in fact, as described by Lorenzi et al. [60], the use of technique on multi-layer concrete structures with different wave propagation velocities (such as bridge decks with overlays) could lead

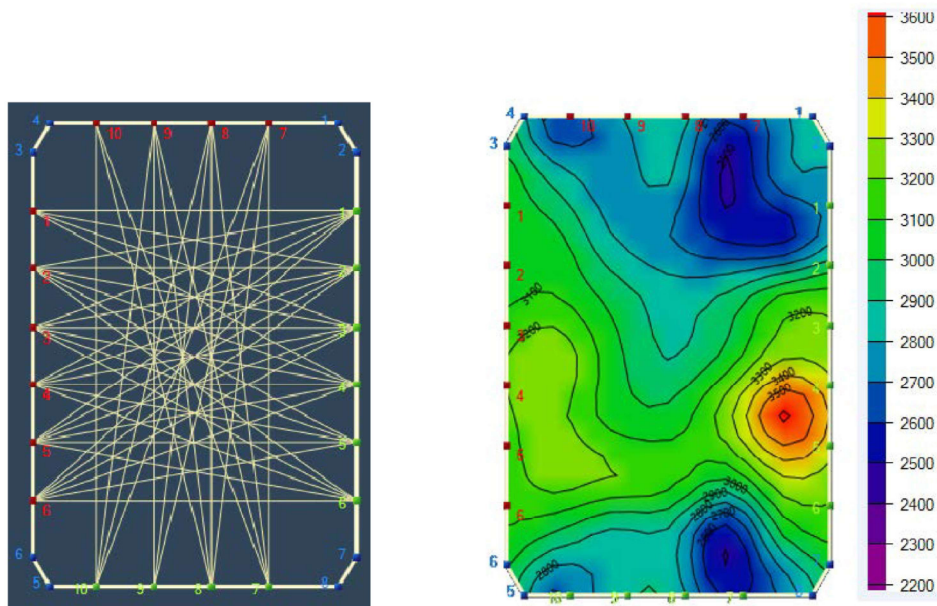


FIGURE 3.10: Tomography test: the speed paths calculated on four surfaces (i.e. column test)

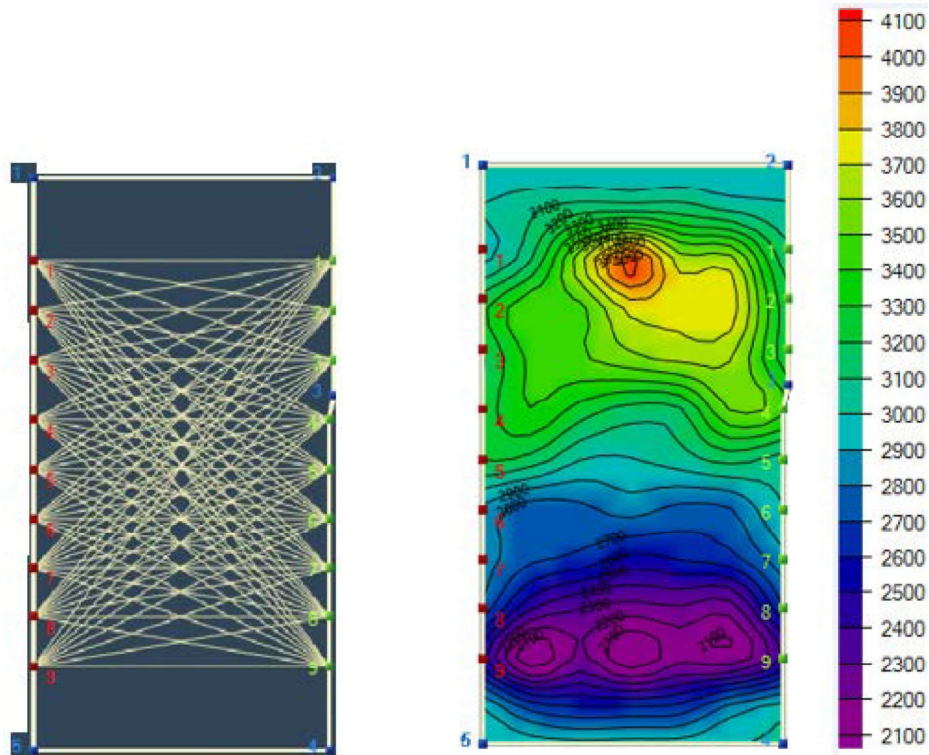


FIGURE 3.11: Tomography test: the speed paths calculated on two surfaces (i.e. beam or wall test)

to unreliable results.

3.4 Experimental techniques for steel reinforcement

The variability of steel reinforcement strength is usually lower than concrete strength; for this reason, non-destructive tests are less common for the steel strength evaluation. However, as seen for the concrete strength evaluation, the destructive tests are the most reliable technique in order to estimate mechanical parameters which cannot be always correlated: *stiffness* and *strength*. This techniques take into account that material samples extracted from a structure/element and the sample test is performed in laboratory. The results are direct and very accurate, but these techniques are very expensive and invasive.

In order to evaluate steel mechanical properties, deformations, yield and ultimate strengths are usually the most representative parameters. In Italy, at the beginning of 20th century (in the same time of the first reinforced concrete applications), the steel mechanical properties are regulated by R.D.L. 1939-XVIII [88]. This code, concerning the smooth steel prescriptions, is remained in force until 1972. The classification proposed in R.D.L. 1939-XVIII divided the steel reinforcement in three categories: soft, semi-hard and hard. Moreover, the posterior code LL.PP. n.1472/57 [85], preserved the same strength limits, but it considered different category names (see Figure 3.12). However, the deformation, yield and ultimate

Normativa	R.D.L. n°2229/1939			LL.PP. n°1472/1957				D.M.30/05/1972					D.M. 30/05/1974			
Tipologia	liscio			liscio		a.m.		liscio		aderenza migliorata (a.m)			liscio		a.m.	
Denominazione	Dolce	Semi duro	Duro	Aq42	Aq50	Aq60		FeB22	FeB32	A38	A41	FeB44	FeB22	FeB32	FeB38	FeB44
Snervamento (kgf/mm ²)	≥ 23	≥ 27	≥ 31	≥ 23	≥ 27	≥ 31	/	≥ 22	≥ 32	≥ 38	≥ 41	≥ 44	≥ 22	≥ 32	≥ 38	≥ 44
Rottura (kgf/mm ²)	42-50	50-60	60-70	42-50	50-60	60-70	/	≥ 34	≥ 50	≥ 46	≥ 50	≥ 55	≥ 34	≥ 50	≥ 46	≥ 55
Allungamento (%)	≥ 20	≥ 16	≥ 14	≥ 20	≥ 16	≥ 14	≥ 12	≥ 24	≥ 23	≥ 14	≥ 14	≥ 12	≥ 24	≥ 23	≥ 14	≥ 12

FIGURE 3.12: Steel classification Italian code evolution

strengths remained the most important mechanical steel properties.

Therefore, the extracted sample is tested in laboratory by the *tensile test* (i.e. MTS tensile testing is usually used, see Figure 3.13). On the steel sample, displacement transducers can be applied in order to evaluate the elastic modulus.

Previously as described, the Italian code (until 1972) allowed the use of the smooth bars; nevertheless, the structural elements with smooth bars show a different behaviour compared to the element with ribbed bars. Verderame et al. [113] developed a mechanical-empirical formulation in order to evaluate the ultimate rotational capacity of columns with smooth bars. Moreover, several experimental tests are performed by Abbiati et al. [6], that proved the high deformation of the smooth bars compared to the ribbed bars. Obviously, the bond



FIGURE 3.13: Tensile steel testing

capacity of smooth bar is very low.

3.5 The case study: an Italian school building

The experimental activity is conducted on five elements, four portions of column and one portion of beam, extracted from an Italian school building that is destroyed in 2003. The building, located in Tuscany (Aulla-MS), was built in 1940 (ground and first floor) and it was completed in 1957 (second floor).

Technical information and constructive details are taken from the design drawing and the technical report provided by the Italian Seismic Office (see Figures 3.14 and 3.15).

The structural system is composed by reinforced concrete frames oriented to two orthogonal directions. The perimeter columns are covered by infill masonry internally.

The experimental testing is conducted in the Structural Laboratory of University of Roma Tre (*PRISMA*), with the aim to evaluate the structural capacities of elements and in order to estimate mechanical material properties through the use of destructive and non-destructive methods. First, station test position was chosen according to preliminary inspection; second, non-destructive tests are performed in order to evaluate the material homogeneity; finally, destructive tests are performed on materials and elements in order to calculate material strengths and structural capacities, respectively.

The experimental testing performed on each structural element is described:

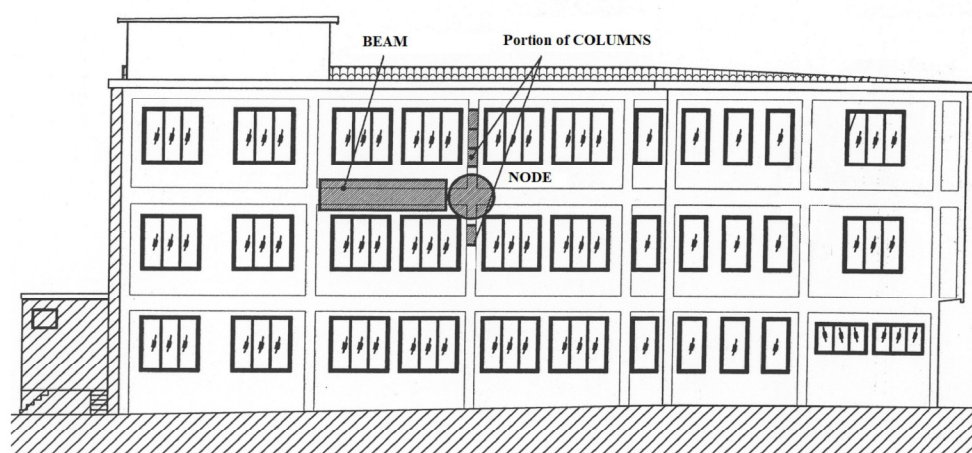


FIGURE 3.14: Front view of Italian school building

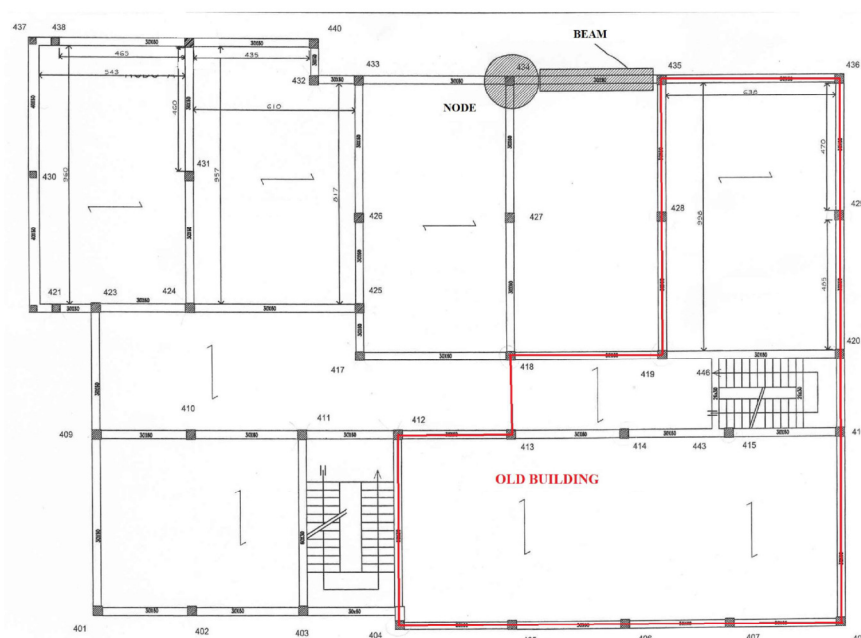


FIGURE 3.15: Plan view of Italian school building

- *Top portion of column - floor I (see Figure 3.17)*

1. Non-destructive tests:
 - a. Rebound and ultrasonic tests
 - b. Sonic tomography
 - c. Pull-out tests
2. Destructive tests:
 - a. Concrete compressive tests
 - b. Tensile steel reinforcement tests

- *Portion of beam - floor II* (see Figure 3.18)
 1. Non-destructive tests:
 - a. Rebound and ultrasonic tests
 - b. Sonic tomography
 2. Bending moment test
 3. Destructive tests:
 - a. Concrete compressive tests
 - b. Tensile steel reinforcement tests
- *Bottom portion of column - floor II* (see left side of Figure 3.19)
 1. Non-destructive tests:
 - a. Rebound and ultrasonic tests
 - b. Sonic tomography
 2. Cyclic axial compressive test
- *Intermediate portion of column - floor II* (see Figure 3.20)
 1. Non-destructive tests:
 - a. Rebound and ultrasonic tests
 - b. Sonic tomography
 - c. Pull-out tests
 2. Destructive tests:
 - a. Concrete compressive tests
 - b. Tensile steel reinforcement tests
- *Top portion of column - floor II* (see right side of Figure 3.19)
 1. Non-destructive tests:
 - a. Rebound and ultrasonic tests
 - b. Sonic tomography
 2. Cyclic axial compressive test

Figure 3.16 shows the lateral structural frame layout of the Italian school building, the five elements are highlighted.

The geometry parameters of each structural element are described on Table 3.1:

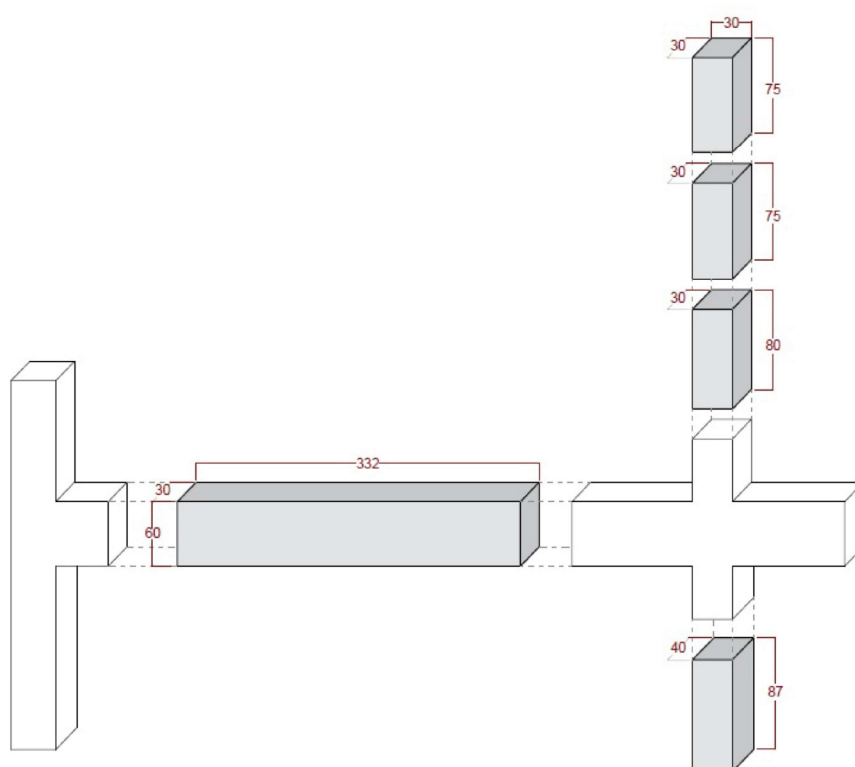


FIGURE 3.16: Five elements extracted from the Italian school building



FIGURE 3.17: The top portion of column - floor I



FIGURE 3.18: The beam portion - floor II



FIGURE 3.19: The column portions related to the floor II: the bottom portion (on the left) and the top portion (on the right)



FIGURE 3.20: The intermediate portion of column - floor II

<i>Element</i>	L [m]	b [m]	h [m]
Top portion of column - floor I	0.87	0.30	0.40
Portion of beam - floor II	3.32	0.30	0.60
Bottom portion of column - floor II	0.80	0.29	0.29
Intermediate portion of column - floor II	0.75	0.29	0.29
Top portion of column - floor II	0.75	0.29	0.29

TABLE 3.1: Geometry of structural elements

3.5.1 Destructive tests for concrete and steel reinforcement

In this Section, the performed destructive tests are described; in this case, the compressive concrete tests and the tensile steel reinforcement tests are performed on the following elements:

- *Top portion of column - floor I*
- *Portion of beam - floor II*
- *Intermediate portion of column - floor II*

3.5.1.1 Destructive tests description

According to the preformed compressive concrete tests and tensile steel reinforcement tests, the sample position, the execution methods and the material characteristics are described.

- *Top portion of column - floor I*

The top portion of column, located on floor I, is built in 1940; the cross-section is equal to 30 cm x 40 cm, the longitudinal steel reinforcement is asymmetric (e.g. $2\phi 18 + 2\phi 20$) and the stirrups are $\phi 6/20$ cm. The concrete surface quality is regular and no degradation state is recognisable (see Figure 3.17).

- a. *n.6 of concrete compressive tests*

The concrete cores are extracted according to non-destructive test stations and the steel reinforcement is avoided. Concrete core extraction layout is shown in Figure 3.21.

The sample characteristics (e.g. sample code, weight, diameter, height, height/-

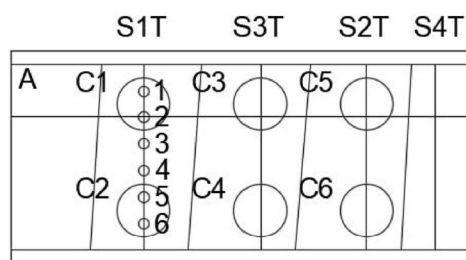


FIGURE 3.21: Concrete core extraction layout of top portion of column - floor I

diameter ratio, area, density and compressive force) are listed in Table 3.2.

Sample code	Weight [g]	Diameter [mm]	H [mm]	H/ϕ [-]	Area [mm ²]	Density [kg/m ³]	Force [kN]
C1	3167	94	205	2.23	6939	2226	71
C2	3232	94	210	2.23	6939	2217	67
C3	3243	94	207	2.20	6939	2257	102
C4	3211	94	208	2.21	6939	2224	83
C5	3246	94	209	2.22	6939	2237	99
C6	3165	94	201	2.14	6939	2268	79

TABLE 3.2: Characteristics of concrete samples extracted from the top portion of column - floor I

b. n.3 of tensile tests on longitudinal bars and n.3 tensile tests on transversal bars

The longitudinal and transversal steel reinforcement are extracted after the concrete core extractions. Steel sample characteristics are describe on Table

Steel type	ϕ [mm]	ϕ_{eq} [mm]	Weight [kg]
L1	18	17.75	1.95
L2	20	19.77	2.41
L3	20	17.95	2.45
T1	6	6.08	0.22
T2	6	6.16	0.23
T3	6	6.09	0.22

TABLE 3.3: Characteristics of steel samples extracted from top the portion of column - floor I

3.3; for each longitudinal (L) and transversal (T) rebar, diameter (ϕ), equivalent diameter (ϕ_{eq}) and weight are shown. Equivalent diameter is needed in order to calculate the yield and ultimate steel strengths (smooth and ribbed bars).

The equivalent area of smooth bar (A_{eq}) is described by the equation 3.6:

$$A_{eq} = \frac{m}{\rho * L_{bar}} * 10^4 \quad (3.6)$$

where:

A_{eq} is the equivalent area of smooth bar with same weight [mm^2];

m is the bar weight [kg];

L_{bar} is the bar length [dm];

ρ is the steel density (assumed $7.81kg/dm^3$, according to new Italian Technical code 2018 [27]).

Thus, the equivalent diameter of smooth steel is calculated by the simple equation 3.7:

$$\phi_{eq} = \sqrt{\frac{4 * A_{eq}}{\pi}} \quad (3.7)$$

- *Portion of beam - floor II*

The beam portion, located in the floor I, is built in 1940; it is $L = 3.32$ m long and the cross-section is equal to 30 cm x 60 cm. The steel reinforcement layout is asymmetric along the beam axis, because it is only a portion of entire element: in the right cross-section there are $6\phi14$ (lower reinforcement) and $2\phi10$ (upper reinforcement), whereas the left cross section shows $3\phi14$ (lower reinforcement), $1\phi14$ (central reinforcement) and $2\phi10 + 3\phi14$ (upper reinforcement), see Figure 3.22. It is clear that

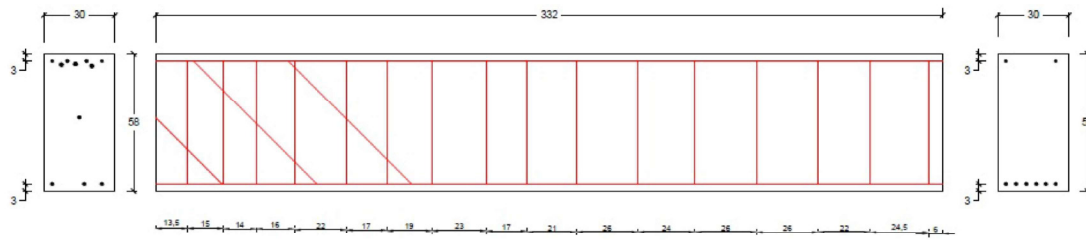


FIGURE 3.22: Geometry and steel reinforcement layout of portion of beam - floor II

some longitudinal rebars change the position along beam axis: usually in the past, the steel reinforcement were optimized according to the stresses (in this case, bending moment and shear). Moreover, the stirrups ($\phi 8$) spacing is variable, from 15 cm to 20 cm.

a. n.10 of concrete compressive tests

After the capacity test (the bending moment test is described in the Section 3.5.4.2), several concrete cores are extracted from the beam. Figure 3.23 shows the sample positions, n.8 on the left side and n.2 on the right side; these positions are chosen according to previous non-destructive test stations (especially related to sonic tomography) and avoiding rebars.

For each concrete core, the colorimetric method is performed, which proved

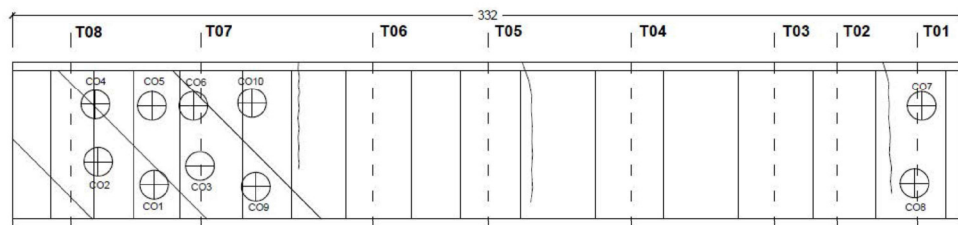


FIGURE 3.23: Position of concrete cores extracted from portion of beam - floor II

that the average of carbonation surface is about 10 cm (see Figure 3.24).

In order to correlate the concrete strength and the ultrasonic pulse velocity, some concrete core are divided in two portions. Concrete core characteristics are shown in Table 3.4. In this case, some samples included steel reinforcement (stirrups and longitudinal bars); however, in order to evaluate the concrete strength evaluation, these disturbance factors are considered. In total n. 17 concrete strengths are estimated and the results are compared with indirect measurements.

b. n.6 of tensile tests on longitudinal bars and n.6 tensile tests on transversal bars

The steel reinforcement bars are extracted after the capacity test. During the



FIGURE 3.24: Phenolphthalein hydroalcoholic solution applied on concrete core *C08* extracted from lower side of portion of beam

Sample code	Weight [g]	Diameter [mm]	H [mm]	H/ϕ [-]	Area [mm^2]	Density [kg/m^3]
C01	3068	94	199	2.11	6939	2227
C02	3215	94	209	2.22	6939	2222
C03	1540	94	101	1.07	6939	2209
C04A	1638	94	101	1.07	6939	2350
C04B	1579	94	98	1.04	6939	2328
C05A	3079	94	199	2.12	6939	2227
C05B	1692	94	111	1.18	6939	2202
C06A	3202	94	203	2.16	6939	2277
C06B	1759	94	116	1.23	6939	2190
C07A	3057	94	194	2.06	6939	2271
C07B	1534	94	97	1.03	6939	2289
C08A	2908	94	186	1.98	6939	2252
C08B	1437	94	96	1.02	6939	2169
C09A	2904	94	190	2.02	6939	2207
C09B	1516	94	97	1.04	6939	2244
C010A	3029	94	196	2.09	6939	2227
C010B	1565	94	98	1.04	6939	2314

TABLE 3.4: Characteristics of concrete samples extracted from portion of beam - floor II

bending test, longitudinal steel reinforcements worked in different ways, according to the strain state of the cross-section. Thus, the same rebar can be differently stressed, according to the position along the beam. For this reason, each steel sample is divided in three parts related to the cross-section beam position: mid-span, intermediate and support.

Moreover, six stirrups are tested: three stirrups located near the mid-span and three near the support.

The steel reinforcement characteristics are shown on Table 3.5. For each rebar,

Steel type	Steel position	ϕ [mm]	ϕ_{eq} [mm]	Weight [kg]
L1	span	14	13.78	0.70
L2	span	14	13.70	0.59
L3	span	14	13.77	0.57
L4	span	14	13.92	0.70
L5	span	14	13.73	0.70
L6	span	14	13.77	0.70
L1	intermediate	14	14.03	0.71
L2	intermediate	14	14.06	0.61
L3	intermediate	14	14.12	0.59
L4	intermediate	14	14.13	0.72
L5	intermediate	14	14.07	0.72
L6	intermediate	14	14.07	0.72
L1	support	14	14.04	0.71
L3	support	14	14.15	0.59
L4	support	14	13.92	0.71
L6	support	14	14.07	0.72
T1	support	8	8.24	0.21
T3	support	8	7.91	0.20
T4	support	8	8.13	0.21
T8	span	8	8.08	0.20
T9	span	8	8.20	0.21
T10	span	8	8.19	0.20

TABLE 3.5: Characteristics of steel samples extracted from portion of beam - floor II

it is shown the diameter, the weight and the equivalent diameter. The rebars located near to mid-span had smaller equivalent diameters than others: probably the deformation related to the bending test influenced the steel reinforcement close to mid-span.

- *Intermediate portion of column - floor II*

The intermediate portion of column, located on floor II, is built in 1957; the cross-section is equal to 29 cm x 29 cm, the longitudinal rebars are 4 ϕ 16 and the helicoidal stirrups are ϕ 6/20. The concrete surface shows evident degradation state: the surface is irregular and slightly damaged.

- a. *n.4 of concrete compressive tests*

Concrete cores are extracted according to non-destructive test stations and the steel reinforcement is avoided. Table 3.6 shows the sample characteristics (e.g. sample code, weight, diameter, height, height/diameter ratio, area, density and compressive force). These data are very different compared to the concrete cores

Sample code	Weight [g]	Diameter [mm]	H [mm]	H/ϕ [-]	Area [mm ²]	Density [kg/m ³]	Force [kN]
C7	3169	94	206	2.19	6939	2216	59
C8	3074	94	200	2.19	6939	2214	51
C9	2994	94	195	2.13	6939	2212	51
C10	1484	94	97	1.10	6939	2204	55

TABLE 3.6: Characteristics of concrete samples extracted from intermediate portion of column - floor II

extracted from the first floor: the density and the compression force show that the concrete quality is very low.

b. n.3 of tensile tests on longitudinal bars and n.3 tensile tests on transversal bars

The steel samples (longitudinal and transversal) are extracted after the concrete core extraction (see Figure 3.25). This phase confirms the low quality of



FIGURE 3.25: Extraction steel reinforcement by means of pneumatic drill

concrete. Table 3.7 shows the longitudinal and transversal steel characteristics.

3.5.1.2 Results

In this subsection, the destructive test results are described.

- *Top portion of column - floor I*

a. n.6 of concrete compressive tests

Concrete core tests are performed on MTS machine testing and the strain-gauges are used to evaluate the deformations. In this way, elastic modulus is determined. Related to the n.6 concrete samples shown in the previous Section

Steel type	ϕ [mm]	ϕ_{eq} [mm]	Weight [kg]
L1 II	16	15.55	1.50
L2 II	16	15.79	1.54
L3 II	16	15.75	1.14
T1 II	6	6.08	0.23
T2 II	6	6.16	0.25
T3 II	6	6.09	0.22

TABLE 3.7: Characteristics of steel samples extracted from intermediate portion of column - floor II

3.5.1.1, Figure 3.26 shows Stress-Deformation ($\sigma - \epsilon$) behaviour for each sample. According to test results, Table 3.8 shows the compressive strength values

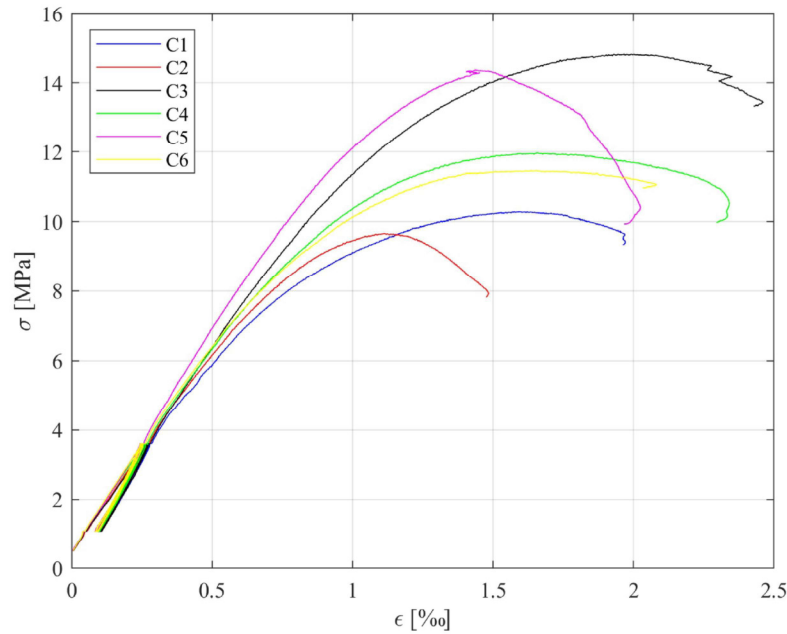


FIGURE 3.26: Stress-Deformation behaviour of concrete samples extracted from top portion of column - floor I

(sample, cylindrical and cubical strength; static e dynamic elastic modulus).

It is noted shows a great variability of concrete strength on the same portion of column (the top portion - floor I). In fact, the coefficient of variation (CoV), related to cylindrical compressive strength, is about 17%; moreover, the mean value is low (≈ 13 MPa).

The compressive concrete tests show a similar elastic static modulus ($E_{c,st}$) but different deformations related to the maximum stress.

Sample code	$f_{c,sam}$ [MPa]	$f_{c,cyl}$ [MPa]	R_c [MPa]	$E_{c,st}$ [MPa]	E_d [MPa]
C1	10.29	10.99	12.78	13939	19593
C2	9.66	10.31	11.99	14609	22980
C3	14.83	15.84	18.42	14316	20880
C4	11.97	12.78	14.86	14515	22115
C5	14.38	15.35	17.85	14992	24093
C6	11.47	12.24	14.23	15390	24002
<i>Mean value</i>		12.92		14627	22277
<i>St. Dev.</i>		2.26		510	1786
<i>CoV</i>		0.17		0.03	0.08

TABLE 3.8: Compressive strength and elastic modulus of core samples extracted from top portion of column - floor I

b. *n.3 of tensile tests on longitudinal bars and n.3 tensile tests on transversal bars*

The tensile test results, about longitudinal and transversal bars, are shown in Table 3.9. Yield (σ_y) and ultimate (σ_u) strengths are evaluated for each rebar;

Steel type	ϕ [mm]	σ_y [MPa]	σ_u [MPa]	ϵ_u [%]	E_s [MPa]
L1	18	359.05	564.94	18	176000
L2	20	330.10	499.13	21	-
L3	20	353.27	428.42	28	-
<i>Mean value</i>		347.47	530.83		176000
<i>St. Dev.</i>		15.32	32.97		-
<i>CoV</i>		0.04	0.06		-
T1	6	343.01	417.95	-	203000
T2	6	331.02	403.80	-	198340
T3	6	332.25	409.21	-	195720
<i>Mean value</i>		335.43	410.32		199020
<i>St. Dev.</i>		6.60	7.14		3687
<i>CoV</i>		0.02	0.02		0.02

TABLE 3.9: Yield and ultimate strengths, ultimate deformation and elastic modulus of steel samples extracted from top portion of column - floor I

moreover, the elastic modulus (E_s) is calculated for all the transversal steel bars and for one longitudinal bar. The ultimate deformation related to the maximum load, is evaluated by the following equation (3.8):

$$\epsilon_u = \frac{\Delta L_{max}}{L_e} * 100 \quad (3.8)$$

where:

ΔL_{max} is the displacement related to the maximum load;

L_e is the length of the measurement (usually $L_e = 5\phi$).

The results shows that the mean values of longitudinal yield and ultimate strengths are higher than transversal ones. In both cases, the variability is low: $CoV < 0.05$ for the yield strengths. The stiffness of the transversal steel bars is higher than longitudinal one.

- *Portion of beam - floor II*

The destructive tests performed on the samples extracted from the beam portion are described.

- a. *n.10 of concrete compressive tests*

Concrete cores are tested on MTS machine and for each sample strain-gauges are applied in order to evaluate the deformations. The Table 3.10 shows the sample ($f_{c,sam}$), cylindrical ($f_{c,cyl}$) and cubical strength (R_c), moreover, the static ($E_{c,st}$) and dynamic (E_d) elastic modulus. The compressive test results show a great variability of concrete strength (in the same element). Mean value of concrete strength is about 12.5 MPa and the coefficient of variation is very high ($\approx 23\%$). However, the mean values of concrete strength and the elastic modulus are comparable with the top portion of column (floor I) results.

According to the non-destructive test results, presented in Section 3.5.2, it is noted that concrete quality is different related to the lower and the upper side of the beam; for this reason, the concrete strengths are taken into account this knowledge. On the lower side, the following samples are extracted: C01, C02, C03, C08A, C08B, C09A and C09B; the mean value of concrete strength is about 11 MPa. Moreover, on the upper side (C04A, C04B, C05A, C05B, C06A, C06B, C07A, C07B, C010A and C010B are considered), the mean value of concrete strength is about 14 MPa; the difference is about 21%. Probably, the concrete casting method wasn't accurate and homogeneous. However, non-destructive tests (in this case sonic tomography) confirmed a good prediction about the concrete quality.

- b. *n.6 of tensile tests on longitudinal bars and n.6 tensile tests on transversal bars*

The longitudinal rebars are evaluated according to different positions: mid-span, intermediate and support. For each steel reinforcement, the yield and ultimate strengths, the ultimate deformation related to the maximum load and the elastic modulus are estimated (see Table 3.11). It is noted that the yield strengths

Sample code	$f_{c,sam}$ [MPa]	$f_{c,cyl}$ [MPa]	R_c [MPa]	$E_{c,st}$ [MPa]	E_d [MPa]
C01	9.52	9.78	11.37	10941	14664
C02	7.89	8.11	9.44	10452	15140
C03	13.34	12.07	14.01	-	17155
C04A	15.88	15.53	18.00	-	20082
C04B	14.87	14.42	16.72	-	17953
C05A	11.84	12.17	14.14	11581	16922
C05B	11.53	10.74	12.47	-	19841
C06A	15.01	15.42	17.88	16734	19683
C06B	11.27	10.63	12.35	-	15776
C07A	13.01	13.37	15.52	19659	24021
C07B	20.78	18.63	21.54	-	21078
C08A	9.55	9.81	11.41	11957	13619
C08B	10.52	9.35	10.87	-	13199
C09A	7.73	7.95	9.25	10813	13871
C09B	11.67	10.45	12.14	-	13695
C010A	11.86	12.19	14.15	17684	22163
C010B	18.70	16.80	19.45	-	20599
<i>Mean value</i>		12.20		14934	17615
<i>St. Dev.</i>		3.08		3756	3370
<i>CoV</i>		0.25		0.25	0.19

TABLE 3.10: Compressive strength and elastic modulus of core samples extracted from portion of beam - floor II

of longitudinal rebars, located in mid-span, are more variable ($CoV > 0.10$) and the mean value is about 293 MPa. This value is lower than the longitudinal steel extracted from the portion of column located in the first floor, see also Table 3.9. The Stress-Deformation behaviours of longitudinal bars, located in mid-span, are shown in Figure 3.27. In this case, longitudinal rebars, located in same position, show different deformations. Moreover, it is noted that elastic modulus of longitudinal steel, located in mid-span, is very variable: starting from the mean value ($E_s \approx 223000$ MPa), the coefficient of variation is about 23%. This result is more evident in the same rebar, as seen for $L1$ sample, the elastic modulus changes according to the steel position: the Stress-Deformation behaviour of sample $L1$ is shown in Figure 3.28, in this case three positions are examined: mid-span, intermediate and support. The sample $L1$ assumes different behaviours according to the position: near the support, the yield stress is lower but deformation is very high ($\epsilon_u \approx 1.4\%$). In latter case, the yield point is more clear than mid-span and intermediate ones.

Figures 3.29 and 3.30 show, for each longitudinal steel sample, the yield and ultimate strength along beam axis, respectively. Figure 3.29 shows that the

Steel type	Steel position	σ_y [MPa]	σ_u [MPa]	ϵ_u [%]	E_s [MPa]
L1	span	338.37	400.14	33	293333
L2	span	332.65	384.69	30	211946
L3	span	261.45	374.13	33	208927
L4	span	282.30	390.70	-	175299
L5	span	273.85	379.03	-	191138
L6	span	271.23	359.74	-	295302
L1	intermediate	279.40	387.56	43	146667
L2	intermediate	239.89	339.56	-	-
L3	intermediate	243.98	345.92	39	-
L4	intermediate	261.05	368.12	44	-
L5	intermediate	239.71	343.98	57	-
L6	intermediate	238.61	327.68	41	-
L1	support	255.00	362.01	43	191304
L3	support	241.48	329.66	46	-
L4	support	297.56	405.63	30	-
L6	support	237.18	341.83	57	-
<i>Mean value</i>	<i>span</i>	293.31	381.41		229324
<i>St. Dev.</i>	<i>span</i>	33.41	13.97		52048
<i>CoV</i>	<i>span</i>	0.11	0.04		0.23
T1	support	374.50	474.74	33	196780
T3	support	344.59	461.59	35	-
T4	support	333.90	431.62	-	-
T8	span	365.95	472.58	28	-
T9	span	370.51	464.45	38	-
T10	span	374.48	472.14	-	-
<i>Mean value</i>		360.65	462.85		196780
<i>St. Dev.</i>		17.22	16.14		-
<i>CoV</i>		0.05	0.03		-

TABLE 3.11: Yield and ultimate strengths, ultimate deformation and elastic modulus of steel samples extracted from portion of beam - floor II

yield strengths are higher in mid-span than intermediate and support positions. Whereas, the ultimate strengths (see Figure 3.30) are not more variable related to the steel positions.

Table 3.11 shows the yield and ultimate strengths of the transversal steel: both the mean values are higher than longitudinal steel strengths. The variability of these strengths is the variability along beam axis, see Figure 3.31. The coefficient of variation is about 5% (a common value for steel reinforcement in the new structure, as described in fib Bulletin 80 [37]).

However, the yield and ultimate transversal steel strengths are higher than the transversal steel extracted from the portion of column (floor I); indeed, the elastic moduli are comparable.

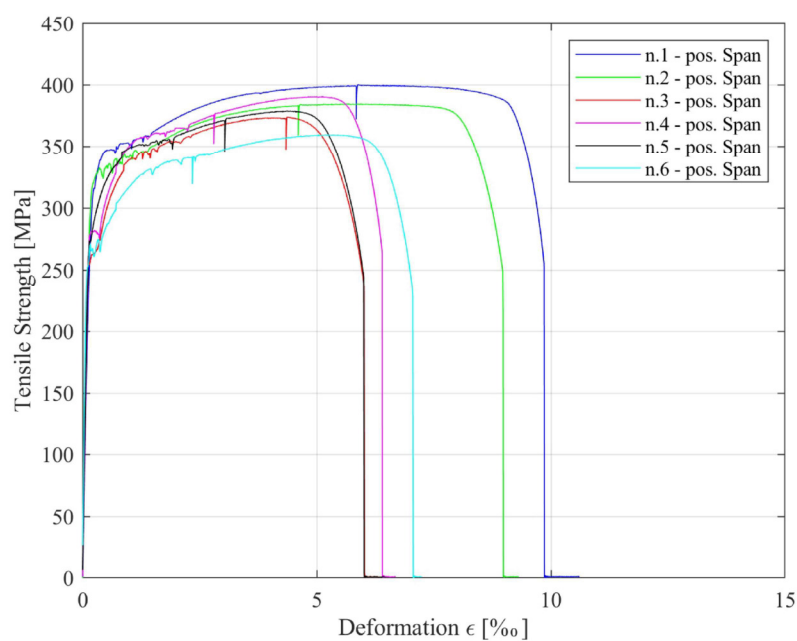


FIGURE 3.27: Tension-Deformation behaviour of steel samples extracted from portion of beam (span) - floor II

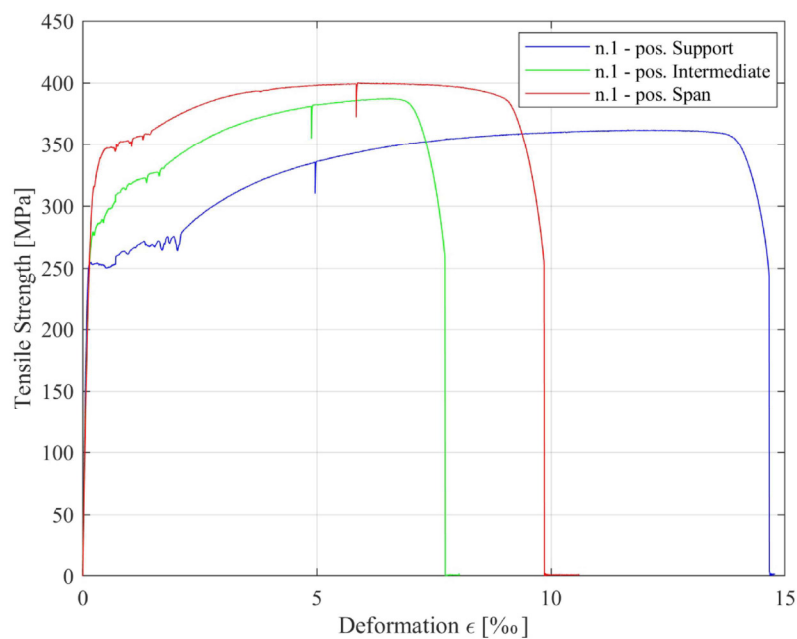


FIGURE 3.28: Stress-Deformation behaviour of steel sample L1 extracted from portion of beam - floor II, according to positions: span, intermediate and support

- *Intermediate portion of column - floor II*

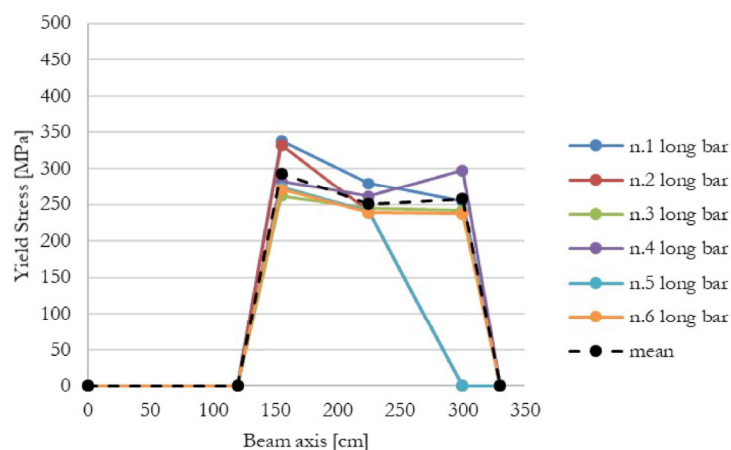


FIGURE 3.29: Yield strengths of longitudinal steel samples extracted from portion of beam - floor II, according to position: span, intermediate and support

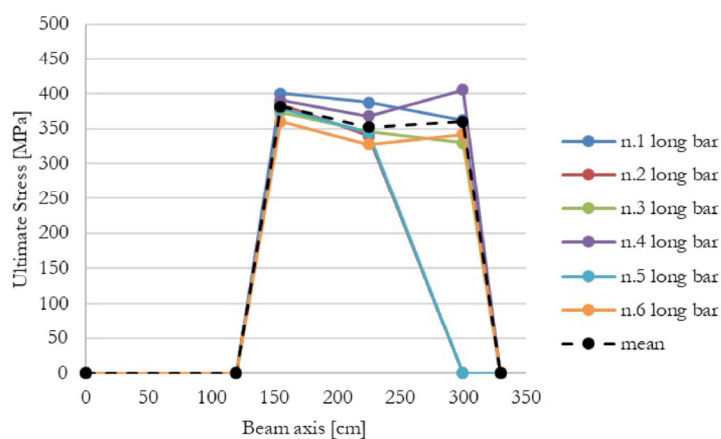


FIGURE 3.30: Ultimate strengths of longitudinal steel samples extracted from portion of beam - floor II, according to position: span, intermediate and support

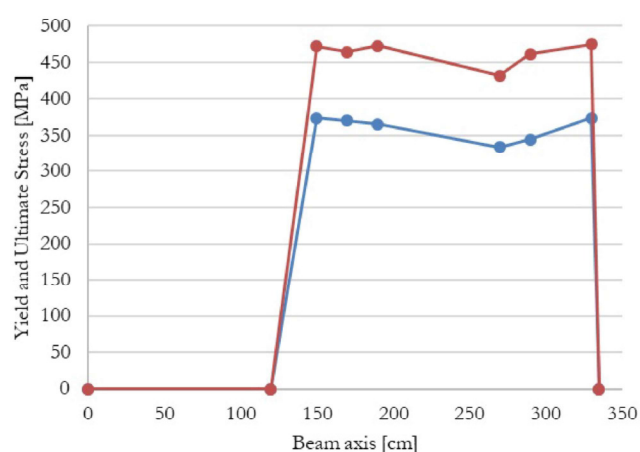


FIGURE 3.31: Yield and ultimate strengths of transversal steel samples extracted from portion of beam - floor II, according to position: span and support

a. *n.4 of concrete compressive tests*

Concrete compressive tests are performed on MTS machine and the strain-gauges are applied on three samples. The Stress-Deformation behaviours are shown in Figure 3.32. It is noted that the samples *C8* and *C9* lead to equal

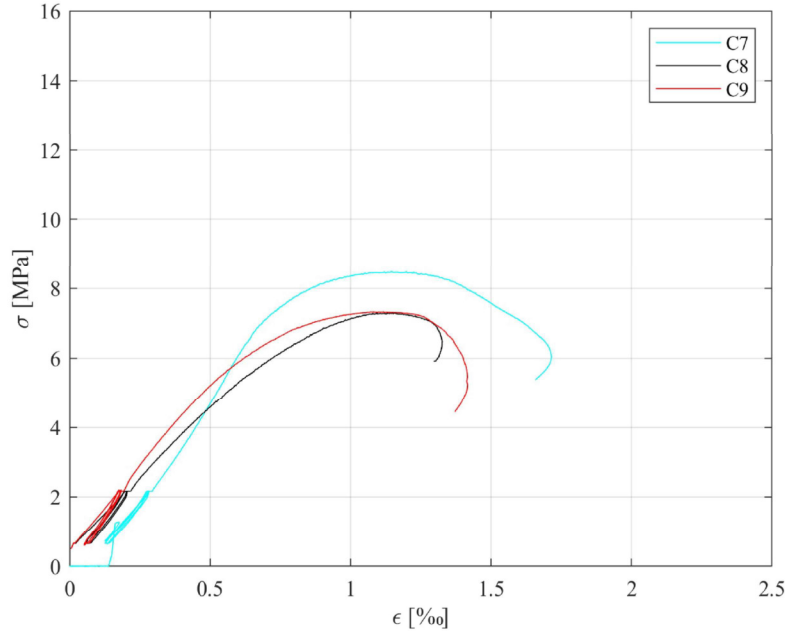


FIGURE 3.32: Stress-Deformation behaviour of concrete samples extracted from intermediate portion of column - floor II

strength, whereas the concrete strength of *C7* is slightly higher. Sample, cylindrical and cubical strength, static and dynamic elastic modulus are listed in Table 3.12. It is noted the smaller variability of the concrete strength ($CoV \approx 7\%$)

Sample code	$f_{c,sam}$ [MPa]	$f_{c,cyl}$ [MPa]	R_c [MPa]	$E_{c,st}$ [MPa]	E_d [MPa]
C7	8.48	9.06	10.53	9846	12557
C8	7.31	7.80	9.07	10578	14652
C9	7.33	7.83	9.10	12297	12847
C10	7.96	7.52	8.75		12358
<i>Mean value</i>		8.05		10907	13104
<i>St. Dev.</i>		0.56		1258	1052
<i>CoV</i>		0.07		0.12	0.08

TABLE 3.12: Compressive strength and elastic modulus of core samples extracted from intermediate portion of column - floor II

than those of portion of beam and column (floor I). Moreover, the static and dynamic elastic modulus are very low.

b. n.3 of tensile tests on longitudinal bars and n.3 tensile tests on transversal bars

Finally, the results of the tensile tests are listed in Table 3.13. Here, it is noted

Steel type	ϕ [mm]	σ_y [MPa]	σ_u [MPa]	ϵ_u [%]	E_s [MPa]
L1 II	16	309.66	438.30	18	191980
L2 II	16	290.85	419.51	21	-
L3 II	16	295.82	422.25	28	-
<i>Mean value</i>		298.78	426.69		191980
<i>St. Dev.</i>		9.75	10.15		-
<i>CoV</i>		0.03	0.02		-
T1 II	6	409.22	544.75	-	193120
T2 II	6	359.94	539.18	-	173150
T3 II	6	413.85	551.12	-	-
<i>Mean value</i>		335.43	545.02		183135
<i>St. Dev.</i>		6.60	5.97		-
<i>CoV</i>		0.02	0.01		-

TABLE 3.13: Yield and ultimate strengths, ultimate deformation and elastic modulus of steel samples extracted from intermediate portion of column - floor II

low variability of the longitudinal and transversal steel strengths ($CoV \approx 0.03$). The yield and ultimate strengths of the transversal steel is higher than longitudinal one. However, the elastic modulus is comparable.

Thus, the destructive tests show very low material strengths on examined elements (probably for this reason the school was destroyed). The concrete compressive tests, performed in total on 20 samples, are different comparing floor I and floor II, probably because they are built in different time. However, the recent one (floor II and especially the intermediate portion of column) shows low values of the concrete strength (the mean value is about 7.5 MPa), whereas on floor II (portion of column and beam) the mean values of concrete strength is about (12.5 MPa) compared them: the different is about 40%. Moreover, considering all elements, the mean value of the coefficient of variation is equal to 15%.

The tensile tests of steel reinforcement proved that the mean value of the longitudinal steel yield strength is about 315 MPa (n. 12 elements are tested) and the coefficient of variation is equal to 6%. Whereas, $f_y = 365$ MPa is the mean value of the transversal steel yield strength ($CoV \approx 0,05$).

Finally, the mean values of elastic modulus are $E_c \approx 14000$, for concrete and $E_s \approx 196000$, for steel.

3.5.2 Non-destructive tests for concrete: description and results

In this Section non-destructive tests, performed on all elements presented above, are described and analysed. Rebound, ultrasonic, tomography and pull-out tests are evaluated according to specific element and specific positions. The following elements, extracted from an Italian school building, are evaluated by non-destructive tests:

- *Top portion of column - floor I*
- *Portion of beam - floor II*
- *Bottom portion of column - floor II*
- *Intermediate portion of column - floor II*
- *Top portion of column - floor II*

For each element, modality and characteristics of non-destructive tests are described in this subsection. Usually, non-destructive tests are performed before destructive tests, but in some case (i.e. ultrasonic pulse velocity), non-destructive tests are performed on concrete cores to evaluate reliable correlation between direct and indirect measurements (herein, ultrasonic tests are performed on concrete cores extracted from portion of beam).

In this work, several ultrasonic tests are performed on all elements: n.13 ultrasonic test stations on portions of column and n.11 ultrasonic test stations on portion of beam. Technical equipment, used in this research, is shown in Figure 3.33, *SolGeo CMS ultrasonic device - BOVIAR* (Integrated systems for diagnostics and monitoring).

In addition, pull-out tests are performed according to concrete core positions: before



FIGURE 3.33: SolGeo CMS ultrasonic device - BOVIAR

drilling concrete core sample, n. 10 inserts post-inserted were extracted (see Figure 3.34).

However, by means of specific instrumental certificate, extraction force F [kN] can be



FIGURE 3.34: Pull-out test: on left side, testing execution; on right side, concrete sample extracted

calculated according to extraction pressure p [bar]. The equation 3.9 is related to instrumentation manufacturer, thus extraction force F [kN] is:

$$F = a * p^2 + b * p + c \quad (3.9)$$

where:

$$a = 1.977E-5;$$

$$b = 3.396E-1;$$

$$c = -2.122E-2;$$

p is the extraction pressure [bar];

Moreover, instrumentation manufacturer provides a relationship between extraction force F and cubical compressive concrete strength (see Figure 3.35). Thus, this relationship is expressed by:

$$1kN = 0.94MPa \quad (3.10)$$

with determination coefficient $R^2 = 0.93$.

This equation is determined correlating extraction tests and compressive tests (with $H/D = 1$) extracted on same points.

- *Top portion of column - floor I*

Geometry and other characteristics related to top portion of column, located on floor I, are described in Section 3.5.1.1.

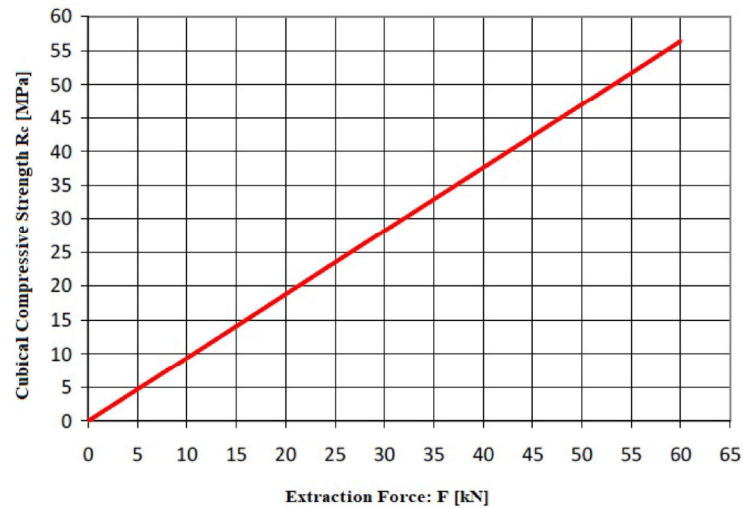


FIGURE 3.35: Relationship between cubical compressive strength and extraction force - BOVIAR

a. Rebound test

According to the International Standard, rebound test have to be performed on two opposed element surfaces, at the same height and for a minimum of 9 measurements. For this reason, two couple of opposed surfaces are analysed: A-C and B-D. For each surface, the measurements are conducted on two stations: up (UP) and down (DN) side. Rebound measurements, calculated for each surface, are shown in Table 3.22. Considering the entire element, mean value of rebound measurements is $I_R = 37$.

b. Ultrasonic test

Four ultrasonic stations (ST1, ST2, ST3 and ST4) are evaluated on top portion of column (floor I), performed for transparency way. For this reason, two couples of surface are investigated: A-C and B-D. Mean values of ultrasonic pulse velocity, performed for each station on two directions (A-C and B-D), are shown in Table 3.14. The results show a not so bad quality of concrete ($3000\text{m/s} < V_{ultr} < 3600\text{m/s}$) and low coefficient of variation equals to 7%.

It is noted that near extreme surface (ST1 and ST4) ultrasonic pulse velocities decrease and probably during the cutting phases of the sample (portion of column) concrete quality is compromised and micro-cracks are generated.

c. Sonic tomography

Sonic tomography test is applied in the same stations that are used to measure ultrasonic pulse velocity. Thus, the stations ST1, ST2, ST3 and ST4 are shown

Station	Section	V_{ultr} [m/s]
ST1	A-C	3237
ST1	B-D	2921
ST2	A-C	3419
ST2	B-D	3019
ST3	A-C	3242
ST3	B-D	3047
ST4	A-C	2885
ST4	B-D	2785
<i>Mean value</i>		3069
<i>St. Dev.</i>		214
<i>CoV</i>		0.07

TABLE 3.14: Ultrasonic pulse velocity of top portion of column - floor I

in Figures 3.36 and 3.37 which present equal ultrasonic velocities maps. It is

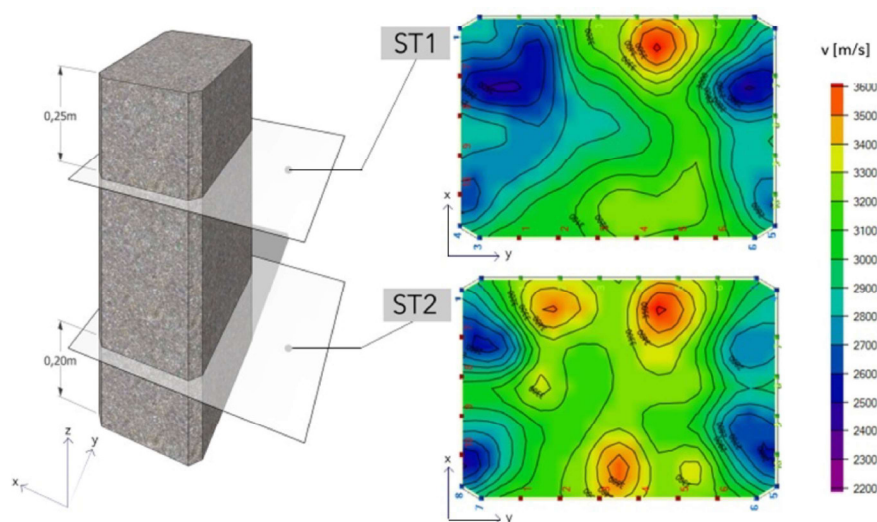


FIGURE 3.36: Sonic tomography tests on top portion of column - floor I: station ST1 ($x=25$ cm) and station ST2 ($x=70$ cm)

noted, in the same map, an ultrasonic pulse velocity variability from 2400 m/s to 3600 m/s: that means an improvement from poor concrete quality to not bad quality. The surface near the extreme boundary (i.e. ST4) shows lower velocities than other ones, so the cutting phase element probably causes a disturbance (e.g. micro-cracks) and ultrasonic velocities decreasing as a consequence.

d. Pull-out test

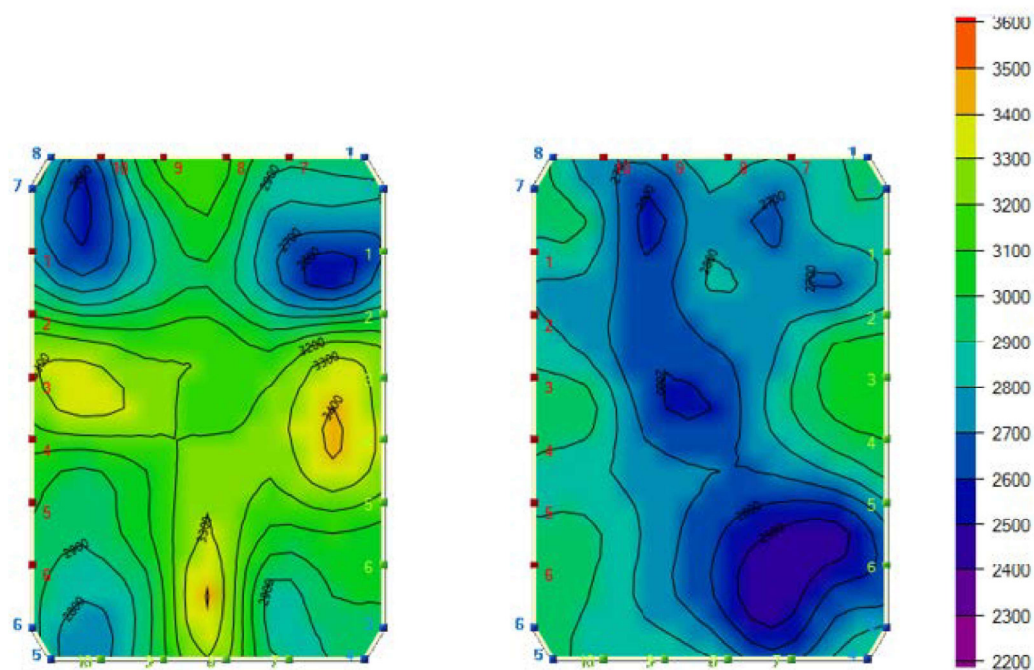


FIGURE 3.37: Sonic tomography tests on top portion of column - floor I: on left side, station ST3 ($x=50$ cm) and on the right side, station ST4 ($x=80$ cm)

Before extracting concrete cores, pull-out tests are performed on the same position (see Section 3.5.1.2); the results are shown in Table 3.15.

The results show a mean value lower than the cylindrical compressive strength

Sample code	Pressure [bar]	$F_{extr.}$ [kN]	R_c [Mpa]	$f_{c,cyl}$ [MPa]
C1	32.7	11.11	10.44	8.98
C2	42.6	14.48	13.61	11.71
C3	46.5	15.81	14.87	12.78
C4	37.5	12.74	11.98	10.30
C5	56.2	19.13	17.98	15.46
C6	35.5	12.06	11.34	9.75
Mean value				11.50
St. Dev.				2.38
CoV				0.21

TABLE 3.15: Pull-out tests on top portion of column - floor I

evaluated by direct testing (12.92 MPa). However, coefficient of variation is very high ($CoV > 20\%$).

- *Portion of beam - floor II*

The geometry and the reinforcement details of beam portion are described in Section

3.5.1.1. Non-destructive tests performed on this element, are described and analysed following.

a. *Rebound test*

On the portion of beam, rebound test is performed on two surfaces (A and C). Five stations are investigated to test rebound measurements and the results are almost constant along the beam axis; the mean value of rebound index is about $I_R = 30$.

b. *Ultrasonic test*

Eleven ultrasonic test stations are evaluated on a beam portion; for each station, mean values of ultrasonic pulse velocity are calculated according to three layers: bottom, intermediate and upper region. The results are shown in Table 3.16. They prove a great variability of concrete quality on the same element,

Station	V_{upper} [m/s]	$V_{inter.}$ [m/s]	V_{bottom} [m/s]
ST01	3590	2986	2659
ST02	3564	3278	2410
ST03	3620	3066	2611
ST03b	3577	3050	2688
ST04	3575	3037	2842
ST05	3362	2968	2926
ST05b	3449	2836	2813
ST06	3245	2925	2742
ST06b	3344	2986	2718
ST07	3139	3027	2597
ST08	2946	2955	2590
<i>Mean value</i>	3401	3010	2691
<i>St. Dev.</i>	219	110	142
<i>CoV</i>	0.06	0.04	0.05

TABLE 3.16: Ultrasonic pulse velocities of portion of beam - floor II

especially according to the three layers evaluated. On the bottom side, mean value of ultrasonic pulse velocity is very low (if $2135 \text{ m/s} < V_{ultr} < 3000 \text{ m/s}$), so it means poor concrete quality; whereas, on the upper side the mean value of ultrasonic pulse velocity is higher (if $3000 \text{ m/s} < V_{ultr} < 3600 \text{ m/s}$), that means not bad concrete quality. However, for each layer the coefficient of variation is low ($CoV \approx 5\%$); obviously, considering the entire beam coefficient of variation increases: it is about 11%.

Previous ultrasonic pulse velocities are evaluated also along beam axis (see Figure 3.38). Here, it is noted that the upper, intermediate and bottom layers show

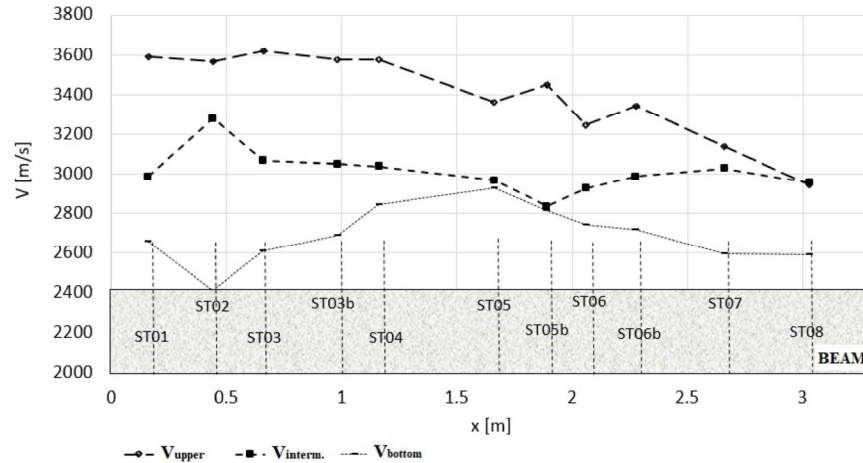


FIGURE 3.38: Ultrasonic pulse velocities along portion of beam axis - floor II

different ultrasonic pulse velocities. All the layers show higher velocities on the left side and lower velocities on the right side (with lower difference among three layers). As already seen in rebound test results, on the left side of beam the variability of measurements is higher.

On the beam portion, ultrasonic tests were performed even on the extracted concrete cores (see Figure 3.39); in this way, direct correlation, e.g. concrete strength-ultrasonic pulse velocity, can be carried out (see Section ??).

The results of ultrasonic pulse velocities on concrete cores are shown in Table

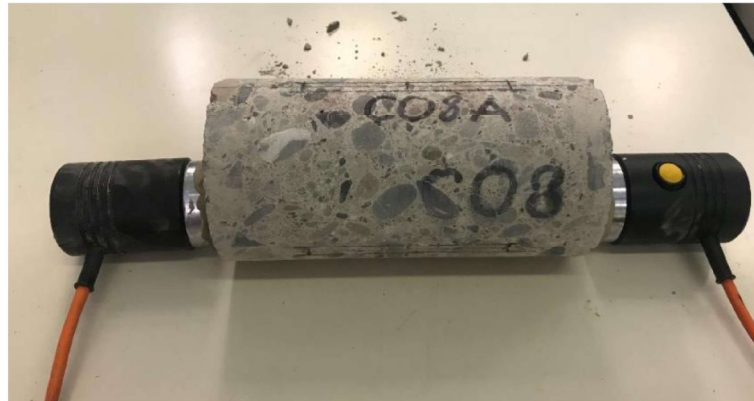


FIGURE 3.39: Ultrasonic test on concrete core extracted from portion of beam axis - floor II

3.17. Considering all the samples, the mean value of ultrasonic pulse velocity shows poor quality of concrete (e.g. $2135 \text{ m/s} < V_{ultr} < 3000 \text{ m/s}$); however, dividing samples according to the upper and bottom side of element, a significant difference of result is obtained. On the upper side *C04A*, *C04B*, *C05A*, *C05B*, *C06A*, *C06B*, *C07A*, *C07B*, *C010A* and *C010B* are considered, mean value of ultrasonic pulse velocities is 3108 m/s ($CoV = 0.06$); whereas,

<i>Sample code</i>	V_{ultr} [m/s]	E_d [MPa]
C01	2705	14664
C02	2752	15140
C03	2938	17155
C04A	3082	20082
C04B	2928	17953
C05A	2905	16922
C05B	3165	19841
C06A	3100	19683
C06B	2796	15776
C07A	3429	24021
C07B	3200	21078
C08A	2593	13619
C08B	2553	13199
C09A	2642	13871
C09B	2605	13695
C010A	3324	22163
C010B	3146	20599
<i>Mean value</i>	2933	17615
<i>St. Dev.</i>	271	3370
<i>CoV</i>	0,09	0.19

TABLE 3.17: Ultrasonic pulse velocities and dynamic elastic modulus of core samples extracted from portion of beam - floor II

on the bottom side the samples $C01$, $C02$, $C03$, $C08A$, $C08B$, $C09A$ and $C09B$ are considered and mean value of ultrasonic pulse velocities is $2684m/s$ ($CoV = 0.05$). Obviously, ultrasonic pulse velocities on concrete cores are performed after bending moment test; for this reason ultrasonic pulse velocities shown in Table 3.17 are lower than those shown in Table 3.16.

c. Sonic tomography

According to ultrasonic measurement stations, the sonic tomography tests are performed on the beam portion. Figure 3.40 shows eleven tomography maps which represent equal velocity curves. Sonic tomography test shows different values of velocity according to the upper and bottom side of beam. The velocities on the bottom side are very low, especially in the mid-span; whereas, velocities decrease near the support. It seems that velocities represent bending moment stresses. In fact, bending moment stress causes micro-cracks in that parts.

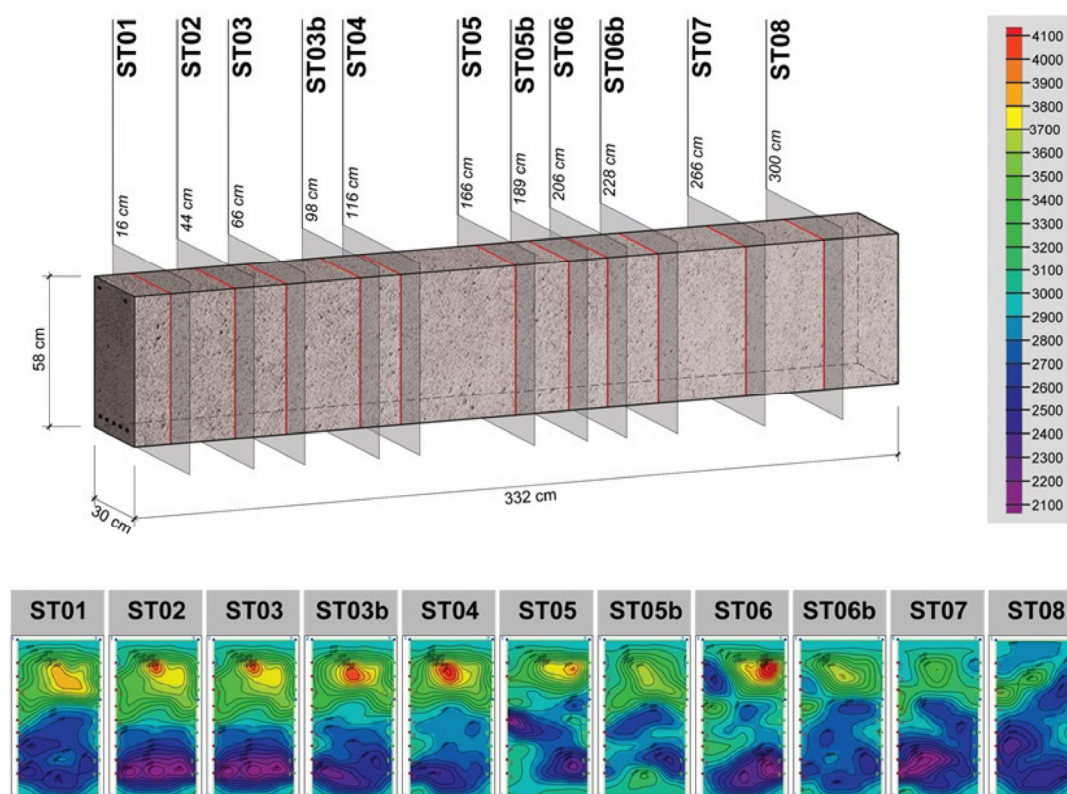


FIGURE 3.40: Sonic tomography test performed on portion of beam axis - floor II

- *Bottom portion of column - floor II*

The second floor is built later compared to the rest of building. For this reason, the bottom portion of column (floor II) presents the anchor steel reinforcement related to the bottom element (see Figure 3.41). However, the cross-section is equal to 29 cm x



FIGURE 3.41: Bottom surface of bottom portion of column located on floor II, anchor steel reinforcement detail

29 cm, $4\phi 16$ are longitudinal bars and helicoidal stirrups are equal to $\phi 6/20$.

a. Rebound test

In this element, two couple of opposed surfaces are analysed: A-C and B-D. For each surface, the measurements are conducted on only one station: down (DN) side (in the other parts, no regular measurement station is available). Rebound measurements, calculated for each surface, are shown in Table 3.23. Considering the entire element, mean value of rebound measurements is $I_R = 31$.

b. Ultrasonic test

Two ultrasonic stations (ST10 and ST11) are evaluated on the bottom portion of column (floor II), performed for transparency way. For this reason, two couples of surface are investigated: A-C and B-D. Mean values of ultrasonic pulse velocity, performed for each station on two directions (A-C and B-D), are shown in Table 3.18. The results show not bad quality of concrete (3000 m/s

Station	Section	V_{ultr} [m/s]
ST10	A-C	2882
ST10	B-D	3101
ST11	A-C	3055
ST11	B-D	3019
<i>Mean value</i>		3014
<i>St. Dev.</i>		94
<i>CoV</i>		0.03

TABLE 3.18: Ultrasonic pulse velocity of bottom portion of column - floor II

$< V_{ultr} < 3600$ m/s) and a very low coefficient of variation, 3%. However, as described above, in this portion of column the anchor reinforcement are added to longitudinal rebars: ultrasonic pulse velocity can be increased according to the high quantity of steel.

c. Sonic tomography

Sonic tomography tests are performed according to previous station (ST10 and ST11); equal velocity maps are shown in Figure 3.42. It is observed that ST11 presents very high velocities in the lower right corner, because the anchor reinforcement causes probably the ultrasonic pulse velocities increasing. It is confirmed by Figure 3.41. Obviously, the measured high velocities change the average of section.

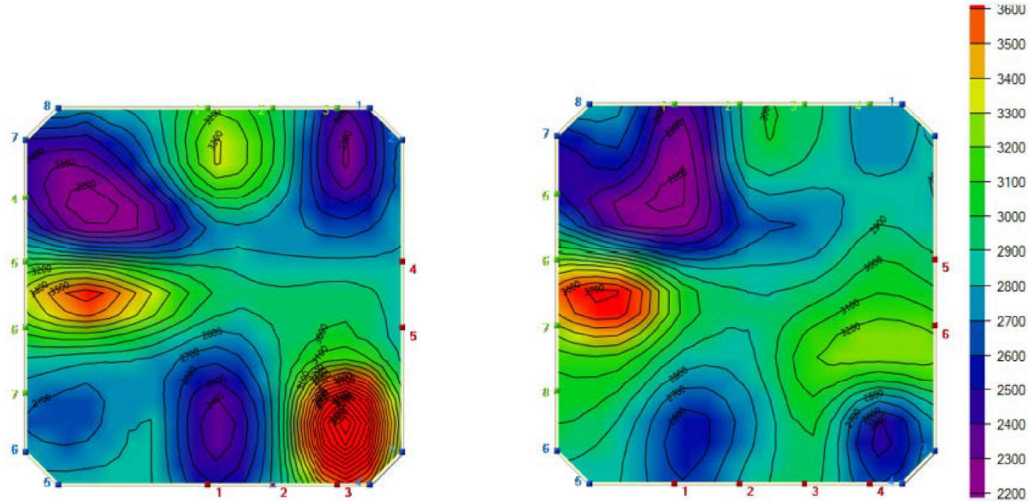


FIGURE 3.42: Sonic tomography tests on bottom portion of column - floor II: on left side, station ST11 ($x=25$ cm) and on the right side, station ST10 ($x=45$ cm)

- *Intermediate portion of column - floor II*

This element is presented in Section 3.5.1.1 which describes his geometry and reinforcement characteristics.

a. Rebound test

As seen in the top portion of column (floor I), two couples of opposed surface have to be analysed: A-C and B-D. For each surface, the measurements are conducted on two stations: up (UP) and down (DN) side. However, rebound measurements are shown in Table 3.25. In this case, considering the entire element, the mean value of rebound measurements is equal to $I_R = 31$ too.

b. Ultrasonic test

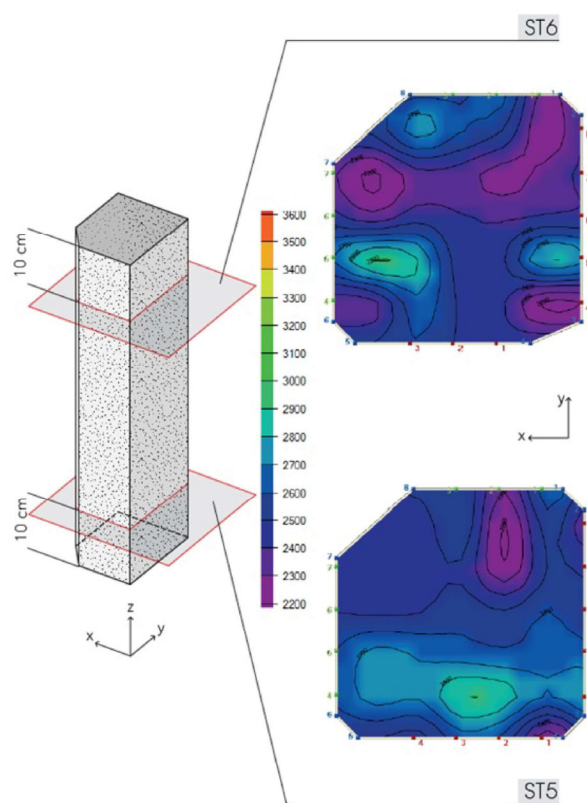
Four ultrasonic stations (ST5, ST6, ST7 and ST8) are evaluated on the intermediate portion of column (floor II), performed for transparency way. For this reason, two couples of surface are investigated: A-C and B-D. Mean values of ultrasonic pulse velocity, performed for each station on two directions (A-C and B-D), are shown in Table 3.19. The results show poor quality of concrete ($2135 \text{ m/s} < V_{ultr} < 3000 \text{ m/s}$) and a very low coefficient of variation, that is 3%.

c. Sonic tomography

The intermediate portion of column, located on floor II, presents damaged surface; the latter region was excluded from tomography analysis. However, according to previous stations, four sonic tomography tests are performed. Equal velocity maps are shown in Figures 3.43 and 3.44. It is noted that ultrasonic

Station	Section	V_{ultr} [m/s]
ST5	A-C	2607
ST5	B-D	2611
ST6	A-C	2522
ST6	B-D	2458
ST7	A-C	-
ST7	B-D	2491
ST8	A-C	-
ST8	B-D	2431
<i>Mean value</i>		2532
<i>St. Dev.</i>		77
<i>CoV</i>		0.03

TABLE 3.19: Ultrasonic pulse velocity of intermediate portion of column - floor II

FIGURE 3.43: Sonic tomography tests on intermediate portion of column - floor II: station ST5 ($x=10$ cm) and station ST6 ($x=75$ cm)

velocities are very low, from 2200 m/s to 2900 m/s; it means poor concrete quality. However, ultrasonic pulse velocities are very homogeneous.

d. Pull-out test

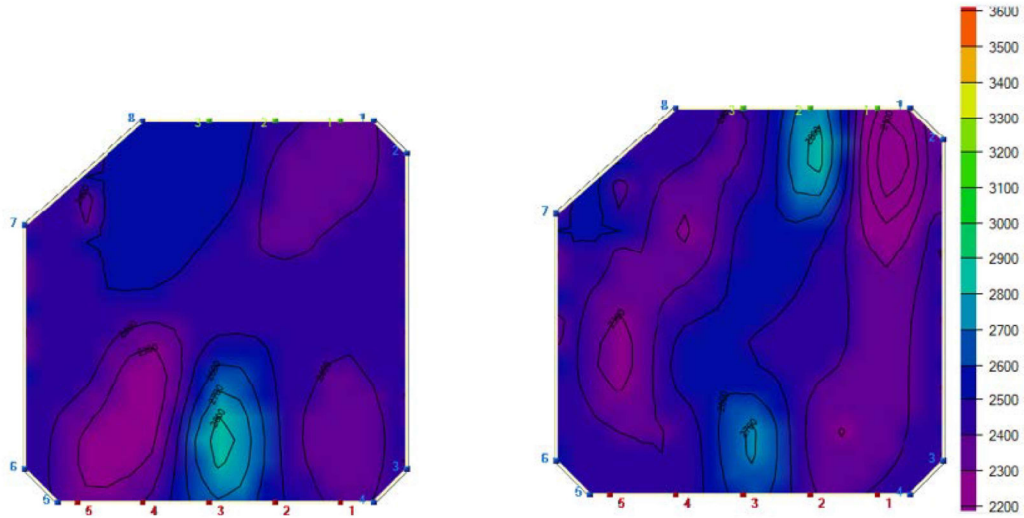


FIGURE 3.44: Sonic tomography tests on intermediate portion of column - floor II: on left side, station ST7 ($x=50$ cm) and on the right side, station ST8 ($x=25$ cm)

As seen on the top portion of column (floor I), also in this element pull-out tests are performed on the same positions used to extract concrete cores. In this way, direct correlation is carried out; however, pull-out test results are shown in Table 3.20.

Sample code	Pressure [bar]	$F_{extr.}$ [kN]	R_c [Mpa]	$f_{c,cyl}$ [MPa]
C7	19.6	6.64	6.24	5.30
C8	27.2	9.23	8.68	7.46
C9	25.2	8.55	8.04	6.91
C10	16	5.42	5.09	4.38
<i>Mean value</i>				6.01
<i>St. Dev.</i>				1.42
<i>CoV</i>				0.24

TABLE 3.20: Pull-out tests on intermediate portion of column - floor II

Here, the results are lower with respect to the results of direct compressive tests; probably the pull-out test results can be influenced by the poor quality of the concrete surface. It is confirmed also according to very high variability ($CoV = 24\%$).

- *Top portion of column - floor II*

Obviously, the top portion of column, located on floor II, presents the same geometry and characteristics of the intermediate and bottom portions.

a. Rebound test

As seen for other portions of column, two couples of opposed surface are analysed: A-C and B-D. For each surface, the measurements are conducted on two station: up (UP) and down (DN) side. However, surface C (UP) is resulted irregular and it wasn't possible to perform rebound test. However, rebound measurements, calculated for each surface, are shown in Table 3.24. Considering the entire element, mean value of rebound measurements is equal to $I_R = 33$.

b. Ultrasonic test

Three ultrasonic stations (ST12, ST13 and ST14) are evaluated on the top portion of column (floor II), performed for transparency way. For this reason, two couples of surface are investigated: A-C and B-D. Mean values of ultrasonic pulse velocity, performed for each station on two directions (A-C and B-D), are shown in Table 3.21. The results show poor quality of concrete (2135 m/s

Station	Section	V_{ultr} [m/s]
ST12	A-C	2541
ST12	B-D	2643
ST13	A-C	2550
ST13	B-D	2605
ST14	A-C	2596
ST14	B-D	2810
<i>Mean value</i>		2630
<i>St. Dev.</i>		109
<i>CoV</i>		0.04

TABLE 3.21: Ultrasonic pulse velocity of top portion of column - floor II

$< V_{ultr} < 3000$ m/s) and a very low coefficient of variation, that is 4%. It can be proved that the ultrasonic pulse velocities performed on the bottom column portion are influenced by anchor reinforcements.

c. Sonic tomography

The stations ST12, ST13 and ST14 are investigated also through sonic tomography. External surfaces of element show advanced degradation state (as seen in Figure 3.19); however, equal velocity maps, related to previous stations, are shown in Figures 3.45 and 3.46. Sonic tomography maps show low velocities, from 2200 m/s to 3100 m/s; this velocity variability is comparable with other portions of column (the intermediate and the bottom one). However, these results confirm poor quality of concrete; in the station ST13 purple region can

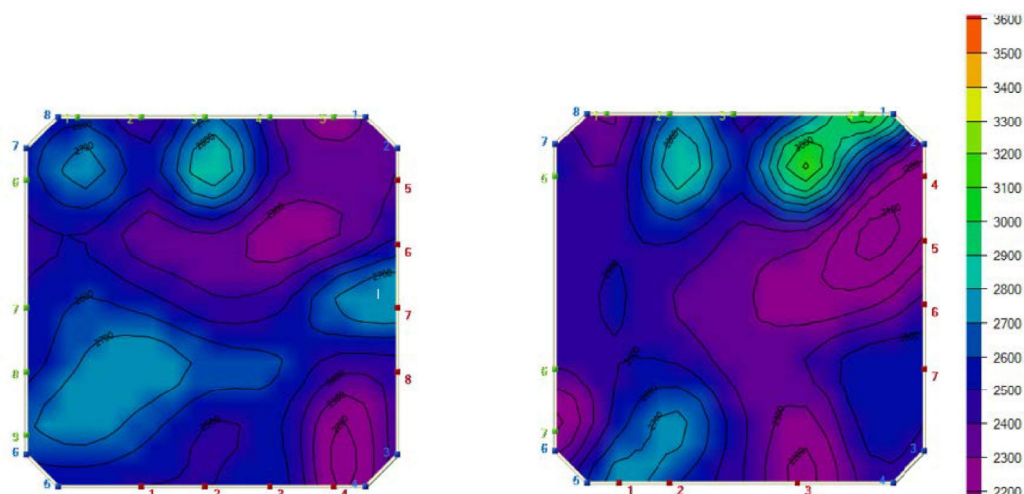


FIGURE 3.45: Sonic tomography tests on top portion of column - floor II: on left side, station ST12 ($x=15$ cm) and on the right side, station ST13 ($x=35$ cm)

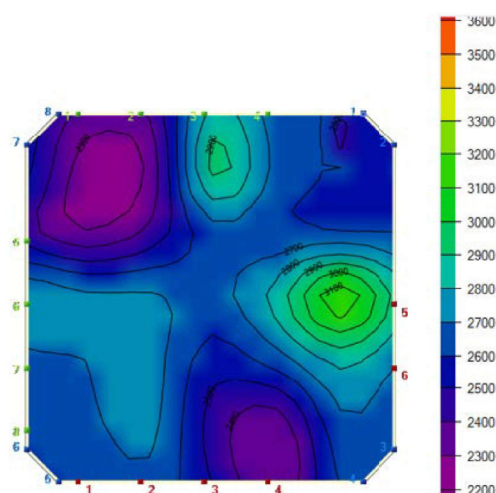


FIGURE 3.46: Sonic tomography tests on top portion of column - floor II: station ST14 ($x=65$ cm)

represent micro-crack area.

Several non-destructive tests are performed on all elements. Rebound test shows a great variability of results and it doesn't provide reliable mechanical information: only surface state is examined.

Many ultrasonic pulse velocity stations are investigated on the examined elements, moreover ultrasonic tests are performed also on concrete core samples. In this way, direct correlation between concrete strength and ultrasonic pulse velocity is analysed.

Ultrasonic test results show a great variability related to the elements, not bad quality of concrete is shown on the top portion of column (floor I) and poor concrete quality is observed on the portions of column located on floor II. The latter case presents an high variability of the concrete quality, especially on the bottom portion better results are observed

(see Figure 3.47). Probably, the ultrasonic pulse velocities, performed in this element, can

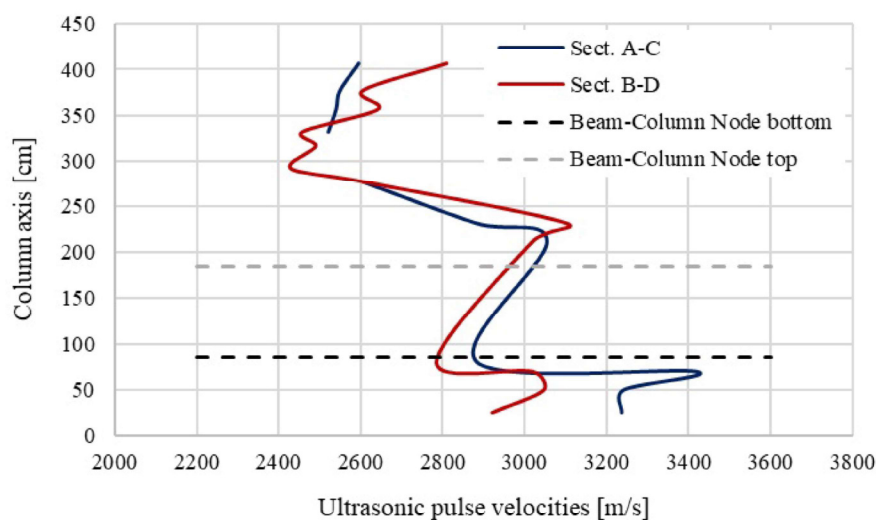


FIGURE 3.47: Ultrasonic pulse velocities along portions of column - floor I and II

be increased by the anchor reinforcement located on the bottom portion of column.

Axial compressive tests, presented in Section 3.5.4.1, will show the real differences of mechanical properties.

Moreover, the beam portion shows different ultrasonic pulse velocity according to the three layers (bottom, intermediate and upper) evaluated. Probably, usual deformation of beam entails micro-cracks on the bottom side; differently, in the past the casting modalities are not accurate and the concrete is not homogeneous.

Surface	Rebound index (I_R) - Top portion of column - floor I																				Mean values
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
A (UP)	36	32	34	34	38	43	31	44	37	33	34	37	34								37
A (DN)	40	38	34	35	40	38	37	37	37	46	40										35
B (UP)	32	39	44	33	35	30	40	33	36	31	31	31	31								38
B (DN)	38	34	31	39	41	39	29	31	41	32	34	33	35								37
C (UP)	36	46	34	37	35	34	34	37	38	33	34	32	51	40	41	40	37	35	36		35
C (DN)	40	36	34	33	31	34	39	31	33	47	31	36	43	38	43	48	39				36
D (UP)	38	34	39	42	41	44	51	37	43	40	36	36									36
D (DN)	35	39	39	39	38	41	34	32	33												39
																					37

TABLE 3.22: Rebound measurements (N) - Top portion of column - floor I

Surface		Rebound index (I_R) - Bottom portion of column - floor II																						Mean values	
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22		23
A (DN)		32	29	26	30	31	26	26	28	30	29	36	28	27	31	31	35	30	32	34	32	36	28	29	30
B (DN)		38	31	31	30	31	30	29	30	27	33	31	30	33	28	29	33	38	33	31	35	32	30	29	31
C (DN)		22	33	32	31	32	32	34	32	30	30	32	31	32	32	32	32	35	32	30	30	36	32	34	32
D (DN)		36	30	34	34	30	30	28	30	33	40	33	42	27	27	34	34	32	29	32	28	30	40	33	32

TABLE 3.23: Rebound measurements (N) - Bottom portion of column - floor II

Surface	Rebound index (I_R) - Top portion of column - floor II																				Mean values	
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20		21
A (UP)	34	34	32	32	38	30	30	37	31	32	36	34	39	31	31	33	32	29				33
A (DN)	34	33	31	33	33	30	32	34	29	31	31	30	28	31	36	32	27	35	29	31	34	32
B (UP)	42	37	34	29	29	28	29	30	28	34	29	33	32	33								32
B (DN)	32	33	29	31	34	29	31	33	31	28	33	33	29									31
C (DN)	30	34	31	30	30	29	31	29	29	30	30	29	28	28	29	27	29	29	29			30
D (UP)	30	27	31	30	32	33	29	30	28													30
D (DN)	29	29	32	31	29	29	29	26	28	30	26	29	30	30	34	30	35					30

TABLE 3.24: Rebound measurements (N) - Top portion of column - floor II

Surface	Rebound index (I_R) - Intermediate portion of column - floor II																				Mean values	
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20		21
A (UP)	27	27	25	28	29	28	27	34	33	27	27	28										28
A (DN)	26	30	30	28	30	33	36	36	35	33	33	30	30									32
B (UP)	28	32	38	32	31	28	28	29	32	34	35											32
B (DN)	28	32	30	34	31	27	29	28	29	30	30	27	35	36								30
C (UP)	32	32	29	32	30	30	31	30	32	28	30	34	29	32	32	32	32	32	32	36	36	32
C (DN)	30	28	28	32	28	30	29	30	29	34	29	31	29	32	32	32	34	32	37	35	36	31
D (UP)	31	32	29	32	31	33	29	35	31	33	34											32
D (DN)	29	29	28	31	29	31	32	33	32	33	31	33	36	33	33	36	28					32

TABLE 3.25: Rebound measurements (N) - Intermediate portion of column - floor II

3.5.3 Correlation between destructive and non-destructive tests

Material strength evaluation, especially for concrete, can be obtained through destructive and non-destructive tests. The first technique is more direct and reliable, but the results are more restricted to few elements and usually they are not representative of entire structure. As seen in Section 3.5.1.2, existing reinforced concrete structures present a great variability about mechanical material propriety, also in the same element.

For these reasons, non-destructive tests represent important tool to increase data tests and to obtain more information about structure, especially material variability. In addition, non-destructive tests are less expensive than destructive tests and no structural damage is caused.

However, non-destructive tests estimate concrete strength through indirect measurements and they are influenced by several factors. Therefore, concrete strength can be evaluated by combined non-destructive methods: several combined non-destructive methods are presented in literature, following parameters must be considered::

- adopted methods can be used on all the reinforced concrete structures, according to dimension and structure order;
- data tests, evaluated through different methods, must be related to several material properties;
- accuracy of concrete strength evaluation must be the same, according to different methods;
- influence of corrective factors, used to convert indirect measurements to concrete strength, must be reduced;
- experimental techniques and respective processing must be easy.

In literature, several empirical formulations are available to evaluate concrete compressive strength by means of combined methods (destructive and non-destructive). However, standard formulations (used for new structures) cannot be adapted to estimate low concrete strength, usual cases consists in existing structures.

In this Section, several combined methods are described and analysed in order to evaluate concrete strength; the aim to reduce destructive tests and improve the knowledge about structure, non-destructive test results (see in Section 3.5.2) are used to create reliable correlation curves, especially for the *case study*.

For this reason, several data tests are collected to increase available information; these data are chosen according to following parameters:

- Age of construction: 1940 – 1970;
- Location: Tuscany (Italy);

- Functionalities (intended use): school building.

According to these parameters, n.134 data are selected: for each data, destructive (concrete compressive tests) and non-destructive tests (rebound and ultrasonic pulse velocity) are available. Within these data there are also some tests performed on other elements extracted by the same school building investigated (e.g. previous case study).

For the purpose of comparing data test results, three sub-samples are considered:

1. Sample A: n. 10 data tests performed on two portions of column (presented in this work, see Section 3.5.1.2);
2. Sample B: n. 7 data tests performed on other elements extracted from the same school building;
3. Sample C: n. 127 data test performed on several school building, located in Tuscany, built from 1940 to 1970.

From *Sample A* the concrete core extracted from portion of beam are excluded: this latter sub-sample is used to correlate concrete strength and sonic tomography.

Thus, assuming a lognormal distribution, probability density functions of cubical compressive strength are determined for each sub-sample (see Figure 3.48). Mean values (μ)

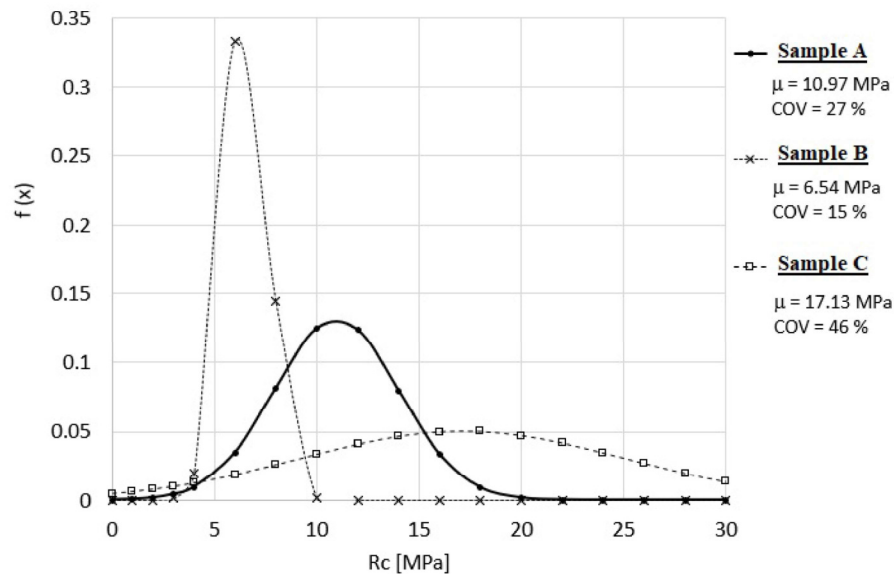


FIGURE 3.48: Probability density function of cubical compressive strength according to the sub-samples: A, B and C

and coefficient of variations (COV) are shown for each sub-sample; it is noted herein that mean value of Sample C is higher than the others but also the variability is very high ($CoV = 0.46$). Moreover, mean value of Sample B is lower than other, the coefficient of variation is comparable with new structures assumption (e.g. $CoV = 0.15$).

Sample code	$V_{ultr.}$ [m/s]	I_R [-]	$f_{c,cyl,pull}$ [Mpa]	f_{sample} [Mpa]	$f_{c,cyl}$ [MPa]
C1	3105	35	11.11	10.29	10.99
C2	3369	35	14.48	9.66	10.31
C3	3183	36	15.81	14.83	15.83
C4	3300	36	12.74	11.97	12.78
C5	3434	38	19.13	14.38	15.35
C6	3404	38	12.06	11.47	12.24
C7	2491	32	6.64	8.48	9.05
C8	2693	30	9.23	7.31	7.80
C9	2522	32	8.55	7.33	7.82
C10	2478	32	5.42	7.96	7.52
Mean value	2998	34	11.52	10.37	10.97
CoV	0.14	0.08	0.37	0.27	0.28

TABLE 3.26: Summary of data tests evaluated from two portions of column extracted from Italian school building: Sample A

In Table 3.26 destructive and non-destructive results are summarized and they are presented in Sections 3.5.1.2 and 3.5.2, according to Sample A. In addition, ultrasonic pulse velocities, rebound index, cylindrical strength evaluated by pull-out test, sample strength and cylindrical strength are presented. Here, coefficient of variations show a great variability of results according to different methods.

3.5.3.1 Ultrasonic velocity vs concrete strength

Several empirical correlations are available in literature (see Figure 3.49) to evaluate concrete strength, starting from ultrasonic pulse velocities.

The main empirical formulations are described in equations 3.11, 3.12 and 3.13, which

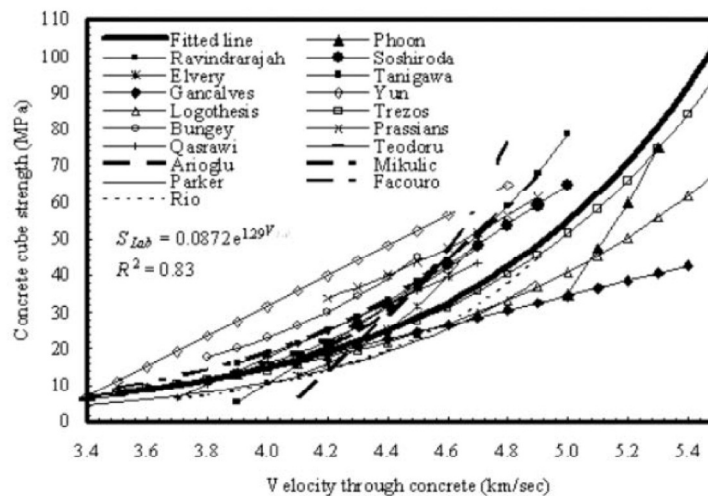


FIGURE 3.49: Calibration curves: ultrasonic pulse velocity and concrete cube strength

correlate different material properties (i.e. dynamic elastic modulus and concrete strength).

$$R_c = 1.88E - 22 * e^{6.184V} \quad (3.11)$$

$$R_c = 3.8E - 5 * V^{9.66} \quad (3.12)$$

$$R_c = \frac{E_d}{1250} \quad (3.13)$$

Usually, these equations can carry out erroneous results because they are not based on the relationship: e.g. good concrete quality thus high concrete strength.

According to equation 3.11, Figure 3.50 shows the correlation between ultrasonic pulse velocity and concrete strength, considering Samples A, B and C.

These correlations are evaluated according to different samples: Sample C (the most numer-

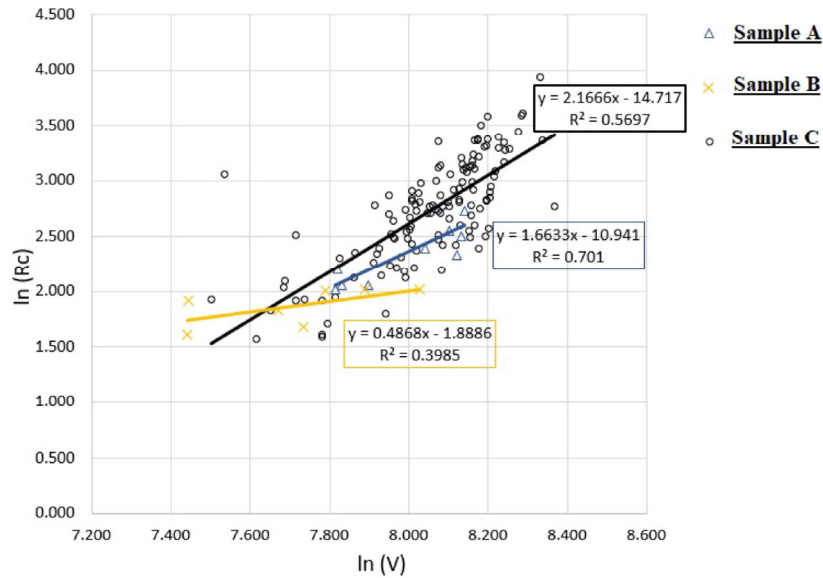


FIGURE 3.50: Calibration curves: logarithmic values of ultrasonic pulse velocity and concrete cube strength, considering Sample A, Sample B and Sample C

ous sample) shows a good correlation between ultrasonic pulse velocity and concrete cube strength, using logarithmic scale. Moreover, Sample A is parallel to Sample C with coefficient $R^2 = 0.70$, which confirms ultrasonic test reliability. Indeed, Sample B (composed by only seven samples) shows a coefficient $R^2 \approx 0.40$: it represents the worst correlation.

Obviously, considering a single sample (Sample A + Sample B + Sample C) correlation result, shown in Figure 3.51, is near to numerous sample (e.g. Sample C). In this case, correlation coefficient is better if only Sample C is considered ($R^2 = 0.60$).

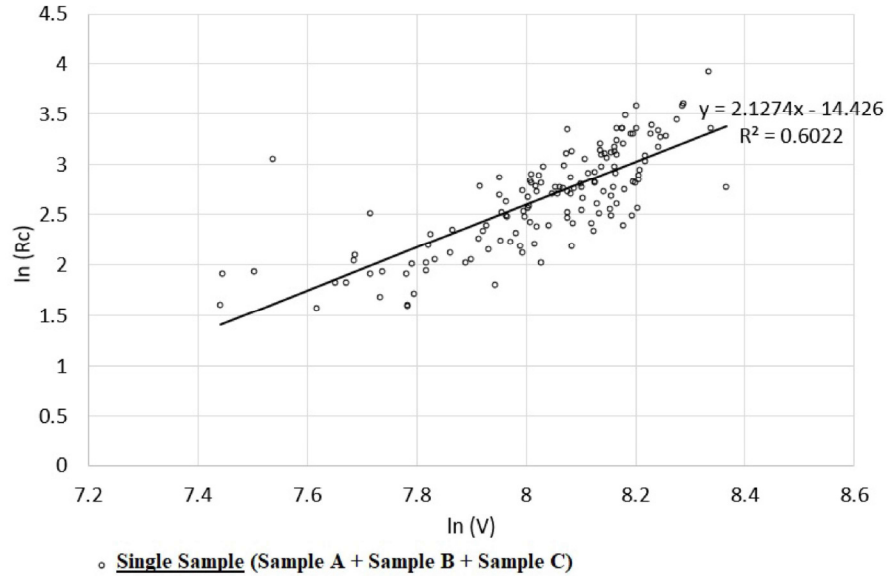


FIGURE 3.51: Calibration curve: logarithmic values of ultrasonic pulse velocity and concrete cube strength, considering Single Sample (Sample A + Sample B + Sample C)

3.5.3.2 Rebound vs concrete strength

In order to evaluate correlation between rebound index (I_R) and concrete strength (R_c) there exist several curves in literature. Usually, this correlation is non-linear (see equation 3.14) and its reliability is still much discussed.

$$R_c = aR^b \quad (3.14)$$

where:

a, b are calibration coefficients;

R is rebound index measured from test;

The main correlation curve, related to rebound index and concrete strength, is designed by Schmidt [93]; however, Figure 3.52 shows calibration curves and experimental test results (this graphic is applied also on rebound machine).

Rebound test is easy and very quick to perform, but its results are unreliable, directly: many factors influence surface hardness of reinforced concrete elements and thus test results. Usually, surface hardness is not correlated with concrete strength and no theoretical formulations are developed. However, considering the previous samples (A, B and C), linear correlations are tested on available data. In the Figure 3.53, three linear correlation curves are shown, according to the samples A, B and C. Adopting logarithmic values, rebound index and concrete strength are compared: Sample A seems to show the best correlation ($R^2 \approx 0.83$), but linearity assumption and the reduced number of considered tests do not

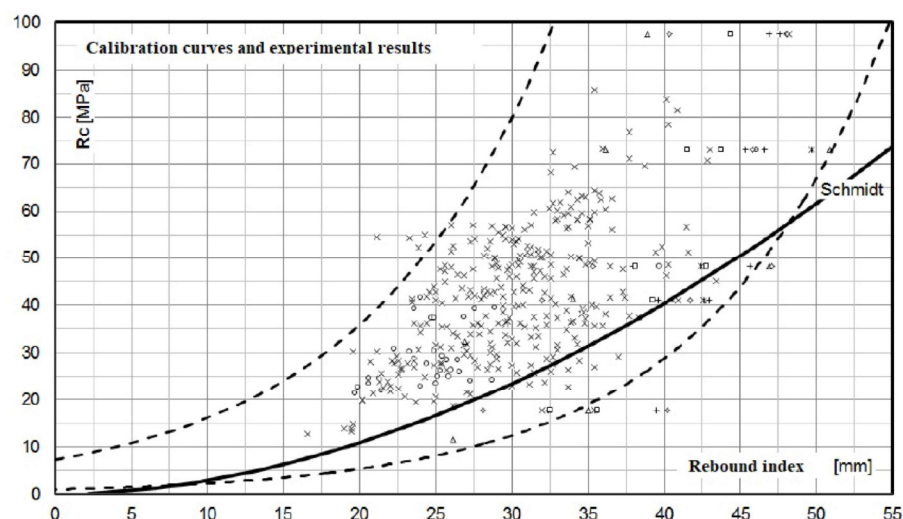


FIGURE 3.52: Calibration curves evaluated by Schmidt: rebound index and concrete cube strength

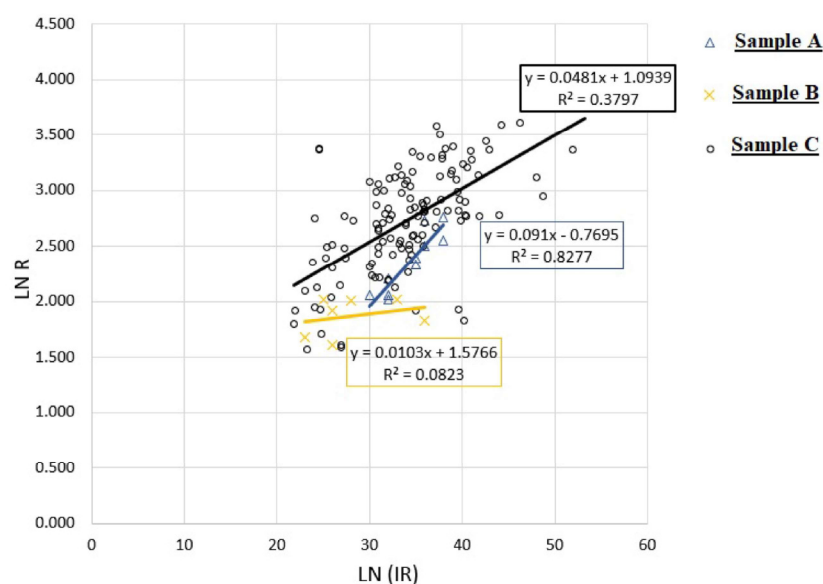


FIGURE 3.53: Calibration curves: logarithmic values of rebound index and concrete cube strength, considering Sample A, Sample B and Sample C

guarantee its reliability.

3.5.3.3 Sonreb method

The combined SONREB method (SONic + REBound = ultrasound + sclerometer) was developed to reduce the errors of two methods described above. In fact, it is noted that the moisture content causes underestimated rebound index and overestimates ultrasonic pulse velocity. Moreover, when concrete age increases, also rebound index increases while ultrasonic pulse velocities decrease. Therefore, as described by Masi [63], combined use of

two tests reduces the errors committed using the two methods individually. Sonreb method application requires evaluation of the local ultrasonic velocity $V_{ultr.}$ and rebound index I_R , from which it is possible to obtain concrete cube strength R_c by means of following general equation:

$$R_c = aV^b I_R^c \quad (3.15)$$

However, the main formulations in literature, made by RILEM [89], Gasparik [43] and Di Leo and Pascale [30] are the equations 3.16, 3.17 and 3.18, respectively.

$$R_{c,1} = 9.27 * 10^{-11} * I_R^{1.4} * V_{ultr.}^{2.6} \quad (3.16)$$

$$R_{c,2} = 8.06 * 10^{-8} * I_R^{1.246} * V_{ultr.}^{1.85} \quad (3.17)$$

$$R_{c,3} = 1.20 * 10^{-9} * I_R^{1.058} * V_{ultr.}^{2.446} \quad (3.18)$$

where:

R_c is concrete cube strength [MPa];

I_R is rebound index;

$V_{ultr.}$ is ultrasonic pulse velocity [m/s].

It is also possible to evaluate concrete strength through equal strength curves ($V_{ultr.} - I_R$ plan), according to previous expressions. In Figure 3.54, according to the equation 3.16, equal strength curves are generated considering different values of ultrasonic pulse velocity and rebound index.

Considering data tests performed in this work (e.g. Sample A), in Table 3.27, concrete

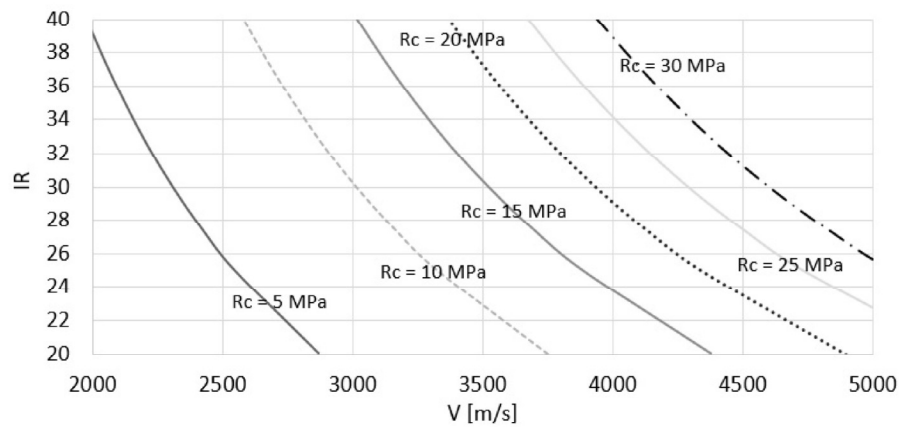


FIGURE 3.54: Equal strength curves, according to RILEM formulation [89]

strengths are evaluated using the expressions 3.16, 3.17 and 3.18. According to these expressions, three concrete strengths are evaluated for each sample; comparing cylindrical

compressive strength ($f_{c,cyl.}$), evaluated by direct test and concrete strength ($R_{c,i}$), estimated by different expression of Sonreb method, a corrective coefficient is determined:

$$C_{S,i} = \frac{f_{c,cyl.}}{R_{c,i}} \quad (3.19)$$

For each Sonreb expression, mean value of corrective coefficients is evaluated; obviously, the closest mean value to $C_{S,i} = 1$ will be the best interpretation of destructive data tests. Considering the Sample A, the equation 3.11 leads to mean value of corrective coefficient $C_{S,1} = 1,10$, which means the best estimate according to mean value of cylindrical compressive strength (10.97 MPa, see Table 3.27).

Other concrete strength evaluations (see equations 3.17 and 3.18) lead to the farthest mean values of corrective coefficients, but with less variability ($CoV = 17\%$, in $C_{S,2}$ and $CoV = 22\%$, in $C_{S,3}$). Thus, RILEM equation 3.16 leads to more accurate results but with higher variability.

Assuming lognormal distribution, density probability functions of concrete strength, evaluated by the three Sonreb formulations, are shown in Figure 3.55. It is noted that RILEM

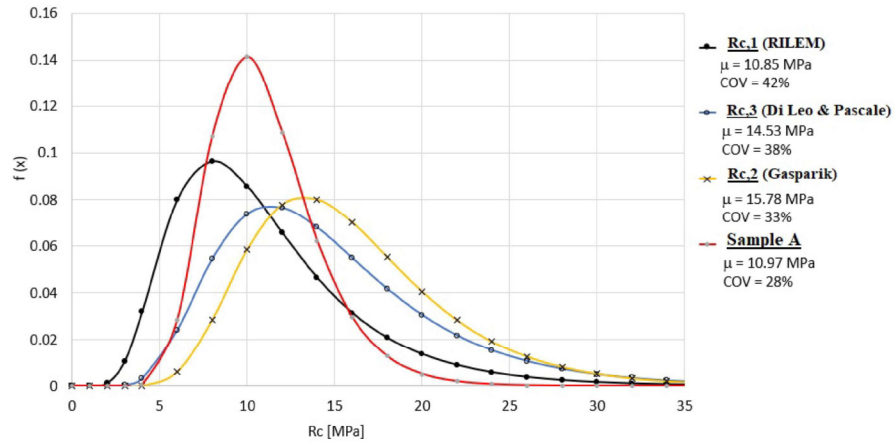


FIGURE 3.55: Density probability functions of concrete strength assuming different Sonreb formulations

formulation leads to a closer mean value related to Sample A, but with high coefficient of variation ($CoV = 0.42$).

However, according to the equation 3.15, in this work, some alternative formulations are proposed: the constants a , b and c are evaluated by multi-regressive analysis (e.g. the least squares method is adopted); whereas, $V_{ultr.}$ and I_R are independent basic variables.

The following formulations, according to the sub-samples presented in Section 3.5.3, are carried out:

1. Sample A:

$$R_{c,(A)} = 5.62 * 10^{-5} * I_R^{2.567} * V_{ultr.}^{0.405} \quad (3.20)$$

2. Sample B:

$$R_{c,(B)} = 6.24 * 10^{-3} * I_R^{0.994} * V_{ultr.}^{0.486} \quad (3.21)$$

3. Sample C:

$$R_{c,(C)} = 4.80 * 10^{-6} * I_R^{0.992} * V_{ultr.}^{1.445} \quad (3.22)$$

4. Sample A+B:

$$R_{c,(A+B)} = 2.98 * 10^{-4} * I_R^{1.466} * V_{ultr.}^{0.682} \quad (3.23)$$

These formulations bring out equal strength curves, according to ultrasonic pulse velocities and rebound index. As seen in Table 3.27, the equations 3.20 and 3.21 lead to mean values of corrective coefficient that are very far from 1.

However, it is possible to note a good correlation between the expressions 3.22 and 3.23; equal strength curve maps, related to latter equations, are shown in Figures 3.56 and 3.57.

Starting from different samples, Figures 3.56 and 3.57 lead to different equal strength

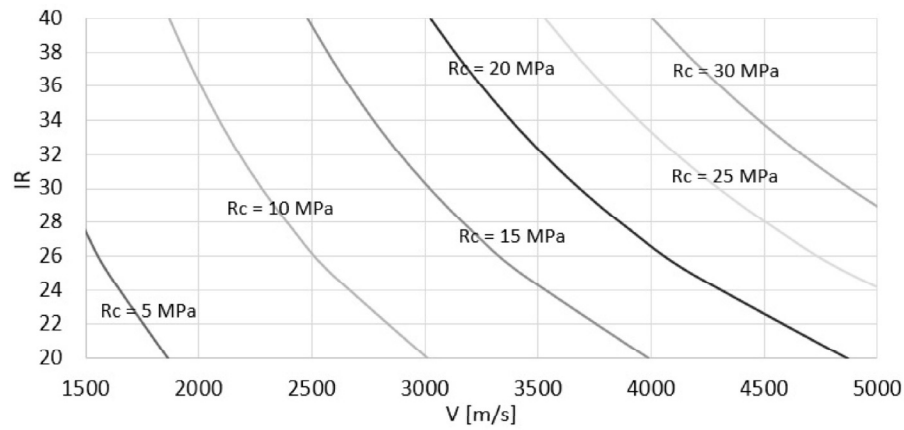


FIGURE 3.56: Equal strength curves, according to Sample C (equation 3.22)

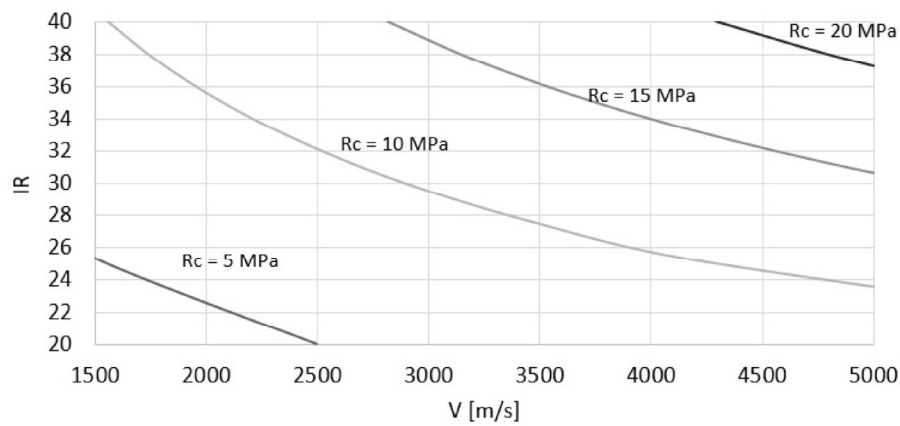


FIGURE 3.57: Equal strength curves, according to Sample A+B (equation 3.23)

curves; particularly, Sample (A+B) considers lower values of concrete strength.

Sample	I_R	$V_{ultr.}$	$f_{c,cyl.}$	$R_{c,1}$	$R_{c,3}$	$R_{c,2}$	$R_{c,(B)}$	$R_{c,(A+B)}$	$R_{c,(A)}$	Corrective Coefficients					
code		[m/s]	[MPa]	[MPa]	[MPa]	[MPa]	[MPa]	[MPa]	[MPa]	$C_{S,1}$	$C_{S,3}$	$C_{S,2}$	$C_{S,(B)}$	$C_{S,(A+B)}$	$C_{S,(A)}$
C1	35	3105	10.99	11.31	15.24	16.52	15.30	11.12	9.08	0.97	0.72	0.67	0.72	0.99	1.21
C2	35	3369	10.31	13.99	18.61	19.22	17.34	11.76	9.45	0.74	0.55	0.54	0.59	0.88	1.09
C3	36	3183	15.83	12.83	16.96	18.26	16.49	12.06	9.59	1.23	0.93	0.87	0.96	1.31	1.65
C4	36	3300	12.78	14.09	18.52	19.52	17.43	12.36	9.76	0.91	0.69	0.65	0.73	1.03	1.31
C5	38	3434	15.35	16.55	21.32	22.12	19.18	13.49	10.37	0.93	0.72	0.69	0.80	1.14	1.48
C6	38	3404	12.24	16.18	20.87	21.77	18.92	13.41	10.33	0.76	0.59	0.56	0.65	0.91	1.19
Mean			12.91	14.16	18.58	19.57	17.44	12.36	9.76	0.92	0.70	0.66	0.74	1.04	1.32
CoV			0.17	0.14	0.12	0.11	0.09	0.08	0.05	0.20	0.19	0.18	0.17	0.15	0.16
C7	32	2491.	9.05	5.62	8.08	9.82	10.12	8.38	7.45	1.61	1.12	0.92	0.89	1.08	1.22
C8	30	2692	7.80	6.54	9.40	10.84	11.08	8.38	7.47	1.19	0.83	0.72	0.71	0.93	1.04
C9	32	2522	7.82	5.81	8.33	10.05	10.32	8.46	7.50	1.35	0.94	0.78	0.76	0.93	1.04
C10	32	2478	7.52	5.56	7.99	9.74	10.05	8.36	7.44	1.35	0.94	0.77	0.75	0.90	1.01
Mean			8.05	5.88	8.45	10.11	10.49	8.39	7.46	1.38	0.96	0.808	0.77	0.96	1.08
CoV			0.08	0.08	0.08	0.05	0.04	0.01	0.01	0.13	0.13	0.11	0.10	0.09	0.09
Mean			10.97	10.85	14.53	15.78	14.62	10.78	8.84	1.10	0.80	0.72	0.76	1.01	1.22
CoV			0.27	0.42	0.38	0.33	0.26	0.20	0.14	0.26	0.22	0.17	0.14	0.14	0.17

TABLE 3.27: Sonreb method applied to data test: concrete strengths and corrective coefficients

3.5.3.4 Pull-out test vs concrete strength

In this work, pull-out tests are performed on two portion of columns, extracted from floor I and II (see Section 3.5.2). The extracted samples are chosen according to concrete core positions; generally, n. 10 pull-out tests are carried out and linear relationship of concrete strength, estimated by direct test and concrete strength and evaluated by pull-out correlation, is shown in Figure 3.58. As seen in Section 3.5.2, pull-out test is a reliable method to

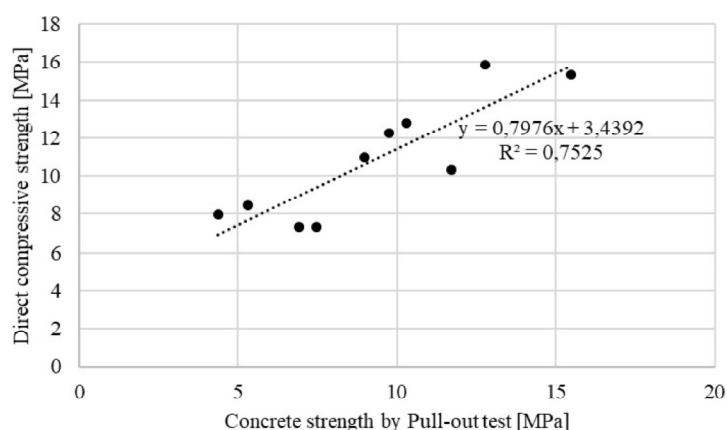


FIGURE 3.58: Linear correlation between direct concrete tests and pull-out tests, related to Sample A

estimate concrete strength. In fact, Figure 3.58 shows a good correlation reliability between the two methods, ($R^2 > 0.75$).

3.5.3.5 Tomography correlation?

In the Section 3.3.3 a recent development of sonic tomography applications on reinforced concrete structures is described. In this research, several tomography maps are carried out on different elements, especially in the portion of beam n.11 tomography maps are evaluated (see Section 3.5.2). Moreover, some concrete cores (e.g. C02, C03, C04, C06, C07 and C08) are extracted according to the tomography maps: ST01, ST07 and ST08 (see Figure 3.59).

Before performing direct compressive tests, ultrasonic pulse velocity test are carried out on each concrete core. In this context, correlation among ultrasonic pulse velocity (V_{ultr}), mean values of ultrasonic velocity evaluated from tomography maps ($V_{m,tomo}$) and concrete strength ($f_{c,cyl}$), are estimated (see Table 3.28).

Assuming logarithmical values, linear correlation between ultrasonic pulse velocities and mean values of tomography velocity is shown in Figure 3.60.

Good correlation of results is displayed comparing ultrasonic pulse velocity and tomography maps ($R^2 \approx 0.73$). It means that tomography test is an useful tool to evaluate concrete

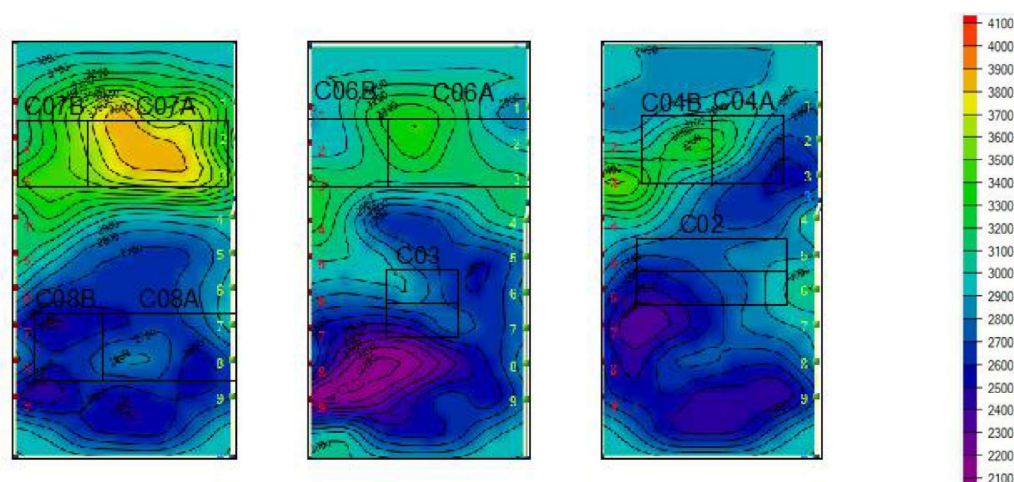


FIGURE 3.59: Concrete core positions according to tomography sections, from left to right: ST01, ST07 and ST08

Sample code	$f_{c,cyl}$ [MPa]	V_{ultr} [m/s]	$V_{m,tomo}$ [m/s]
C02	13.37	2752	2924
C03	12.07	2938	2890
C04A	15.53	3082	2996
C04B	14.42	2928	2996
C06A	12.17	3100	3228
C06B	10.63	2796	3228
C07A	13.37	3429	3698
C07B	18.63	3200	3698
C08A	9.81	2593	2724
C08B	9.35	2553	2724
Mean value	13.26	2937	3111
St. Dev.	2.90	275	354
CoV	0.22	0,09	0.11

TABLE 3.28: Cylindrical compressive strength, ultrasonic pulse velocities and mean values of ultrasonic velocity evaluated on tomography maps of core samples extracted from portion of beam - floor II

quality on structural element.

However, correlations between concrete strength, evaluated by direct compression, and ultrasonic pulse velocity (firstly, evaluated on concrete core and then using tomography maps) are shown in Figures 3.61 and 3.62, respectively.

Obviously, ultrasonic pulse velocities (evaluated on concrete cores) present a better correlation with concrete strength with respect to mean values of velocity evaluated from tomography maps.

The latter methods represent a reliable tool to evaluate concrete quality on cross section and to estimate concrete strength.

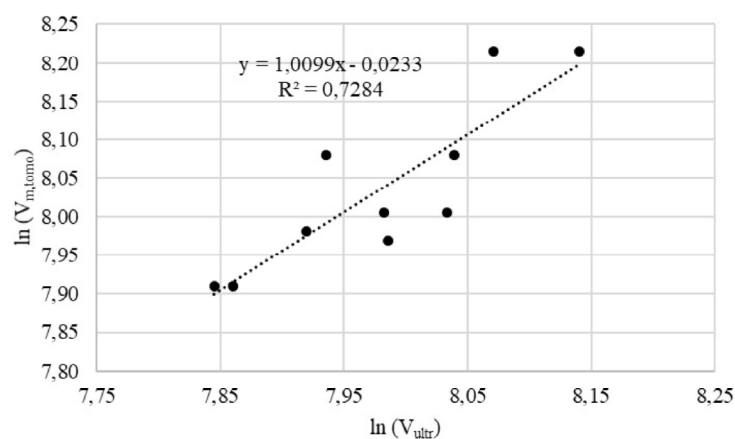


FIGURE 3.60: Linear correlation between ultrasonic pulse velocities and mean value of sonic velocities evaluated by tomography maps, logarithmic values

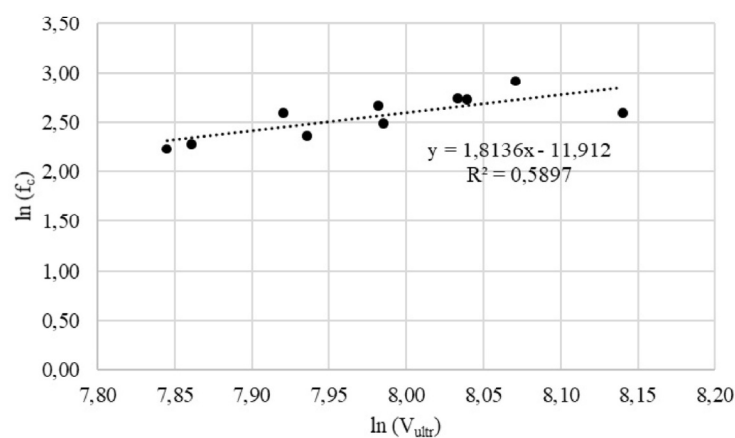


FIGURE 3.61: Linear correlation between ultrasonic pulse velocities and cylindrical concrete strength, logarithmic values

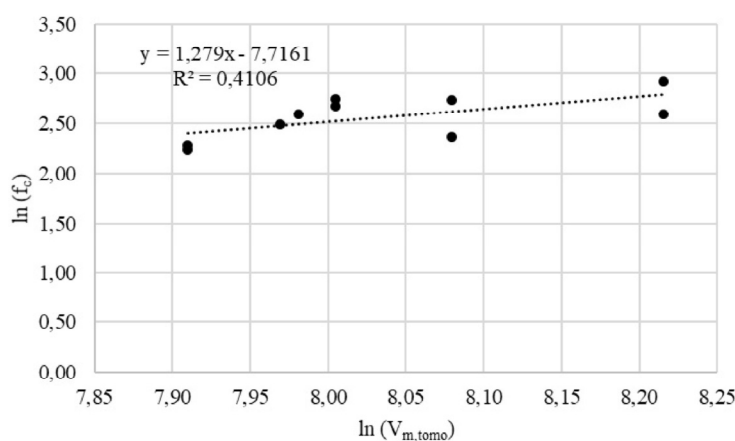


FIGURE 3.62: Linear correlation between mean value of sonic velocities evaluated by tomography maps and cylindrical concrete strength, logarithmic values

In the following Section 3.5.4, tomography maps are used as preliminary information in forecast F.E.M. analysis.

3.5.4 Structural element tests

Structural safety estimation is the aim of researchers and professionals in the field of assessment of existing structures. As seen in the previous sections, reinforced concrete existing structures show a great variability of material properties, within the same structure but also in the same structural element. For this reason, structural capacity tests represent the best prove to understand real capacity of elements according to geometry and material properties.

In this Section, three structural capacity tests are presented and analysed. Starting from the presented case study (Italian school building located in Tuscany), two cyclic axial compression tests are performed on portions (the bottom and the top) of column located in floor II and a bending moment test is performed on portion of beam extracted from the same floor.

The experimental tests are conducted on Structural Laboratory of University of Roma Tre (*PRISMA*), with aim of analysing mechanical failure of column and beam and comparing structural capacity with material investigations.

Finally, experimental test results are compared with forecast F.E.M. analyses performed according to non-destructive test information.

3.5.4.1 Cyclic axial compression tests

The collapse of frame structures often results from a deficiency in concrete strength, which strongly affects the capacity of the columns to carry vertical loads, as described by Kaltakci et al. [50]. For this reason, Braga et al. [13] investigated that problem evaluating cyclic compression tests on RC columns confined with stirrups or retrofitted with FRP jackets. In axial compression test, the size effect of the column with circular and square cross sections assumes a fundamental role on structural behaviour, as investigated by Du et al. [32].

In this Section, the bottom and the top portions of column extracted from floor II of Italian school building are tested and cyclic axial compression is performed. Geometry and material properties of these elements, evaluated by non-destructive tests, are presented in Section 3.5.2.

Moreover, FEM forecast analyses, implemented using non-destructive test results, are compared with experimental capacity tests.

Description of test setup and results The column located on the floor II presents square cross section (29 cm x 29 cm), longitudinal bars equal to 4 ϕ 16 and helicoidal stirrups

equal to $\phi 6/20$ cm. The experimental tests aimed to evaluate structural capacity and to compare test results with material properties estimated by destructive and non-destructive tests.

Axial compression test is performed on *press test machine* which allows to control applied force (3000 kN is the maximum allowed force), manually. Moreover, technical instrumentation, used to measure displacements is described as follows (see also Figure 3.63):

- Four vertical transducers of global type (potentiometers), applied on each surface, in order to measure the top and bottom surfaces dip: initial measurement is equal to 400 mm;
- Four vertical transducers of local type (strain-gauges), applied on each surface, to measure local deformations (i.e. elastic modulus can be estimated): initial measurement is equal to 80 mm;
- Two horizontal transducers of local type (strain-gauges), applied on two next surfaces, to measure local deformations (i.e. Poisson modulus can be estimated): initial measurement is equal to 80 mm;
- Two vertical transducers of global type (potentiometers), applied on press plates, to evaluate any eccentricity.



FIGURE 3.63: Technical instrumentation applied on portions of column

During the test, the axial compression cycles are repeated for 90% of the previous maximum load. Before testing, predicted collapse load is calculated:

$$N_{max} = (A_c + nA_s) * f_{cm} \approx 775kN \quad (3.24)$$

where:

A_c is concrete cross-section: 84100mm^2 ;

A_s is longitudinal steel reinforcement area: 804mm^2 ;

n is homogenization coefficient, usually it assumes $n = 15$;

f_{cm} is mean value of concrete strength, evaluated from destructive tests (see Section 3.5.1.2).

1. Bottom portion of column - floor II

Seven axial cycles are performed on this portion of column (see Figure 3.64) with reached maximum load is equal to 864 kN; the latter value is higher than the predicted collapse load (see equation 3.24).

The displacement, shown in Figure 3.64, is calculated as mean value of two vertical

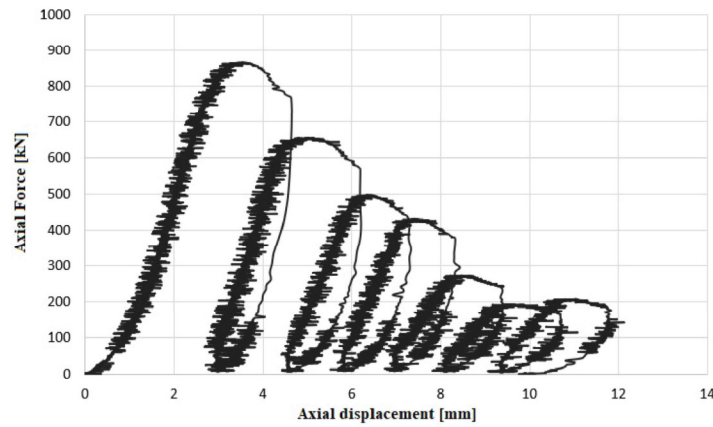


FIGURE 3.64: Force-Displacement curve of cyclic axial compression test of bottom portion of column (floor II)

transducers of global type, applied on press plates.

Table 3.29 shows the maximum load for each cycle. After the second cycles, portion

Load cycles	Load [kN]
1 (failure)	864.04
2	653.60
3	495.64
3	443.73
4	312.79
5	240.46
6	191.78

TABLE 3.29: Load cycles of axial compression test performed on bottom portion of column (floor II)

of column showed evident cracks on east-surface (see left side of Figure 3.65) and

finally (seventh cycle) longitudinal bars showed buckling phenomena (see right side of Figure 3.65), according to confined helicoidal stirrups.

Axial stress are calculated considering axial compression force and homogenised



FIGURE 3.65: Damage states during the axial compression test: on left, third cycle state; on right, seventh cycle state

concrete cross-section (see the first terms of equation 3.24). Figure 3.66 shows stress-deformation behaviour according to the four vertical potentiometers, applied on each surface (e.g. south, east, north and west). All the potentiometers, except west surface,

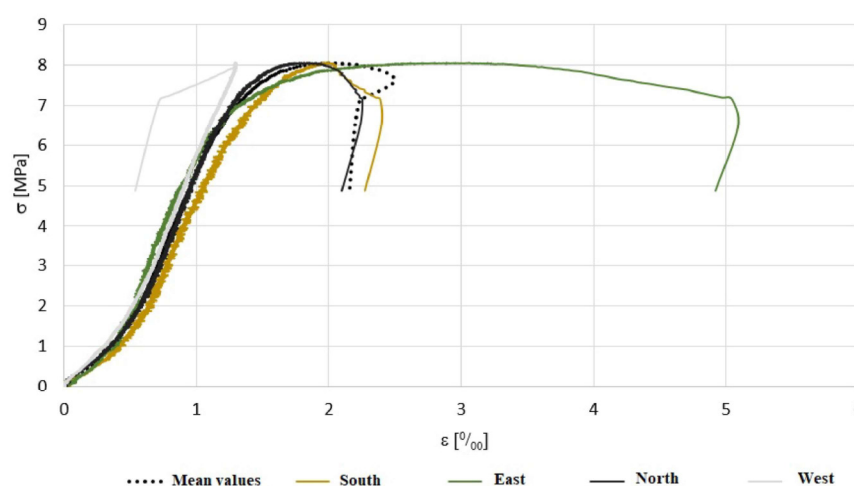


FIGURE 3.66: Stress-Deformation curves according to four vertical potentiometers - cyclic axial compression test of bottom portion of column (floor II)

show the same behaviour: registered high axial stress on east surface entails damage on west potentiometer; for this reason, after elastic phase it doesn't work.

Also strain-gauges behaviour confirms that west surface is damaged: Figure 3.67

shows stress-deformation behaviour according to four vertical strain-gauges applied on each surface (also in this case: south, east, north and west).

It is possible to note that after elastic phase all the strain-gauges stopped to work:

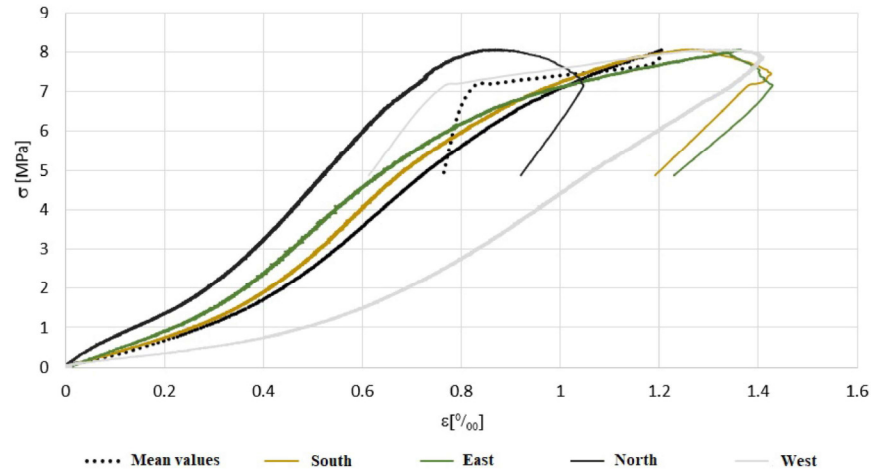


FIGURE 3.67: Stress-Deformation curves according to four vertical strain-gauges - cyclic axial compression test of bottom portion of column (floor II)

surface micro-cracks damaged these instrumentals.

In this test, horizontal strain-gauges are used to estimate Poisson coefficient; they are applied on two orthogonal surfaces (e.g. south and east) and its stress-deformation graphics are shown in Figure 3.68. Here, horizontal strain-gauges show non-linear

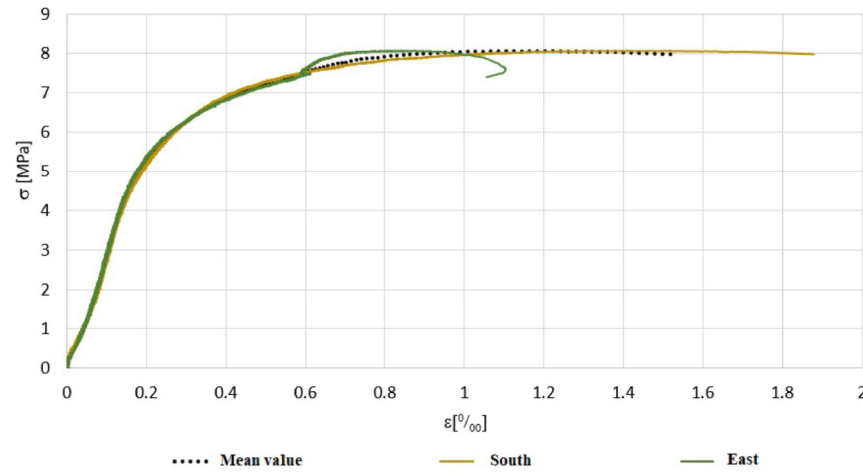


FIGURE 3.68: Stress-Deformation curves according to two horizontal strain-gauges - cyclic axial compression test of bottom portion of column (floor II)

behaviour of element.

According to National Standard [106], secant elastic modulus ($E_{c,sec}$) are evaluated

through the following equation:

$$E_{c,0} = \frac{\Delta\sigma}{\Delta\epsilon_0} = \frac{\sigma_a - \sigma_b}{\epsilon_a - \epsilon_b} \quad (3.25)$$

where:

σ is stress applied [MPa];

σ_a is upper stress $\sigma_{max}/3$;

σ_b is lower stress $\sigma_{max}/10$

ϵ_a is deformation related to σ_a ;

ϵ_b is deformation related to σ_b .

Considering that the maximum stress value is equal to $\sigma_{max} = 8.05$ MPa, parameters related to secant elastic modulus evaluation are shown in Table 3.30. Here, secant

	Potentiometers		Strain-Gauges
σ_a [MPa]	2.68	σ_a [MPa]	2.68
ϵ_a [%]	0.67	ϵ_a [%]	0.50
$E_{c,sec}$ [MPa]	4035	$E_{c,sec}$ [MPa]	5214
		$\epsilon_{a,hORIZ}$ [%]	0.10
		ν	0.188
σ_a [MPa]	2.68	σ_a [MPa]	2.68
ϵ_a [%]	0.67	ϵ_a [%]	0.50
σ_b [MPa]	0.81	σ_b [MPa]	0.81
ϵ_b [%]	0.30	ϵ_b [%]	0.20
$E_{c,sec}$ [MPa]	5136	$E_{c,sec}$ [MPa]	6640

TABLE 3.30: Parameters related to secant elastic modulus $E_{c,sec}$, bottom portion of column - floor II

elastic modulus is calculated according to potentiometers or strain-gauges measurements. Moreover, $E_{c,sec}$ is estimated also without considering the lower stress (e.g. σ_b), see the upper side of Table 3.30.

2. Top portion of column - floor II

As carried out for the bottom portion of column, seven axial compressive cycles are performed on this element; axial force and axial displacement are shown in Figure 3.69. In this case, the reached maximum load is equal to 874 kN, probably material properties are slightly better than the bottom portion of column. This result proves also that there are not relevant differences about concrete quality in the column located in floor II (note that tomography maps show the best concrete quality of the

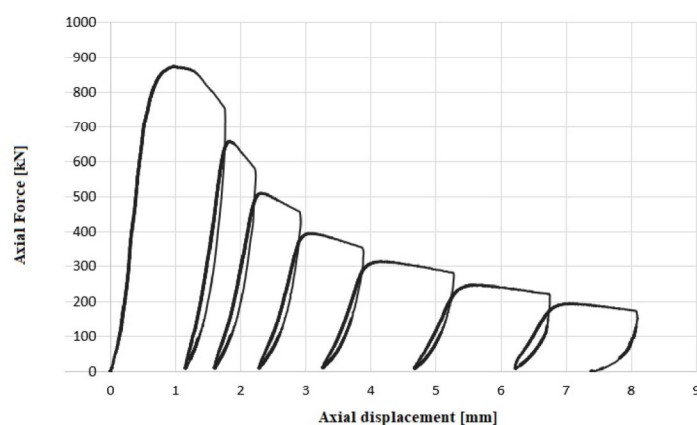


FIGURE 3.69: Force-Displacement curve of cyclic axial compression test of top portion of column (floor II)

bottom portion of column which presented also anchor reinforcements). Moreover, deformation value is calculated as mean value of four vertical transducer measurements, for this reason maximum displacement is different related to the bottom portion (see Figure 3.64).

Damage state evolution is shown in Figure 3.70, which presents the greatest damage on west-surface. Vertical cracks occur after load failure (see left side of Figure 3.70),



FIGURE 3.70: Damage states during the axial compression test: on left, second cycle state; on right, seventh cycle state

whereas on seventh cycle shear cracks and buckling phenomena of longitudinal steel reinforcement interest the lower side of portion of column (see right side of Figure 3.70).

Stress-deformation behaviour, evaluated according to the four vertical potentiometers

(e.g. south, east, north and west), is shown in Figure 3.71 All the potentiometers,

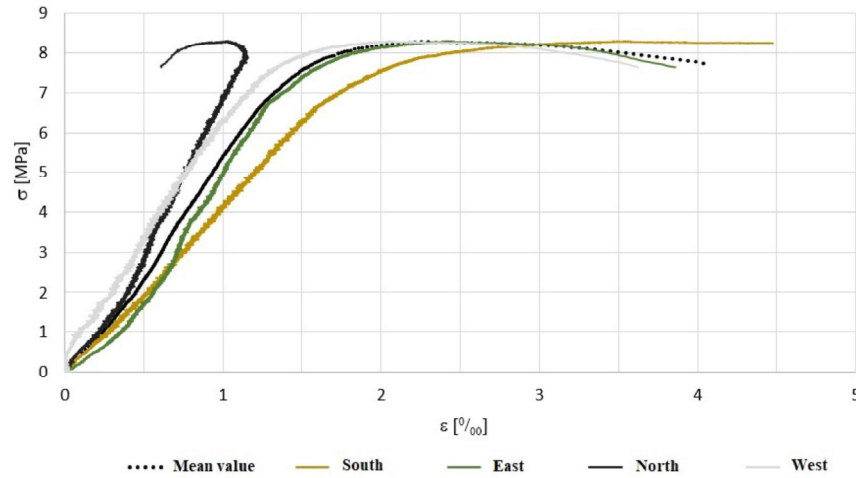


FIGURE 3.71: Stress-Deformation curves according to four vertical potentiometers - cyclic axial compression test of top portion of column (floor II)

except north-surface, work in the same direction. Deformation values are comparable with the bottom portion of column (see Figure 3.66).

Also vertical strain-gauges are applied on four surfaces and the strain-deformation behaviour is shown in Figure 3.72. After elastic behaviour, all the strain-gauges stopped

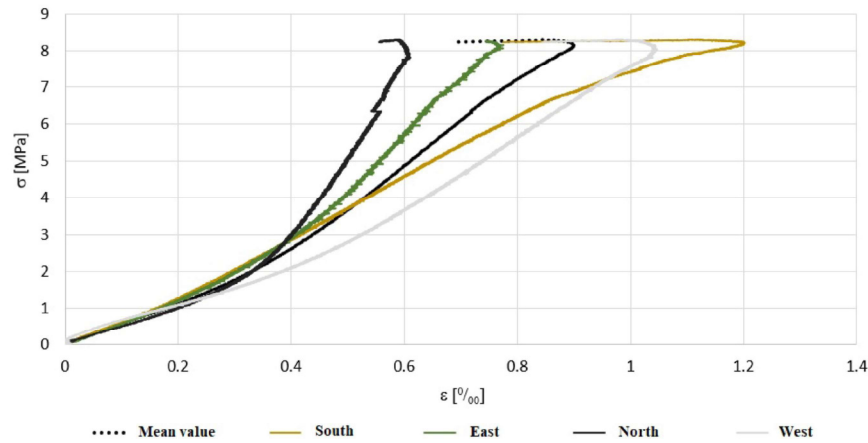


FIGURE 3.72: Stress-Deformation curves according to four vertical strain-gauges - cyclic axial compression test of top portion of column (floor II)

to work. However, deformation results are comparable with those of the bottom portion of column.

Therefore, two horizontal strain-gauges are applied to estimate Poisson coefficient; in this case only south and north surface are available. Stress-deformation graphic is shown in Figure 3.73. In this case, deformation values are comparable with strain-gauges applied on the bottom portion of column east-surface.

Considering all test results, secant elastic modulus ($E_{c,sec}$) is evaluated from equation

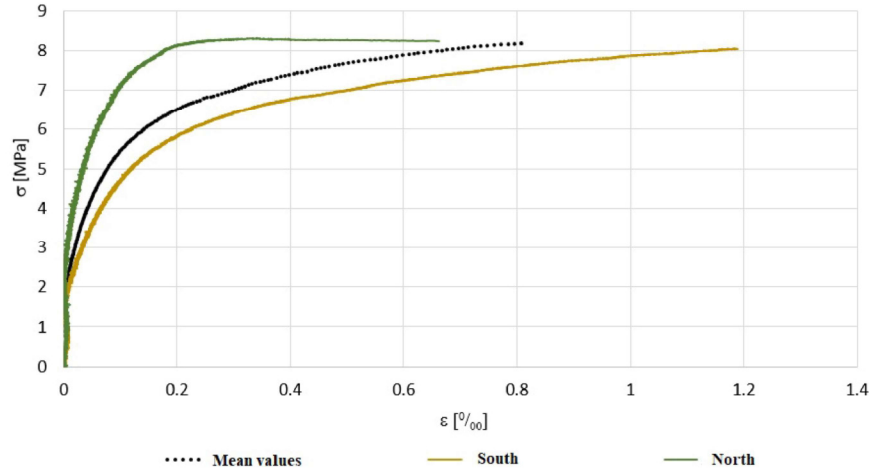


FIGURE 3.73: Stress-Deformation curves according to two horizontal strain-gauges - cyclic axial compression test of top portion of column (floor II)

3.25. For a maximum stress value equal to $\sigma_{max} = 8.30$ MPa, parameters and relative secant elastic modulus results are shown in Table 3.31. Secant elastic modulus, esti-

	Potentiometers		Strain-Gauges
σ_a [MPa]	2.77	σ_a [MPa]	2.77
ϵ_a [%]	0.6	ϵ_a [%]	0.40
$E_{c,sec}$ [MPa]	4799	$E_{c,sec}$ [MPa]	6655
		$\epsilon_{a,hORIZ}$ [%]	0.30
		ν	0.065
σ_a [MPa]	2.77	σ_a [MPa]	2.77
ϵ_a [%]	0.6	ϵ_a [%]	0.40
σ_b [MPa]	0.83	σ_b [MPa]	0.83
ϵ_b [%]	0.2	ϵ_b [%]	0.10
$E_{c,sec}$ [MPa]	5114	$E_{c,sec}$ [MPa]	7228

TABLE 3.31: Parameters related to secant elastic modulus $E_{c,sec}$, top portion of column - floor II

mated for the top portion of column, is greater than the bottom portion of column; whereas, stress and strain are almost comparable. Thus, the top portion of column is slightly stiffer than the bottom one.

Finally, both cases show important deformation increasing after steel reinforcement buckling phenomenon; the importance of confined reinforced concrete columns can be confirmed.

Forecast F.E.M. analysis Usually, non-linear analyses for three-dimensional structural models are computational onerous; for this reason mono-dimensional models could represent a reliable solution with accurate results in order to solve beam and column problems (Neuenhofer and Filippou [75]).

Fibre finite element modelling divides a single beam element in several fibres which are stressed by axial deformation only. In order to create RC element modelling, the longitudinal fibres represent concrete and reinforcement steel material. The stress-strain relationship of the cross-section came from the stress-strain relationship for each fibre integration. The non-linear element behaviour depends on non-linear stress-strain relationship related to concrete and steel fibres. For this reason, the quality of the results depends on the model choices.

In this work, fibre finite element analyses are performed using Midas GEN software, which considers orthogonal sections, related to beam axis, during the load application.

The formulation about flexibility of element is described by Spacone et al. [96]; in this case, the flexibility formulation is equal to Eulero-Bernoulli, but it is diversified according to load interpolation functions (e.g. the load on section and the flexibility on section).

The fibre modelling considers distributed non-linearity on structural element; for this reason, fibre sections must be assigned for each different cross-sections and/or for different steel reinforcement layout.

In order to evaluate the axial compression behaviour of column portion through forecast F.E.M. analysis, Kent and Park [51] model is adopted as concrete non-linear model. This latter is one of the most accurate model that represents cyclic compression behaviour of confined reinforced concrete elements; in fact, experimental tests demonstrated that for high axial compressive load, the confined element assumes completely different behaviour related to the unconfined one. However, in this model, tensile strength of concrete is neglected.

The theoretical diagram of concrete stress-strain, described by Kent and Park and used in Midas GEN software, is shown in Figure 3.74.

The relationship between stress and strain is described by the following equations:

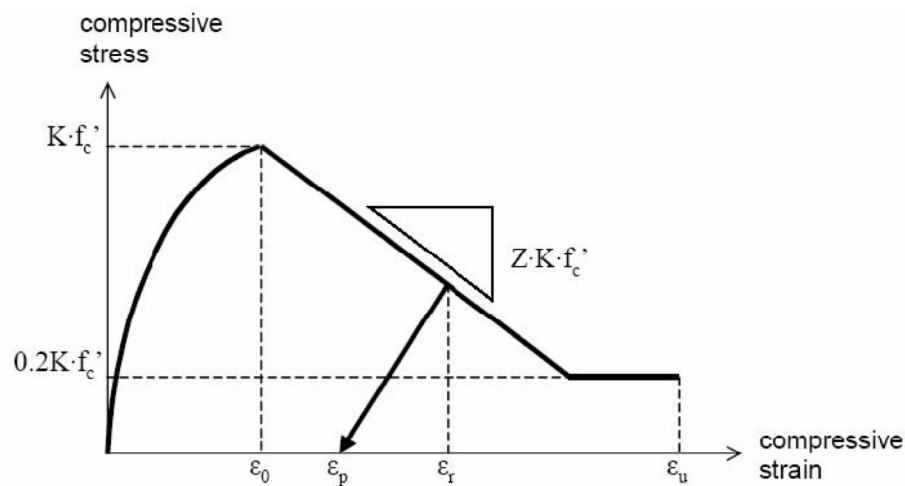


FIGURE 3.74: Stress-Strain diagram of concrete non-linear behaviour, according to Kent and Park model

for $\epsilon_c \leq \epsilon_0$:

$$\sigma_c = K f'_c \left[2 \left(\frac{\epsilon_c}{\epsilon_0} \right) - \left(\frac{\epsilon_c}{\epsilon_0} \right)^2 \right] \quad (3.26)$$

for $\epsilon_0 \leq \epsilon_c \leq \epsilon_u$:

$$\sigma_c = K f'_c [1 - Z(\epsilon_c - \epsilon_0)] \geq 0.2 K f'_c \quad (3.27)$$

where:

$$\epsilon_0 = 0.002 K \quad (3.28)$$

$$K = 1 + \frac{\rho_s f_{yh}}{f'_c} \quad (3.29)$$

$$Z = \frac{0.5}{\frac{3+0.29f'_c}{145f'_c-1000} + 0.75\rho_s\sqrt{\frac{h'}{s_h}} - 0.002K} \quad (3.30)$$

The parameters shown in the previous equations are described as follows:

ϵ is the concrete fibre strain;

σ is the concrete fibre stress;

ϵ_0 is the strain related to maximum stress;

ϵ_u is the ultimate strain;

K is the factor that represents the concrete strength increasing according to concrete confinement;

Z is the softening curve slope;

f'_c is the cylindrical compressive strength [MPa];

f_{yh} is the yielding steel strength [MPa];

ρ_s is the transversal reinforcement percentage (e.g. volume of stirrups and volume of concrete core ratio);

h' is the width of concrete core (e.g. the longer side);

s_h is the stirrup spacing.

The non-linear steel reinforcement behaviour used in this analysis is described by Menegotto and Pinto [67] and successively modified by Filippou et al. [38]. This model is reliable and accurate to replace the steel reinforcement behaviour related to cyclic loads. The theoretical diagram of steel reinforcement stress-strain, described by Menegotto and Pinto and used in Midas GEN software, is shown in Figure 3.75. The relationship between stress and strain

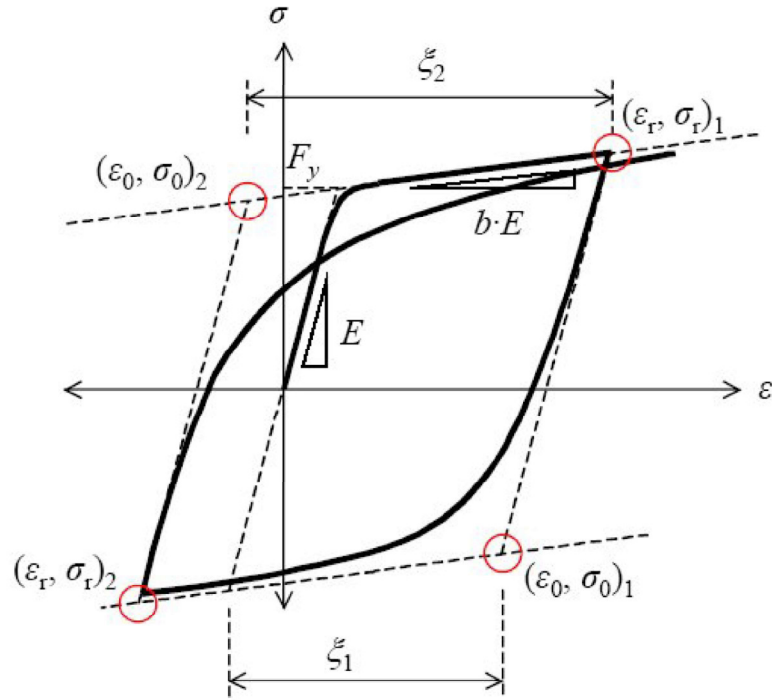


FIGURE 3.75: Stress-Strain diagram of steel reinforcement non-linear behaviour, according to Menegotto and Pinto model

is described by the following equation:

$$\sigma^* = b\epsilon^* + \frac{(1-b)\epsilon^*}{(1 + \epsilon^{*R})^{(1/R)}} \quad (3.31)$$

where:

$$\epsilon^* = \frac{\epsilon - \epsilon_r}{\epsilon_0 - \epsilon_r} \quad (3.32)$$

$$\sigma^* = \frac{\sigma - \sigma_r}{\sigma_0 - \sigma_r} \quad (3.33)$$

$$R = R_0 - \frac{a_1 \zeta}{a_2 + \zeta} \quad (3.34)$$

The parameters shown in the previous equations are described as follows:

ϵ is the steel reinforcement fibre strain;

σ is the steel reinforcement fibre stress;

(ϵ_r, σ_r) is the unloading point (e.g. in the elastic phase is equal to $(0, 0)$);

(ϵ_0, σ_0) is the intersection of the two asymptotes (e.g. load-unload path);

b is the stiffness reduction factor;

R_0 is the coefficient determined by Filippou et al. (it is equal to 20);

a_1 is the coefficient determined by Filippou et al. (it is equal to 18.5);

a_2 is the coefficient determined by Filippou et al. (it is equal to 0.15);

ζ is the difference between maximum value of strain on load path, or unload path, and ϵ_0 .

In this work, fibre finite element modelling of column portion is carried out according to two different test information about concrete:

1. Model n.1: the homogeneous cross-section is made by destructive test results (e.g. concrete strength is evaluated as mean value of compressive concrete tests, $f_{cm} = 8.05$ MPa as seen in Table 3.12). Fibre finite element modelling and non-linear concrete model parameters are shown in Figure 3.76;
2. Model n.2: the heterogeneous cross-section is made by non-destructive test results (e.g. concrete strength is evaluated according to sonic tomography map *ST12*, described in the Section 3.5.2). The concrete cross-section is divided in four regions and for each region, mean values of ultrasonic pulse velocities are calculated; the concrete strengths are evaluated according to the correlation law determined by Sample A (e.g. $y = 1.663x - 10.94$, see Section 3.5.3). Figure 3.77 shows the four regions (e.g. A, B, C and D) according to tomography map *ST12*. Moreover, for each region, the mean values of ultrasonic pulse velocities and the estimated concrete strengths are described in Table 3.32.

According to ultrasonic velocities, related to tomography map *ST12*, non-linear

Region	$V_{ultr.}$ [m/s]	f_c [MPa]
A	2450	7.67
B	2250	6.66
C	2500	7.94
D	2300	6.91

TABLE 3.32: Mean values of ultrasonic pulse velocities and estimated concrete strength related to each region of section *ST12*

concrete model parameters are shown in Figure 3.78.

In order to represent the non-linear steel reinforcement model parameters, evaluated according to tensile test results shown in Table 3.13 (e.g. $f_{ym} = 298.78$ MPa and $E_s = 191980$ MPa), the same model is used both in Model n.1 and in Model n.2.

Non linear static analysis is performed both for Model n.1 and Model n.2. In Figure 3.79, fibre stress-strain behaviours related to both Model n.1 and Model n.2 are compared. In Model n.2, fibre stress-strain behaviours are carried out for each region; the Figure 3.79 shows different behaviours of Model n.2 fibres. Moreover, the behaviour of Model n.1 fibre and Model n.2 fibres (e.g. regions A and C) are very similar to vertical potentiometer

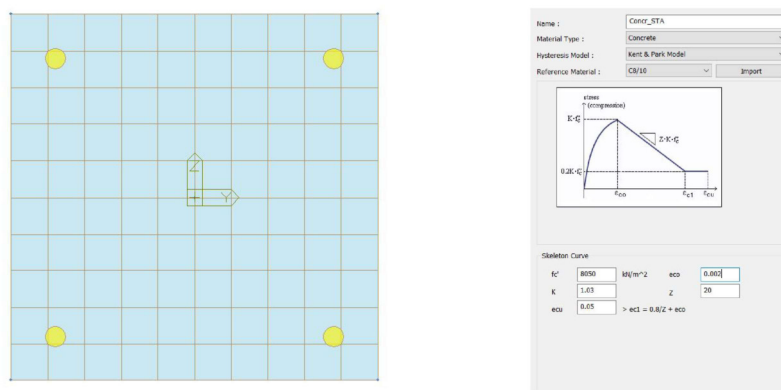


FIGURE 3.76: Model n.1: fibre finite element modelling, on the left side; non-linear concrete model parameters, on the right side

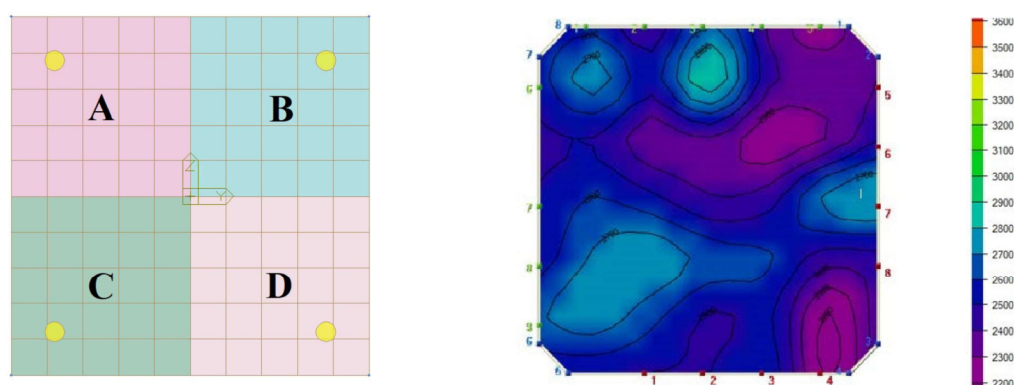


FIGURE 3.77: Model n.2: fibre finite element modelling, on the left side; sonic tomography map *ST12*, on the right side

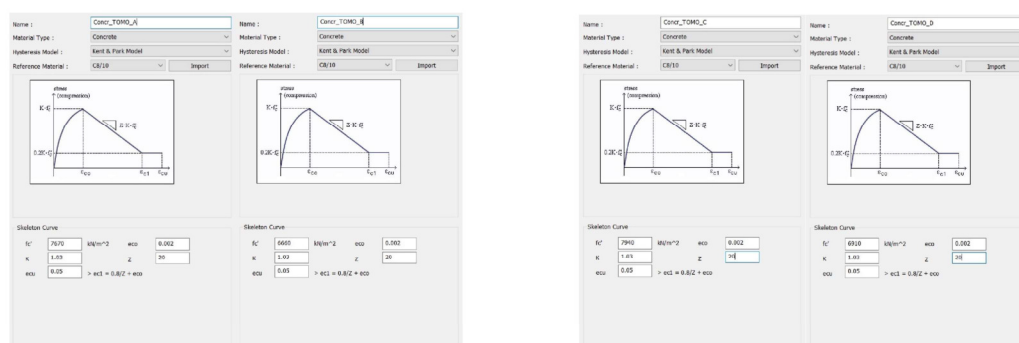


FIGURE 3.78: Non-linear concrete model parameters of Model n.2: regions A and B, on the left side; regions C and D, on the right side

results, shown in Figure 3.71; in both cases, the plastic phase starts in $\epsilon = 0.002$.

However, element behaviour related the axial compression force and displacement are

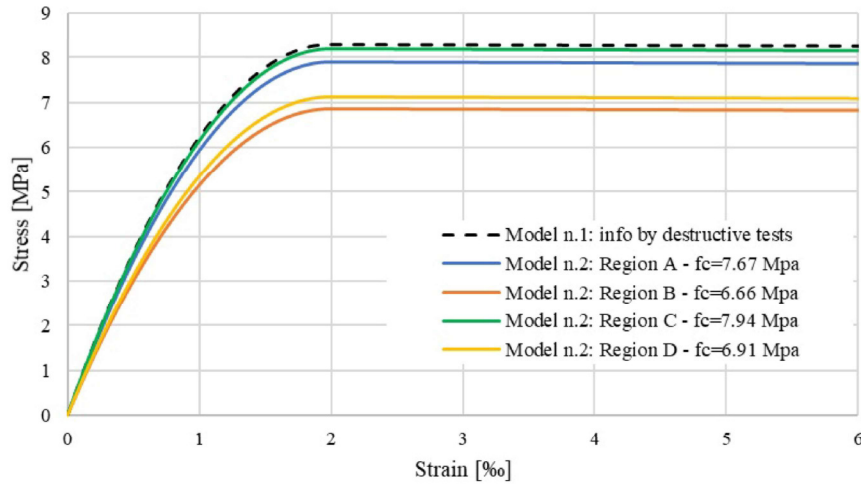


FIGURE 3.79: Fibre stress-strain behaviour related to Model n.1 and Model n.2

evaluated for each Model; in the Figure 3.80, Model n.1 shows an higher axial compression force than one of Model n.2. Moreover, comparing with experimental test results, shown in Figure 3.69, both Models show a smaller stiffness.

However, the axial compression force, evaluated with Model n.2, are very close to exper-

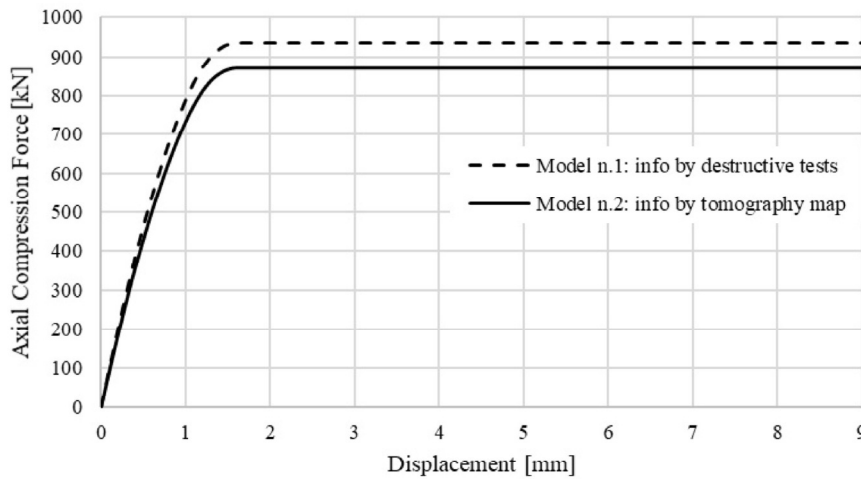


FIGURE 3.80: Element axial compression force-displacement behaviour related to Model n.1 and Model n.2

imental axial load collapse (e.g. $N_{max(test)} \approx 874$ kN). A focus of axial load collapse is shown in Figure 3.81 for both Models; in this case, for Model n.2 axial force collapse is equal to 871 kN, whereas for Model n.1 is equal to 936 kN.

Therefore, assuming information by concrete destructive tests (e.g. Model n.1), it leads

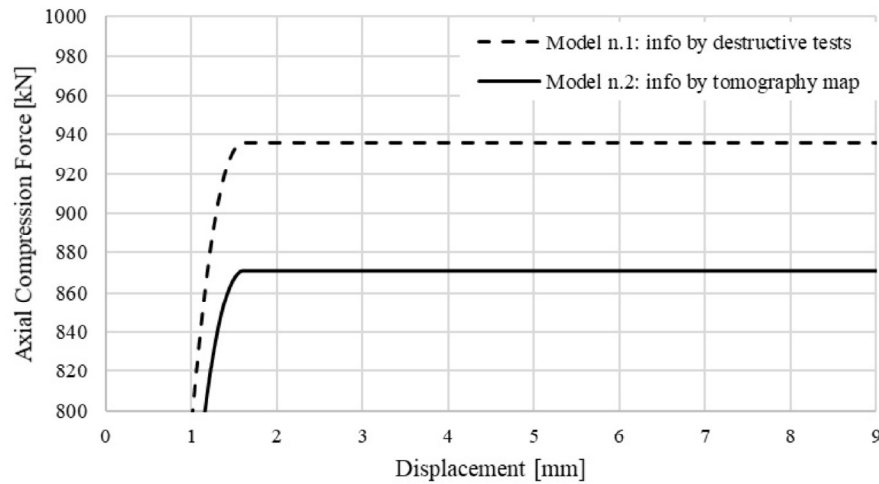


FIGURE 3.81: Element axial compression force-displacement behaviour related to Model n.1 and Model n.2, focus on axial load collapse

to overestimate the axial load result. However, fibre stress-strain behaviours are very similar to vertical potentiometer results. Tomography map, according to a reliable correlation law, leads to accurate information to conduct forecast F.E.M. analysis without performing destructive tests on structural elements. An alternative tool could be represented in order to evaluate the structural capacity on existing structure assessment.

3.5.4.2 Bending moment test

Structural capacity evaluation of existing reinforced concrete beams is a topic frequently investigated: usually, steel reinforcement in existing RC beams is designed with low safety tolerances (at beginning of XX century material cost was very high). Often, as described by Lavorato et al. [55] shear strength of reinforced concrete beams reveals insufficient transversal steel reinforcement, mainly due to design or construction defects or increased design load requirements. For this reason, retrofitting solutions by carbon fibre reinforced polymer (CFRP) reinforcements are very common, as shown in Lavorato et al. [57] and Nuti et al. [77].

Herein, portion of beam extracted from floor II of Italian school building is tested and its structural capacity is evaluated. In the Section 3.5.1.1, geometry and material properties, evaluated by non-destructive methods before capacity testing, are presented. The results of experimental test of portion of beam are investigated also by means of *Digital Image Correlation* (D.I.C.). Finally, FEM forecast analysis is conducted using non-destructive test information and its results are compared with experimental test.

Description of test setup and results The portion of beam extracted from the case study (an Italian school building) is located on floor II, as presented in the Section 3.5. The rectangular cross section is equal to 30 cm x 60 cm but destructive tests show different concrete quality according to the upper and the lower side of beam (the upper side presents a better concrete quality).

Longitudinal steel reinforcement layout shows evident asymmetry, as presented in Figure 3.22; it means that examined portion of beam was closer to joint. Figure 3.82 shows beam cross sections towards the joint (left side) and towards the span (right side). Thus, in



FIGURE 3.82: Cross section of portion of beam located on floor II: on left side, cross section towards joint; on right side, cross section towards span

span cross-section there are 6 ϕ 14 (lower reinforcement) and 2 ϕ 10 (upper reinforcement); whereas, in joint cross-section there are 3 ϕ 14 (lower reinforcement) and 2 ϕ 10+3 ϕ 14 (upper reinforcement). It means that 3 ϕ 14 reinforcement bend towards the joint side (with 45 degrees); it was very common in existing RC structures to optimize steel reinforcement according to the stress (shear and bending moment).

Transversal steel reinforced is determined by means of pacometer analysis, stirrups ϕ 8 with variable distance (it is almost equal to 20 cm).

The portion of beam presents some relevant cracks: the element was probably damaged during the extraction and transport operations. The main cracks are shown in Figures 3.83 and 3.84. Here, vertical cracks, which interest almost the entire beam depth, could suggest a crisis due to tensile or shrinkage stress. However, this situation is very common in existing RC element.

Before experimental test is performed, longitudinal steel reinforcement are fixed to permit steel bonding (see Figure 3.85); smooth rebars without reliable anchorage cannot guarantee complete concrete steel bonding.

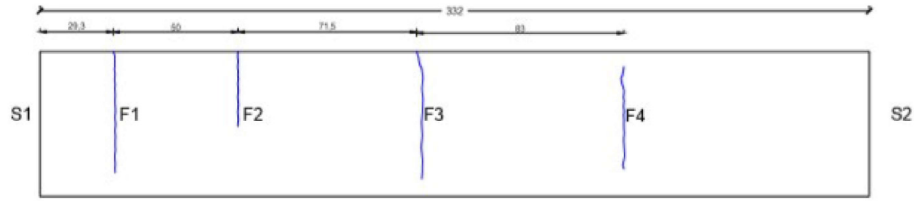


FIGURE 3.83: Cracks layout in the portion of beam extracted from Italian school building



FIGURE 3.84: Crack detail: on left side, crack F3; on right side, crack F4

Preliminary study is conducted to evaluate failure mechanism of portion of beam: bending moment and shear resistances are estimated according to static sketch (trolley - hinge), geometry (concrete cross section, longitudinal and transversal steel) and material properties (estimated from non-destructive test results).

According to static sketch, trolley - hinge with concentrated force in the mid-span, the collapse load is:

$$F_{collapse} = \frac{M_{rd} * 4}{L} \approx 211 kN \quad (3.35)$$

where:

M_{rd} is the resisting bending moment capacity [kNm], cross-section in the mid-span is evaluated;

L is the length of static sketch, 3.00 m;

The force, evaluated in equation 3.35 and applied on the mid-span, leads to bending moment failure; whereas, residual safety tolerance is safeguarded to shear stress.

According to static sketch and mechanical failure, the following technical instrumentation are considered:



FIGURE 3.85: Longitudinal steel reinforcement fixing operations: on left side, concrete cover removing; in right side, steel plate welding

- Two trestles, by means of steel cylinders trolley and hinge support are created (see Figure 3.86), on steel cylinders, steel and rubber plates are applied;



FIGURE 3.86: Beam supports: on left side, hinge; on right side, trolley

- Crossbars steel structure, anchored to the ground by two tie rods;
- Hydraulic pump to increase the load, connected to a cylindrical jack;
- Four potentiometers to measure any failure supports;
- Six vertical potentiometers to evaluate vertical displacements along beam axis, located three of them located for each surface;
- Four horizontal potentiometers, located in the middle of the beam, on upper and lower side, see Figure 3.87;
- Eight diagonal potentiometers to measure shear deformations, see Figure 3.87;
- A control unit for data acquisition;

- National Instruments 6281 analogical-to-digital conversion board with 18-bit resolution;
- LabView software;



FIGURE 3.87: Potentiometers applied on portion of beam: on left side, horizontal type; on right side, diagonal type;

The complete test layout, with the presented technical instrumentations, is shown in Figure 3.88; obviously, load area surface is made uniformly to avoid dangerous local stress.

Vertical potentiometers are applied on the lower side of beam to evaluate vertical dis-

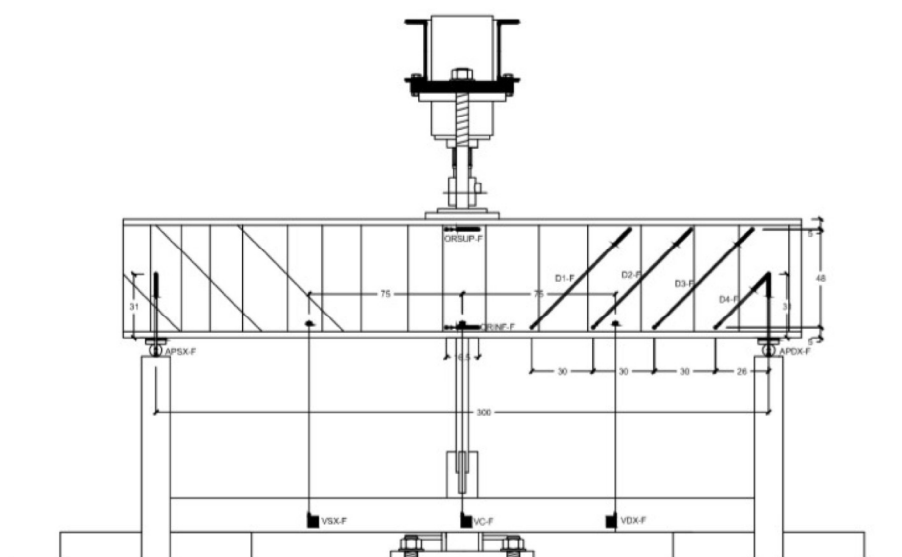


FIGURE 3.88: Test layout of portion of beam

placements: starting from hinge position (left side of beam), three vertical potentiometers ($P1$, $P2$ and $P3$) are located at $L/4$, $L/2$ and $(3/4)L$, respectively. Vertical potentiometers behaviour, on force-displacement plan, is shown in Figure 3.89. Potentiometers show per-

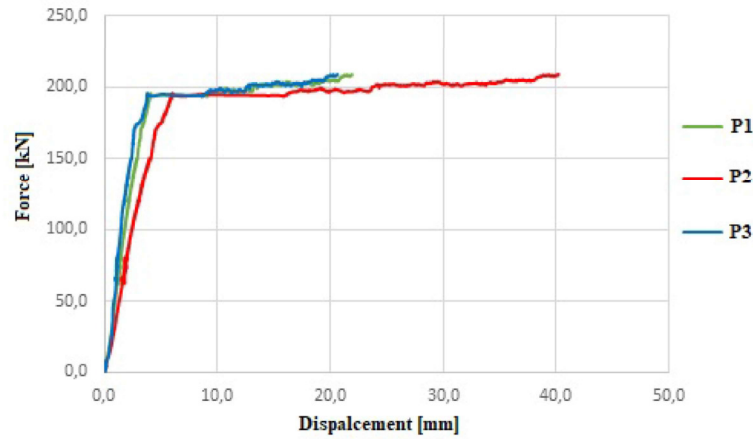


FIGURE 3.89: Force-Displacement graphic of bending moment test: three vertical potentiometers applied at $L/4$, $L/2$ and $3L/4$

fect elastic-plastic beam behaviour, with slight hardening, which shows elastic phase until $F \approx 190$ kN and then plastic phase until failure load, $F = 209$ kN.

Ultimate displacement, measured on P2, is almost equal to 4 cm, whereas for P1 and P2 the maximum displacement is almost equal to 2.2 cm.

Moreover, four diagonal potentiometers are applied where beam shear resistance is lower (e.g. there is no diagonal steel reinforcement and greater distance between stirrups). According to diagonal potentiometers ($D1$, $D2$, $D3$ and $D4$), force-deformation behaviour is shown in Figure 3.90. Diagonal potentiometer $D4$ is located closer to support, it shows less

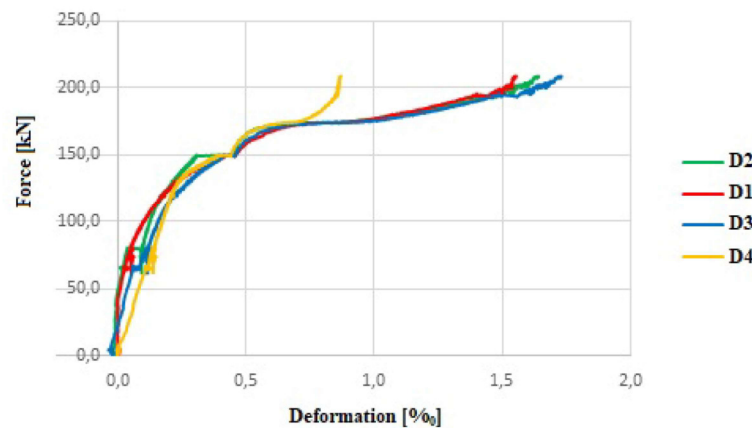


FIGURE 3.90: Force-Deformation graphic of bending moment test related to the four diagonal potentiometers

deformation; whereas, potentiometers ($D1$, $D2$ and $D3$) show the same behaviour, their maximum displacements are $D1 = 1.04$ mm, $D2 = 1.10$ mm and $D3 = 1.17$ mm (see Figure 3.92).

Two horizontal potentiometers are applied in the mid-span, especially on the upper and lower side (UP and DN). Force-deformation behaviour is shown in Figure 3.91. Also in

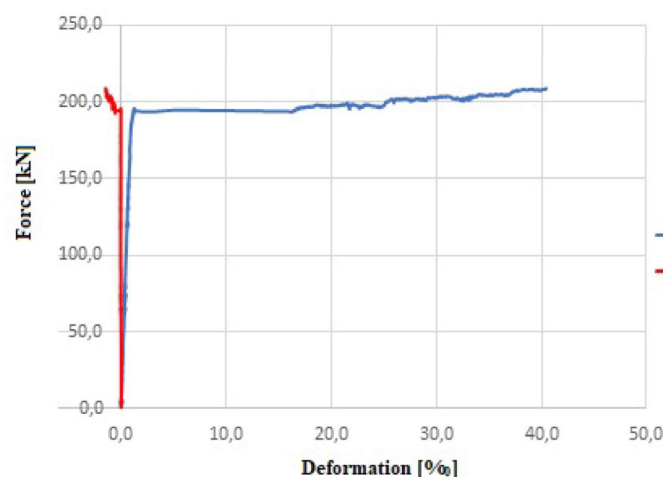


FIGURE 3.91: Force-Deformation graphic of bending moment test related to the two horizontal potentiometers

this Figure, perfect elastic-plastic behaviour is shown; obviously, in the upper side compression deformations are shown. The horizontal potentiometer *DN* leads to a maximum displacement equal to 6.6 mm (see Figure 3.93); whereas, 0.2 mm (negative) for the potentiometer *UP*.

Experimental test leads to comparable result related to preliminary study which considers



FIGURE 3.92: Shear cracks measured by diagonal potentiometers

material properties according to non-destructive tests performed on the beam but even destructive tests performed on other elements (e.g. portions of column).

Capacity testing shows initial diagonal cracks, related to shear stress that according to load value was almost equal to 150 kN: it is the point where the curves change slope, see Figure 3.90. However, increasing load force, diagonal cracks remained constant and bending cracks increased until failure (see Figure 3.94).

In this Figure, it is noted that beam surface are painted white with black dots; it is necessary to evaluate beam behaviour through Digital Image Correlation (D.I.C.).

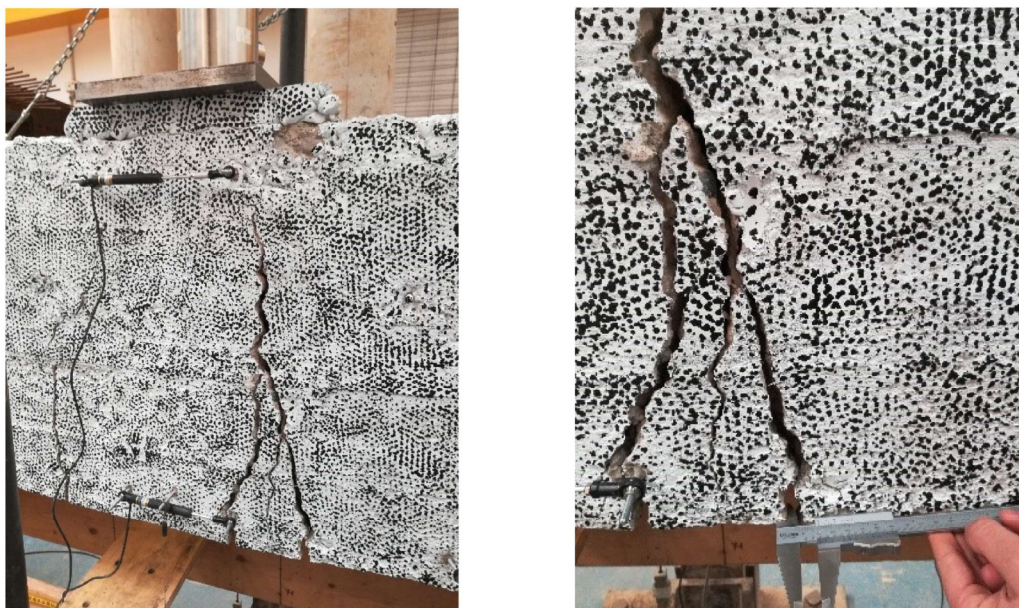


FIGURE 3.93: Bending moment cracks measured by horizontal potentiometers and manually

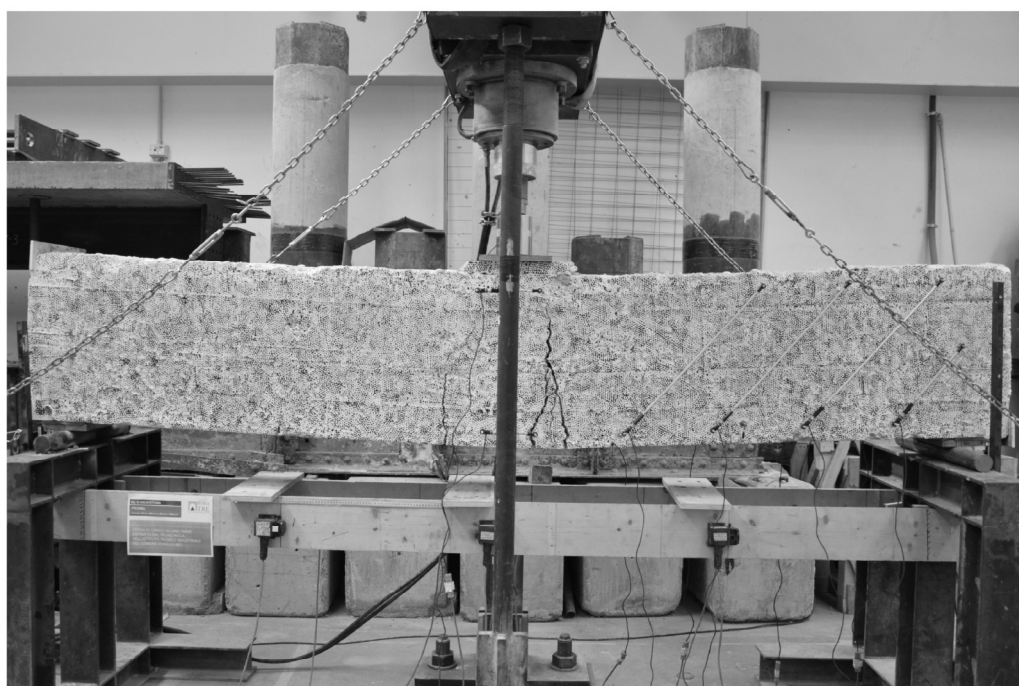


FIGURE 3.94: Final step of experimental test: bending moment capacity failure

Digital Image Correlation analysis Digital Image Correlation (D.I.C.) is a contactless optical method to measure displacements (and/or strains) in experimental testing, based on the correlation of the digital images taken during execution. As described by Tekieli et al. [112], it is used in several fields of experimental solid mechanics and recently on composite reinforcements.

This method, directly provides full-field displacements and strains by comparing the digital

images of the specimen surface with in the un-deformed (or reference) and deformed states respectively. Thus, DIC is an optical metrology based on digital image processing and numerical computing.

During recent years, the DIC method has been extensively investigated and significantly improved for reducing computation complexity: the two-dimensional (2D) DIC method using a single fixed camera is limited to in-plane deformation measurement of planar object surface (see Figure 3.95).

Compared with other optical techniques used for in-plane deformation measurement, the

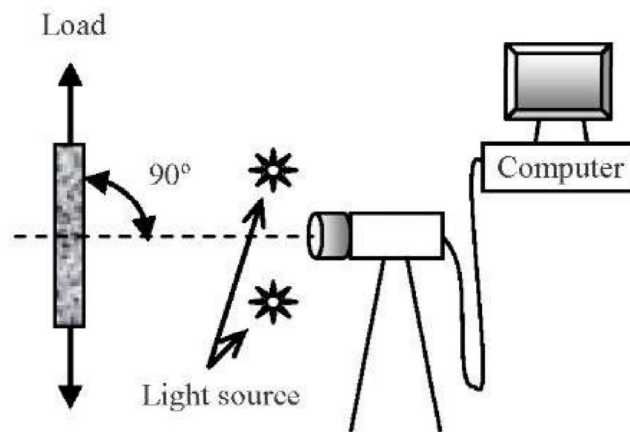


FIGURE 3.95: Typical optical image acquisition system for the 2D DIC method

2D DIC method has some limitations:

- Simple experimental setup and specimen preparation;
- Low requirements in measurement environment (2D DIC does not require a laser source);
- Wide range of measurement sensitivity and resolution.

However, as highlighted by [83], the 2D DIC method suffers some disadvantages:

- Test planar object surface must have a random gray intensity distribution;
- Measurements depend heavily on the quality of the imaging system;
- Strain measurement accuracy of the 2D DIC method is lower than that of interferometric techniques and it is not recommended as a reliable tool for non-homogeneous small deformation measurement.

Moreover, in order to introduce DIC to a wide community of researchers, Blaber et al. [11] have developed an open-source, freely available 2D subset-based digital image correlation software package (*NCorr*) that amalgamates modern algorithms proposed in literature.

In this work, Digital Image Correlation method is used to investigate displacements and strains during bending moment capacity test. Therefore, a random speckle (white base coat and black dots) on the specimen is obtained to capture the points identification, during the test execution (see Figure 3.96).

The capture distance is directly related to the camera focusing and dimensions of specimen



FIGURE 3.96: Speckle pattern used on portion of beam

to be studied. Concerning the reinforced concrete beam, the image capture was decided at 3.0 m distance from surface beam. The camera Nikon 5300 is used for the test, equipped with an interval timer. The latter is essential because it allows to take pictures continuously according to pre-set time interval.

In order to calculate the in-plane displacement of the specimen surface, a computational grid is defined on the picture or on a portion of it, named *Region Of Interest* (ROI). The pictures taken before and after deformation are correlated and the points of the grid in the ROI are matched, i.e., their position after deformation is identified as that associated to the peak of the correlation coefficient.

The correlation criteria are used to determine a measure of similarity between the subset in the reference configuration and that one in the current configuration. It is obtained comparing grayscale values of final subset in reference image with those obtained from the subset in current image. The two methods commonly used are *Normalized cross correlation criterion* and *Normalized least squares criterion*; an example of correlation criteria reliability is shown in Figure 3.97.

Moreover, according to several non-linear method to solve a system of equations on a set of real unknown variables, it is possible to obtain the subset displacement from the initial undeformed condition to the final deformed condition, in the reference image. Figure 3.98 shows subset displacement value evaluated as the sum of several displacements.

In this work, region of interest (ROI) is chosen according to used technical instrumentation; Figure 3.99 shows the considered ROI. The area is limited because diagonal potentiometers on the right side of the beam and the tied road disturbed image processing.

In this work, DIC method is performed in two step, according to load force value:

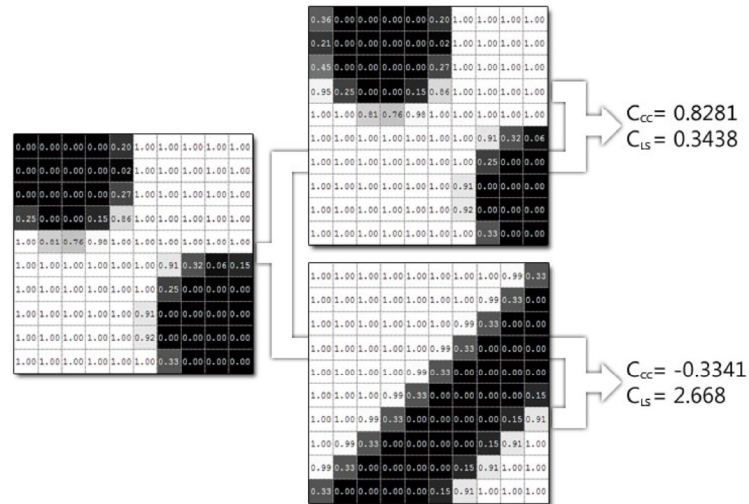


FIGURE 3.97: Example of correlation criteria reliability: gray scale values of the figure on the left are compared with gray scale values of the images on the right

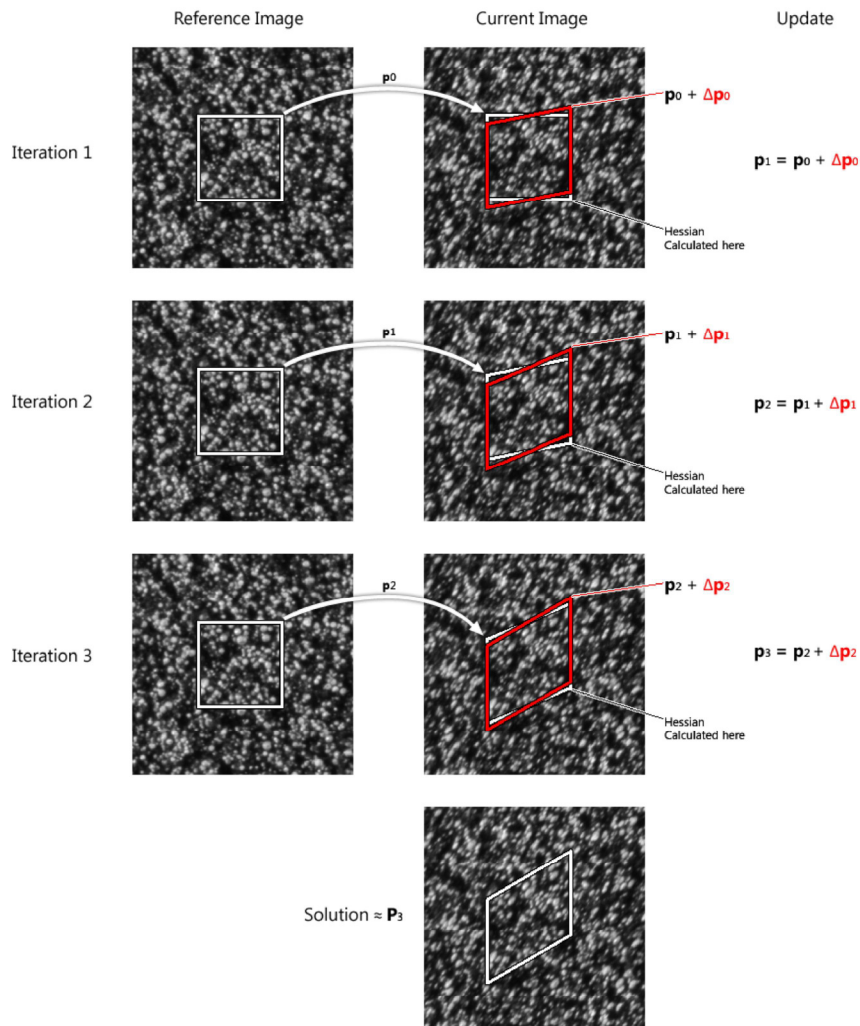


FIGURE 3.98: Example of subset displacement value evaluation

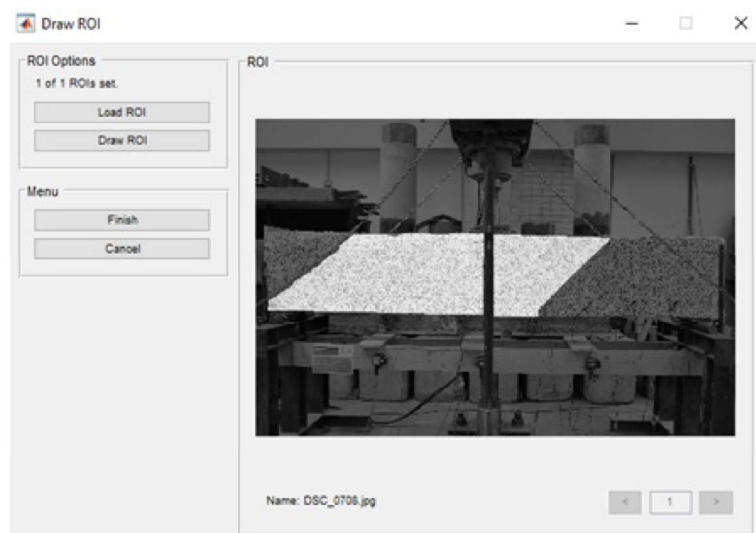


FIGURE 3.99: Region of Interest considered in bending moment test

- From 0 to 175 kN;
- From 175 kN to load failure (≈ 209 kN).

In the following results, only the second step is shown. In order to compare DIC analysis results with traditional measurements (e.g. potentiometers applied on the beam) two stations are investigated (see Figure 3.100). Here, two stations are highlighted (A and B);

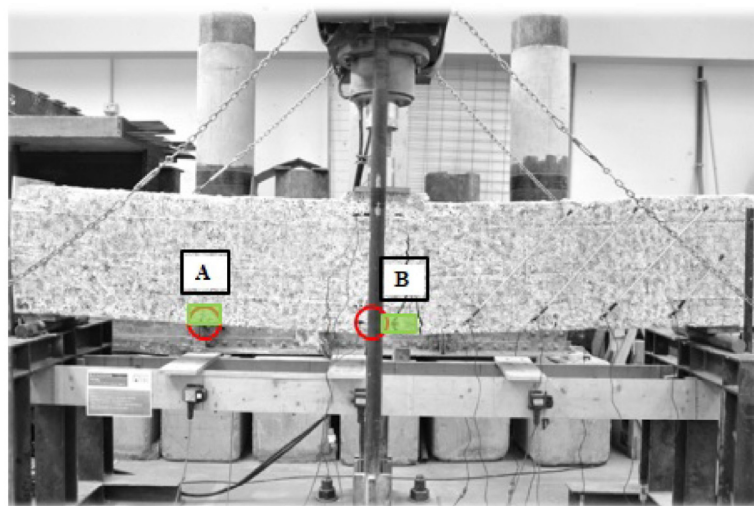


FIGURE 3.100: Stations investigated in DIC analysis

for each station green area represents the pixel considered in DIC analysis and red circle indicates potentiometer position. In this way, direct comparison of results is possible.

In the Figure 3.101, station A is investigated: displacements evaluated with potentiometer P1 and with DIC analysis are compared. In the graph, horizontal translation between two curve equal to 3.75 mm can be observed. Probably, it depends on difficulty to obtain at

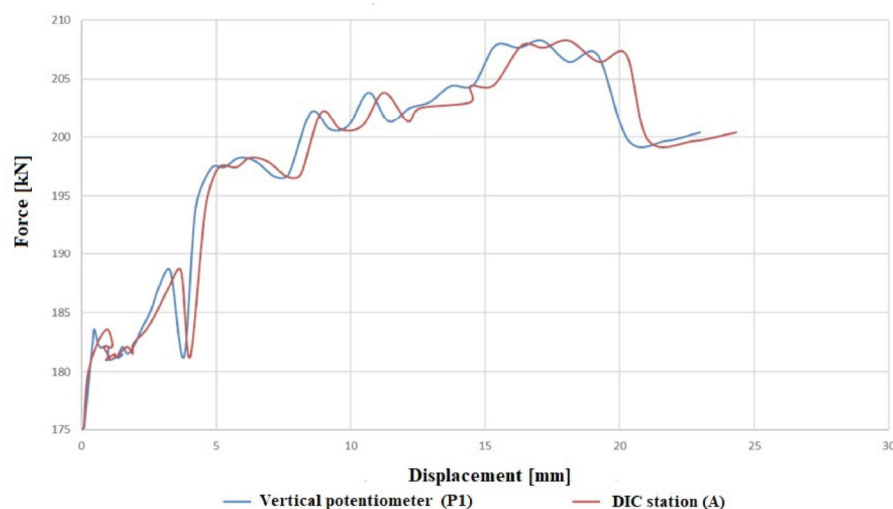


FIGURE 3.101: Vertical displacements related to station A

the same time, force values and relative photo (the time between two consecutive photos is 5 seconds). Moreover, rubber plate, located on beam support, is slightly deformed during the test; it means that there are some differences in reference image.

In the Figure 3.102, vertical displacements of station B are evaluated. Vertical displace-

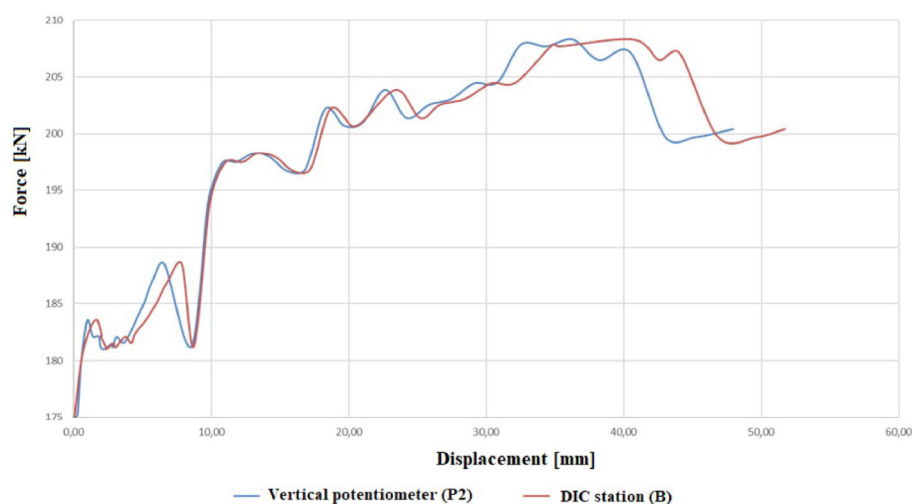


FIGURE 3.102: Vertical displacements related to station B

ments of pixel, located in station B, are compared with vertical potentiometer (P2) measurements. Also in this case, the difference between the two curves is equal to 3.73 mm, probably for the same reason described above.

However, in both graphics DIC analysis results are comparable to the traditional measurements.

Finally, Figure 3.103 shows maximum vertical displacements at the end of experimental test (e.g. 209 kN). Maximum horizontal deformations, related to beam Region of Interest, are shown in Figure 3.104. It is noted that horizontal deformations are concentrated on

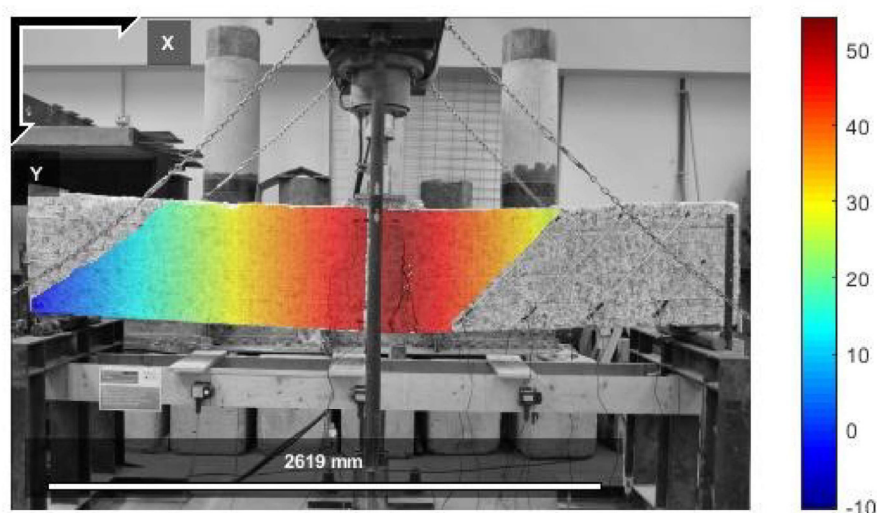


FIGURE 3.103: Maximum vertical displacements related to beam Region of Interest

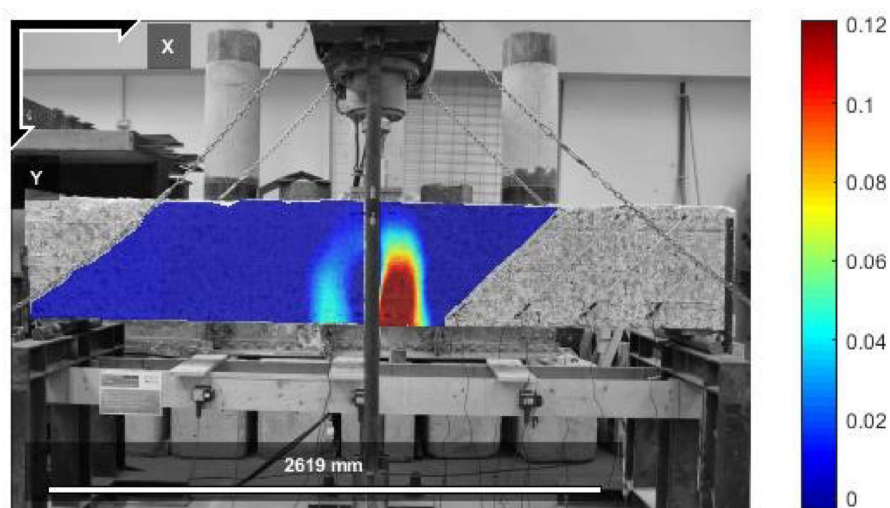


FIGURE 3.104: Maximum horizontal deformations related to beam Region of Interest

mid-span, according to evident bending moment cracks.

Forecast F.E.M. analysis Forecast F.E.M. analysis is conducted to evaluate the bending moment behaviour of beam according to non-destructive test result information.

As seen in axial compression analysis, fibre modelling of section is performed on Midas GEN software. In order to reproduce non-linear behaviour of materials, simple inelastic material models are used for concrete and steel reinforcement.

Nagoya Highway Corporation Model is used to represent inelastic behaviour of concrete

(see Figure 3.105), which considers also a little tensile stress.

The parameters that characterise this model are described as follows:

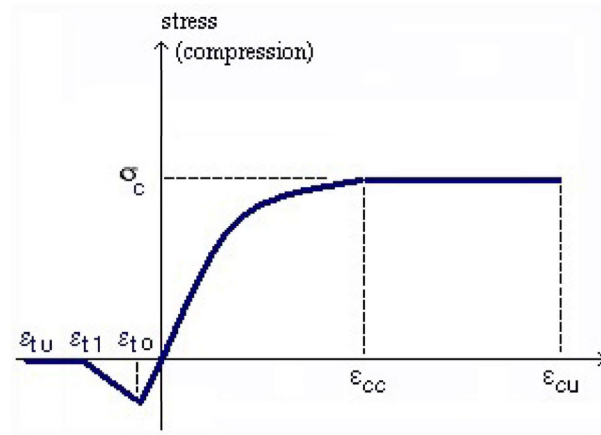


FIGURE 3.105: Inelastic material model for concrete

σ_c is the maximum concrete compressive strength;

ϵ_{cc} is the concrete strain according to the maximum concrete compressive stress;

ϵ_{cu} is the ultimate concrete compressive strain;

ϵ_{t0} is the concrete strain according to the maximum concrete tensile stress;

ϵ_{t1} is the concrete strain related to crack phase starting;

ϵ_{tu} is the ultimate concrete tensile strain.

In this model, confinement contribution on concrete behaviour is neglected.

A simplified inelastic model for steel reinforcement is considered (see Figure 3.106); the bilinear model for steel reinforcement is described by following parameters:

f_y is the yield strength of steel reinforcement;

$E1$ is the elastic modulus of steel reinforcement;

$E2$ is the inelastic path slope.

In this work, fibre finite element modelling of beam portion is carried out according to two different test information about concrete:

1. Model n.1: the homogeneous cross-section is made by destructive test results (e.g. concrete strength is evaluated as mean value of concrete compression tests, $f_{cm} = 12.51$ MPa, as seen in Table 3.10). Fibre finite element modelling and non linear concrete model parameters are shown in Figure 3.107;

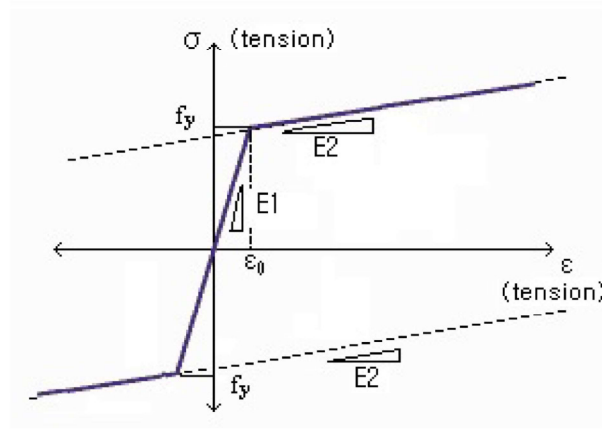


FIGURE 3.106: Inelastic material model for steel reinforcement

2. Model n.2: the heterogeneous cross-section is made by non-destructive test results (e.g. concrete strength is evaluated according to sonic tomography map *ST05b*, described in Section 3.5.2). According to performed tomography tests, the concrete cross-section is divided in two regions (upper and lower side); for each region, mean value of ultrasonic pulse velocities are calculated. The concrete strengths are determined according to the correlation law calculated in the subsection 3.5.3.5 (e.g. $y = 1.279x - 7.716$, see Figure 3.62). Figure 3.108 shows the two regions (e.g. UP and DN) according to the tomography *ST05b*. Moreover, for each region, the mean values of ultrasonic pulse velocities and the estimated concrete strengths are described in Table 3.33. The parameters of non-linear concrete model are shown in Figure 3.109.

Region	$V_{ultr.}$ [m/s]	f_c [MPa]
UP	3300	14.07
DN	2800	11.50

TABLE 3.33: Mean values of ultrasonic pulse velocities and estimated concrete strength related to each region of section *ST05b*

However, in order to represent the non-linear steel reinforcement model parameters, evaluated according to tensile test results shown in Table 3.11 (e.g. $f_{ym} = 293.3$ MPa and $E_s = 229324$ MPa), the same inelastic steel model is used in both Model n.1 and Model n.2.

In order to compare F.E.M. analysis results and experimental test measurements, the behaviour of steel and concrete fibres are evaluated in the same position of horizontal potentiometers applied on the beam (see the left side of Figure 3.87). For this reason, the F.E.M. analysis results are compared with the horizontal potentiometer behaviours; the force-deformation graphic, shown in Figure 3.91, highlights the maximum deformation measured by the two horizontal potentiometers. Related to the maximum vertical force (e.g. $F_{max} = 208.9$ kN), the lower potentiometer measures the deformation equals to

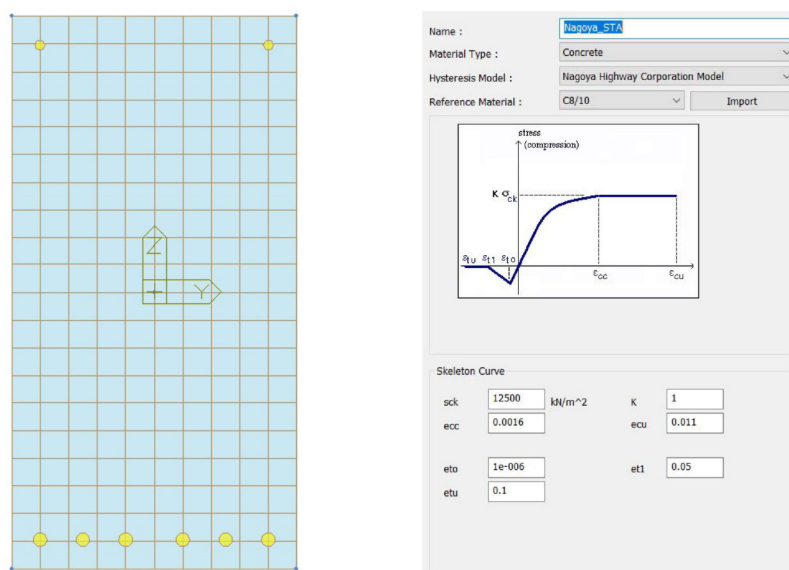


FIGURE 3.107: Model n.1: fibre finite element modelling, on the left side; non-linear concrete model parameters, on the right side

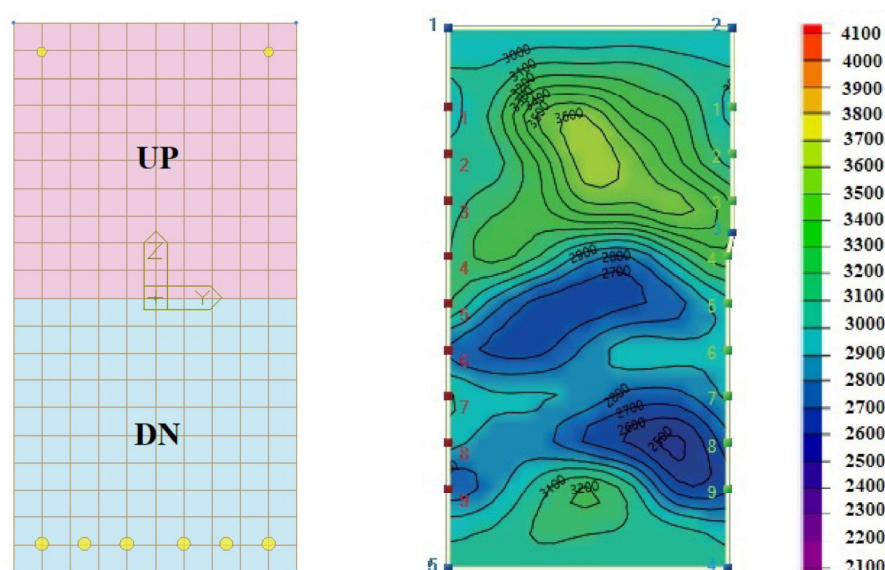


FIGURE 3.108: Model n.2: fibre finite element modelling, on the left side; sonic tomography map ST05b, on the right side

4.04%, whereas the upper potentiometer measures the deformation equals to -0.15% .

The F.E.M. analysis results are shown in Figure 3.110, related to both Model n.1 and Model n.2; steel and concrete fibres are evaluated according to force-strain graphic. The fibre behaviours is very similar about the two models: with regard to the maximum force measured during the experimental test (e.g. $F_{max} \approx 209$ kN), steel fibre strains are equal to 3.99% and 3.70% for the Model n.1 and the Model n.2, respectively.

Moreover, considering maximum force measured during the experimental test, the concrete fibre strains are equal to -0.28% for the Model n.1 and -0.24% for the Model n.2; in this case the difference between experimental test and analysis results are grater.

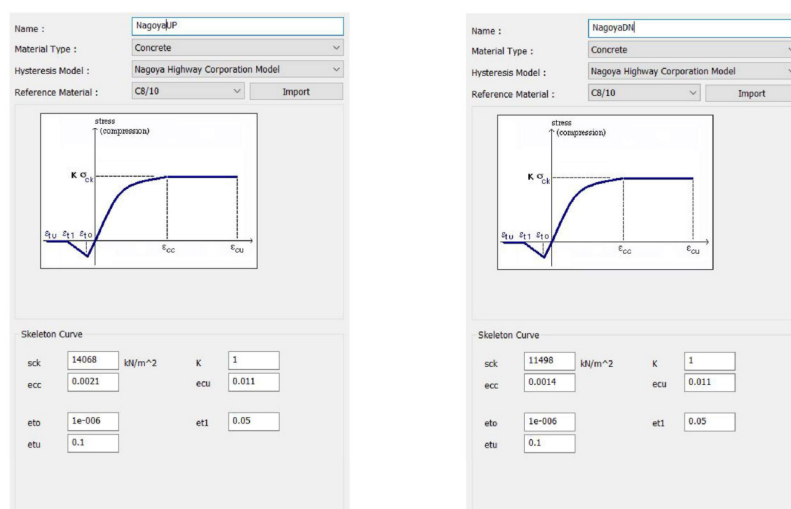


FIGURE 3.109: Non-linear concrete model parameters of Model n.2: region UP, on the left side; region DN, on the right side

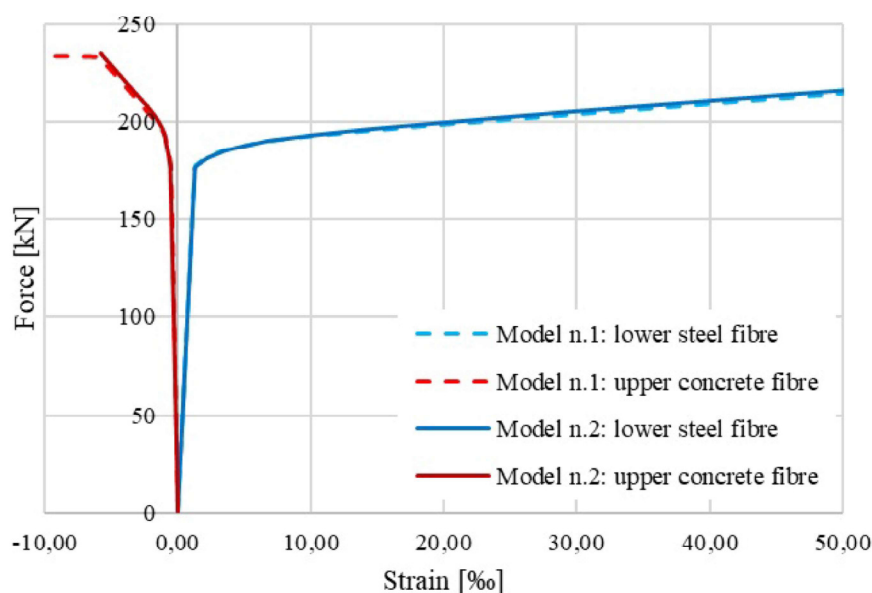


FIGURE 3.110: Force-strain fibre behaviours of steel (lower side) and concrete (upper side), related to Model n.1 and Model n.2

It is noted that accurate concrete strength modelling on bending moment capacity doesn't influence the analysis results, however using tomography maps concrete fibre strain results nearer to experimental measurement are obtained.

3.6 Conclusions

Experimental tests assume a crucial role in the diagnostic process, constituting the indispensable activities to evaluate the structural safety of existing structures. Experimental

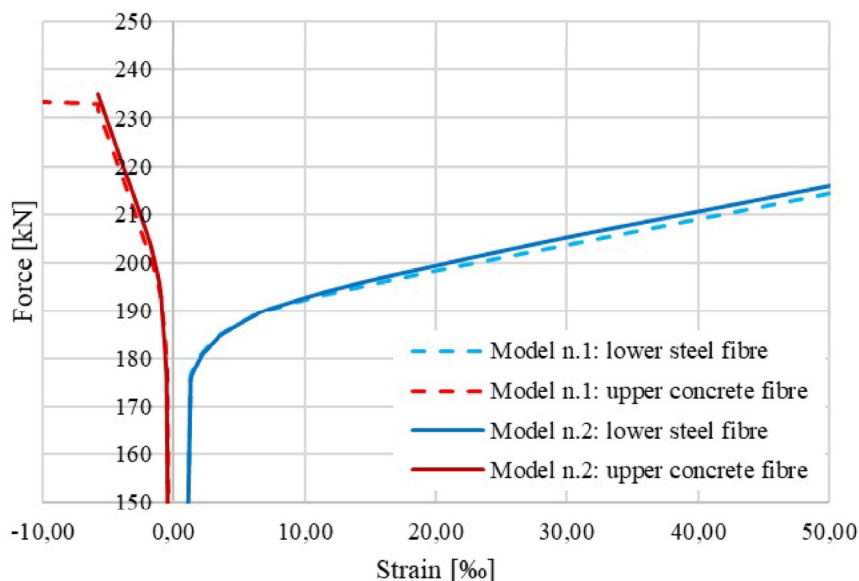


FIGURE 3.111: Focus of force-strain fibre behaviours of steel (lower side) and concrete (upper side), related to Model n.1 and Model n.2

tests aim at knowing the material properties of a structure, and especially its strength variability. Usually, reinforced concrete existing structures show a great variability of material strengths, particularly related to the concrete. The evaluation of in-situ concrete compressive strength can be carried out using accurate techniques which are not extremely onerous and invasive. For this reason, combining destructive tests with indirect measurements could lead to reliable results. Destructive tests provide locally reliable results, but they are invasive and only few samples are accepted; for this reason, they couldn't be very representative of the structure. Alternatively, using only non-destructive tests provide absolutely unreliable results but, according to a larger number of application possibilities, it is possible to perform numerous measurements and to better reproduce the in-situ concrete variability.

In this Chapter, a wide experimental activity is conducted on some structural elements extracted from an Italian school building built in 1940. This structure represents an usual case of reinforced concrete building built at that time.

The experimental results highlighted an high variability of the material properties, between the examined elements, but also within the same structural element. Particularly, the mean values of concrete strengths related to the portion of column (first floor), the portion of beam (second floor) and the portion of column (second floor) are equal to 12.92 MPa, 12.40 MPa and 8.05 MPa, respectively. However, in the same order, the coefficients of variation of these concrete strengths are equal to 17%, 19% and 9%. The elastic modulus of concrete, related to the portion of column (first floor) and the portion of beam (second floor), are equal to $E_c \approx 14900$ MPa; whereas, related to the portion of column (second floor), the elastic modulus of concrete is equal to $E_c = 11723$ MPa.

Steel reinforcement yield strength is less variable than concrete strength: concerning the same elements, the mean value of longitudinal steel strengths is equal to 313.2 MPa and the coefficient of variation is equal to 6%. The mean value of transversal steel yield strengths is equal to 363.5 MPa (it is higher than longitudinal one) and the coefficient of variation is equal to 5%. The mean value of elastic modulus is equal to $E_s \approx 199000$ MPa in longitudinal steel, while it is equal to $E_s \approx 193000$ MPa in transversal steel. Moreover, tensile tests performed on longitudinal steel samples, extracted from the portion of beam, show variable mechanical properties according to the position: close to the mid-span, the yielding phase is smaller than close to the beam support. However, steel reinforcement close to the beam support shows higher ductility than steel portion close to the mid-span.

In order to estimate concrete mechanical parameters, non-destructive tests show very similar results compared to destructive tests. In fact, ultrasonic tests, performed on all elements, show variable sonic pulse velocities, according to the direct compression tests on extracted concrete core. The mean values of ultrasonic pulse velocities, performed on the portion of column (first floor) and on the portion of beam (second floor) are equal to $V_{ultr.} \approx 3000$ m/s; whereas, the mean value of ultrasonic pulse velocities performed on the portions of column (second floor) is equal to $V_{ultr.} \approx 2600$ m/s. The correlations obtained through the calibration procedures, between destructive and non-destructive tests, well estimate the mean values of the concrete compressive strength. For this reason, the developed correlation laws are used to estimate the concrete strength applied on the F.E.M. analyses.

Nevertheless, the experimental tests on structural elements show different mechanical properties compared to material test results. Especially, the elastic moduli evaluated through the axial compression tests, performed on the portions of column, are equal to $E_{c,col.} \approx 6800$ MPa; whereas, the mean value of elastic moduli evaluated according to the concrete core samples (extracted from the portion of column of second floor) is equal to $E_c = 11723$ MPa. Therefore, useful indications derived from this work lead to conduct a wide non-destructive survey in order to identify, in relative terms, the characteristics of the concrete in several structural elements. According to non-destructive test results, one or more homogeneous zones can be identified and the number and location of the cores can be established.

It should also be considered that the results of the compression tests could be influenced by possible cracking conditions of the material or damage caused by the extraction method. For this reason, sonic tomography represents a reliable tool which estimates the quality of concrete; moreover, its results can be used also to conduct forecast analyses, in line with the good results described in this work. In fact, F.E.M. analysis, carried out to evaluate the axial compression behaviour of the column portion, confirms that the use of sonic tomography with reliable correlation laws is an accurate method to estimate the concrete strength. The maximum axial compression force estimated from Model n.2 (according to tomography map results) is closer to experimental test result ($N_{max} = 874$ kN) than Model n.1 estimate (according to destructive test results). However, on the bending moment evaluation,

the concrete strength contribution results very limited; the distance between the concrete strains estimated from F.E.M. analysis (Model n.2) and the experimental measurement (related to upper potentiometer) is equal to 61%. Also, the distance between F.E.M. analysis (Model n.1, according to destructive test results) and the experimental measurement is high (it is about 88%); probably, the upper potentiometer measurement is influenced by inner frictions.

Chapter 4

Structural capacity evaluation: applications

4.1 Abstract

Semi-probabilistic approaches (Design Value Method and Adjusted Partial Factor Method) and full-probabilistic approaches (Monte Carlo simulation and Sensitivity Analysis) are applied to the structural capacity assessment of reinforced concrete existing elements (see Figure 4.1). These methods are applied to the structural bending moment of a beam portion and to the axial compression of a column portion, considering a variable number of material tests and consequently strength values. The structural capacities evaluated with semi and full-probabilistic approaches are compared with experimental tests carried out on the structural elements. Moreover, the Italian code approach, which considers a reduction factor (e.g. Confidence Factor) for mean values of strengths, which is determined according to the degree of knowledge (Knowledge Level) of the structure/element and it is applied to the same structural capacities.

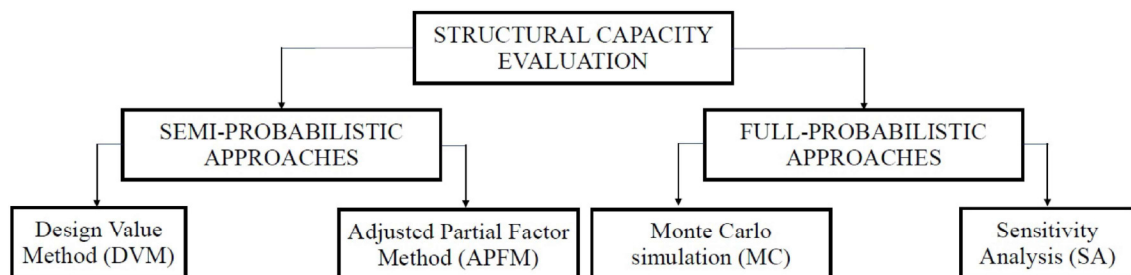


FIGURE 4.1: Flowchart of applied methods to evaluate the structural capacity

Approcci semi-probabilistici (Design Value Method e Adjusted Partial Factor Method) e puramente probabilistici (simulazione Monte Carlo e Analisi di Sensitività) sono applicati

per la valutazione della capacità strutturale di elementi esistenti in calcestruzzo armato. In questo Capitolo, tali applicazioni hanno l'obiettivo di stimare la capacità strutturale di momento flettente di una porzione di trave e la compressione assiale di una porzione di pilastro, considerando un numero variabile di test sul materiale e quindi differenti valori di resistenza. I risultati dei metodi sono confrontati con i test sperimentali realmente condotti sugli elementi strutturali. Inoltre, tali capacità strutturali sono valutate anche utilizzando l'approccio normativo italiano, che considera un fattore di riduzione delle resistenze medie (Fattore di Confidenza), determinato in funzione del grado di conoscenza (Livello di Conoscenza) della struttura/dell'elemento.

4.2 Partial Factor Methods

According to the Partial Factor Methods, which has been presented previously, design values of bending moment capacity and axial compression capacity are evaluated using the Design Value Method and Adjusted Partial Factor Method. The main steps of these method are summarized in Figure 4.2.

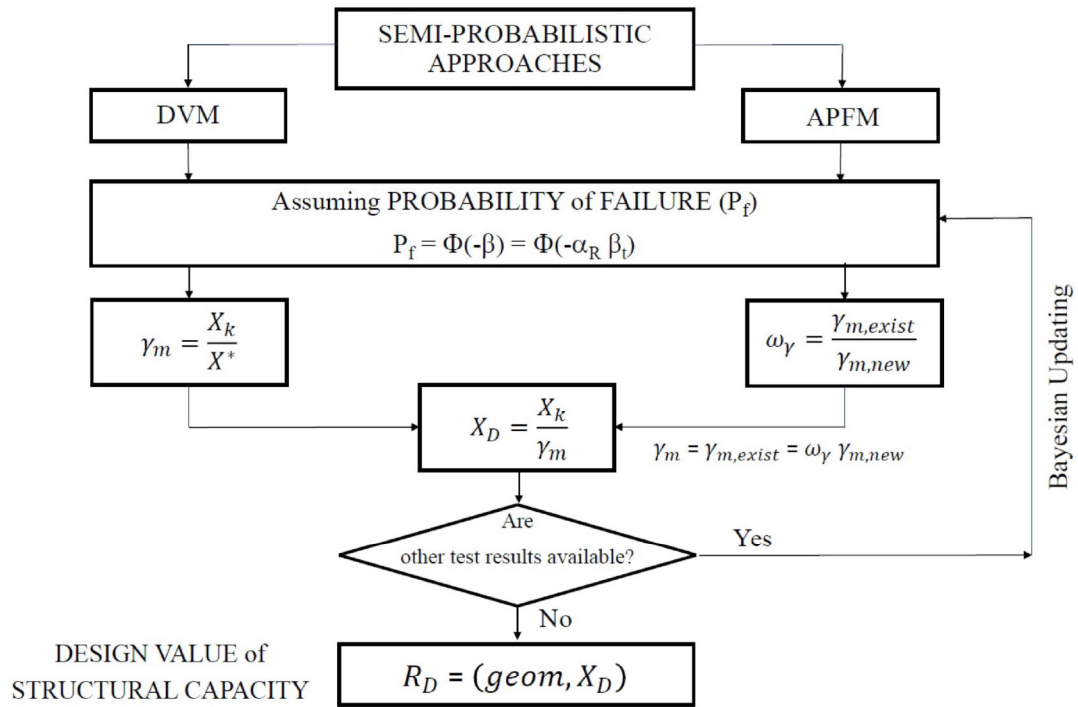


FIGURE 4.2: Flowchart of the Partial Factor Methods applied to evaluate the structural capacity

4.2.1 Applications

In this Section, the structural capacity of the RC elements extracted from an Italian school building (presented in the previous Chapter) is evaluated with the *Partial Factor Methods*.

Starting from the experimental test results (e.g. concrete and steel reinforcement strengths), the structural capacities are calculated according to different knowledge levels of materials: in other words, starting from the complete sample of experimental tests, all the possible sub-samples are considered to estimate the structural capacity. In this way, all possible combinations of data tests are evaluated to estimate the structural capacity.

The reasons of this choice are described:

- The number of experimental tests performed on the presented case studies are unrealistic for the same element, where only few destructive tests are permitted;
- Considering all the combinations, it is possible to evaluate the real variability of the material properties (thus structural capacity variability), according to the different samples extracted.

The structural capacities examined in this section are:

1. *Resisting bending moment capacity*, related to the beam portion extracted from a school RC building built in 1940 (placed on I floor). The capacity model is described:

$$M_{rd} = A_s * f_{yd} \left(d - 0.5 \frac{A_s * f_{yd}}{b * f_{cd}} \right) \quad (4.1)$$

where:

A_s is the longitudinal area of the steel reinforcement of the beam;

d is the effective depth of the beam cross section;

b is the width of the beam cross section;

f_{yd} are the design values of the steel reinforcement yield strength, according to the different test samples;

f_{cd} is the design value of the concrete compressive strength;

2. *Resisting axial compressive capacity*, related to the column portion extracted from a school RC building built in 1940 (placed on II floor, 1957 - built later). The Capacity model is described in the equation 4.2.

$$N_{rd} = 0.8 * f_{cd} * A_c + A_s * f_{yd} \quad (4.2)$$

where:

A_s is the longitudinal area of the column steel reinforcement;

A_c is the area of the column cross-section;

f_{yd} is the design value of the steel reinforcement yield strength;

f_{cd} are the design values of the concrete compressive strength, according to the different test samples;

Starting from the data tests described in the *experimental activity*, the design value of the material properties is calculated according to the equation 2.44; the design values of the steel reinforcement yield strength and concrete compressive strength are shown through the equations 4.3 and 4.4, respectively.

$$f_{yd} = \frac{f_{yk}}{\gamma_S} \quad (4.3)$$

$$f_{cd} = \frac{f_{ck}}{\gamma_C} \quad (4.4)$$

4.2.1.1 Resisting bending moment capacity

Resisting bending moment capacity is calculated according to the data tests evaluated previously. Mechanical characteristics, geometry and experimental test of the portion of beam are described in the same chapter.

With the aim of evaluating the structural safety, according to the same probability of failure for the new structures, the results of the partial factor methods (material strengths and structural capacities) are calculated with target reliability index $\beta = 3.8$. In the Figures 4.3 and 4.4 the design values of the steel reinforcement yield strength and concrete compressive strength, respectively are shown. Starting from 5 steel reinforcement samples, all

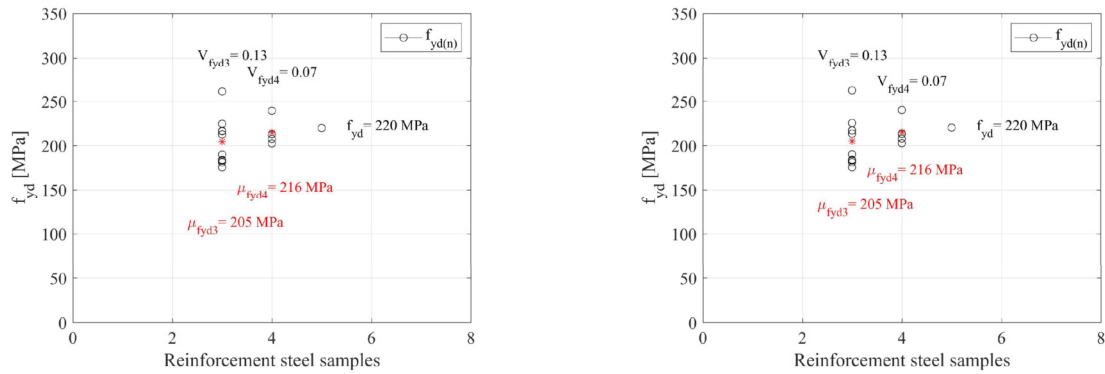


FIGURE 4.3: Design values of the steel reinforcement yield strength according to different number of extracted samples: on the top DVM, on the bottom APFM

the possible sub-samples are evaluated and respective design values of the yield strength are determined. Figure 4.3 shows that DVM and APFM results are almost the same: the uncertainty of the results related to 3 samples is about 13% ($V_{fyd3} = 0.13$).

In the Figure 4.4, starting from 6 concrete compressive tests, all the possible combinations of sub-samples are considered to evaluated the design values of concrete strength. It is noted the great dispersion of results considering few samples: for example with 3 tests,

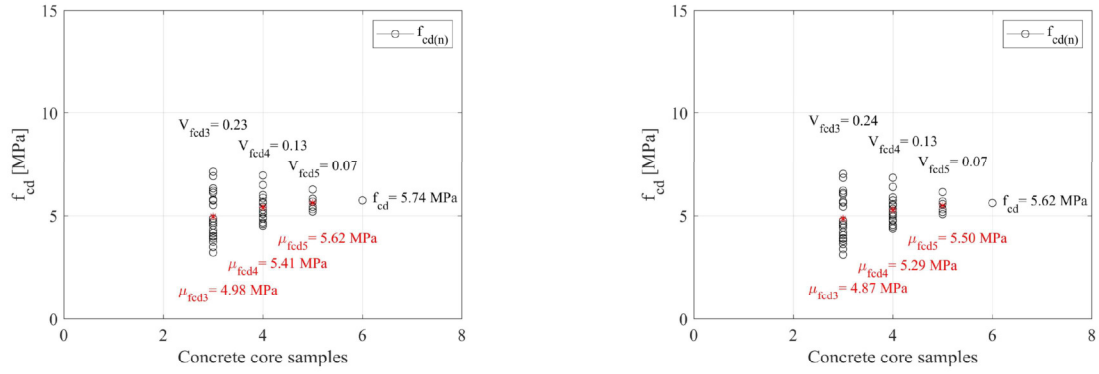


FIGURE 4.4: Design values of the concrete compressive strength according to different number of extracted samples: on the top DVM, on the bottom APFM

the coefficient of variation is $V_{f_{cd3}} = 0.24$, for DVM and $V_{f_{cd3}} = 0.23$, for APFM. Moreover, the difference between the results according to the maximum available data test is $f_{cd}^{APFM}/f_{cd}^{DVM} \approx 0.98$. These results prove that there are no differences (from the engineering point of view) between two methods.

On the resisting bending moment capacity, the dominant variable is the steel reinforcement strength; for this reason in the following results the yield strength is the only variable. Concrete strength f_{cd} is assumed according to the maximum data tests (e.g. 6 test results). In Figure 4.5, the resisting bending moment capacity is evaluated according to a series of data tests. The capacity results show less variability than basic variables (see Figure 4.3):

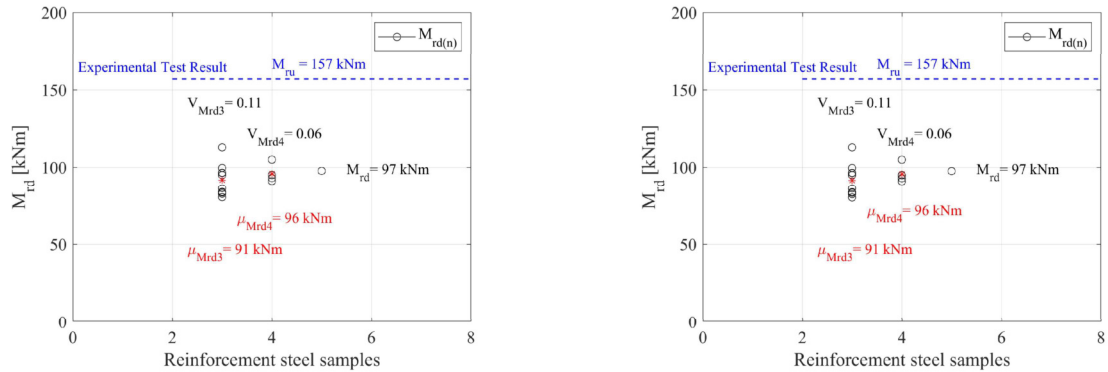


FIGURE 4.5: Design values of the bending moment capacity according to different number of extracted samples: on the top DVM, on the bottom APFM

the coefficient of variation, related to sub-sample of 3 tests, is about $V_{M_{rd3}} = 0.11$. Also the mean values, for each sub-sample, is less variable: from 3 tests to 5 tests (maximum number of available data) the values are over $M_{rd} = 90 \text{ kNm}$.

Figure 4.5 shows the resisting bending moment capacity calculated with DVM and APFM; the results from these methods are compared with the experimental test result, described in the previous Chapter.

For each sub-sample of steel reinforcement tests, the maximum, minimum and mean val-

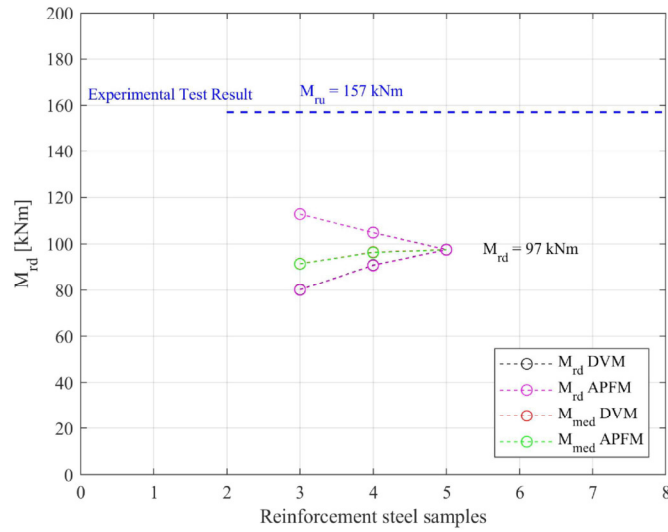


FIGURE 4.6: Comparison between DVM and APFM: resisting bending moment capacity related to portion of beam extracted from school RC building, built in 1940

ues of the resisting bending moment capacity are evaluated and their results are shown in Figure 4.6. In this Figure DVM and APFM are compared in the same graphic and it is possible to notice the distance between partial factor method results and experimental test. It is well known that partial factor format is more conservative than real failure. Indeed, the ratio capacity is $M_{rd}/M_{ru} \approx 0,62$ (where M_{rd} is related to the maximum available tests).

4.2.1.2 Resisting axial compressive capacity

Starting from the same material test results considered in the subsection 4.2.1.1, the resisting axial compressive capacity is calculated for the portion of column described in the *experimental activity*.

Also in this case, the material strengths and structural capacities are calculated assuming the target reliability index $\beta = 3.8$. Design values of steel and concrete strengths are shown in Figures 4.3 and 4.4, respectively.

Concrete compressive strength is a dominant variable on the resisting axial compressive capacity, thus the capacity results are evaluated assuming f_{cd} as only one variable. Steel strength contribution is not considered variable and it is assumed $f_{yd} = 220MPa$ (yield strength related to 5 samples, see Figure 4.3).

Figure 4.7 shows the resisting axial compressive capacity calculated according to a series of concrete samples; for each sub-sample, the mean value and coefficient of variation of axial compressive capacity are evaluated. All possible results of the axial compressive capacity

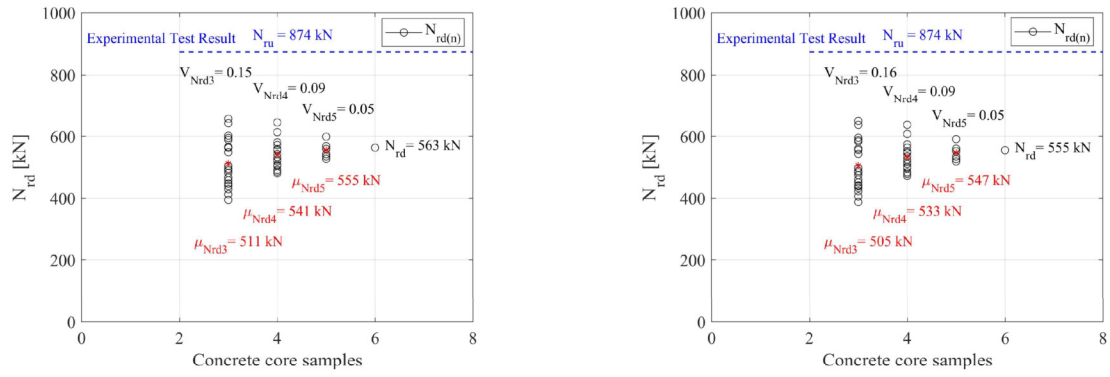


FIGURE 4.7: Design values of the axial compressive capacity according to different number of extracted samples: on the top DVM, on the bottom APFM

are determined using DVM and APFM; furthermore, the experimental test result is displayed on both graphics. Also in this case, the results evaluated with DVM and APFM are very similar: the coefficient of variation considering 3 samples is about $V_{N_{rd3}} \approx 0.15$ and the capacity ratio evaluated for the maximum material test result is $N_{rd}^{APFM}/N_{rd}^{DVM} \approx 0.98$. For each sub-sample of concrete strength, the maximum, minimum and mean values of the resisting axial compressive capacity are shown in Figure 4.8.

Figure 4.8 shows a good distance from the resisting axial compressive capacity to the

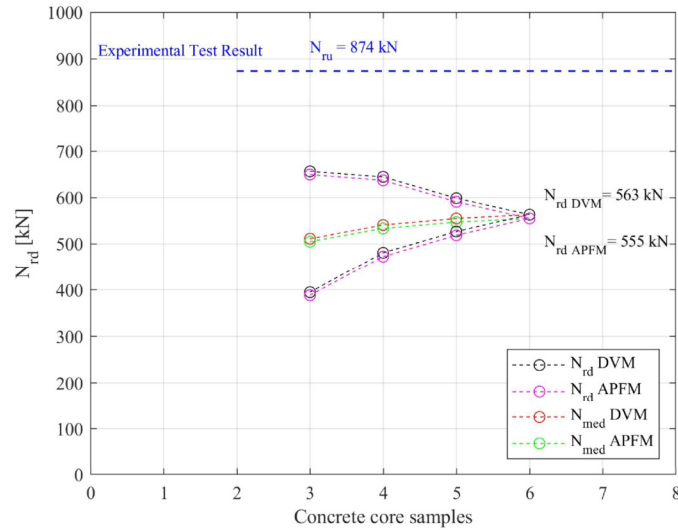


FIGURE 4.8: Comparison between DVM and APFM: resisting axial compressive capacity related to a portion of column extracted from a school RC building, built in 1940

experimental test result line: indeed, the ratio capacity $N_{rd}/N_{ru} \approx 0,64$ (where N_{rd} is related to the maximum available tests).

In this Section two alternative methods are presented to obtain partial factors for existing structures: the design value method (DVM) and the adjusted partial factor method

(APFM). Both methods basically assess the partial factor γ_m in a similar way while differing slightly in estimating γ_{Rd} . From the engineering point of view DVM and APFM lead to the same results: the mean values and the coefficient of variation of capacities (for each sub-sample considered herein) are almost the same. However, APFM is slightly more conservative than DVM.

4.3 Advanced Statistical Methods

According to the Advanced Statistical Methods, which has been presented previously, a design values of bending moment capacity and axial compression capacity are evaluated using Monte Carlo simulation and Sensitivity Analysis. The main steps of these method are summarized in Figure 4.9.

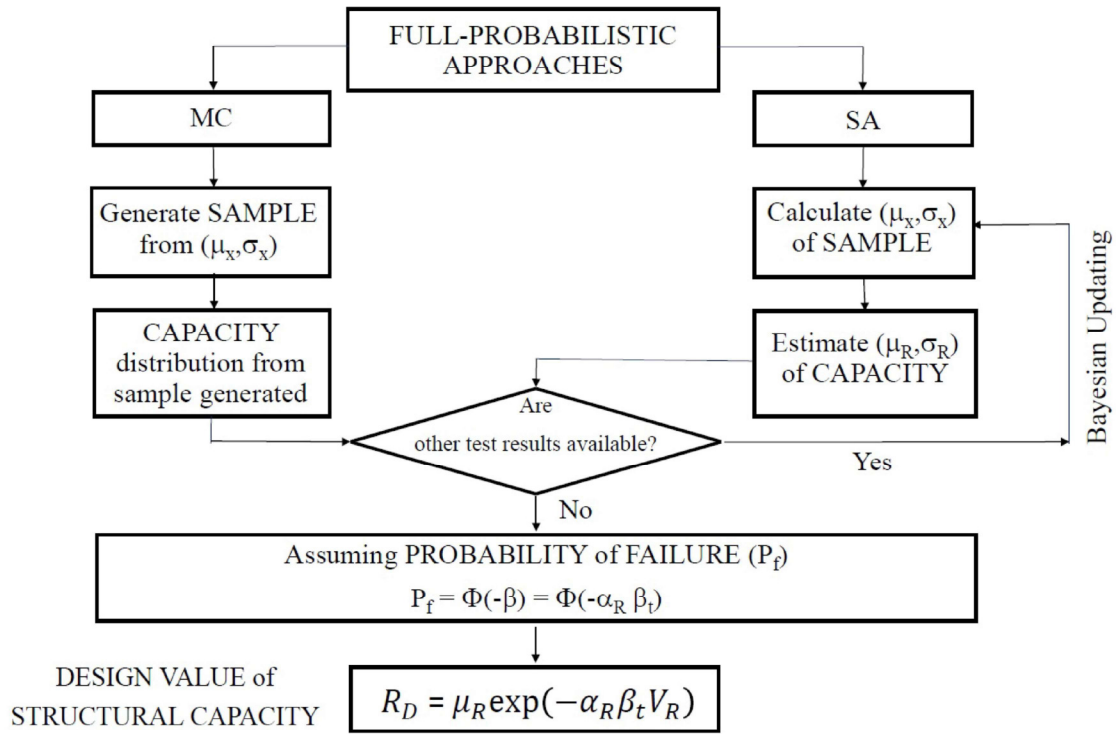


FIGURE 4.9: Flowchart of Advanced Statistical Methods applied to evaluate the structural capacity

4.3.1 Applications

In this Section structural capacities, which have been evaluated above with the partial factor method, are determined with advanced statistical methods: Monte Carlo simulation and Sensitivity Analysis.

The two methods, presented in 2.7.1 and 2.7.2, are applied to the following structural capacities:

1. *Resisting bending moment capacity*, related to the beam portion extracted from a school RC building built in 1940 (placed on I floor). The capacity model is described:

$$M_{rd} = A_s * f_y \left(d - 0.5 \frac{A_s * f_y}{b * f_{cm}} \right) \quad (4.5)$$

where:

A_s is the longitudinal area of the beam steel reinforcement;

d is the effective depth of the beam cross section;

b is the width of the beam cross-section;

f_y is the steel reinforcement yield strength (basic variable both in Monte Carlo generation and in Sensitivity Analysis);

f_{cm} is the mean value of the concrete compressive strength, according to the maximum number of available tests;

2. *Resisting axial compressive capacity*, related to the column portion extracted from a school RC building built in 1940 (placed on II floor, 1957 - built later). The Capacity model is described in the equation 4.6.

$$N_{rd} = 0.8 * f_c * A_c + A_s * f_{ym} \quad (4.6)$$

where:

A_s is the longitudinal area of the column steel reinforcement;

A_c is the area of the column cross-section;

f_{ym} is the mean value of the steel reinforcement yield strength, according to the maximum number of available tests;

f_c is the concrete compressive strength (basic variable both in Monte Carlo generation and in Sensitivity Analysis);

With the aim to assume the same probability of failure both in the Partial Factor Methods and in the Advanced Statistical Methods, FORM-based method is applied to the structural capacities presented above (see equations 4.5 and 4.6). Assuming a Lognormal distribution for the structural capacities, the design values are calculated by the equation 2.58, for Monte Carlo method and by the equation 2.62, for Sensitivity Analysis. On both methods, a target reliability index $\beta = 3.8$ is assumed.

4.3.1.1 Resisting bending moment capacity

The results of bending moment capacity shown in Figures 2.21 and 2.22 are compared in the same graphic (see Figure 4.10). In this Figure, Monte Carlo simulation (MC) and Sensitivity

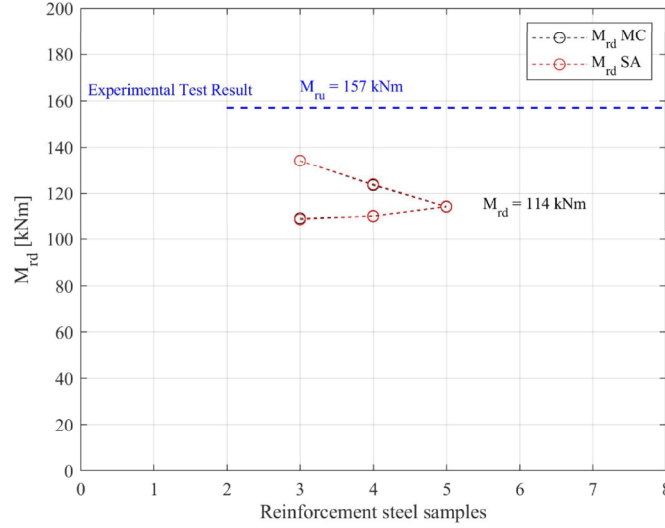


FIGURE 4.10: Comparison between MC and SA: resisting bending moment capacity related to portion of beam extracted from school RC building, built in 1940

Analysis (SA) are compared with the experimental test result presented in Chapter 2. It is easy to understand that both methods lead to the same results. As seen in partial factor application (especially in the subsection 4.2.1.1), the design values of the bending moment are lower than in the experimental test result. Moreover, the ratio capacity $M_{rd}/M_{ru} \approx 0,73$ (where M_{rd} is related to the maximum available tests).

4.3.1.2 Resisting axial compressive capacity

The design value of the axial compressive capacity is evaluated using the equations 2.58 and 2.62 for Monte Carlo simulation and Sensitivity Analysis, respectively. The possibility to assume the Lognormal distribution for the axial compressive capacity is illustrated in Figure 4.11, where the capacity, calculated with the generated sample of f_c , fits the probability density function of lognormal distribution.

According to a series of sample tests, the design values of the axial compressive capacity are shown in Figure 4.12. Also in this case, both methods lead to the same results. In Figure 4.12, it is clear that for few concrete core samples (i.e. $n = 3$), where usually the capacity variability is great, estimated structural capacity crosses the experimental test result line. The latter situation represents an unsafety condition. However, the ratio capacity $N_{rd}/N_{ru} \approx 0,86$ (where N_{rd} is related to the maximum available tests).

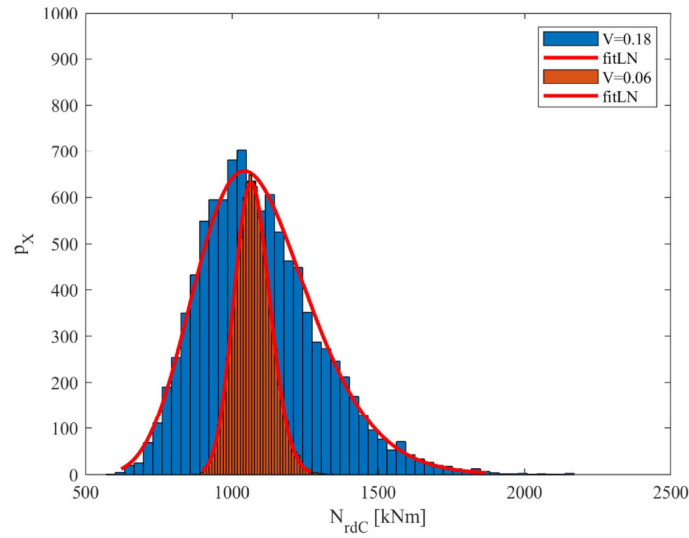


FIGURE 4.11: Lognormal distribution fitting of probability density functions of resisting axial compression capacity calculated from two samples generated according to the maximum and the minimum coefficient of variation of concrete compressive strength for the sub-sample with $n = 3$

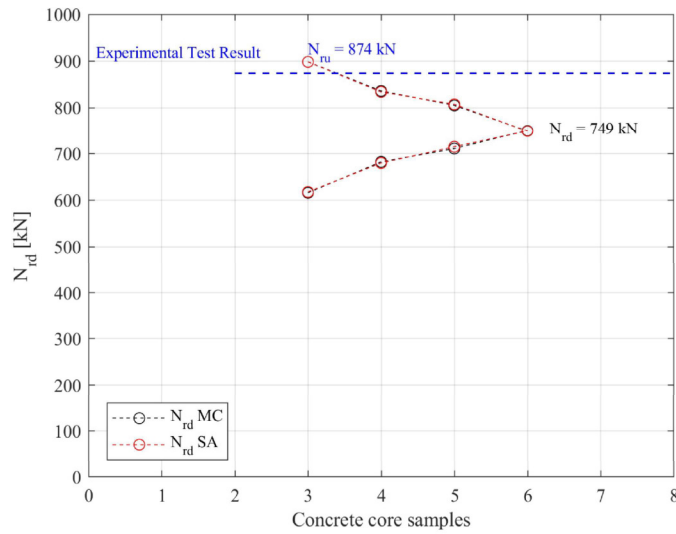


FIGURE 4.12: Comparison between MC and SA: resisting axial compressive capacity related to a portion of column extracted from school RC building, built in 1940

4.4 Comparison between Partial Factor Methods and Advanced Statistical Methods

Starting from the previous results shown in the Sections 4.2.1 and 4.3.1, it is possible to notice that *Design Value Method* and *Adjusted Partial Factor Method* lead to the same results. Moreover, if we compare *Monte Carlo method* and *Sensitivity Analysis* very similar results are obtained (from the engineering point of view).

Thus, in Figure 4.13 four alternative methods are presented to obtain design values of the

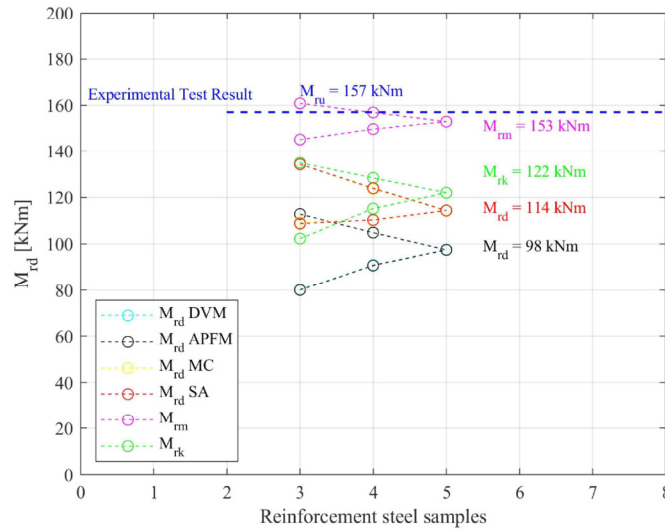


FIGURE 4.13: Comparison between DVM, APFM, MC and SA: resisting bending moment capacity related to portion of beam extracted from school RC building, built in 1940

bending moment capacity: two partial factor methods (DVM-APFM) and two advanced statistical methods (MC-SA). Both statistical approaches basically assess the design values of the capacity model in similar ways, they calculate the mean value and the standard deviation of the bending moment (in this case only steel reinforcement is considered as variable). DVM and APFM assess the capacity model starting from the design value of the material resistances, but only the dominating material is considered as variable (i.e. steel reinforcement for bending moment capacity). As previously seen, both statistical approaches lead to the same results and both partial factor methods lead almost to the same results. However, the latest methods are more conservative. Moreover, bending moment evaluated with mean values (M_{rm}) and characteristic values (M_{rk}) are calculated. The results of DVM, APFM, MC and SA compared to the capacity models evaluated with the mean and the characteristic values of material properties lead to lower values. However, the results of MC and SA (evaluated for probability of failure $P_f = 0.001$) and the results of *characteristic capacity model* M_{rk} ($P_f = 0.05$) are not very distant. In Figure 4.14, all methods are applied to the resisting axial compressive capacity, related to a portion of column extract from the II floor of school RC building. The latter was built in 1940, but the II floor was built later (1957).

On a design value capacity evaluation, n. 6 concrete core samples (extracted from the I floor of the same building) are considered in order to evaluate the resisting axial compression. However, the mean values of the concrete strength from the I floor are higher than those from the II floor: in the first case, f_{cm} is about 12 MPa. For this reason, the results of the resisting axial compressive capacity, evaluated with mean value (N_{rm}) are higher than the experimental test result ($N_{ru} = 874$ kN). Moreover, *mean axial compression capacity*

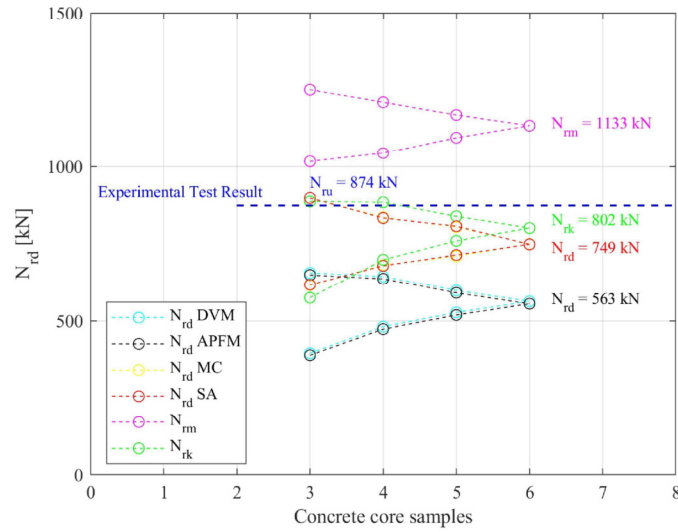


FIGURE 4.14: Comparison between DVM, APFM, MC and SA: resisting axial compressive capacity related to a portion of column extracted from school RC building, built in 1940

(N_{rm}) leads to higher results, related to N_{rd} calculated with partial factor methods (the difference is about 100%). Also in this case, both statistical approaches and the partial factor methods lead to very similar results, but latest methods are more conservative. Finally, it can be affirmed that assuming a FORM-based method on basic variables (as presented in Section 4.2) leads to very different results related to assume the FORM method on structural capacity (as described in Section 4.3); it means that assuming the same probability of failure on both the basic variables and the structural capacity model don't lead to the same results.

It is also true that the capacity model evaluated with the advanced statistical approaches is only sensitive to one basic variable (the dominant one), whereas on partial factor methods the probability of failure is applied to all the basic variables (material strength) included in the capacity model.

4.5 Italian Code approach - NTC2018

According to the Italian code approach, which considers a reduction factor (e.g. Confidence Factor) for mean values of strengths, determined according to the degree of knowledge (Knowledge Level) of the structure/element, the structural capacities of the bending moment and axial compression are evaluated.

4.5.1 Applications

In this Section, the Italian Code approach is applied on the same cases studied with partial factor methods and advanced statistical methods. In order to evaluate the structural capacity, the analytical model considers mean values of material properties reduced according to the *confidence factor* (FC). The following structural capacities are evaluated:

1. *Resisting bending moment capacity*, related to the beam portion extracted from a school RC building built in 1940 (placed on I floor). The capacity model is described:

$$M_{rd} = A_s * \frac{f_{ym}}{FC} \left(d - 0.5 \frac{A_s * f_{ym}/FC}{b * f_{cm}/FC} \right) \quad (4.7)$$

where:

A_s is the longitudinal area of the steel reinforcement of the beam;

d is the effective depth of the beam cross section;

b is the width of the beam cross section;

f_{ym} are the mean values of the steel reinforcement yield strength, related to a series of samples;

f_{cm} is the mean value of the concrete compressive strength, according to the maximum number of available tests;

FC is the confidence factor according to the desired knowledge level;

2. *Resisting axial compressive capacity*, related to the column portion extracted from a school RC building built in 1940 (placed on II floor, 1957 - built later). The Capacity model is described in the equation 4.8.

$$N_{rd} = 0.8 * \frac{f_{cm}}{FC} * A_c + A_s * \frac{f_{ym}}{FC} \quad (4.8)$$

where:

A_s is the longitudinal area of the steel reinforcement of the column;

A_c is the area of the column cross-section;

f_{ym} is the mean value of the steel reinforcement yield strength, according to the maximum number of available tests;

f_{cm} are the mean values of the concrete compressive strength, related to a series of samples;

FC is the confidence factor according to the desired knowledge level;

4.5.1.1 Resisting bending moment capacity

As seen on the presented methods (DVM, APFM, MC and SA), the resisting bending moment capacity is evaluated according to different reinforcement steel samples. In Figure

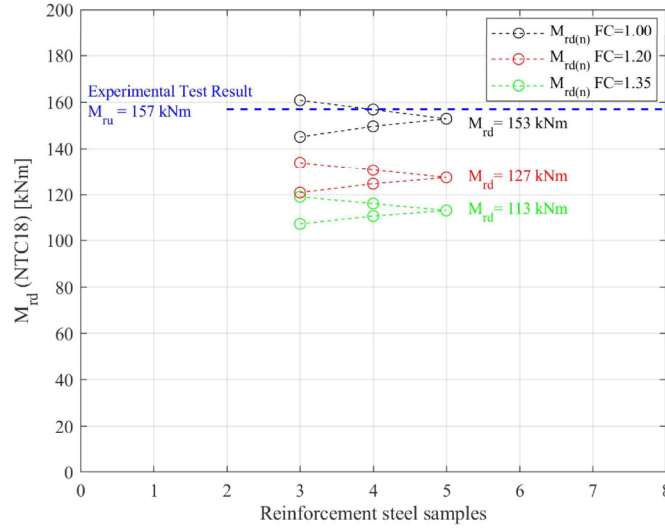


FIGURE 4.15: The Italian Code approach applied to the resisting bending moment capacity related to a portion of beam extracted from a school RC building, built in 1940

4.15, three different structural capacities are shown according to the desired knowledge level (LC1, LC2 and LC3). For a few steel samples, the result variability is smaller than that determined with other methods; in this case, mean values of yield steel strength f_{ym} are less variable. Obviously, assuming $FC = 1.00$ means to evaluate the structural capacity with the mean values of the data tests (usually, these results are very close to the experimental test). However, the ratio capacities are $M_{rd}/M_{ru} \approx 0,81$, assuming $FC = 1.20$ and $M_{rd}/M_{ru} \approx 0,72$, assuming $FC = 1.35$ (where M_{rd} is related to the maximum available tests, in this case n. 5 reinforcement steel samples).

4.5.1.2 Resisting axial compressive capacity

Starting from the same data tests used to estimate the resisting bending moment capacity and the resisting axial compression capacity are calculated according to the Italian Code approach, related to a series of test samples. In this model capacity, concrete strength is the only variable.

As previously described, the tested portion column was extracted from the II floor of a school building which was probably built after the I floor. For this reason, the capacity results are very high: in Figure 4.16, the experimental test line is between the model capacity evaluated with $FC = 1.20$ and the model capacity evaluated with $FC = 1.35$. It is proved that the Italian Code approach is very sensitive to the test results, thus it is

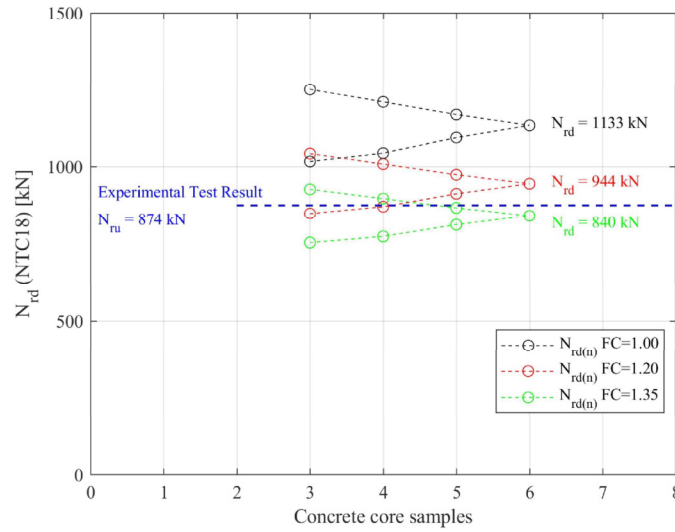


FIGURE 4.16: The Italian Code approach applied to the resisting axial compressive capacity related to a portion of column extracted from a school RC building, built in 1940

important to identify the part of the concrete structure that can be considered homogeneous in order to correctly evaluate the structural capacity.

4.6 Conclusions

The structural capacity assessment of existing reinforced concrete structures is a key point for the evaluation of a proper repair and retrofitting interventions. In this framework, the experimental activity shows a great variability of material properties, especially regarding the concrete strength of the reinforced concrete structures built at the beginning of the 20th century. Moreover, destructive tests on concrete and steel specimens provide the most reliable measurement of strength. However, the number of specimens usually accepted is often limited because these methods are expensive and invasive. For this reason, the processing of the required data represents a crucial step of the structural capacity evaluation.

In this Chapter, four alternative methods have been presented in order to obtain design values of the structural capacities: two Partial Factor Methods (Design Value Method and Adjusted Partial Factor Method) and two Advanced Statistical Methods (Monte Carlo simulation and Sensitivity Analysis).

Design Value Method (DVM) and Adjusted Partial Factor Method (APFM) allow the assessment of the capacity model starting from the design value of the material resistances, but only the dominating material is considered as variable (i.e. a steel reinforcement for the bending moment capacity). Both methods basically assess the material partial factor γ_m in a similar way, while differing slightly in the estimate of the model partial factor γ_{Rd} . From the engineering point of view, DVM and APFM lead to the same results (e.g. the difference is about $X_{D,APFM}/X_{D,DVM} \approx 0.98$). Furthermore, the mean values (μ_R) and

the coefficients of variation (V_R) of structural capacity, evaluated for each sub-sample, are almost equal; however, APFM is slightly more conservative than DVM.

The advanced Statistical Methods basically assess the design value of the structural capacity in a similar way, calculating the mean value and the standard deviation of the capacity model R ; also in this framework, the dominating material strength is considered as variable. However, both Advanced Statistical Methods and Partial Factor Methods lead to almost the same results, but the latest ones are more conservative.

In bending moment capacity evaluation, DVM, APFM, MC and SA, compared with the capacity model evaluated with mean and characteristic values of material properties, lead to lower values. Moreover, the results of MC and SA, evaluated for $P_f = 0.001$ (e.g. $\beta = 3.8$), are not very distant from the results of characteristic capacity model (M_{rk}), evaluated for $P_f = 0.05$. If we compare the experimental test results, Partial Factor Method results (evaluated for the total number of material samples) are almost 62%, whereas Advanced Statistical Methods (evaluated for complete number of samples too) are almost 73%. The distance, in terms of percentage, can be reduced by using lower values of failure probability, that is the target reliability index β (see Appendix).

In the resisting axial compression evaluation, six concrete core samples extracted from a first floor column are considered (the mean values of the concrete strength are higher than for the second floor column, f_{cm} is about 12 MPa). For this reason the results of the resisting axial compression with the mean values (N_{rm}) are higher than the experimental test result ($N_{ru} = 874$ kN). Resisting axial compression evaluated with the mean values leads to higher results related to N_{rd} calculated with the Partial Factor Methods (the difference is about 100%). Also in this case, DVM, APFM, MC and SA, compared with the capacity model evaluated with mean and characteristic values of the material properties, lead to lower values. However, the results of MC and SA, evaluated for $P_f = 0.001$ (e.g. $\beta = 3.8$), are very close to the results of characteristic capacity model (M_{rk}), evaluated for $P_f = 0.05$. Moreover, considering few samples with high variability, that is sub-sample with n.3 test and V_{max} (e.g. upper bound of spindle), the Advanced Statistical Method results cross the experimental test result line (e.g. unsafe condition). Compared to the experimental axial compression test result, the Partial Factor Method results (evaluated for complete number of material samples) are almost 64%, whereas the Advanced Statistical Methods (evaluated for complete number of samples too) are almost 86%.

Therefore, the Partial Factor Methods are the most conservative approaches, because its results are almost 65% related to experimental tests. On the other hand, the Advanced Statistical Methods are more sensitive to the choice of the test samples.

Finally, the Italian code approach for the existing structure evaluation is applied to the same cases; according to this approach, a Confidence Factor ($FC = 1.00$, $FC = 1.20$ or $FC = 1.35$), determined according to the Knowledge Level ($LC1$, $LC2$ and $LC3$, respectively), is applied to mean values of the material strengths. However, also the Italian code

approach is very sensitive to the test samples: assuming $FC = 1.35$, its results are very similar to the Advanced Statistical Method ones. Obviously, assuming $FC = 1$ the Italian code approach results coincide with structural capacity evaluated with the mean values of the material strengths.

Chapter 5

Appendix: structural capacity evaluation assuming different values of β

In this appendix, the structural capacity evaluated with previous methods are compared assuming different probabilities of failure. It means that for partial factor methods (PFM) and statistical methods (SM), it was assumed different values of β : from $\beta = 3.8$ (medium failure consequences in Ultimate Limit State - $P_f = 0.001$) to $\beta = 2.3$ (very low failure consequences in Ultimate Limit State - $P_f \approx 0.07$).

In the Figure 5.1, bending moment capacity evaluated with different methods are compared

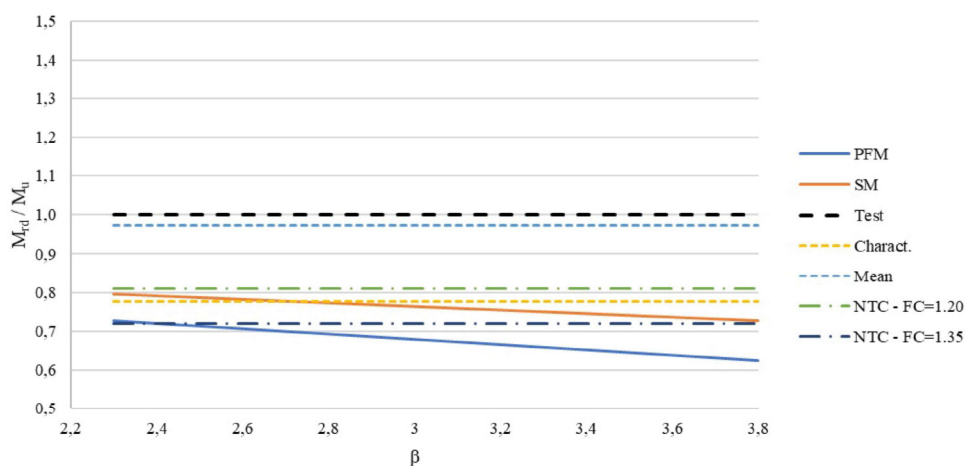


FIGURE 5.1: Resisting bending moment capacity assuming different values of β : portion of beam extracted from school RC building, built in 1940

to experimental test result, according to different values of β . Structural capacity results (related to the complete available sample tests) are divided with experimental test result; in this way, it is possible to check the distance from experimental test result according to different probabilities of failure.

It is noted that assuming $\beta = 2.3$, partial factor methods results (PFM) increase more than those of statistical methods (SA). With $\beta = 2.3$, PFM assume same value of Italian Code approach with $FC = 1.35$ (e.g. assuming limited knowledge level); with the same target reliability index, SM assume same value of Italian Code approach with $FC = 1.20$ (e.g. assuming extended knowledge level).

Assuming the same procedure seen above, in the Figure 5.2 axial compressive capacity evaluated with different methods are compared to experimental test result, according to different values of β . Also in this case, structural capacity results (related to the complete available sample tests) are divided with experimental test result. Assuming different values

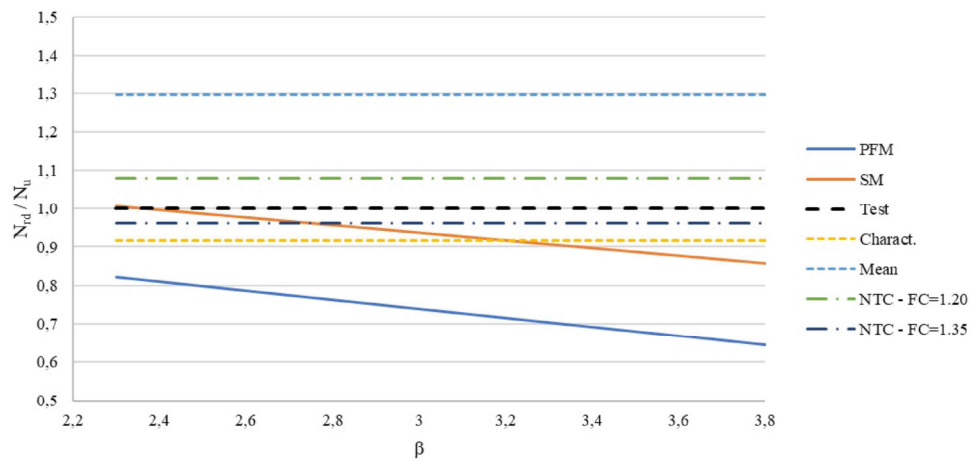


FIGURE 5.2: Resisting axial compression capacity assuming different values of β : portion of column extracted from school RC building, built in 1940

of β , the methods that don't cross experimental test result line are partial factor method (PFM), Italian code approach with $FC = 1.35$ (NTC - $FC = 1.35$) and axial compression evaluated with characteristic values of material properties (*charact.*)

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