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Safety and sustainability of school buildings

By

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Declaration

I hereby declare that, the contents and organization of this dissertation constitute my own original work and does not compromise in any way the rights of third parties, including those relating to the security of personal data.

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* This dissertation is presented in partial fulfillment of the requirements for **Ph.D. degree** in the Graduate School of Politecnico di Torino (ScuDo).

I would like to dedicate this thesis to my loving parents because, despite the distance, they have been a constant source of motivation, and to my beloved Giada, who has always cheered me on.

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Abstract

School facilities play a key role in civil society. They are first and foremost the place where everybody learns, grows and shapes the future, and where students spend a significant part of the day. Besides, schools provide strategic functions for local communities. Therefore, ensuring their safety should be a paramount concern, even if, in Italy, school buildings are too frequently neglected. However, the numerous collapses occurred in schools, have widely proven that this attitude represents a serious risk to the safety of students. Structural deficiencies are closely related to the ageing of Italy's school building stock, which in turn depends on the fact that the trend of new school constructions has always been closely linked to demographic trends. As a result, about 60% of Italian schools were built before 1975, i.e., according to codes rules that did not consider the current safety and structural standards. In addition, energy performance is also generally fairly low, resulting in a waste of money, and in a high environmental impact as well.

The relationship between the historical evolution of pedagogical models on the design approach of school buildings is evaluated herein by analysing both the worldwide context and the Italian case. In particular, the role of the standard module (classroom) and the organisation of teaching and connection spaces on the structural layout is investigated, also considering the technological evolution of structural materials and of construction techniques. The advancement of Italian structural and school building regulations, as well as the improvements and shortcomings progressively introduced, are also reviewed.

A survey on concrete and steel reinforcement extracted from reinforced concrete (RC) existing schools located in the Provincia di Torino is then carried out, comparing the results with a database of tests performed at Politecnico di Torino. The results reveal that the compressive strength of the extracted concrete is significantly lower than expected, especially in schools built between 1970 and 1990. Afterwards, the most frequent degradation factors and structural deficiencies in school buildings, alongside the relevant repair and retrofitting techniques, are illustrated. For each techniques, the pros and cons are discussed, focusing on the troubles in their implementation in schools.

A new and more affordable technique for reinforcing existing beams against bending moment using precast UHP-FRCC panels is also proposed. According to the results of the tests, this method proved to be promising also from a sustainability point of view.

Concerning environmental impact, a comparison of the equivalent CO_2 (CO_2 - eq.) emissions assessed with the LCA methodology of a new timber school and of a refurbished existing RC school is proposed. The aim is to define a solution that ensures the lowest greenhouse emissions over the life of the buildings. Despite the lower volume of new materials required for the refurbishment of the existing school, resulting in lower embodied CO_2, the long-term impact of this building is higher than that of the new school. However, the energy retrofitting enables offsetting the CO_2 - eq. emissions of the new materials within 14 years, while saving on the energy demand costs.

Finally, best practices are proposed on design approaches, retrofitting methods for structural and non-structural elements, along with recommendations on data collection and sharing between authorities, school building owners and Universities.

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Chapter 1

Introduction

The school building is a complex system. In contrast to other building types such as residences or offices, a multitude of functions and services are intertwined within them. In fact, besides the learning spaces for students, a school needs offices, meeting rooms, conference and sports halls, laboratories and, sometimes, healthcare facilities (infirmary). Furthermore, the characteristics required for such spaces have changed over the time, as like teaching techniques, construction and equipment technology, and local school population trends. In other words, the requirements of safety, accessibility, and hygiene rules have evolved, therefore school construction cannot be framed within a well-defined and unchanging building type.

Safety in school buildings is addressed herein as well. Nevertheless, safety is another complex topic to deal with, covering a wide range of fields (e.g. safety against natural disasters, fire safety, workplace safety, structural safety and safety against external factors and actions). Moreover, safety is an abstract concept. Indeed, human activity is broken down into objectives to be achieved by pursuing a series of actions. When the desired result (e.g. the construction of a building or the holding of an event) and the relevant deadline are well defined and measurable, planning the activities is relatively straightforward. Achieving the goal is perceived as a "reward" for the work done and thereby a stirring for reaching further goals. Maintaining safety runs in the opposite direction: in fact, it consists of a series of actions aimed at ensuring that something (perceived as negative) does not happen, such as injury or loss of human life during a natural disaster, or due to a collapse, or external factors. It can be said that ensuring the integrity and health of people day-by-day is the "reward" to strive for. In other words, all efforts should be pursued to avoid further causalities due to design mistakes, poor maintenance and, in general, by low safety levels. As the “zero risk” goal is impossible to achieve, the structural designer must strike a dauting balance between an adequate level of safety and the feasibility of implementing measures to obtain it.

In this work, the heterogeneous and complex Italian school building stock is analysed, investigating critical issues and flaws that lead to safety deficiencies. Safety is addressed mainly from a structural standpoint, analysing the shortcomings of past and current construction techniques and rules for structural and non-structural elements. The retrofitting methods are reviewed, yet innovative approaches were proposed as well. Finally, sustainability is also considered. Indeed, building a new school, or retrofitting an existing one, represents a twofold opportunity. On the one hand, it allows innovative materials and techniques (with a low carbon footprint) to be applied and tested. On the other hand, energy-saving measures can be implemented to reduce both greenhouse gas emissions and the operating costs of school buildings.

1.1 Aims

This PhD thesis is intended to be a dissemination document, thus also aimed at a non-engineer audience. The topics are addressed with a simple approach, avoiding numerical models and complex mathematical expressions, which can be further investigated by referring to the extensive relevant technical literature. Indeed, safety is a matter that involves everyone, as the effects of low safety have an impact on the whole community. Moreover, the school is an iconic place, because all of us have frequented it, and many will still have to deal with it, like workers, parents and pupils. School users are particularly sensitive and vulnerable, as it hosts children and teenagers (i.e., the future generation), therefore, the culture of safety awareness becomes even more crucial in these buildings

The novelty introduced to this work, compared to the existing literature, consists of addressing each topic through its implementation in the specific case of school buildings. For instance, for each type of structural vulnerability, the frequency and degree of the related risk in a school building are both specified. Similarly, a structural retrofitting technique is discussed by analysing potential conflicts with the use of pedagogical spaces. More details are only illustrated in the sections presenting the studies and investigations carried out during the PhD programme.

1.2 Methods

This PhD thesis consists of three parts:

1. To understand the critical issues affecting the Italian school buildings, an overview of the school building stock is provided. Specifically, an analysis of the historical evolution of construction techniques and functional models is discussed, starting from the international context and then focusing on the Italian case. Indeed, such factors guide the decision-making processes concerning the distribution of space and, thus, the structural parts of the buildings. The state-of-the-art of school construction is assessed both using the general information included in the open data published by the Ministry of Education, University and Research (MIUR), and by analysis a specific dataset of school structures collected during the PhD programme. As considering all the types of school buildings is not feasible, the typical issues of reinforced concrete buildings are investigated herein. The updating process of Italian structural and school building standards is analysed as well, to understand the reasons for the structural defects.
2. In the second part of the thesis, the most critical issues related to structural and non-structural elements that could lead to safety problems for students and school staff are discussed. In particular, research is devoted to non-structural elements, both because they are prone to significant vulnerability and because the current literature is poor on this argument. In fact, they were often roughly designed, constructed, inspected and maintained, because there was a lack of dedicated standards until a decade ago. Methods of strengthening structural elements and retrofitting strategies for enhancing the structural response are illustrated, pointing out their advantages and drawbacks in school buildings. Accordingly, a new method of strengthening existing reinforced concrete beams with precast Ultra High Performances – Fiber Reinforced Cementitious Composites (UHP-FRCCs) is proposed. The experimental activity carried out in the Life Cycle Engineering Laboratory (LCEL) at Tohoku University in Sendai, (Japan) to test the effectiveness is illustrated and analyses. Concerning the non-structural elements, the current Italian rules are analysed, identifying their limitations and proposing improvements. A review of established and innovative methods for retrofitting existing non-structural elements is also presented. Finally, recommendations are

proposed on inspection procedure for school buildings and data management methods.
3. The last part of this thesis is dedicated to the environmental impact of school buildings, albeit the sustainability is taken into account throughout the work. Specifically, a new timber school and an existing refurbished school are compared, assessing both the environmental impact related to the carbon footprint of building materials, and the greenhouse gas emissions due to energy consumptions. In other words, the Life Cycle Assessment (LCA) methodology extended to the entire lifespan of two schools was used to assess the short-term and long-term environmental impacts.

1.3 Limits

As already argued, safety covers several aspects, hence it was necessary to narrow down the analysis. Specifically, the safety of the construction components of the school building was mainly addressed, by investigating the structural and non-structural vulnerability, and studying the relevant mitigation methods. Nevertheless, accurate analysis of the vulnerability of the heterogeneous school building stock was not feasible. Therefore, the research only focused on reinforced concrete buildings because they are the most common structures in Italian schools built after the Second World War.

Although this study was initially aimed at gathering data on existing school buildings and retrofitting interventions of whole Italy, due to the considerable bureaucratic barriers, the research was restricted to the Turin Province.

Chapter 2

School building heritage: an historical background

Schools have always played a crucial role in the cultural growth of a nation, often being considered as an element of national identity. It is not surprising that wherever there was a change of mass education, schools gained prominence in political debate as a means of conveying new ideas and raising future generations, regardless of judgement ascribed to the intentions of the governments throughout history [1]. Although educational models, and school buildings as well, have been affected by local cultural influences and historical events, some common stages in their evolution can be identified, especially in Western countries. England was the first nation experienced the industrial revolution and, thus, made its way towards modern education. The English case was taken as a benchmark, referring to the detailed English Heritage report [2]. In the analysis of the evolution of pedagogical models, the case of the United States was also analysed later [3], because many educational theories on design approaches for educational spaces, still used today, originated in USA and then exported to other countries. Finally, a brief examination of the current European situation, is also presented.

Within this context, a historical survey of the Italian case is also studied, focusing on the design of learning environments.

2.1. A worldwide overview

In Western countries, until the mid-19th Century, elementary education for poor children was provided in modest school built and supported either by religious foundations, or by philanthropic entrepreneurs and benefactors. Classrooms were often inadequate, as well as the teaching staff, which often consisted of the oldest students. There were no official and uniform guidelines or rules for the construction of school buildings, therefore the layout of teaching spaces arbitrarily defined by the institutions and foundations that funded them.

In the case of England, between the 1600s and 1700s, education was provided in large classrooms within buildings, often including almshouses, hospitals, and workplaces for self-financing [2]. At the beginning of the 19th Century, Industrialization and the expansion of provincial cities led to the *Factory Act* of 1802, namely the first to be required to mill owners in order to provide elementary education for their working children. Although the Factory Act was initially scarcely used due to lack of inspections, its gradual tightening, along with increased child survival, led to a growing demand for education. To address the challenge of providing education to large masses of pupils cheaply, Joseph Lancaster published instructions on the layout of classrooms (Fig. 1), which advocated for fixed desks facing the teacher, whereas the sides of the room were clear for group work [4].

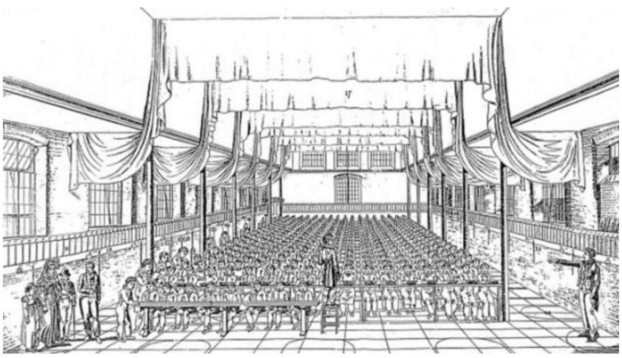

Fig. 1: classroom layout of the monitorial education model proposed by Lancaster [5]

This approach was exported in the USA in 1818 by the *British and Foreign School Society*. Lancaster's work was also followed by the 1815 report of the *National Society for Promoting the Education of the Poor in the Principle of the Established Church* which proposed, by contrast, movable desks arranged along the edges of classrooms to allow for most flexible small-group teach in the center. These approaches, still based on the so-called *monitorial system*, in which a single teacher controlled about 300 students (the older students delivered set lessons to the smaller ones), were challenged by the "*simultaneous method*". The letter, being more suitable especially for infants, encouraged a direct teaching between the mister/mistress and the student, to be carried out in the main classroom, to which a smaller classroom was added for the group teaching. Some specialists proposed classrooms with "galleries", featuring stepped seating, for handling large groups of students [6]. In 1840, the government of Education Committee (appointed in 1839) published 16 standard plans designed for the "mixed method" by merging monitored and simultaneous instruction. However, these classrooms did not take off because the monitoring method was preferred. As a matter of fact, until the 1880s, teaching was carried out in large rooms with a set of desks in the center, separated by curtains and, wherever possible, steps with seats. In the second half of the 1800s, government funding grew, but while day schools were built in large cities, boarding schools were mostly developed in rural areas. In the United States, until the first half of the 19th Century, schools were badly conceived and not properly located in the urban buildings because there were no design standards. Nevertheless, as cities grew, more debates was devoted to schools, seen as a springboard for the renewal of society. The first proposal for a standard model came from Horace Mann, who designed the classroom shown in Fig. 2 for about sixty students, with rows of desks, a teacher's desk on a platform and windows on two sides.

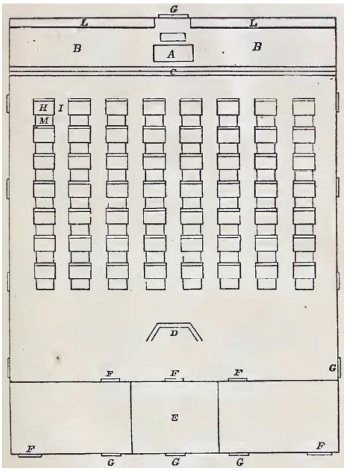

Fig. 2: one-room schoolhouse plan recommended by Horace Mann [7]

In 1870, the *Education Act* was applied by the British government, which led to the gradual raising of the compulsory school age. The so-called *school boards*, i.e. elementary schools managed by a school board composed of ratepayers, were introduced and gradually took over from church schools. Most board schools were arranged in separate classrooms (for infants, girls, boys) which could host up to 80 children each (Fig. 3), and a gallery, both made by brick or stone, having a simple shape and large windows. The school projects had to comply with the requirements imposed by the *Educational Department*, which approved them. Later, a central hall was also introduced, and soon became a standard for all board schools. Schools were often composed by a single-story, but could reach three stories when built in small-size sites. Also starting in the 1870s, the first high schools were built in large British cities to cope with the increasing need for teachers and office employees. These schools, being lower in number but having to serve large urban areas, were usually centrally located. Moreover, they consisted of large buildings as they contained many classrooms, albeit smaller in size, to provide for several specialization courses. In the United States, the

Kalamazoo Decision[1], the broadening of child labour laws, the end of the Civil War and the nation's growing momentum in the industrial revolution, played a key role in spreading education to the older age ranges. Yet, the resulting increase in demand for enrolment at the turn of the Century was tackled hastily by building 'factory-like' schools, featured by learning spaces designed to house as many students as possible, with little care of hygienic and pedagogical issues.

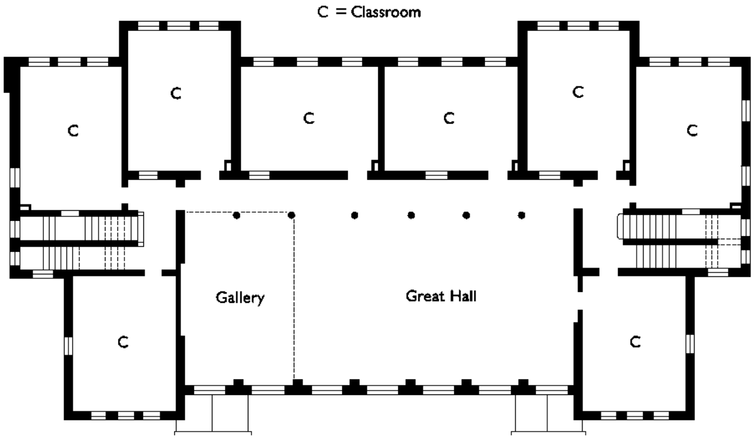

Fig. 3: the T Roger Smith's plan adopted for Jonson Street School, Stepney, by the London Board, which was the first to experiment with separate classrooms with a central schoolroom

In 1907, the *British Board of Health* introduced mandatory medical inspections in schools, which was a turning point because, for the first time, authorities put the needs of pupils and teachers first. As a result, the approach to school planning shifted substantially, having to be based on the students' and teachers' standpoints, ensuring their wellness. The architect George Widdows to develop an earlier idea of John Hutchings, built several and innovative schools featuring a row of classrooms, with usually south-facing windows, accessed by a covered verandah. The central classrooms were smaller, whereas those set at the ends accommodated about 60 students. Natural light and cross-ventilation was provided by dormer windows, although electric lighting was also provided. This type of layout quickly spread across the country, marking the end of central halls. Lagging behind Germany and the United States, the British government also increased its efforts in building secondary schools over the same period. However, such schools usually remained based on the old principles and emphasized social status, rather than the needs of students. Indeed, unlike elementary school, the

[1] The Kalamazoo Decision, or Kalamazoo Case, a landmark ruling in an 1874 case involving Kalamazoo Union High School in Michigan and local property owners filed, was a milestone in US public education, which until then had been reserved for common schools. It established that funding high schools with local property tax revenues was legal, effectively boosting the construction of new high schools [175].

students mainly came from wealthy families. Widdows was an exceptions, who extended the same principles previously introduced in elementary school to secondary schools. At the beginning of the 20^{th} Century, the first US rules on daylighting in classrooms were also published. They required that windows had to be set up on the long-side wall. The total window size had to be calculated in relation to both the floor area and the area of the of the wall, avoiding dark spots, resulting in large rows close-up windows. Some standards on artificial lighting were also introduced, although the letter was barely used until 1930 when fluorescent lights began to be used. The letter replaced the more energy intensive and problematic incandescent lights introduced at the beginning of the Century. Artificial ventilation also took its first steps at this time, especially in schools located in large cities.

The inter-war period between 1914 and 1940 saw a new generation of school reformers emerge, led by scholars such as John Dewey in the United States and Maria Montessori in Italy. These pioneers of modern teaching proposed a child-centred view, fostering active teaching instead of the traditional 'sitting-at-a-desk' method. On the other hand, budget cuts for public buildings due to the American Depression and European downturn, along with studies on school lighting, pushed the use of steel and timber frames. This context become an opportunity for progressive architects, such as Alvar Aalto and Walter Gropius, who supported the new ideas on schools. The schools they designed gave rise to *open air school* movement, so called because of the emphasis on airy spaces, light, easy circulation through spaces and outdoor teaching. Among the solutions developed in the UK, which were sometimes inspired by Frank Lloyd Wright's projects, the "*finger plan*" is an example of a school layout commonly used, which was also taken up in the US in 1940. It consisted of long rows of single-story classrooms, connected by lower corridors, which formed fingers spread out across the plan of the school. It is also worth mentioning the large use of prefabrication in school buildings by C. G. Stillman in 1936, who introduced a steel module system derived from the structure of caravans, which could be assembled together. In this period, two tiers of education were gradually defined: elementary schools, up to 11 years old, and high schools, with *grammar, technical* or *scientific* courses. In science laboratories modern architecture was largely applied, by using light, highly glazed buildings with a reinforced concrete structure and white painted bricks.

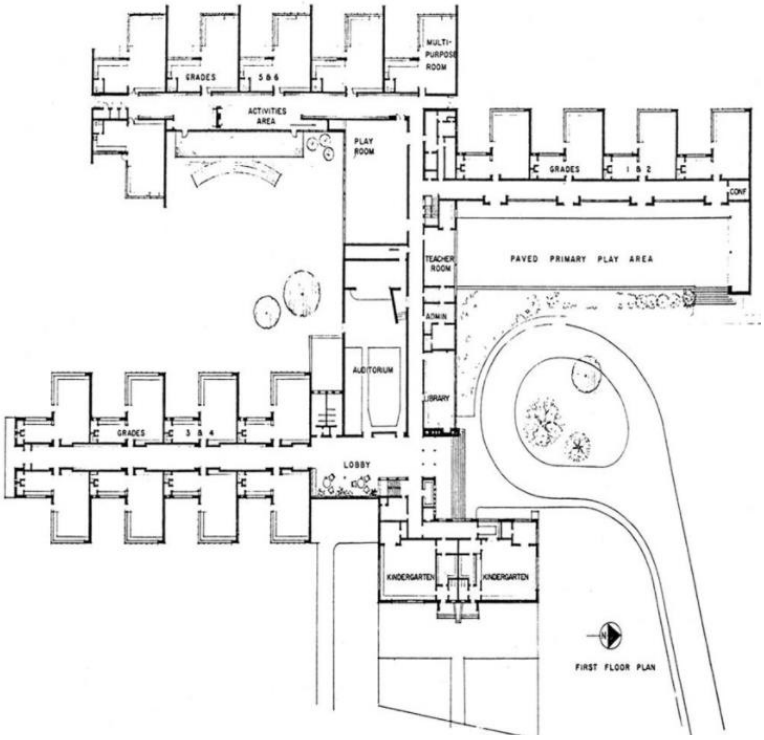

Fig. 4: the Crow Island School of Perkins & Will Architects: an early example of a *Finger plan* School [8]

In the post-war period a steep rise in school enrolment and, consequently, a boom in school construction occurred in the United States, driven by massive public funding. As a result, a new era of innovation in school architecture, with a mushrooming of standardised schools both in plan and architectural style, began. Classical masonry schools were finally replaced by modern schools, with steel or reinforced concrete structural system approach, full-height ribbon windows and a flat roof. In particular, the finger plan, which spread all over the world, was capable of providing great floor plan flexibility and excellent classroom lighting [9]. In addition, while modern schools offered a number of advantages, such as fulfilling the new rigorous construction standards, as well as being more functional, cheaper and easier to build, they had a shorter life expectancy. This lifespan reduction was justified by the fact that schools had to be rebuilt periodically [10]. In Europe, the large number of destroyed or damaged schools, and the significant rise in birth rates, dramatically increased the demand for schools in the post-war period, which had to cope with the war-damaged economy. To face this challenge, the *British Ministry of Education* recommended in 1943 the use of standardized prefabrication, usually made with lightweight steel frame systems. The first systems used for small one-story buildings; however, the system was later upgraded using aluminium and concrete, extending its use to multi-storey buildings. The implementation of prefabrication in the US schools was promoted by the *School Construction Systems Development Program* (SCDS), a joint effort by university researchers. This programme, which aimed at developing more economical building technologies by importing industrial standardization and techniques, played a key role in building experimentation in the 60s [11, 12].

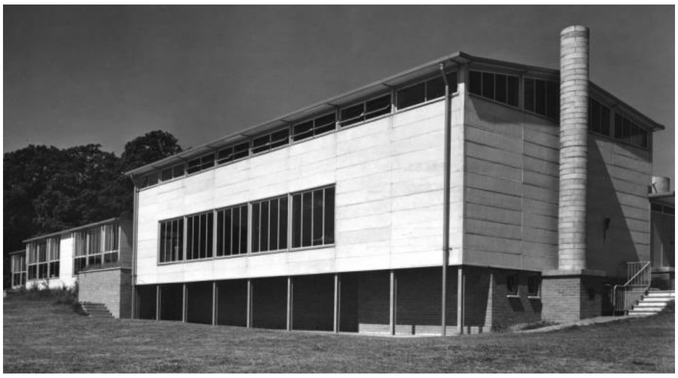

Fig. 5: Essendon Primary School, Hertfordshire: an early example of prefabrication in school buildings, made of light steel-framed system [13]

The *RA Bulter's Act* of 1944 saw the adoption in UK of the three-tier system of schools *grammar*, *technical grammar*, and *secondary modern*[2]. The three school levels were often grouped into a single site (to save money and space), paving the way for the so-called *Comprehensive Schools*. However, a problem arose concerning the size of these school facilities, which had to accommodate up to 2,000 students. Where lands were limited, school blocks often had only workshops areas and halls set out separately. As a result, the need for more flexible and centralized plans became increasingly clear. Instead of traditional classrooms, wide open teaching areas around libraries or service centres were preferred, while crafts and science labs were held in proper open plan units, marking the beginning of *open planning* schools. Open planning were particularly successful in primary schools located in rural areas where classrooms usually accommodated few students of different ages. In 1967 the *Department of Education and Science* (DES) developed two model plans, either circular or pavilion in shape, which were shared with the other authorities [14]. The presence of open schools were widely debated in the United States from the 60s, when the reduction of school populations, the problem of desegregation and new studies on the influence of the size and design of the physical environment on student learning, forced to rethink about existing school spaces. The *Educational*

[2] The three tier system was widely adopted by British educational authorities, albeit not everywhere, from 1945. Children were allocated according to their intelligence test scores at the age of 11. The best students were admitted to grammar schools, whereas secondary modern schools provided training for trades. The system was criticised in the 60s because considered iniquitous by progressive authorities, then abandoned in the 70s when it gave way to comprehensive schools.

Facilities Laboratory (EFL), namely the main research organization in the field of theory and practice of school design, argued that large classrooms with a little definition of space inside stimulated creativity, interaction and interest among pupils (Fig. 6). The work of the EFL influenced open plan schools in other countries, such as Australia [15, 16, 17], Canada [18] and Israel [19]. Nevertheless, this system was short-lived. In fact, in the 70s the work of the EFL was considered as uneconomical and restrictive, as well as receiving very conflicting feedback from researchers regarding its impact on teaching. Besides, this innovative teaching method was not supported by an appropriate training of the teaching staff, who tended to retain traditional educational methods and to advocate a physical and clear division of space [20]. Consequently, from the 80s onwards there was a step backwards in the design of school spaces, thus returning to the cheaper traditional "*cells and bells*" factory model, with classrooms arranged around shared activity spaces and circulation areas. Secondary schools were particularly affected by the lack of care taken in the design of learning spaces, being conceived as a set of spaces resulting in lower and longer buildings, gathered around a central atrium or courtyard. In the United States, the revival of the conservative movement, and the downward trend in confidence in the experimental models developed during the 60s and 70s, were fostered by the drastic drop in enrolment and by the decrement of public funds for building new school facilities and renovating existing ones. As a result, some reports depicted a school building stock in the 90s consisting of energy-intensive buildings, often dilapidated or failing to provide adequate indoor environmental quality [21, 22]. Since the end of the last Century, the design of schools has therefore been strongly influenced by the need for renovation of school buildings and the growing awareness of the climate change issue. The worldwide efforts of scholars and designers have been mainly focused on the design of new energy-efficient and low-impact schools and on the energy upgrading of existing facilities [23, 24].

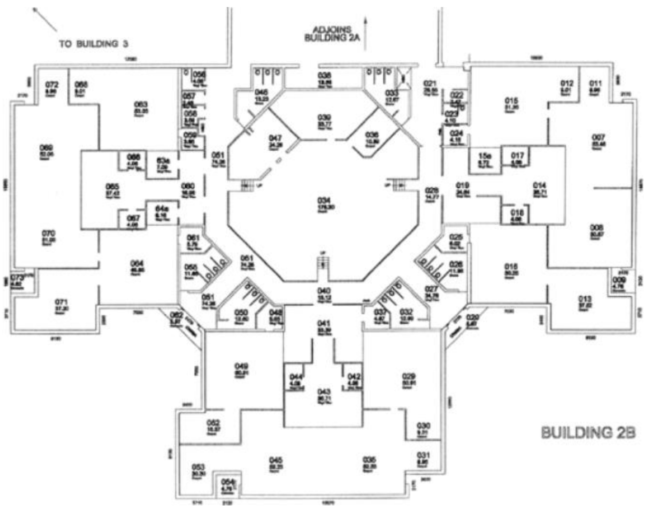

Fig. 6 Rokeby Primary School, Tasmania: an example of open planning school [17]

2.2 The Italian situation

The school building conceived as a facility and intended exclusively for the education of students is a relatively recent concept in Italy. In fact, for a long time education was reserved for the clergy and the nobility, hence teaching was carried out in convents and private residences. As already illustrated for UK and the US, the industrial revolution imposed the teaching of a trade on a large number of children, thereby requiring the design and construction of buildings exclusively for schools. Early examples of schools often consisted of single, overcrowded classrooms, designed with the purpose of educating large masses of students in a very a short time and with little regard for hygiene and functional aspects.

The first Italian law on schools were issued when the country was unified. The *Casati Law* (1859) entrusted municipalities with the construction of schools, whereas the first regulations was devoted to solving hygiene issues and ensuring the healthiness of the environment, rather than applying of pedagogical activities [25]. Nevertheless, the small economic resources of municipalities led to an extremely slow development of school buildings, especially in southern Italy. As a result, education was often conducted in convents and private schools, where functional and hygienic requirements were usually not ensured. The first guidelines on the urban, typological and constructive characteristics of school buildings were issued in 1888. In particular, it established that buildings had to be arranged in classrooms containing at most 50 students, whose ceilings had to be at least 4.5 m high. Hallways were preferably to face north and acted as a connective space between classrooms, running along the wings of the building. Therefore, the most common building type was the so called "*German-matrix barrack scheme*", which was either in a line or L-shaped or C-shaped according to the shape of the constricting site (Fig. 7). To avoid rising moisture from the ground, mezzanines floors were built, sometimes above a basement floor. In urban areas, schools were multi-story buildings with no more than three stories and a distinctive architectural typology. Whereas in rural areas, schools reflected local building typologies. Such a building layout, called "*block layout*" and derived from a frontal instruction concept, identified the classroom as a functional unit, according to which spaces and structure were arranged. Despite many pedagogists at the beginning of the twentieth Century wished for an *active teaching*, assuming a different distribution of space and shape of classrooms, the school layout was almost unchanged throughout the following decades.

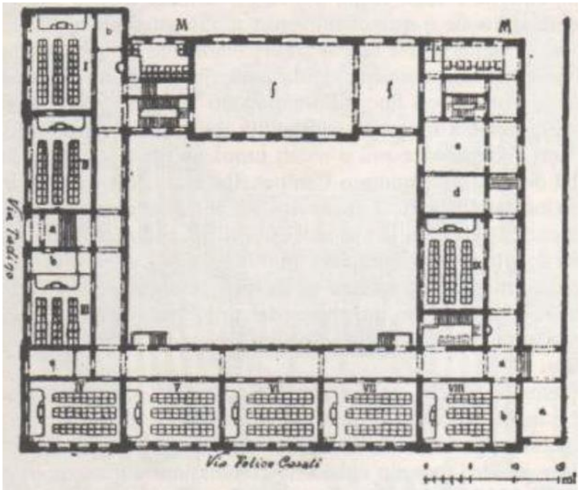

Fig. 7: "Scuola del Lazzaretto" built in 1890 in Milan (Italy) [26]

Upon the establishment of Fascism, school construction was being facing a shortage of classrooms and inadequate teaching environments, especially in southern Italy [27]. To address these deficiencies, the newly formed fascist regime enacted the *Standards for the design and construction of school buildings* (of May 4) and the *Royal Decree* of July 7, 1925, which provided less strict requirements for both new school buildings and for the adaptation of old schools [28]. However, over the years, the school was identified by the regime as a powerful tool for propaganda and training of the new fascist generation, thereby assuming a increasingly key role [29]. In this regard, the Royal Decree No. 875 of May 27, 1939, entitled "*Standards for the design of kindergartens and elementary schools*" was issued. It introduced a multi-purpose building, which included, in addition to the classrooms, the library, the teachers' room and, in particular, large spaces (such as sport hall and screening rooms as shown in Fig. 8). However, if on the one hand innovative construction techniques were experimented to cover the long spans that characterized these large spaces, on the other hand the barracks scheme was maintained in all schools of that period. Indeed, frontal instruction was perfectly consistent with the fascist educational model.

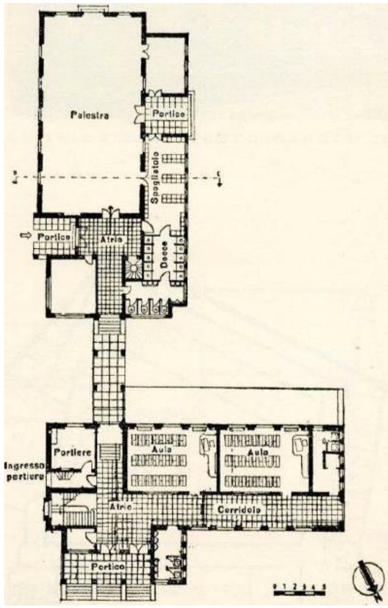

Fig. 8: Primary school of Alseno Piacenza, (Italy), as an example of fascist architecture in schools [30]. The key role of physical activity can was gathered from the size of the sport hall (compared to the whole school).

After the war, the economic recovery and the increasing demands for schools, especially in Northern Italy, led to an intensive school construction. Traditional school building was again suitable for such a context, because it was cost-effective and easily applicable on a large scale. As a matter of fact, the architectural design textbooks still conceived the school as a series of rectangular classrooms [31]. A first attempt to break with traditional models occurred when the *Ministry of Education* announced a public call for the design of a school building in 1949, which encouraged the disregard of current rules. Ciro Cicconcelli won the competition by submitting a project that conveyed a new idea of school, no longer based on the classroom as a functional unit, but rather as a set of equally valuable spaces, including classrooms [32]. The heated debate that ensued, although it led to a conspicuous collaboration between architects and educators, did not lead to a significant renewal of the regulatory framework for school construction, which continued to aim at satisfying more quantitative than qualitative criteria. The new standard issued in 1975 finally introduced the concept of flexibility [33]. The functional unit of the classroom was definitively abandoned and replaced by the parameter of *student space*. In other words, the classroom could no longer be a basic element to be repeated along a corridor, but had to be tailored to the students' needs and activities, and had to be effectively integrated and connected with other spaces. The use of long hallways was also reduced, and replaced with spaces that were not only for distribution, but also for meetings and other activities. As a result, the parameters that defined the shape and layout of the interior spaces of schools depended on the education stage of the students, and therefore varied as the study cycle changes [34]. The standards issued between the 60s and the 1975s provided the basic principles still used today for the design of school buildings. The need for flexible areas, as required by these standards, along with the increasing demand for school buildings due to the strong demographic growth, led to the massive use of precast structures between the 60s and 90s (Fig. 9). In fact, precast school buildings were affordable, easy to build and enabled spaces to be adapted to the current requirements, as well as to those imposed by the ongoing updates in standards and new pedagogical theories, such as the Montessori method [35, 36, 37].

Fig. 9: Precast school in Piazza Mancini, Rome (1975). The interior consists of different spaces to ensure maximum flexibility, according to the pedagogical models of the time [38].

However, the precast technique was not pursued in the following decades, because of the unsolved debate concerning the relationship between industrialization of building and architectural design [39]. Therefore, reinforced concrete frame systems were used once again, and today they are the most common buildings in Italy. Besides, in the beginning the fervour and confidence in new materials, construction techniques and pedagogical models of 80s, in addition to the economy slowed and birth rates reduction, making the need for new schools less urgent. Similarly to other countries, in Italy innovation in pedagogical sciences was not supported by adequate training of designers and teaching staff. For these reasons, the design of large spaces for group teaching and creative activities is still limited to preschools, and partly in the primary schools, whereas teaching in secondary schools is almost exclusively face-to-face even today. Thus, the design of spaces is currently traditional. Nowadays, the Italian school building heritage ranges from historical masonry buildings (dating back to before the 20th Century) to experimental buildings (both in terms of space design and construction techniques) of the 60s, 70s and 80s, up to the most recent buildings conceived to meet safety and energy saving criteria.

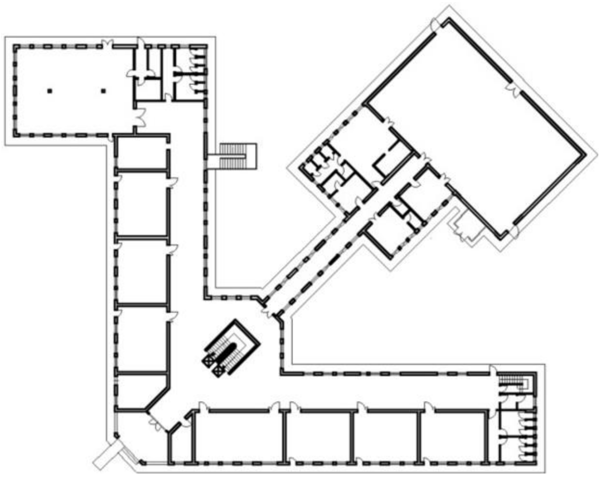

Fig. 10: “Alberto Manzi” Comprehensive school built in Grosseto in 1981 (by kind permission of Comune di Grosseto). A return to the "finger plan" model can be observed, with the classroom as the basic space unit.

2.3 Comparing the Italian school buildings with those of Europe and Japan

Currently in Europe educational facilities represent 17% of non-residential buildings, more than 30% of which were built before 1960 [40]. Although the evolution of school buildings has moved through some common stages in many western countries, the different historical vicissitudes, the uneven economic and social conditions, the very varied climate and the different types and degrees of natural risks throughout the wide territory of the old continent, make the school building landscape very diversified. For instance, some northern European countries have been focusing for several decades on energy saving, and on the quality of the indoor environment, in terms of thermo-hygrometric comfort, materials and finishes [41]. On the other hand, countries like Italy and Greece are lagging behind on these issues. The seismic-prone territories have led them to focus the efforts mainly on earthquake-resistant design, starting from the most seismic areas and then throughout the country [42]. Furthermore, Eastern European countries have only been introduced to Western educational models in the last 30 years, thus the legacy of the Soviet educational model is still present in their school structures [43]. Nevertheless, the lack of unified policies concerning school curricula, interventions, funding and legislation on schools suggests that a settlement to these differences is still a long way off [44].

Outside Europe, the case of Japan is very significant, compared to Italy. In fact, Japan has faced similar challenges since the post-war period, albeit often with a different approach. Based on the work of Kawano et al. [45], the main historical stages of Japanese school constructions can be described and compared to the Italian case. As known, the country was heavily damaged during the World War II, hence it had to cope with rebuilding issue similar to Italy. Furthermore, economy and population rapidly growth from the 50s onwards, imposing the massive construction of public buildings, including schools. To provide rules for new buildings, including schools, the *Fundamental Law of Education* (1947), the *Building Standard Act* (1950) and the *Standard design of reinforced concrete school buildings* (1950) were issued. The latter, increased fire resistance requirements for schools and led to the switch from timber to reinforced concrete as construction material for new schools. This is similar to the Italian case, where the transition from masonry to reinforced concrete occurred in the same period, although Japanese timber schools were demolished or abandoned, whereas Italian masonry schools were kept on and widely used. The second baby boom at the end of the 70s and the sharp expansion of large towns, due to the concentration of the population in urban areas, required the construction of large school facilities designed to enable future building additions. As well as in Italy, a large number of schools currently in use were built between the 70s and 80s. In 1981, the revision of the *Building Standard Act* led to the adoption of seismic-resistant design standards, followed (in 1995) by the *Act for Promoting of Renovation for Earthquake Resistant Structure*, which made the adoption of earthquake-resistant measures mandatory for schools. To support the implementation of these measures, the Japanese government began two subsidy plans: in 1994 for the seismic reinforcement of schools, in 2002 for safety control facility development works, which included school building structures. This major and audacious economic and regulatory effort enabled Japan to achieve the 99.2% of public elementary and lower secondary school earthquake-proofed in 2018. This valuable result has not been achieved yet in Italy, and in the rest of Europe as well, because the Italian standard does not require a mandatory seismic retrofitting of schools, but merely recommends to local public authorities to provide seismic strengthening of schools, especially in high seismic areas.

However, the *Great East Japan Earthquake* of 2011 highlighted two fundamental points. On the one hand, it showed the key role played by schools as shelters for the population, further emphasising the need to make them safe. On the other hand, it revealed a vulnerability towards the tsunami according to their structural materials, number of floors, distance from the coast and local topographical characteristics [46]. For instance, the two primary schools of Okawa and Kodowaki in the Miyagi Prefecture, were barely damaged by 2011 tsunami disaster. At the Okawa Elementary School, there were 84 causalities [47]. The low-rise two-storey school building, while suffering limited direct damage due to the earthquake, turned out to be very vulnerable to the tsunami. The level of water reached 8.6 m due to the school's unfavourable location in relation to the orography of the surrounding area. Conversely, in the Kodowaki elementary school students and staff escaped in time and survived. However, the school, which did not suffer any major direct damage from the earthquake, was hit hard by the tsunami and a fire that broke out from debris dragged into the building by the ocean. The heavy legacy of the disastrous events of 2011 is a key lesson that shows how not all the risks can be tackled by acting on a simple building level. Indeed, the fundamental safety measures (e.g. against earthquakes or fire) must be combined with effective Disaster Risk Reduction (DDR) strategies, which include prevention and information for pupils and teachers [48]. The two school buildings have been decommissioned, but the Japanese authorities want to preserve them as a reminder of what happened for future generations. (Fig. 11 and Fig. 12).

Fig. 11: Okawa Elementary school after being damaged by 2011 tsunami**.**

Fig. 12: Kodowaki Elementary school after being damaged by tsunami (November 2019)

Japan and Italy also share a significant ageing population that has been going on for more than 30 years. Japan has been experiencing declining birth rates since the 80s, leading to a progressive surplus of classrooms. As a result, the theory of school design followed by the authorities, which before 2000 advocated the construction of new school facilities, has changed in recent years towards the refurbishment and intended use change of existing school buildings. Even in other countries with declining populations such as South Korea, Greece and Spain, many schools are facing under-use or even disuse. Therefore, in the last few years the problems are those of renovating existing schools and designing the school spaces to be easily tailored to users [49].

Chapter 3

State of art

3.1 Design approaches and structural system

According to Chapter 2, advances in building materials and technologies, as well as the new pedagogical models and functions within schools, significantly changed the way spaces were conceived and organised. Materials, arrangement and type of structural system mainly depend on the following factors:

- period of construction;
- design approaches, (Fig. 13);
- size and layout of the school plan;
- Intended use of spaces (e.g., classroom, laboratory, circulation and socialising space, sport and conference hall).

The following paragraphs list the most common structural types of the Italian schools, with comments on their main characteristics, strengths and weaknesses.

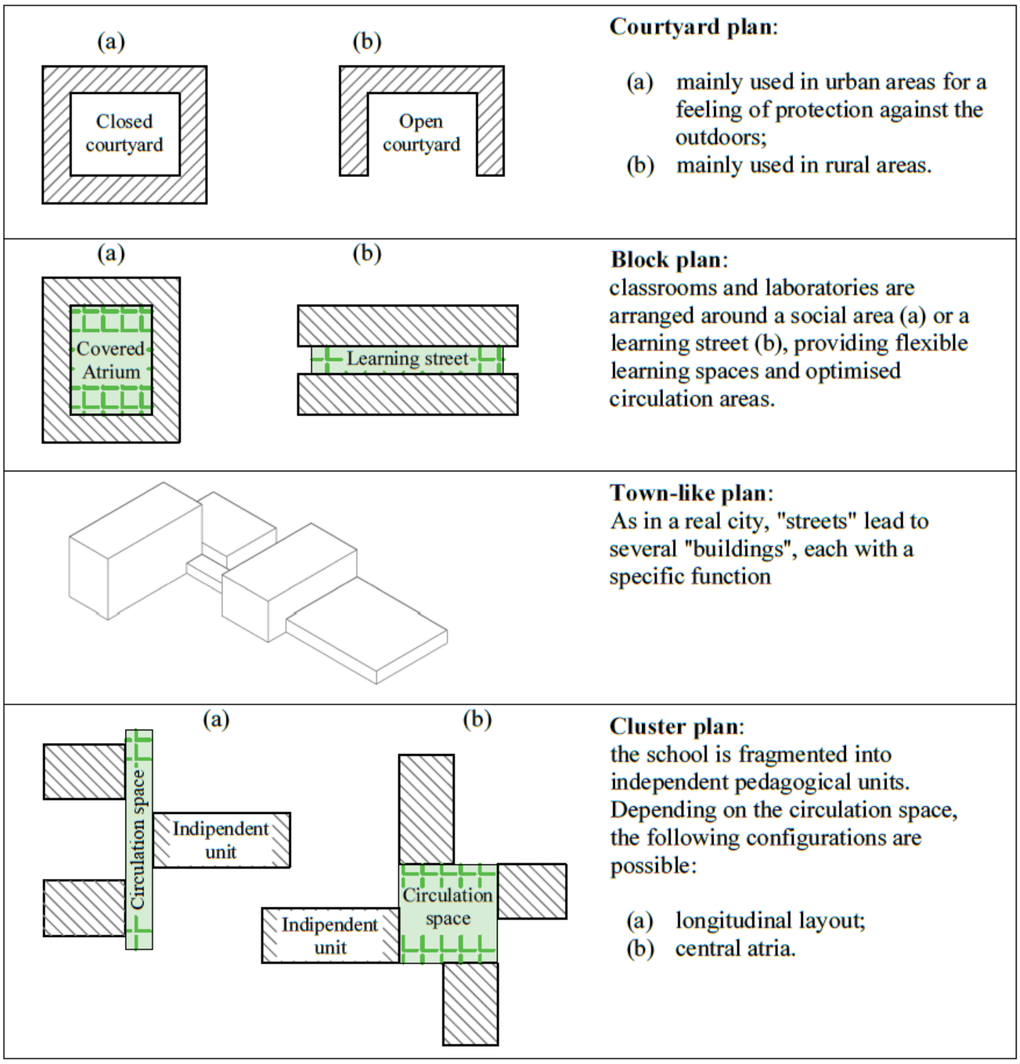

Fig. 13: School building design types and spaces arrangement [50]

3.1.1 Masonry structures

Masonry structures were the most commonly used for schools the end of World War II (and until the 60s in rural areas), when they were superseded by reinforced concrete schools. Masonry structure could be made of brick or stone, thus the structural strength was strongly affected by the material (e.g. type and shape of the stones and mortar) and the masonry texture, which in turn depended mainly on local building customs [51]. Slabs are usually in masonry vaults or in brick with steel joists, therefore limited spans was covered. Partitions separating main spaces, such as classrooms and corridors, often also correspond to structural elements (Fig. 14a).

Strengths:

- in the case of good quality masonry, bi-directional load-bearing walls provide box behaviour of the building, with good performance against horizontal actions;
- fairly high durability;
- buildings are usually of good aesthetic quality;

Drawbacks:

- Low plan flexibility, which makes impossible modifying the plan layout;
- space lighting is poor because window sizes are generally small;
- seismic retrofitting is invasive and expensive, especially in presence of low masonry performances.

3.1.2 Reinforced Concrete (RC) structures

Schools with reinforced concrete frames are the most common in Italy. They first appeared in the industrial cities in the early of 20th Century, sometimes combined with masonry. Nevertheless, the massive implementation of this structural system began after the Second World War alongside the Italian economic boom. The structural design quality and performances of RC school buildings depend on the standards at the time of construction, as well as on the quality and type of structural materials, e.g. the mix design of the concrete and the type of reinforcing (smooth or deformed). Furthermore, aggressive environments and lack of maintenance can reduce the structural performances, as RC is vulnerable to degradation.

Strengths:

- Columns provided moderate plan flexibility, along with the capability of being modified over time, as the brick or plasterboard internal partitions can be moved or demolished (Fig. 14b);
- the frame structure allowed for larger openings and windows than masonry, thus ensuring good space lighting;
- complex building shapes were designed.

Drawbacks:

- a weak capacity against seismic load because, until the early of 21th Century, schools were designed without considering any horizontal actions. Indeed, bracing elements are often missing,

frames have only one direction and frequently weak column – strong beam are present;
- In contrast to masonry, whose mechanical performance can be reliably predicted, in RC buildings expensive destructive tests is often necessary as the type and quality of the structural material depends on many parameters, as described in Section 3.4.

3.1.3 Precast structures

Schools with precast concrete panels were widely used for a relatively short period, roughly between the 60s and the 80s, when a great confidence was placed in the industrialisation of the building sector. They usually consist of one or two-storey buildings, with fiber-cement panels, embedding thermal insulation in case of external walls (sandwich panels), sometimes combined with columns and beams made of steel profiles or precast reinforced concrete elements (Fig. 14c).

Strengths:

- faster to build and less expensive than cast-in-situ structures;
- certified initial mechanical performance due to industrial-type prefabrication system;
- flexibility in the design of the rooms, whose sizes are usually multiples of the precast panels' module size;
- small footprint of structures and walls;
- Thermal insulation provided by the embedded insulation inside the panels (excluding thermal bridges at the joints);

Drawbacks:

- often were made of asbestos cement;
- panel-to-panel and panel-to-frame joints were usually designed to support vertical loads only, thus structures have low strength against horizontal actions [52].
- a low durability of the structures was often reported, being prone to degradation mainly due to water leakage between the joints [39];
- the aesthetics of this type of building have always been debated, as they are often considered unattractive.

3.1.4 Structures made of precast RC columns and beams

These structures are still used today, because they are suitable for large spans such as sport and conference halls and auditoriums. They consist of columns up to ten metres high supporting “T” or rectangular beams, covering spans of up to 30 m or more, when accommodating large conference halls or sports grounds with spectator stands.

Strengths:

- capable of covering large spans with a high degree of plan flexibility;
- fast to build;
- certified initial mechanical performance due to industrial-type prefabrication system;
- fairly constant retaining over time of mechanical performances.

Drawbacks:

- until Italy was declared entirely seismic, structures were designed to cope with dead loads only, with friction-based connections among structural elements (e.g. simply supported beam-to-joist and beam-to-column connections). Therefore, failures have been observed after several seismic events, mainly due to the loss of support of beam elements and relative movements of elements [53, 54, 55];

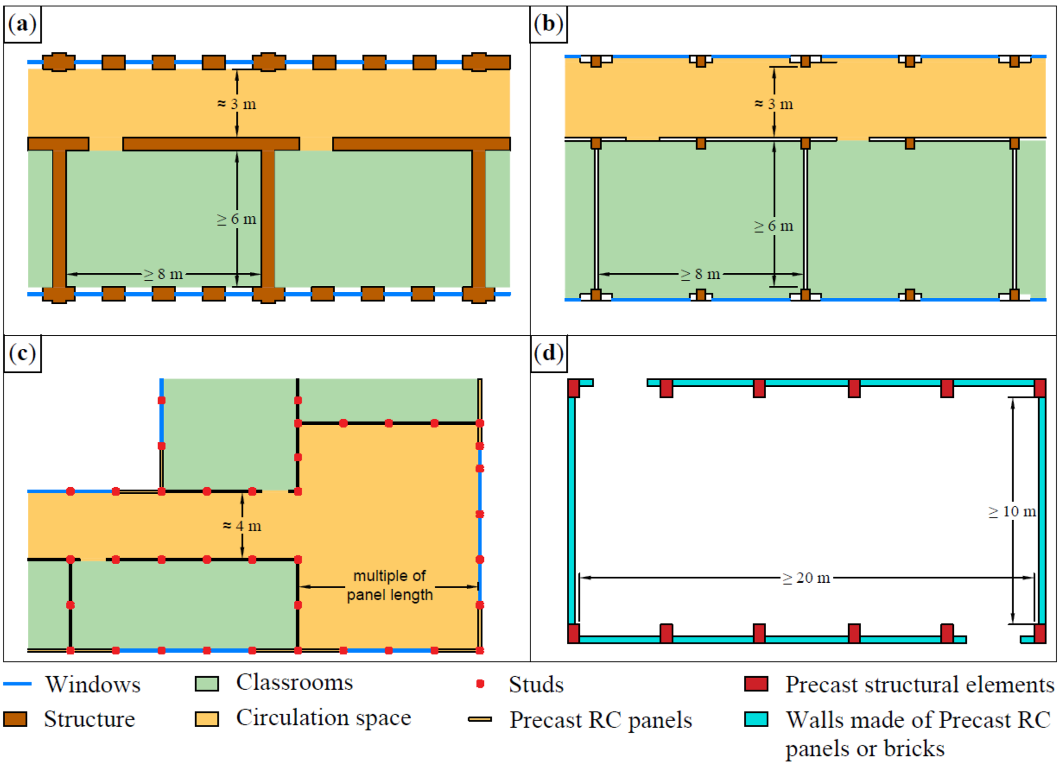

Fig. 14: **Typical structural layouts** of: (**a**) masonry school; (**b**) RC frame school; (**c**) school made of precast concrete panels; (**d**) precast RC columns and beams**.**

3.2 Regulatory framework

The regulatory framework includes both general standards for all constructions and public facilities, and specific laws for school buildings. The main laws that have been introduced in the last 50 years in Italy, regarding both school building and structural design, are listed and briefly described on below.

3.2.1 Specific standards for school buildings

- Circular no. 425/1967 "*Residential Standards*" of the Ministry of Public Works, which prescribes location, size, characteristics and equipment that schools must have according to the type of school and the number of inhabitants in the surrounding urban context [56];
- Ministerial Decree of December 18, 1975 "*Updated technical standards relating to school buildings*" which updates the general requirements relating to the location and size of schools, and the school site as well. Furthermore, it contains specific instructions on the technical requirements for acoustics, lighting, air quality and thermo-hygrometric comfort of individual pedagogical units, as well as laboratories, physical education spaces, classrooms and services, and the school as a whole. The constant advancement in pedagogical methods was strongly taken into account, as reflected in the flexibility requirements with which spaces must be designed [57];
- Ministerial Decree of August 26, 1992 "*Fire prevention standards for school buildings*" provides specific instructions for school facilities, such as prescriptions on the accessibility of sites to emergency vehicles, the compartmentalisation of buildings, the fire resistance of materials, systems and fire safety signs [58];
- Law no. 23/1996 "*Standards for school building*", mainly aimed at satisfying the need for classrooms, upgrading and adapting the existing heritage to the standards of use, safety and hygiene. This is a key law because: i) it assigns competence for school building to the territorial authorities, defining their tasks (i.e. assigning the management of nursery, primary and secondary schools to the municipalities, whereas managing of high schools is attributed to the provinces); ii) it sets rules on planning, procedures and funding modalities of interventions; iii) it creates the *Observatory for school building*; iv) it creates the *school building registry*, i.e. an information system that gathers and keeps up-to-date data on the situation and consistency of the school building heritage [59]. The registry collected data, such as general information on the school, its spaces, building structures and components, safety, surroundings, transport and connection systems, through the *ARES forms*. The ARES forms are

filled by the technical staff that carries out surveys in the school facilities assisted by the head of the Prevention and Protection Service of the specific school administration. The form format was updated over the years by adding new sections;

- Inter-ministerial Decree of April 11, 2013 "*School building guidelines*". It gathers the most recent pedagogical models, redefining learning spaces, introducing new concepts such as "*group space*" and "*individual space*", and provides guidelines for the design of classrooms (which lose their centrality as they should no longer be conceived as the only space where teaching), laboratories, halls, central spaces (agora), sports facilities, services and administration offices [60];
- Law no. 107/2015 “*Good School Law*”. The decree, alongside significant changes to school curricula, provided for the allocation of 4 billion euros to improve the safety some of 36,000 Italian schools [61];
- Ministerial Decree of October 11, 2017 “*Minimum environmental criteria (CAM) for the assignment of design services and works for the new construction, renovation and maintenance of public buildings*”. It defines the typology, characteristics and minimum criteria from an environmental sustainability and circular economy standpoint that the materials, energy performance and energy supply sources of new public buildings must provide for [62].

3.2.2 Standards on structural design

- Law no. 1086/1971 and Ministerial Decree of May 30,1972, which provided the technical standards for normal and prestressed RC buildings and steel structures [63];
- Law no. 64/1974, which provided specific requirements for buildings within seismic zones [64];
- Ministerial Decrees of January 9 and 16, 1996, which provided technical standards for the assessment, construction and inspection of RC and steel structures, including specific standards for buildings in seismic areas, and general criteria for the safety assessment of constructions and loads and overloads [65, 66];
- OPCM of March, 2003 n. 3274 “*General criteria for the seismic classification of the national territory and technical standards for constructions within seismic zones.*”: the key point is the introduction for public authorities of compulsory structural checks on strategic buildings, thus including schools. Nevertheless, retrofitting the structure which do not fulfil the seismic standards is not mandatory: such evaluation is carried out by public authorities according to the

risk level. Moreover, it provides a new seismic hazard map of Italy, whereby the entire national territory is classified as seismic and divided into 4 hazard zones [67].

- Ministerial Decrees of January 14, 2008 "*Technical Standards for Construction (NTC 2008)*" and Circular no. 617/2009 was the first code rule which gathers all the standards (especially the Eurocodes) for all types of new and existing ordinary structures. In particular, it imposed structural strengthening when buildings did not comply with static analysis, whereas seismic retrofitting was not mandatory. However, in the seismic retrofitting of an existing building, the same safety level for a new building was required [68, 69].
- Ministerial Decrees of January 17, 2018 "*Technical Standards for Construction (NTC 2018)*" and Circular no. 7/2019 (namely, the standards currently used for structural design) updates NTC 2008 by introducing novelty and improvements aimed at solving the issues raised over the 10 years of NTC 2008 use. In particular, it defines the ζ_E coefficient, i.e. the ratio between the maximum seismic action that an existing structure supports and the minimum seismic action that a new structure must withstand according to the seismic hazard level of a specific zone. In particular, in contrast to the previous NTC 2008, achieving the same level of safety as a new building is no longer required when renovating a school. It is sufficient to provide $\zeta_E = 0{,}6$. Indeed, a lower but still fairly high safety level against seismic action means enabling public authorities to deal with a higher number of schools strengthening because the cost of each intervention is lower [70, 71].
- Eurocodes series: namely, a set of 10 standard volumes issued by the European Commission that provide instructions for the safety of several construction types, which largely provide the framework of Italian standards [72].

3.3 An overview on the Italian school building stock

As stated in section 3.2.1, Law no. 23/1996 established the *Observatory for school building* and the *school building registry* which collect records for the Ministry of Education (MIUR). The aim is to monitor the school building stock. The dataset is published on a dedicated institutional web page and can be downloaded as raw data [73]. These data provide an overview of the current situation of Italian school buildings, without focusing in detailed technical information. In section 3.3.1 the most relevant data provided by MIUR, alongside the results of a survey carried out by *Fondazione Angelli* about age and structural safety of school building, are briefly commented on, also taking into account some tragic events that have occurred in recent decades. In section 3.3.2 data about energy performances aspects provided by both MIUR and ENEA (National Agency for New Technologies, Energy and Sustainable Economic Development) are evaluated.

3.3.1 Age and safety of the Italian school building stock

According to data provided by MIUR, there are more than 50000 school buildings in Italy, mainly located in the most populous regions of northern Italy (Fig. 15).

Fig. 15: Geographic distribution of schools throughout Italy [74]

As mentioned before, the period of construction of schools covers more than two centuries, but the construction trend is not homogeneous over the time.

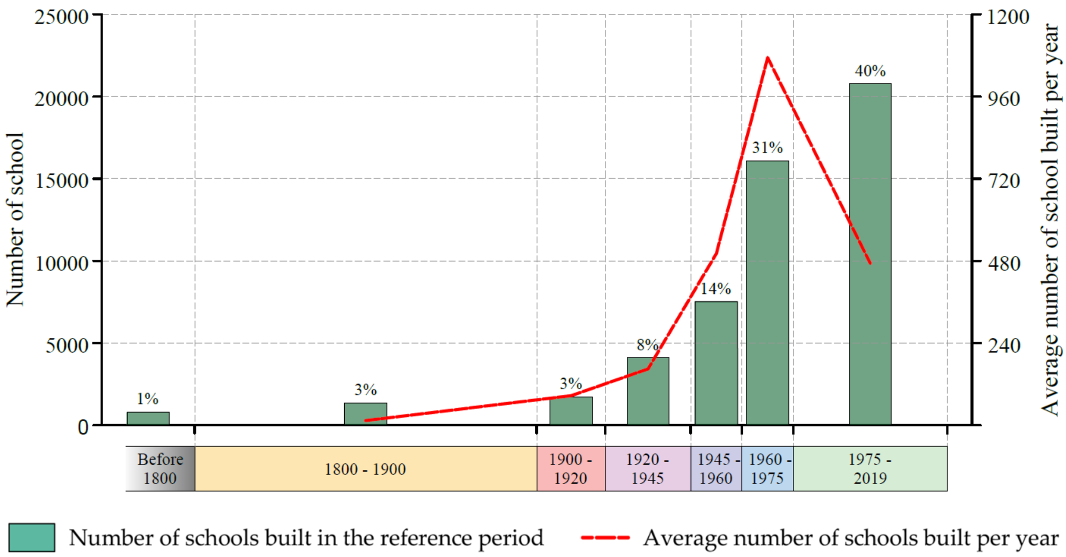

Fig. 16: Construction trend of new school buildings over the time [74].

Approximately 60% of the schools are more than 45 years old. In other words they were built in periods when no seismic standards existed, except in very restricted areas recognised as highly seismic. It is also worth noting that although the last period 1975 - 2019 has the highest percentage of schools built (40%), the most prolific period for school construction was 1960-1975 with more than a thousand schools built every year. As a result, even though 46% of schools are located in medium or high seismic zones (Fig. 17a), only 25% and 20% of schools were seismic-resistant designed in high and medium seismic zones, respectively (Fig. 17b). Beyond seismic risk, more than half of schools have a certificate of static sustainability (Fig. 17c), whereas about 55% of schools do not have a certificate of use and occupancy (Fig. 17d). On the other hand, a survey carried out by the Agnelli Foundation in 2019 reported that almost 10% of schools revealed structural problems, with an estimated cost of around € 200 billion to renovate and reinforce the entire Italian school building stock. [75].

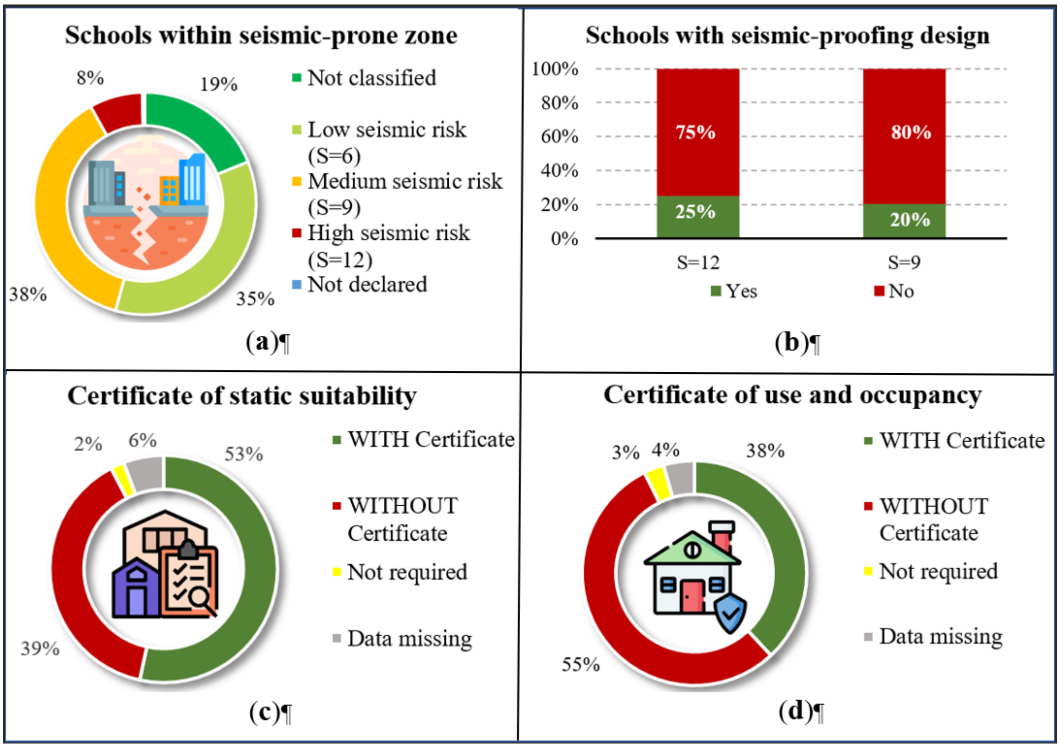

Fig. 17: Key indicators concerning the safety of school buildings based on data provided by MIUR [74]: (**a**) school distribution throughout the 4 seismic-prone zones; (**b**) share of schools with seismic-resistant structural systems in high and medium seismic areas; (**c**) share of schools with a certificate of static sustainability; (**d**) share of schools with a certificate of use and occupancy.

Regrettably, the fragility of school buildings leads to tragic events, such as the collapse of a entire Primary School in San Giuliano di Puglia on October 31, 2002 (Fig. 18a), where 27 children and one teacher died, and the collapse of the ceiling at the 'Darwin' High School in Rivoli on November 22, 2008, which caused Vito Scafidi's death and one student was seriously injured (Fig. 18b). In particular, these two cases show the tragic effects of two different aspects of the safety deficiency. On the one hand, the collapse of the school in Molise, after an earthquake of magnitude 6 Richter, was caused by the lack of anti-seismic measures in the design of the school, which was not addressed despite being aware that the school was in an seismic-prone area. On the other hand, the collapse of the brick ceiling in the Darwin High School is not caused by a triggering factor, but rather by both a lack of knowledge about the type and mechanical performance of the ceiling and by the heavy debris rested upon it. These tragic events show that the safety of schools depends not only on design and safety aspects, but also on a thorough knowledge of school buildings, which includes construction techniques, as well as all the events and interventions that occurred during their lifetime, along with their constant monitoring. These disastrous events have prompted the authorities to increase their efforts on the school safety issue in recent years, with funding lines being created for seismic retrofitting and, in general, for increasing the safety of schools. However, according to the report of "*Cittadinanza Attiva*", during 2020/2021 academic year there were 35 collapses in schools, showing that the road ahead is still long [76].

(a)¶ **(b)**¶

Fig. 18: Some school collapses in Italy: (**a**) "*Francesco Iovine*" Primary School in San Giuliano di Puglia [77]; (**b**) "Darwin" High School in Rivoli [78].

3.3.2 Sustainability and environmental costs of the Italian school building stock

To assess the school building energy demand, MIUR evaluated the adoption of a list of energy saving measures, including double glazed windows, insulation of roofs and walls and the implementation of solar and photovoltaic systems. The situation revealed by this survey is rather negative. Indeed, less than 60% of the schools have at least one of these measures (Fig. 19a). Around 70% of schools have double-glazed windows and only slightly more than 40% have an insulated roof. The percentage drops to 15% when insulation of the walls is considered. Finally, both solar panel systems for Domestic Hot Water (DHW) and photovoltaic panel systems are absent in almost all the schools (Fig. 19b). Only in around 300 schools, i.e. less than 1% of the total building stock, all the five measures considered are implemented. (Fig. 19a).

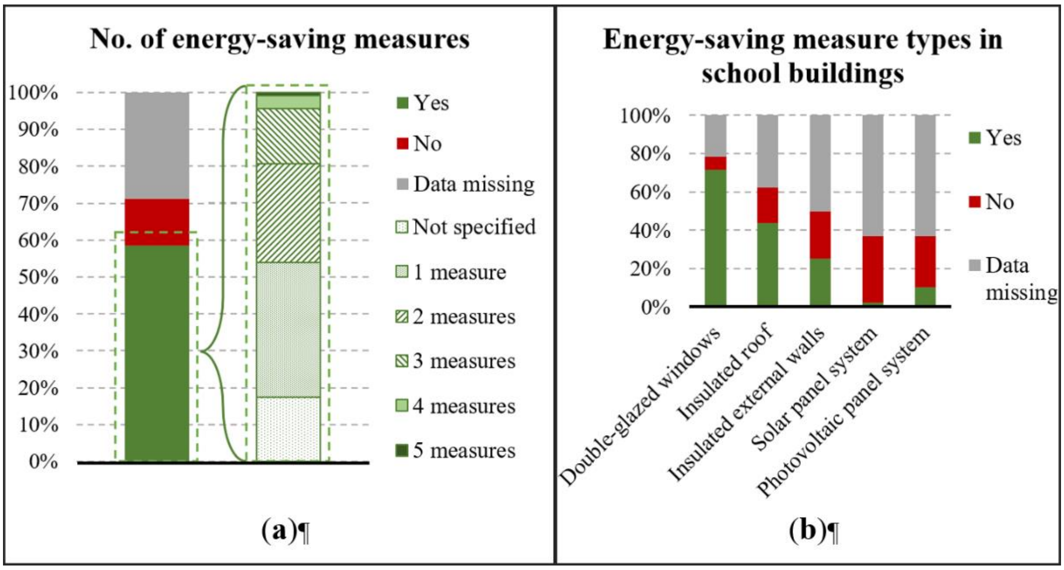

Fig. 19: Energy saving measures; (**a**) number of schools with one or more measures; (**b**) types of measures and their implementation in schools

As a result, the cost of energy consumption in the school buildings is very high. According to an ENEA report based on the last study carried out in 2007, 87% of the thermal energy consumption in public buildings is due to school buildings, with a total consumption of 12.5 million MWh per year. ENEA also estimates that by targeting 35% of the oldest buildings, almost 20% in heating energy can be saved easily [79]. The high energy waste, due to both the high heat loss of the building envelope and the energy-intensive aging heating systems which furthermore still largely use fossil fuels, obviously imply a high environmental impact. The environmental footprint of the old school building stock in Italy is further raised when considering building materials in a broader circular economy perspective. The use of recyclable, renewable or reusable materials is fairly recent, whereas concepts such as *design-for-disassembly* are still struggling to catch on. Even more concerning, is the fact that 4,3% schools built between the 50s and the 90s still have building components containing asbestos [80]. Therefore, demolishing an existing school in order to build a more eco-friendly one often implies a high environmental cost, as many of the materials cannot be recycled or reused, creating a waste stream to landfills.

3.4 A survey on the performance of RC schools in Turin

The data provided by MIUR, despite being useful for outlining a general overview, does not provide technical details to analyse the state of the art of structural material and school structures. To collect valuable data for large-scale structural evaluations, a survey based on the results of destructive tests on structural material drilled from Italian schools was carried out. However, performing a study covering the whole of Italy was not feasible due to the difficulties involved in collecting and managing a huge amount of data, as well as for privacy issues. Therefore- the analysis was focused on schools in the province of Turin. Furthermore, only reinforced concrete (RC) structures have been taken into account, as they represent the most common Italian schools.

3.4.1 Research significance

The case of schools in Turin is particularly significative. Similar to other cities in northern Italy, Turin experienced massive immigration from southern Italy during the economic boom years (from 50s to 70s). The resulting increment of population, combined with a fairly high birth rate, led to an upsurge in the construction of schools, which were built almost exclusively with cast-in-situ concrete or with precast RC panels and frame.

When dealing with existing building, the assessment is carried out by starting with mechanical characteristics of the materials stated in the original project. When they are not available, the requirements in force when the structure was built are used. However, in this way the values of mechanical performance could be overestimated [81]. Indeed, concrete carbonation and steel rebars corrosion are examples of degradation phenomena that structures built in the second half of the 20th Century are particularly prone to suffer throughout their lifespan [82, 83]. Also, wrong practices, frequently performed on the construction site, cause the poor quality of the structural material. This is the case of water addition to the concrete mixture to avoid reducing the workability when begins setting. This common practice led to a reduction in mechanical strength, and durability as well [84, 85]. As a result, a high share of buildings need structural checks to evaluate the material properties, and the capability of the structure to withstand the static and dynamic actions required by the current standards. Therefore, in severe cases, structural retrofitting interventions are needed [86]. This is particularly relevant in school buildings, where the utmost safety for students must be ensured. However, during the early stages of a project and in feasibility studies, the often low budget allows only cursory inspections to be carried out, which do not allow sufficiently reliable parameters to be gathered for an accurate evaluation of the intervention cost.

The survey reported in this thesis was carried out in cooperation with *Direzione Regionale Opere Pubbliche* of Turin, which provided structural reports of static and seismic strengthening interventions performed on 45 existing schools in the Province of Turin between 2014 and 2020. Destructive test results were taken into consideration from the structural reports. The aims was providing an estimation of the concrete compressive strength and the rebars tensile strength as a function of the time of construction of the building. Indeed, these parameters can be entered as the input data in the structural analysis, when detailed investigations on the mechanical characteristics of the reinforced concrete are missing. Although the preliminary results need to be integrated with further structural investigations, it should allow more reliable technical and economic evaluations even during the early stages of a retrofitting project. Moreover, the outcomes of these structural analyses can be useful when performing large-scale seismic vulnerability assessment, as structural survey campaigns covering many buildings are not feasible. Finally, the results of static analyses were also obtained from the structural reports to study a potential relationship with the average performance of the concrete and the behaviour of the structures against dead loads of a certain period.

3.4.2 Historical Reinforced Concrete

As is well known, many parameters affect the compressive strength of concrete and the tensile strength of steel rebars. With respect to concrete, its compressive strength can be tailored at the concrete plant by setting the mix design, the type and size of aggregates and the type of cement. As advances in production techniques and research have occurred since the introduction of concrete, it is possible to state that its physical and chemical properties also depend on the period of production. On the other hand, the control over the phases of casting and curing, as well as throughout the lifespan of the building (i.e., the conditions of the surrounding environment which trigger degradation phenomena) is important. These aspects imply that the results of compression tests carried out to control specimens before casting, made for assessing the strength of the concrete, often differ significantly from those obtained by cores extracted from the actual structure [87].

Actually, the relationship between the time of construction of the building and the strength of the concrete has already been observed in a previous study carried out on concrete cores collected from public buildings located in a moderate seismic zone in Tuscany [88]. Furthermore, these results were compared with the strength-for-age curves obtained using an huge internal database stored at the Politecnico di Torino, which concerns compression tests on cubic control specimens covering almost the entire 20th Century (Fig. 20). An increasing trend in the average concrete strength over the years are observed in both the curves obtained from both the database of the Politecnico and the concrete extracted from the buildings. However, a deviation, fairly constant over time, was found between the two sets of data, as the strength of the cores is between the 5th and 25th percentile curves of the database.

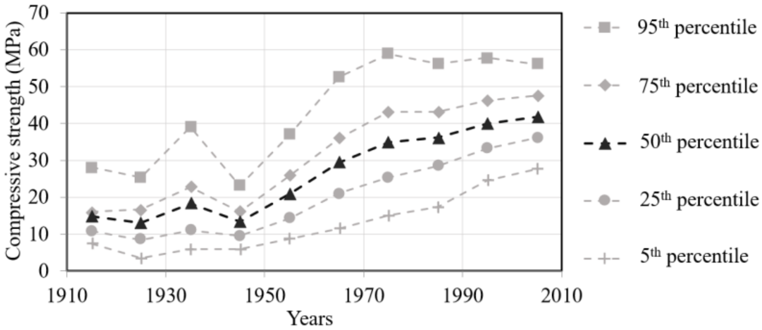

Fig. 20: Strength-for-age curves of concrete obtained from the database of the Politecnico di Torino.

The database stored in the Politecnico di Torino was also compared with the results of compression tests on concrete cores extracted from two bridges built in 1914 and 1975, respectively. The most relevant result was that the average compressive strength of the oldest bridge was close to 95th percentile curve plotted with the database, whereas the strength of the 70s bridge was slightly above the 25th percentile curve. This is due to the finer grinding of the cement and the reduction in the amount of C_2S to the detriment of C_3S which occurred as production techniques progressed, as a result of a lower increase in the long-term strength of concrete [89].

As well as for concrete, predictive strength-for-age curves were obtained both for tensile and yield strength of rebars by using the database of the Politecnico di Torino, also covering the whole last Century (Fig. 21). A comparison with the tensile strength of rebars extracted from two bridges of different ages (the first built in 1935, whereas the other is the same 1975 bridge mentioned above) was carried out to test the soundness of the predictive laws. The values relating to the rebars extracted from the actual structures were generally close to the average values calculated with the predictive curves. An exception is the ultimate strength of the rebars from the oldest bridge, whose values are affected by a greater dispersion, as well as resulting in a lower average strength than that calculated with the predictive curves. As a matter of fact, a larger number of specimens to correctly estimate the tensile strength is necessary for bridges built before World War II [90]. As steel is an industrial product, uncertainty about its mechanical properties is low. Indeed, they depend on the rebar type and the steel grade, usually stated in the structural report or inferable from the time of construction. The level of corrosion is usually the only unknown, because it depends on the age of structure, the conditions of the surrounding environment and the correct sizing of the concrete cover.

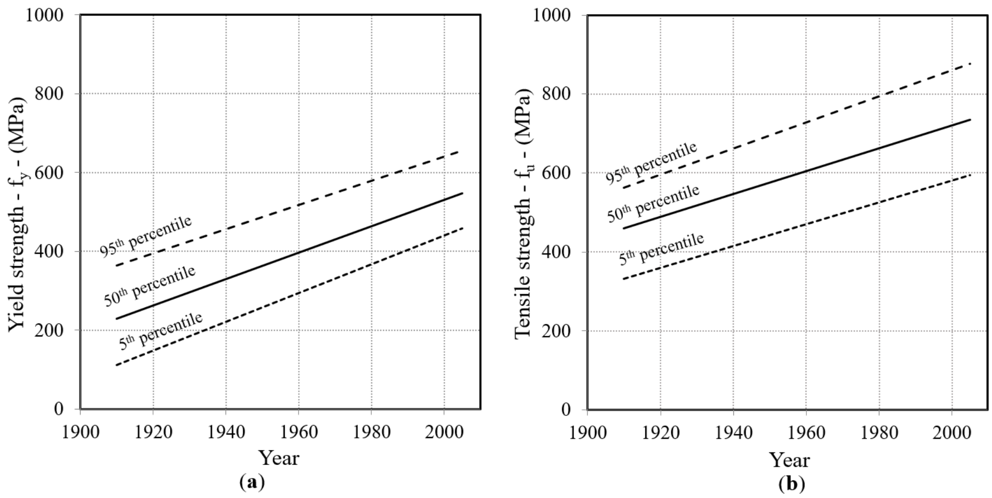

Fig. 21: Predictive strength-for-age curves of steel reinforcement obtained from the database of the Politecnico di Torino in the case of: (**a**) Yield strength; (**b**) Tensile strength.

3.4.3 Existing school buildings in the Province of Torino and data processing methods

When performing structural interventions, code building requires experimental tests on the existing structure according to the level of knowledge to be achieved [91]. Hence, a report concerning the experimental tests, indicating the test type, the results and the time of construction where the test is carried out, must be considered as a part of the project. Therefore, the experimental results provided by the structural reports of 45 structural retrofitting and seismic strengthening interventions carried out on RC schools built between 1950 and 2000 have been collected. Namely, 40 schools with cast-in-situ structural elements and 5 schools with prefabricated elements were investigated. Within this study, only destructive tests on concrete cores and rebar segments, extracted from actual structures, have been taken into account. To account for factors affecting the experimental test results (e.g., length-to-diameter ratio, moisture condition, effect of damage sustained during drilling), the rules provided by the ACI 214.4R-03 standard has been used to evaluate the equivalent in-situ strength f_{core} from the strength of each core f_c. [92]. The values were then grouped by decade, according to the year of construction of the school, and the average equivalent strength was calculated for each decade. Concerning the rebar segments, both yield strength f_y and ultimate strength f_u are taken from the experimental tests. As in the case of concrete, the average values f_{ym} and f_{um} for each decade have been calculated for steel rebars.

The structural material performances affect the behaviour of the structure. If the quality of the concrete is significantly lower than that stated in the original structural report, structural weaknesses even against static loads are expected.

Depending on the type and the target of the intervention, the structural report may include the result of the static analysis. With the aim of correlating the average strength of the structural material and the potential static shortcomings, the result of the static analysis has been extrapolated for each school (when available). Table 1 summarises the details of each structural intervention.

Table 1: Results of destructive tests and static analyses performed on each school($\overline{f_c}, \overline{f_y}, \overline{f_u}$, represent the average concrete compressive strength, the average rebars yield and tensile strength, respectively, of the samples extracted from each building or building block).

8. I I	**Struct. type**	**Type of intervention**	**Date of building**	**Conc. core**	$\bar{f}_c$	**Steel rebar**	$\bar{f}_y$	$\bar{f}_u$	**Static analysis**
			Year	**n.**	**MPa**	**n.**	**MPa**	**MPa**	
#1	RC frame	Seismic strengthening	1980	5	17.3	0	[-]	[-]	Not performed
#2	RC frame	Local strengthening	1970	5	18.1	0	[-]	[-]	Verified
#3	RC frame	Seismic strengthening	1960	3	13.5	0	[-]	[-]	Not Verified
#4	RC frame	Seismic strengthening	1982	6	12.2	2	449.5	596.5	Verified
#5	RC frame	Seismic strengthening	1981	11	9.8	0	[-]	[-]	Not Verified
#6	RC frame	Seismic strengthening	1971	2	14.8	1	438.0	598.0	Not Verified
			1976	2	15.2	1	475.0	678.0	
#7	RC frame	Extraordinary repair	1979	3	23.1	1	472.9	727.5	Verified
			1991	3	38.8	4	438.7	614.3	
			1999	3	19.9	1	528.5	608.4	
#8	RC frame	Seismic retrofit	1960	3	15.7	1	287.0	386.0	Verified

8. II	Struct. type	Type of intervention	Date of building	Conc. core	$\bar{f}_c$	Steel rebar	$\bar{f}_y$	$\bar{f}_u$	Static analysis
			Year	n.	MPa	n.	MPa	MPa	
#9	RC frame	Extraordinary repair	1967	8	8.5	5	431.0	576.9	Verified
			1970	9	13.5	7	450.8	606.4	
			1971	11	11.4	3	364.7	536.2	
#10	Mixed RC and masonry	Seismic retrofit	1986	2	14.6	1	531.6	784.7	Not performed
#11	Mixed RC and masonry	Local strengthening	1950	2	17.5	2	327.8	505.5	Not performed
#12	Mixed RC, wood and masonry	Demolition and rebuilding	1984	2	20.8	0	[-]	[-]	Not performed
#13	RC frame	Renovation and Building extension	1969	4	21.4	1	369.5	539.0	Verified
#14	RC frame	Extraordinary repair	1965	5	28.3	12	471.9	679.2	Not Verified
#15	RC frame	Renovation	1980	4	15.0	1	488.0	748.0	Not Verified
#16	RC frame	Building extension	1999	2	34.8	1	526.9	629.4	Not performed
#17	RC frame	Structural rehabilitation	1982	5	20.7	2	392.2	608.0	Verified
#18	RC frame	Seismic strengthening	1969	3	20.9	1	328.0	468.0	Not performed
			1972	3	24.6	1	403.0	565.0	

8. I I	Struct. type	Type of intervention	Date of building	Conc. core	$\bar{f}_c$	Steel rebar	$\bar{f}_y$	$\bar{f}_u$	Static analysis
			Year	n.	MPa	n.	MPa	MPa	
#19	RC frame	Seismic retrofit	1979	6	11.7	5	410.6	573.6	Not Verified
#20	RC frame	Structural strengthening	1985	2	14.9	1	300.4	399.5	Not Verified
#21	Steel frame	Extraordinary repair	1972	3	20.1	1	568.3	807.5	Not performed
#22	RC frame	Seismic retrofit	1968	6	19.8	1	393.7	560.4	Verified
			1975	4	15.1	2	352.5	493.1	
#23	RC frame	Seismic strengthening	1984	8	18.5	0	[-]	[-]	Verified
#24	RC frame	Local strengthening	1976	3	17.3	1	491.4	635.5	Verified
#25	RC frame	Seismic retrofit	1973	4	8.7	1	613.1	759.0	Verified
			1984	5	15.7	0	[-]	[-]	
#26	Mixed RC and masonry	Seismic strengthening	1961	4	20.7	8	435.2	607.4	Not Verified
			1961	12	20.6	5	450.3	645.5	Verified
			1967	12	16.8	4	374.7	571.3	
#27	RC frame	Seismic strengthening	1978	4	18.8	0	[-]	[-]	Verified
#28	RC frame	Structural rehabilitation	1965	6	22.9	2	364.9	528.5	Not Verified

8. II	Struct. type	Type of intervention	Date of building	Conc. core	$\bar{f}_c$	Steel rebar	$\bar{f}_y$	$\bar{f}_u$	Static analysis
			Year	n.	MPa	n.	MPa	MPa	
#29	RC frame	Renovation	1961	15	19.8	6	390.2	519.8	Verified
#30	RC frame	Renovation	1955	3	10.6	1	369.5	504.7	Verified
#31	Mixed RC, wood and steel	Seismic retrofit	1976	7	16.5	2	501.5	680.0	Not Verified
#32	Mixed RC and masonry	Seismic retrofit	1963	7	15.1	2	366.5	505.5	Not performed
			1974	2	10.9	2	368.0	521.5	
			1982	2	17.8	0	[-]	[-]	
#33	RC frame	Seismic retrofit e Building extension	1979	3	13.1	3	426.4	636.7	Verified
#34	RC frame	Seismic retrofit	1964	48	16.6	11	401.6	573.5	Not performed
#35	RC frame	Seismic retrofit	1971	2	26.3	0	[-]	[-]	Not Verified
#36	RC frame	Seismic strengthening	1974	3	23.6	0	[-]	[-]	Not performed

8. I I	Struct. type	Type of intervention	Date of building	Conc. core	$\bar{f}_c$	Steel rebar	$\bar{f}_y$	$\bar{f}_u$	Static analysis
			Year	n.	MPa	n.	MPa	MPa	
#37	Mixed RC and masonry	Structural rehabilitation and building extension	1974	2	18.9	2	347.2	502.6	Not performed
			1975	2	18.9	0	[-]	[-]	
			1979	3	12.1	2	469.9	678.2	
#38	Cast-in-situ and precast RC	Seismic strengthening	1978	3	24.4	3	464.0	721.9	Verified
#39	Cast-in-situ and precast RC	Seismic retrofit	1979	4	24.4	6	490.8	722.3	Not Verified
#40	RC frame	Seismic retrofit	1982	10	24.6	6	443.0	648.5	Not performed
#P1	Precast RC	Seismic retrofit	1997	2	28.8	0	[-]	[-]	Verified
#P2	Precast RC	Seismic strengthening	1985	3	37.9	3	511.3	759.9	Not Verified
#P3	Precast RC	Seismic strengthening	1984	2	24.1	3	489.0	750.7	Verified
#P4	Precast RC	Seismic retrofit	1981	6	26.6	6	498.3	600.5	Verified
#P5	Precast RC	Seismic retrofit	1979	5	29.9	0	[-]	[-]	Not Verified

3.4.4 Results of the survey on the school buildings

The trend in the mechanical properties of structural materials over time was assessed by calculating the average values per decade of the compressive strength (f_{cm}) of concrete cores, as well as the yield (f_{ym}) and tensile strength (f_{um}) of rebar segments. Table 2 groups the results and also reports the outcomes of the static analysis. In Fig. 22 the comparison of the percentiles obtained using the database of the Politecnico di Torino and the average compressive strength of the cores are plotted.

In Fig. 22a the results for schools with cast-in-situ structures are depicted separately from those for schools with precast structures, whereas Fig. 22b illustrates the overall results for all the buildings.

Table 2: Average values per decade obtained by the tests performed on samples extracted from school buildings.

9. Decade	Schools	Conc. samples	f_{cm}	Rebars samples	f_{ym}	f_{um}	Static analysis					
							Verified		Not verified		Not perf.	
Years	n.	n.	MPa	n.	MPa	MPa	n.	%	n.	%	n.	%
50s	2	5	13.86	3	341,7	505,2	1	50	0	0	1	50
60s	13	136	18.09	59	417,3	589,4	6	46	4	31	3	23
70s	24	91	16.57	47	441,5	626,7	9	48	5	26	5	26
80s	15	62	16.80	25	466,1	654,7	4	33	3	25	5	42
90s	5	9	29.13	6	468,4	615,8	2	67	0	0	1	33

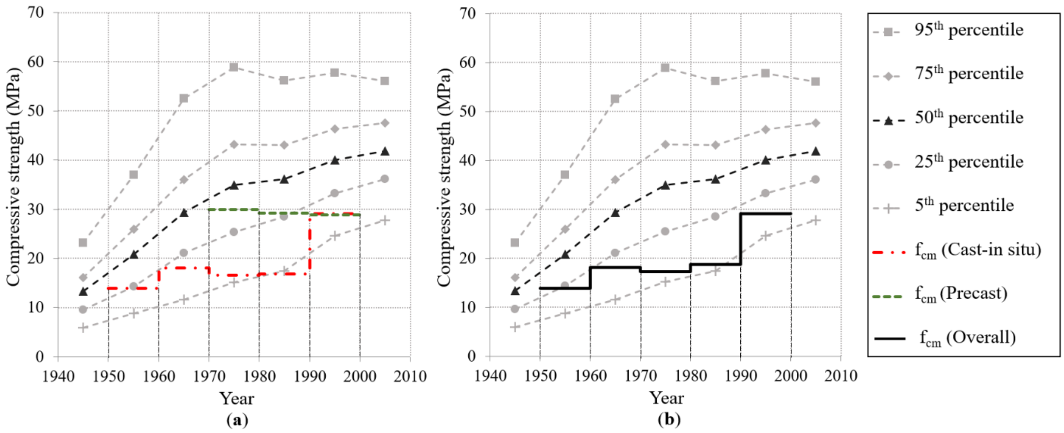

Fig. 22. Comparison of the strength-for-ages curves obtained with the database of the Politecnico di Torino and the average compressive strength computed per decade: (**a**) schools with cast-in-situ reinforced concrete structure and schools with precast structural elements separately; (**b**) all schools combined.

As illustrated in Fig. 22a, the average compressive strength of concrete increases between 1950 and 1970, with values close to the 25th percentile curve, showing a similar result to the trend observed for buildings in Tuscany [88]. Conversely, a reduction in the average mechanical strength of cast-in-situ concrete of about 8% is observed in the 70s. Thereafter, compressive strength remains constant throughout the 80s, before rising sharply by 70% in the 90s. As a result, the mechanical performance of the concrete used in the structures of the schools built between 1970 and 1990 is far lower than the average strength of the concrete tested in the laboratory of the Politecnico di Torino over the same period, and even below the 5th percentile curve in the 80s. Values deviating from the average strength, and moreover slightly above 15 MPa, could be significantly lower than stated in the structural report, and may therefore lead to shortcomings in the structure Concerning dead loads. On the other hand, the average compressive strength of concrete extracted from precast elements is nearly constant over time, with values of about 30 MPa. Such a result is deemed to be rather reliable as the concrete of precast structures has been cast and cured under controlled boundary conditions, although being less statistically robust due to the small sample of precast schools. Nevertheless, Fig. 22b shows that the overstrength of precast concrete, compared to the cast-in situ concrete, barely influences the overall strength due to the significantly lower number of schools with precast structures.

Fig. 23 illustrates the comparison of the predictive curves obtained with the database of the Politecnico di Torino and the average yield and tensile strength values of the rebar segments extracted from the structures. The error bars represent the scatter of the results.

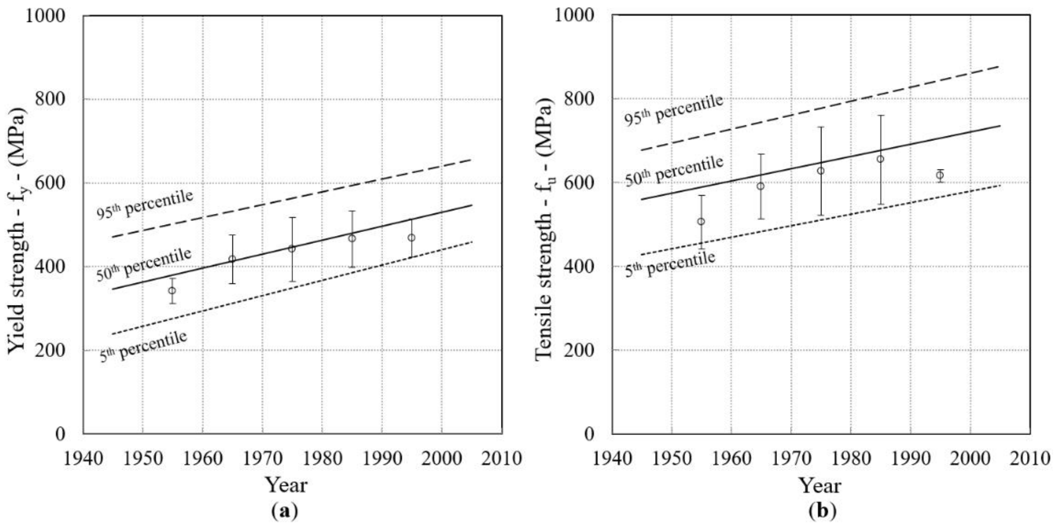

Fig. 23. Comparison of the strength-for-age curves and the results of experimental tests carried out on rebar segments extracted from schools: (**a**) yield strength; (**b**) tensile strength.

Concerning yield strength, the reliability of the strength-for-age curves are corroborated by the experimental results, since they tend to match the 50th percentile curve and the error bars fall within the 5th to 95th percentile curve. A substantial matching between the predictive curves and the experimental data for tensile strength can be claimed as well, although the average values are slightly below the 50th percentile curve and the wider error bars show a higher uncertainty of the results.

Obviously, the behaviour of the structure is strictly related to the mechanical performance of the concrete. In the case of unsound concrete, the structure can be expected to have issues against static loads besides seismic actions. In such cases, a high level of risk could be reached with only the service loads currently applied to the structure. Based on the results reported in Table 2, the histogram in Fig. 24 shows the proportion of positive and negative outcomes of the static analysis of schools.

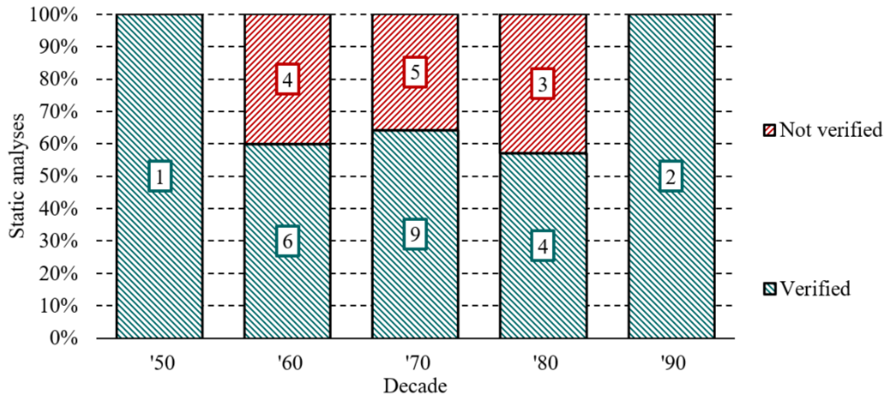

Fig. 24. Results of the static analyses in the school buildings analysed in this thesis.

The small number of schools investigated in the static analyses does not enable a clear and statistically robust correlation between concrete performances and the results of the analyses to be extrapolated. However, it is quite clear that approximately 40% of the schools built between the 60s and the 80s have issues Concerning dead loads. Throughout this period, both low mechanical performance of the concrete and a significant gap with respect to the average values obtained by the database of the Politecnico di Torino have been observed. Therefore, it can be stated that it would be worthwhile conducting an extensive campaign of vulnerability analyses on schools focusing especially on structures built during these years. Structural strengthening, renovation and rehabilitation of schools built over these years may be more cumbersome and expensive than other school buildings.

Chapter 4

Structural vulnerability and retrofitting of structural and non-structural elements in school buildings[3]

4.1 Structural vulnerability as a risk factor

Seismic risk is a function of three factors: seismic hazard, exposure and seismic vulnerability [93]. The hazard is related to the magnitude of the seismic event. The exposure considers direct and indirect effects on both humans and the surrounding area: this aspect is of paramount importance in schools, as they are considered strategic facilities for the local population, especially when identified as a shelter after a natural disaster or emergency. The vulnerability depends on the intrinsic characteristics of the construction, i.e. the robustness of the structure and the overall condition of the building. Nevertheless, whilst hazard and exposure factors can be evaluated with adequate accuracy by using available data on the

[3] *Part of this chapter has been previously published in:*

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building, such as location and intended use, the assessment of seismic vulnerability involves extensive structural analysis. Indeed, structural ductility and seismic risk are principles closely related to the progressive evolution of the design approach and construction techniques. As described in section 3.2, the seismic design of structures was introduced in 1975 in Italy, whereas only in 2003 Italy was assumed to be entirely seismic. As a result, most constructions built decades ago are prone to structural failure as they were designed according to standards neglected horizontal forces due to seismic action [94].

The following sections provide a brief overview of the shortcomings concerning both structural (with particular regard to reinforced concrete buildings) and non-structural elements.

4.1.1 Structural elements

The main problems of reinforced concrete structures can usually be ascribed to design flaws and wrong practices in construction plats. In addition, the missing or poor awareness of the structural behaviour, as well as the chemical and physical phenomena affecting reinforced concrete structures, produced damage and decay in the existing buildings such as:

Corrosion phenomena [95]: corrosion of steel rebars occurs either due to concrete carbonation or exposure to chlorides. In the first case, the carbon dioxide (CO_2) in the environment reacts with the calcium hydroxide ($Ca(OH)_2$) resulting in calcium carbonate ($CaC0_3$) and water (H_2O). This chemical transformation drops the strongly basic and steel rebars-protective pH provided by calcium hydroxide, triggering the steel oxidation (*depassivation* of the steel reinforcement). Therefore, it is common in schools located near carbon dioxide-producing sources, such as in large cities and close to roads. In the second case, depassivation is due to Cl^- ions, which in turn damage the protective iron oxide film of the reinforcement, thereby leaving it vulnerable to the effects of oxygen and water. Although this phenomenon is less common, as it is typical of semi-immersed marine structures, it can still occur in school buildings near the coast. Nowadays, we know that when passivation and, therefore, corrosion of the reinforcement occurs, then the concrete cover was not built thick enough and the concrete mix was not properly tuned to avoid CO_2 and chloride penetration.

Fatigue damage or collapse [96]: it occurs in structures exposed to prolonged cyclic loads over time. Although schools generally are not prone to fatigue phenomena, there are particular situations, such as teaching laboratories in technical schools containing mechanical equipment, where vibrations and cyclical loads should be taken into account.

Brittle behaviour [97]: a structure exhibits brittle behaviour when it suddenly collapses after reaching ultimate strength (brittle failure). Conversely, a ductile structure provides a significant advantage against earthquakes, since even when the stress exceeds the resistance capacity of a structural element, it is damaged without collapsing, providing more time for people to escape the building. Most RC buildings designed without any seismic actions are not ductile because the structural elements are designed to support the static design stresses only. In other words, the ratio between the concrete and the reinforcement area of the cross-section does not allow the plastic reserves of the structural elements and joints (plastic hinges) to be exploited.

In addition, the external loads affecting the building were often underestimated and, as mentioned above, the seismic action tended to be neglected altogether.

Slender columns [98]: for a long time, RC structures were associated with the idea of lightness and plan flexibility, which could not be achieved with masonry buildings. Therefore, structural frames were designed with slender elements in order to save space. However, slender columns, which are designed to withstand the axial compressive load only, are prone to second-order effects, i.e. the additional bending moments caused by eccentric loads. The lateral deflection reduces the ultimate load, hence the column collapses for smaller stresses than it was designed to. Schools affected by this issue are rather usual, as large windows for natural light and wide inner spaces suitable for pedagogical needs were often achieved by reducing the column footprint;

Lack of confinement of columns [99]: the transversal reinforcement (e.g. stirrups) creates a confining effect of the concrete core, which enhances both the ductility and ultimate load of the column. Thus, the high spacing among the stirrups reduces the confining effect, prompting the buckling of longitudinal reinforcement and thereby the compression crushing of the column.

Plan irregularity [100]: buildings with complex and asymmetrical layouts are extremely vulnerable to seismic action. As a matter of fact, the eccentricity between the centre of mass of the slabs and the centre of rigidity of the columns and vertical elements, induces torsional vibration modes resulting in significant displacements, especially in the edge columns. Most school buildings are characterised by an irregular floor plan: in fact, as described in sections 2.1 and 2.2, several architectural movements advocated complex layouts, such as the 'finger plan' type.

Vertical irregularity [101]: it consists of discontinuities between structural elements of consecutive storeys, e.g. misaligned or missing columns in relation to lower levels. A beam supporting column results in an extreme vulnerability to external actions, especially if it has not been designed with a proper ductility.

Schools rarely show this design flaw either because they often have a limited number of floors (especially kindergartens and primary schools), or because the distribution of space tends to be repeated among levels. Atriums and conference rooms, usually without columns, may be an exception when there are upper floors with classrooms.

Soft storey effects [102]: it occurs when the ground floor, in contrast to the upper storeys, has partitions and infill walls either half-height or completely missing, such as in buildings with *pilotis floors*. In this scenario, the bracing provided by the infill walls is lacking, thereby reducing the shear resistance of the columns with respect to the horizontal seismic action. In other words, in contrast to the good practice, the ground floor turns out to be the weakest storey of the structure, resulting in an extremely high vulnerability regarding the entire building. This structural defect is not usually found in school buildings because they have no pilotis at all, or the portions of the floor plan without walls are limited to porches or entrances.

Strong beam - weak column mechanism [103]: it occurs when the ultimate bending moment of the beams is higher than that of the columns at the beam-to-column joint. This design approach was typically used when the seismic actions were not considered, because columns are assessed basically on axial loading, thus resulting in a lower moment resistant capacity than beams whose design is based on bending moment. A column failure can trigger a chain effect that leads to the collapse of the entire structure, whereas a beam failure results in a local collapse. This design weakness is fairly common in schools. In fact, on the one hand, the design bending moment of the beams is usually high because the beam spans and the dead loads on the slabs are both significant; on the other hand, schools are usually low-storey buildings, hence axial loads on the columns are rather low. Furthermore, implementing the geometric constraints required by the code rules for beam-to-column joints is complex, so errors in the construction phase are rather common [104].

One-way RC frame [105]: still concerning structures designed without seismic criteria, ordinary buildings such as schools were usually designed with one-way slabs supported by 2D frames. Therefore, the only connection among frames frequently consisted of slabs themselves, whose negligible stiffness did not provide any resistance contribution against the seismic action perpendicular to the vertical planes of the frames.

As far as wrong practices carried out on the construction site are concerned, the following list reports the most common:

***Improper mixing of concrete components and irregular casting*:** arbitrary changes to the components of the concrete were often made before casting by modifying the mix design. For instance, adding water to increase workability was a common practice, resulting in increased porosity and reducing strength and proofing of the final concrete. The decrease in strength was not recorded because test samples of the cementitious compound were often cast before adding water. As stated in Section 3.4.4, this could partly explain the substantial discrepancy between the strength values of the cores extracted from the real buildings and those obtained from Politecnico di Torino's database, measured on the test samples. Delayed casting of concrete when the setting was already at an advanced stage, void formation due to ineffective vibration of concrete, segregation of aggregates, casting in improper weather conditions (too hot or cold) are other wrong practices.

***Inconsistency with the original project*:** frequently school structure is different from that defined in the structural design. For instance, missing or differently arranged structural elements can be observed, as well as different types and amounts of steel reinforcement. Changes were carried out either during construction, or during the lifespan of the building. In all the cases, these practices can cause serious problems, because the structural behaviour can significantly differ from that evaluated in the original project.

Project variation and building modifications: foreseeing all boundary conditions and problems during the design phase is a challenge. As a matter of fact, design changes often have to be introduced during the construction phase. Structural shortcomings can occur when substantial changes are superficially planned, for cost and time reasons, which are therefore not properly designed, or whose influence on the overall structure was not well analysed. On the other hand, increasing school populations, changing school grades, and upgrading pedagogical activities were common reasons for a school building to be modified during its lifespan. These changes, such as storey additions and plan expansions, along with structural modifications, can be harmful as well, if the behaviour of the entire structure, and not just the added or modified structure, are not properly analysed. Table 3 summarises the structural problems analysed so far, indicating whether they represent deficits against static loads in addition to seismic actions, as well as the impact grade on school buildings. Frequency has been evaluated by analysing the original structural project and the retrofitting interventions shared by the technical offices of "*Direzione Regionale Opere Pubbliche*" of the Piedmont Region and of the Municipality of Grosseto.

Table 3: Type, effects on static capacity and frequency of structural deficits found in schools.

Structural issue	Static weakness	Frequency*
Corrosion phenomena	Yes	low
Fatigue damage or collapse	Yes	low
Brittle behaviour	No	high
Slender columns	Yes	medium
Lack of confinement of columns	Yes	low
Plan irregularity	No	high
Vertical irregularity	Yes	low
Soft storey effects	No	low
Strong beam - weak column mechanism	No	high
One-way RC frame	No	medium
Improper mixing of concrete components and irregular casting	Yes	high
Inconsistency with the original project	Yes	medium
Project variation and building modifications	Yes	high

* *low* = The structural deficiency is rarely observed in school buildings; *medium* = it is quite common to detect the structural deficiency in school buildings; *high* = the structural deficiency is inherent in the approach used in the past for the structural design of schools, thus it is frequently observed.

4.1.2 Non-structural elements

Although the structures are affected by numerous defects and problems, designing structures is a long-established practice. In addition, static tests on building have been required since 1970. Conversely, the static capabilities and dynamic behaviour of the non-structural elements were usually assessed either by rough dimensioning based on common practice, or by verifying a model-type without properly tailoring to the specific situation. However, in several cases they are not assessed at all. Along with a lack of proper design of non-structural elements, there is often a poor awareness of their material, components, fastening system and condition. Sometimes their presence is not even known during the design stage. Particular attention should be devoted when the entire project (or part of it) is missing, or when construction changes have been carried out over the years without providing the details. *FEMA E-74* is a guideline on the risk sources from non-structural elements, which provides methods and best practices to enhance the safety [106].

The risk fields affected by the failure or damage of non-structural elements are:

- Life safety;
- Functional loss and condemned buildings;
- Economic losses;

The critical aspects resulting from a collapse or failure of each of the main non-structural elements of school buildings are discussed below:

Dropped ceiling: in school buildings, as well as in offices and other public buildings, there are different types of systems necessary to fulfil workplace, lighting and health requirements. Hence, suspended ceilings with the aim of conveying and hiding the systems are quite common in schools. The ceiling types are numerous, depending on the time of construction, the function and the aesthetic requirements. Nowadays, ceiling panels are lightweight (usually plasterboard) and easy to be removed for inspection and replacement. Moreover, the substructure is redundant and the fastening systems are certified and assessed according to the standards. Among the obsolete typologies, timber or reed ceilings, rarely used in pre-twentieth-Century schools, have been almost completely removed for fire safety reasons. On the other hand, brick tile ceilings are still rather common. This ceiling type was widely used after World War II until the 70s in new and existing school buildings. The construction system usually consists of brick tiles supported by a steel wire hanger system fastened to the slab, or supported by steel profiles fixed to the walls or suspended from the slab. This ceiling system is considered unsafe because the brick tiles are extremely heavy and brittle, hence their collapse, besides being sudden, can cause severe injuries to people. Moreover, the fastening system is hidden by fixed tiles, which cannot be easily removed, thus making inspections invasive and costly. Besides, the steel wire hanger system was often fixed to the hollow brick instead of to the slab joists, which could lead to the hollow brick failing. When the plastered and painted brick slabs appear as the intrados of a brick ceiling and the building information is lacking, it is not unusual that the presence of the ceiling is not even known. In this respect, the collapse of a brick ceiling tile in the *Darwin high school* in Rivoli, Turin, Italy, which resulted in the death of student Vito Scafidi, was the tragic result of a series of construction shortcomings and carelessness (Fig. 25). To begin with, the authorities were not informed of the brick ceiling tiles, which resulted in the absence of inspections and maintenance. Furthermore, heavy rubble and abandoned plant system sections were reported following the disaster [107]. Note that the ceiling collapsed suddenly and without a well-identified reason, i.e. it was not caused by an earthquake or other disasters.

Fig. 25: Illustration of the ceiling collapsed at the Darwin High School in Rivoli, Turin, Italy.

External walls and partitions: In RC structures, the infill walls have the mere function of insulation and separation between the internal and external environment, whereas the internal walls act as partitions to create internal rooms. Therefore, these walls were not subject to structural assessment, because they are expected to carry only the self-weight. This represents a risk factor in the case of seismic shaking, as masonry are extremely weak and brittle. In fact, the walls are prone to fail and overturn if they are not adequately fastened to the structure, which could cause injury to people and can block the escape routes [108]. Retaining systems have only recently been introduced as good practice in seismic areas. Obviously, partitions made with lighter construction technologies, such as plasterboard partitions, can also be susceptible to overturning as well, if they are not properly fixed to the slabs and lateral structures. Undesired behaviour of the structure can arise from the *frame-infill interaction effects*, i.e. walls arranged between columns that do not cover their full height. Indeed, walls between columns act as bracings, hence a horizontal opening in the masonry results in a change of stiffness in the column that may induce a shear collapse under horizontal forces [109].

Doorway getting stuck: a risk factor is door jamming due to the frame warping following an earthquake or a fire, trapping people inside the building without allowing them to escape [110].

Cornices, decorative friezes and other protruding elements: building parts such as cornices, statues, friezes, corbels and signs with complex shapes, are unique non-structural elements and, therefore, there is not always a standard method for verifying their stability. Moreover, they are often located outdoors, thus being particularly susceptible to ageing aspects. Furthermore, in the case of friezes, statues or similar, checking the condition of the fastening system without damaging them is tricky, as well as being difficult to access. Yet, the risk of downfall of these elements is significant, especially when they are placed above entrances, transit and meeting places.

Shelving, furniture, equipment and hanging elements or components: heavy furniture and shelving are prone to overturning if they are unanchored or weakly anchored. The contents of shelves, devices and equipment without restraints are a risk to people as well. Finally, lightweight but high-placed elements such as lamps, plant terminals and suspended pipes also have considerable damage potential due to the energy they gain on falling.

Table 4 aims at providing an overview on the type of risk resulting from the collapse or damage of the non-structural parts of school buildings described above (injury to occupants, or risk of escape routes being blocked) and the level of risk they represent, expressed on a scale from 1 to 3.

Table 4: Overview of non-structural elements representing a potential risk in relation to injuries to persons, influence on escape routes and overall risk level

Non-structural part	Injuries	Escape routes blocking	Risk level*
Dropped ceiling	Yes	Yes	3
External walls and partitions	Yes	Yes	2
Doorway getting stuck	No	Yes	2
Cornices, decorative friezes and other jutting out elements	Yes	Depends on size	2
Shelving, furniture, equipment and hanging elements or components	Yes	Depends on size	1

* 1 = Low risk due to the limited weight of the element, low number of people involved and modest exposure level; 2 = Moderate risk due to the non-negligible weight of the element, the moderate exposure and number of people involved; 3 = High level of risk due to the significant damage potential of the element, the large number of people involved and the high level of exposure.

4.2 Retrofitting methods for structural elements

In this section, the most common methods for the retrofitting of the RC structures are briefly presented, by analysing their main strengths and drawbacks Concerning their implementation in school buildings. They are grouped according to their main function (local strengthening of beams or columns, or global reinforcement of the structure), illustrating the strengths and weaknesses while also considering their implementation in school buildings. Methods for the consolidation of non-structural elements will be explained in Section 4.4.

4.2.1 Retrofit systems for RC columns

These methods, when applied locally on a single column, usually aim at addressing the static weaknesses of a specific structural element. On the other hand, if they are extended to several columns, they represent a valuable strategy for the seismic retrofitting of the whole structure.

Steel caging system: is a widely used technique to strengthen RC column having rectangular cross-section, by applying angle-section steel profiles at the corners of the column joining them by welded steel plates (Fig. 26a). Angle-section profiles are fastened to the existing column by means of expansive mortar or epoxy resin [111]. The main **advantages** of this retrofitting approach are:

- rapid implementation, as no formwork has to be assembled and curing of concrete is not necessary;
- confining effect on the existing column, increasing the ultimate strength, as well as ductility;
- increasing shear strength of the column provided by horizontal plates.

On the other hand, the main **shortcomings** of this strengthening system are:

- corrosion susceptibility, which requires an appropriate protective coating, especially for edge columns;
- welding between vertical profiles at the corners and horizontal plates is carried out in situ, i.e. under uncontrolled boundary conditions, and therefore less reliable than industrial welding;
- potential debonding of the steel profiles from the existing column [112].

In this regard, ACM (Active Confinement of Masonry) technology represents an emerging approach, originally conceived in Italy for seismic retrofitting of masonry and later tailored for the strengthening of columns, beams and beam-to-column joints. It consists of folded steel angle profiles with a rough internal surface, whereas the cross-connection is obtained by means of pre-tensioned strips [113]. Compared to the steel cage system, this method increases the confinement effect while reducing the debonding between the steel elements and the existing concrete.

Concrete jacketing: is a long-standing method for reinforcing rectangular or circular cross-sections of an existing column by wrapping up a RC jacket (Fig. 26b). The construction process starts with the removal of any deteriorated layer of the existing column, thus resulting in a rough surface. Then the steel reinforcement is assembled and finally the new concrete is cast after creating the formwork [114]. Since an adequate concrete cover must be provided, the new layer should have a thickness of at least 4-5 cm. The main **advantages** of this retrofitting approach are:

- the substantial increase of strength (in compression, bending and shear) and stiffness due to the cross-section and rebar amount;
- the confining effect of the jacket increases ductility as well;
- the good adhesion between the core (existing column) and the jacket prevents debonding;
- if the cover is properly designed, both core and additional reinforcement are protected by environmental aggressions.

On the other hand, the main **shortcomings** of the strengthening system are:

- the significant increase in the footprint of the column affects the use of the construction;
- it consists of a sequence of complex, time-consuming steps, such as the new reinforcement and formwork assembling and waiting for the concrete to cure;
- modelling the behaviour of strengthened column is complex, as the jacket is unloaded while the core is loaded [115];
- fire resistance could be a problem.

More recently, jackets made with HPC (High Performance Concrete) or UHPC (Ultra High Performance Concrete) have been developed, which reduce or eliminate rebars, thereby decreasing the footprint of the column (UHPC will be see in section 4.3).

FRP jacketing: Fiber Reinforced Polymer jacket system consists of wrapping the rectangular or circular cross-section of a column with strips, made by high-or-ultra-high-strength polymers (Fig. 26c).. The most used are polymers based on glass (GFRP), carbon (CFRP) or aramid (AFRP). Bonding to the existing column is carried out using epoxy resin. The main **advantages** of this retrofitting approach are:

- the implementation of the strengthening technique does not affect the use of construction;
- the confining effect increases the shear and the compressive strength, and the ductility as well;

- Polymer fibres are not prone to corrosion and can be easily applied;
- extremely lightweight system, hence the increase of permanents actions on the structure is negligible

On the other hand, the main **shortcomings** of this strengthening system are:

- possible delamination of polymer strips from the existing column, especially in cases of shear stresses. As a result, the increase in the bending strength could be small;
- the unit cost of the material is quite high, so extensive application to several columns is expensive;
- this technique is quite young, thus long-term durability is still an open debate;
- fire resistance has to be assured through specific protection.

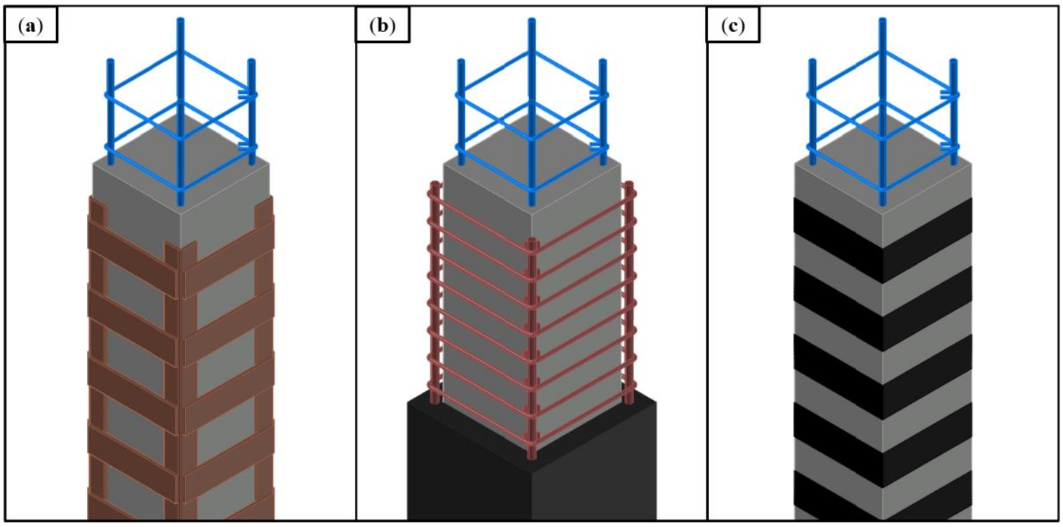

Fig. 26: Most common methods for the retrofitting of the RC columns: (**a**) steel caging system; (**b**) concrete jacketing; (**c**) FRP jacketing.

4.2.2 Retrofit system for RC beams

Beam strengthening systems are based on the same materials and principles already described for columns, with more or less the same advantages and disadvantages. In addition, implementing retrofitting systems to the lateral sides of beams could be tricky in the presence of slabs.

Steel plates: consists of applying steel plates to the zone in tension to enhance ultimate bending moment strength (Fig. 27a). Similarly, applying plates to the lateral faces of beam increases shear strength. Steel plates are usually bonded by means of epoxy resin, possibly combined with dowels [116]. The main **advantages** of this retrofitting approach are:

- rapid to be implemented, as no formwork has to be assembled and curing of concrete is not necessary;
- substantial increase of bending strength (and shear when vertical plates are used);

On the other hand, the main **shortcomings** of this strengthening system are:

- corrosion susceptibility, which requires an appropriate protective coating, especially for edge columns;
- steel plates on the tension surface can lead to a decrease in ductility, as it is equivalent to oversizing the bottom steel reinforcement;
- potential debonding of the bottom steel plates from the existing column;
- difficult application of plates on the lateral surface when the beam supports a slab, especially in beams within the thickness of the slab;
- fire resistance is not always assured.

As well as for steel cages of the columns, the ACM method also brings advantages in the case of beams. In fact, the thin pre-tensioned strip, besides making the beam more ductile due to the confinement effect, requires a less invasive hole for their passage through the slab.

Increase in RC cross-section of the beam: is a well-established method consisting in increasing the beam section through additional reinforcement and concrete in the tension zone at least to increase the bending moment capacity of the beam (Fig. 27b). By adding stirrups and increasing the cross-section width, an increase in shear strength is achieved as well. The new rebar are fastened to the existing beam using dowels.

The main **advantages** of this retrofitting approach are:

- besides increasing bending moment and shear strength, an increase in ductility can be achieved, if the new steel reinforcement is adequately designed;

- the good adhesion between the core (existing beam) and the new concrete layer prevents debonding;
- if the concrete cover is properly designed, there is good protection of both core and additional reinforcement, and also against fire.

On the other hand, the main **shortcomings** of this strengthening system are:

- the beam significantly increases in volume, with aesthetic problems or conflicts with the use of space;
- consisting of a sequence of complex, time-consuming steps, such as assembling the reinforcement and formwork and waiting for the curing of concrete. Besides, the formwork is suspended, requiring a shoring system;
- modelling the behaviour of strengthened beam is complex, as the jacket is unloaded while the core is loaded.
- lateral reinforcement is cumbersome or unfeasible where there is a slab.

As in the case of jackets for columns, HPC and UHPC can also be used in the case of beams to reduce or avoid using additional reinforcement and the volume of new materials.

FRP strips: This is currently one of the most used methods, indeed there exists an extensive literature (Fig. 27c). As on this topic for columns, FRP strips are applied with epoxy resin. If the strips are applied along the tension zones of the beam, the bending moment is increased. On the other hand, the application of the strips on the lateral surfaces of the beam, when it is possible, leads to an increase of shear strength. The main **advantages** of this retrofitting approach are:

- FRP strips do not lead to an increase in beam size;
- there is an improvement of ductility;
- Polymer fibres are not prone to corrosion;
- it can be applied easily;
- it is a lightweight system, hence the increment of loads on the structure is negligible.

On the other hand, the main **shortcomings** of this strengthening system are:

- possible delamination of strips from the existing beam, especially in the cases of shear stresses [117];
- the unit cost of material is quite high, thus an extensive application is not always possible;
- the long-term durability is still an open issue.
- the strips cannot be easily applied when the beam is totally within the thickness of the slab.

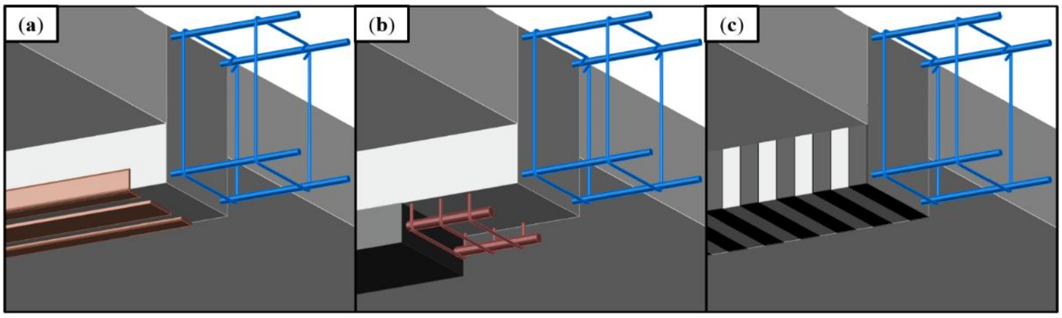

Fig. 27: Most common methods for the retrofitting of the RC beams: (**a**) steel plates system; (**b**) increase in RC cross-section of the beam; (**c**) FRP strips.

4.2.3 Retrofitting strategies for improving the overall structural behaviour

As well as the application of the retrofitting techniques to the structural members illustrated in the previous paragraphs, there are methods of strengthening and improving structural behaviour of whole frames, especially against horizontal forces.

Reinforced concrete walls: it consists of RC walls to stiffen the frames against horizontal action (Fig. 28a). The length is tailored according to the applied horizontal loads. The longitudinal steel reinforcement and stirrups must be properly designed to provide adequate shear resistance while ensuring the required ductility. The position depends on the location of the centre of masses and the centre of stiffness. The main **advantages** of this retrofitting approach are:

- RC walls, also provide a valuable contribution to bear gravity loads;
- the construction cost is consistent with that of RC structures;
- if the concrete cover is properly designed, it is barely prone to degradation phenomena such as corrosion.

On the other hand, the main **shortcomings** of the strengthening system are:

- a structural wall has a significant footprint, hence it may compromise the use of the construction. Furthermore, RC walls cannot include openings, therefore previously existing doors and windows in envelopes and partitions must be removed;
- consisting of a sequence of time-consuming steps, such as assembling the reinforcement and formwork, as well as waiting for the curing of concrete;
- structural models including the addition of RC wall are complex to model, as the new walls are unloaded.

Steel bracing: like RC walls, the main target of this retrofitting system is both to stiffen the structure against horizontal forces and to decrease storey drift [118]. X-shape is the most typical geometry of bracing, but there are other solutions that allow for windows to be placed, such as inverted K-shape bracing (Fig. 28b). On the other hand, to achieve optimum performance, steel bracing usually has to be located between two columns, covering the entire span of the beam. Therefore their footprint depends on the geometric characteristics of the RC frame. Steel bracings are less invasive than RC walls because they may require only partial demolition of the masonry and openings, or even no demolition when they are located outside the frame. Tube profiles, with circular or square cross-section shapes, or open profiles, such as IPE and HE types, are usually used.

The main **advantages** of this retrofitting approach are:

- faster to setup than RC walls;
- comply with the "*design-for-disassembly*" approach, representing a more sustainable solution;
- assembly is entirely or partly carried out at the industrial level, under controlled boundary conditions, thus the system ensures more reliable performances.

On the other hand, the main **shortcomings** of this strengthening system are:

- when applied on envelope walls, bracing is prone to corrosion, thus must be properly protected;

- it could compromise the use of the structure (as it reduces the openings).

Strengthening and confinement of structural nodes: it is a key retrofitting technique to improve the behaviour of the structure with respect to seismic action [119]. The strengthening of non-seismically designed joints increases their shear strength and confinement effect, as well as it ensured their ductile behaviour to prevent brittle collapse. The retrofitting of joints is usually carried out by using the same strengthening techniques, as illustrated for columns and beams, thus it consists of strengthening the beams and columns over the joint (Fig. 28c). In this case, the strong beam-weak column mechanisms can be reversed to achieve plastic hinge formation, which is the main source of inelastic response and ductile behaviour of a structure [120]. Accordingly, each strengthening technique shows the same advantages and disadvantages of the corresponding beam and column retrofitting systems.

Base isolation and damping systems: Base isolation systems aim at isolating the above structure from the ground shaking generated by the earthquake (Fig. 28d). They are used in high-seismicity areas, where the strengthening systems described above are not sufficient. There are several base isolators, which differ on the producer and the type of use [121, 122]. However, studies have revealed that these systems could be prone to near-fault and far-fault earthquakes [123]. In these cases, they must be combined with damping systems, which aim at modifying the natural vibration frequency of the building when this tends to match the frequency of the seismic action [124]. In spite of the very high protection levels that can be achieved, these techniques are rarely applied due to the high cost of the devices and to the difficulty of implementation at the foundations of existing buildings. Moreover, as performances decay over time, the associated maintenance costs could be unsustainable. Since the implementation of insulation systems should be as uniform as possible throughout the building foundations, costs tend to increase in the case of school facilities, also considering that schools are often low-rise buildings with a large footprint. Therefore, their use in Italy is generally limited to the high-seismic zones [125].

Exoskeleton structures: it is a promising approach for the strengthening of existing buildings that has gained increasing interest in recent years. It consists of self-supporting structures fastened around the existing building (Fig. 28e). This technique contributes to supporting both horizontal and vertical actions, also resulting in a sustainable solution because it can be disassembled. The main limits to the use of this system are the high cost and its aesthetic impact.

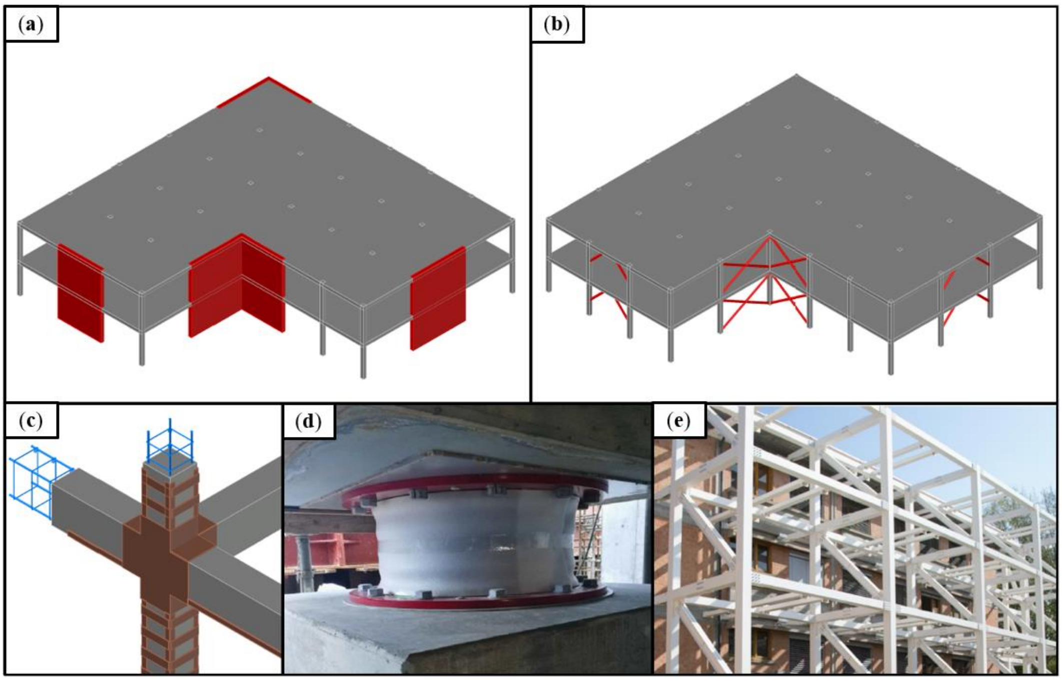

Fig. 28: Most common retrofitting strategies for improving the overall structural behaviour: (**a**) Reinforced concrete walls; (**b**) steel bracing; (**c**) strengthening and confinement of structural nodes (picture shows node confinement in the case of steel caging and plate for retrofitting columns and beams); (**d**) base isolation and damping systems; (**e**) Exoskeleton structures ("Corazza" Primary school in Parma [126]);

4.3 A new approach to strengthen existing RC beams

In the previous chapter, the use of UHPCs to reduce the thickness of jackets and layers in some strengthening approaches of columns and beams was mentioned. A further step forward in the application of these techniques is the use of Ultra High Performance Fiber Reinforced Cementitious Composites (UHP-FRCCs), i.e. composites based on a highly dense cement matrix containing fibres to increase tensile strength. In addition to the high tensile and compressive strength, UHP-FRCCs have remarkable waterproofing properties and therefore can protect structures not only from water, but also from aggressive agents. For this reason, UHP-FRCC have recently been used in the rehabilitation of existing buildings [127, 128, 129].

Cast-in-situ coating layers, made of UHP-FRCC and cured at ambient conditions, are used to enhance the bearing capacity and stiffness of exiting RC beams [129], and to repair those damaged [130]. Nevertheless, this strengthening procedure requires laborious frameworks to be built and long casting procedures compared to the use of FRP. In addition, it is not easy to apply cast-in-situ layers on the bottom of beams because of the gravity action. For these reasons, some studies have been devoted to the mechanical performance of precast HP-FRCC slabs [131] used to strengthen RC structures. For instance, Jongvivatsakul et al. significantly increased the shear capacity of RC beams by applying Steel Fiber-Reinforced Precast Panels to the faces [132]. However, when a cast-in-situ or precast panel overlays an existing structure, the effectiveness of the strengthening depends on the bond condition at the interface between new and old structures.

To avoid the delamination phenomena produced by weak adhesion, roughening treatments of the existing concrete surfaces ensure better performances than using bonding agents, such as epoxy resin [133]. In particular, sandblasting and chipping enable a higher surface roughening than grooves and drill holes [134]. Moreover, the environmental impact of the reinforcing layers made with UHP-FRCC is also an important aspect that has been taken into consideration. In fact, the massive content of cement and the presence of large volume of steel fibers make UHP-FRCC a high carbon footprint material. Thus, the Material Substitution Strategy (MSS), which consists of replacing a large part of clinker with mineral additives, can be an effective way to reduce the embodied CO_2 [135].

Accordingly, a new approach for retrofitting concrete structures is proposed. It consists of enhancing the resisting bending moment, and therefore the lifespan, of existing concrete beams by adding UHP-FRCC in the tensile zone. The work described here is a comparative study in which not only the mechanical performances (such as bearing capacity, bond conditions and ductility), but also the environmental impact of different UHP-FRCC layers are taken into account.

4.3.1 The concept behind the new retrofitting technique and the experimental test campaigns

The proposed idea for repairing and strengthening existing beams against bending moments is to apply a precast UHP-FRCC panel, produced under controlled boundary conditions, to the bottom face of the beam. The fastening is carried out by means of screws and dowels, leaving an empty gap to be filled with a layer of UHP-FRCC. An effective adhesion between the filling layer, the existing beam and the precast panel is achieved by using screws that are tightened after casting. In other words, the precast panel, besides providing a significant increment of strength for the beam, also acts as a formwork for casting the filling material, thereby avoiding the use of complex shoring systems and formwork. Besides, by using of UHP-FRCC the following beneficial effects can be obtained:

- avoiding detachment from the existing structure, which on the contrary could occur with FRP strips or steel plates;
- increasing the stiffness of the retrofitted beam more than that achieved with FRP and steel plates. An increase in stiffness would reduce the beam strain due to vertical loads. In addition, stiffer beams result in stiffer frames, thereby reducing inter-storey drifts. Both of these beneficial effects result in less stress on non-structural elements, such as external walls, partitions, windows and doors, which would be less damaged by the seismic load;
- tailoring the mix of UHP-FRCC to achieve the required performance even while reducing the embodied CO_2 of the precast panel.

Fig. 29 is an indicative sketch of the proposed retrofitting technique. The steps of the panel implementation and fastening procedure to the existing beam are also described.

To move from the basic concept to a high readiness level, several tests are needed to verify all the parameters that come into play. The work carried out so far has consisted of two successive test campaigns. In the first one, the mechanical performance of different types of panels obtained by changing the density of the embedded nails were studied on small-scale mortar specimens. In this first stage, the results were compared with the current method of beam reinforcing with a cast-in-situ layer, also assessing the intrinsic carbon footprint of each type of panel. In the second test campaign, larger scale RC beams retrofitted with panels were tested for evaluating the scale effects and the influence of steel reinforcement.

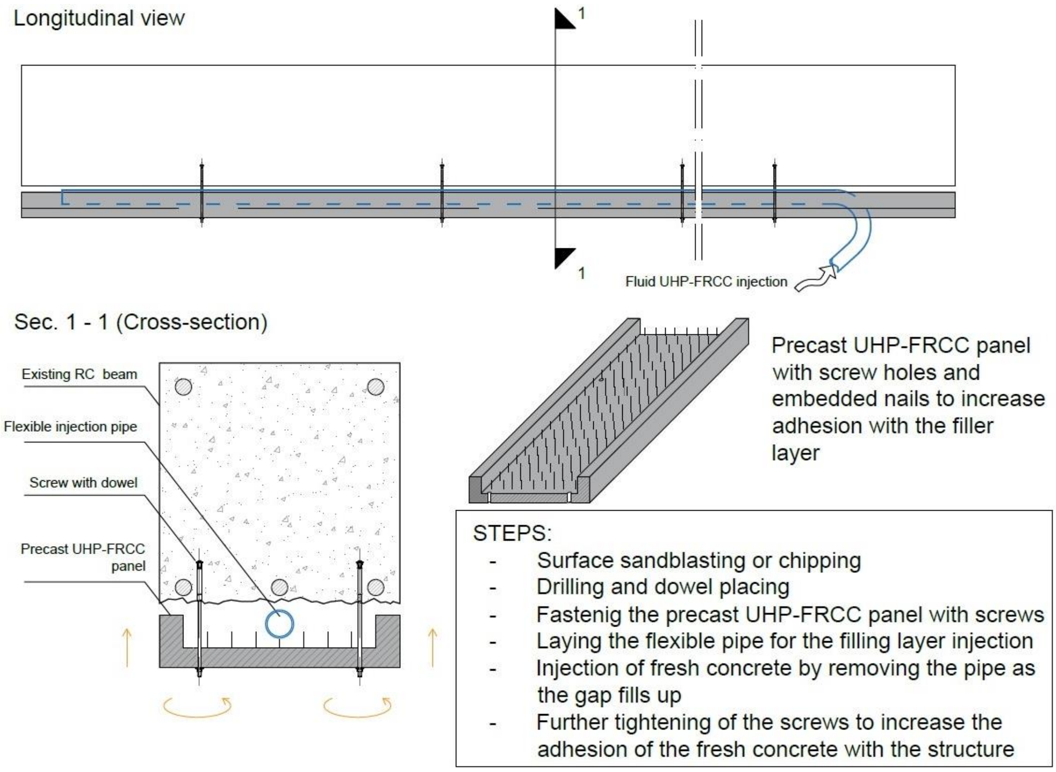

Fig. 29: The proposed method for the retrofitting of an existing RC beam and the steps for assembling and fixing the panel to the existing structure.

4.3.2 Materials and Test Procedure

As known, compressive strength of UHP-FRCC is generally higher than 150 MPa and, in the pre-softening stage, the energy absorption capacity is larger than 50 kJ/m^3 [136]. These performances are achieved not only by adding fibers, but also with a dense microstructure, which is in turn tailored with an extremely low water-binder ratio (lower than 0.2) and by using ultra-fine additive, such as silica fume, wollastonite, and fine sand. Table 5 reports the density of all the materials used to tailor three series of mortar.

Table 5: Materials used to cast the samples (data sheet of producers).

Material	Symbol	Density [kg/m^3]
High Early Strength Portland Cement	HESPC	3140
Low Heat Cement	LHC	3240
Crushed Sand	S1	2610
Land Sand	S2	2580
Silica Fume	SF	2200
Silica Sand	Ss	2600
Wollastonite	Wo	2900
Water	W	1000
Superplasticizer	SP	1050
Defoaming Agent	DA	1010
Macro-fibers (30 mm long)	HDR	7850
Micro-fibers (6 mm long)	OL	7850

Existing beams have been cast with a normal strength mortar, according to mix proportion shown in Table 6. Three UHP-FRCC (namely, FA0, FA20, and FA70) have been used to strengthen the existing beams by means of precast panels. According to the "*material substitution strategy*", the reduction of the carbon footprint of cementitious composites is achieved by replacing cement with materials having a lower embodied CO_2. As reported in Table 7, with respect to the reference FA0, containing only cement and silica fume as a binder, in the mix proportions of FA20 and FA70, 20% and 70% of cement have been replaced by fly ash, respectively. In fact, fly ash is a waste product of coal combustion in industrial processes. Finally, the mix design of the filler layer is shown in Table 8. To reduce the viscosity and facilitate the injection, this filler is obtained from FA0 with macro-fibers only. The mechanical properties of all the mixtures are reported in Table 9. Compressive strength and Young's modulus have been determined by testing cylindrical samples in uniaxial compression, whereas tensile strength has been measured with uniaxial tensile tests on dumbbell shaped specimens [137].

Table 6: Composition of the normal strength mortar simulating the existing beams (kg per m^3 of concrete).

HESPC	S[1]	W
485.6	1456.7	291.4

[1] S = S1 (50% weight) + S2 (50% weight).

Table 7: Composition of the retrofitting UHP-FRCC layers.

Series	W[1]	LHC[1]	SF[1]	FA[1]	Ss[1]	Wo[1]	SP[1]	DA[1]	HDR[2]	OL[2]
FA0	201	1197	263	0	511	190	32.1	0.3	1.5	1
FA20	195	928	255	232	495	184	31.0	0.3	1.5	1
FA70	181	323	236	753	459	170	31.5	0.3	1.5	1

[1] kg per m^3 of concrete.; [2] % Vol of concrete.

Table 8: Composition of the filler layer.

W[1]	LHC[1]	SF[1]	FA[1]	Ss[1]	Wo[1]	SP[1]	DA[1]	HDR[2]	OL[2]
201	1197	263	0	511	190	32.1	0.3	0	1

[1] kg per m^3 of concrete.; [2] % Vol of concrete.

Table 9: Mechanical proprieties of UHP-FRCC.

Parameter	Normal Strength Mortar	FA0	FA20	FA70
Compressive strength [MPa]	45.1	204.7	193.4	150.7
Young's modulus [MPa]	24.9×10^3	46.4×10^3	45.5×10^3	40.1×10^3
Tensile strength [MPa]	2.6	16.5	17.8	7.4

The use of UHP-FRCC panels is particularly effective in the refurbishment of existing buildings, due to their dual function: structural strengthening and protection against aggressive agents. Nevertheless, due to the high cost and to the environmental impact, UHP-FRCC panels are not used to cover the entire perimeter of the cross-section, or through a three-side jacket [138], but rather they are located only the bottom part of a RC beam. Accordingly, in this research project, small-size and medium-size beams with a UHP-FRCC layer in the tensile zone are investigated with two different tests of Campaign 1 and Campaign 2.

In the Test Campaign 1, several series UHP-FRCC strengthening layers (Type 2 in Fig. 30b), differing in FA amount to reduce the embodied CO_2 (series FA0, FA20, and FA70) and in node density (Fig. 30c) to evaluate different bonding conditions, were cast. These panels were fastened to the mortar beam following the procedure shown in Fig. 31. The results obtained from these specimens are benchmarked with those of unreinforced beams (Type 0 in Fig. 30b) and with the samples representative of the current strengthening method (i.e., Type 5 in Fig. 30b). The letter consists of a UHP-FRCC layer, cast-in-situ on the surface of the beam without any screws and plugs. On the other hand, in Test Campaign 2, the focus was on scale effects on larger RC beams.

4.3.3 Test Campaign 1

In the first campaign, mortar beams and precast layers were cast, and the assembling procedure illustrated in Fig. 31 was followed to create the composite structure. Not only the different types of the layers shown in Fig. 30b are tested, but also the environmental performances of the UHP-FRCC are analysed. Such layers have been made by FA0, FA20, and FA70 mixtures shown in Table 7. The mortar beams were stored in the curing room for 28 days, whereas the precast layers have been steam cured and then stored in the curing room for the same lapse of time. To simulate the current reinforcing approach, the bottom surfaces of some mortar beams are chipped and, subsequently, reinforced with a layer of UHP-FRCC cast on the bottom surface, as shown in Fig. 30b (Type 5 layer). The remaining mortar beams are strengthened by applying the precast UHP-FRCC layers (Type 2 in Fig. 30b,c) by means of chipping, plugs, screws, and the filling layer, as illustrated in Fig. 31. Fig. 30a shows the general scheme of the 120 mm high specimens tested by means of a 1000 kN Universal Testing Machine, whereas Table 10 summarizes the composite beams investigated in Test Campaign 1. As every type of specimen counted four samples, a total of 40 beams were realized and tested.

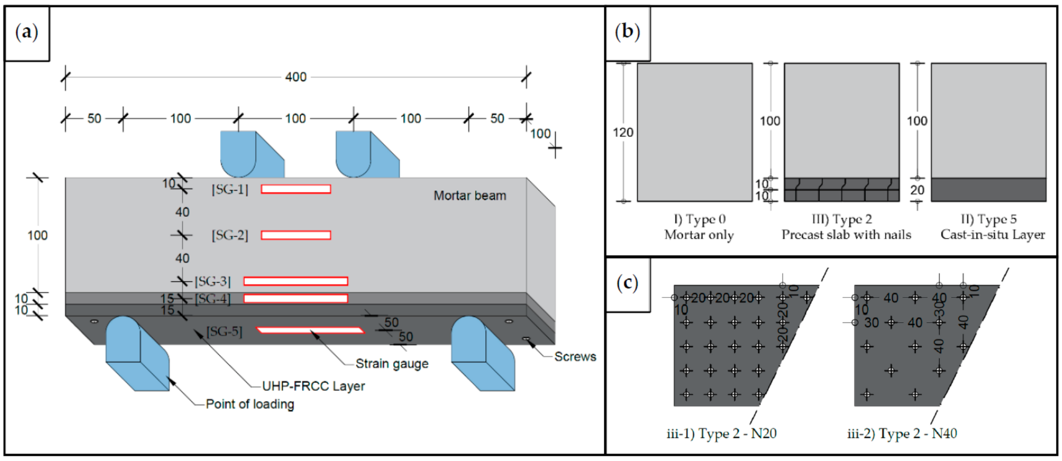

Fig. 30: Mortar beams retrofitted by UHP-FRCC layer. (**a**) Specimens shape, strain gauges arrangement, and test configuration regarding the first test campaign; (**b**) Precast UHP-FRCC layers used to reinforce concrete beams; (**c**) Nail density on the surface of the type 2 precast layers for improving the connection between the parts.

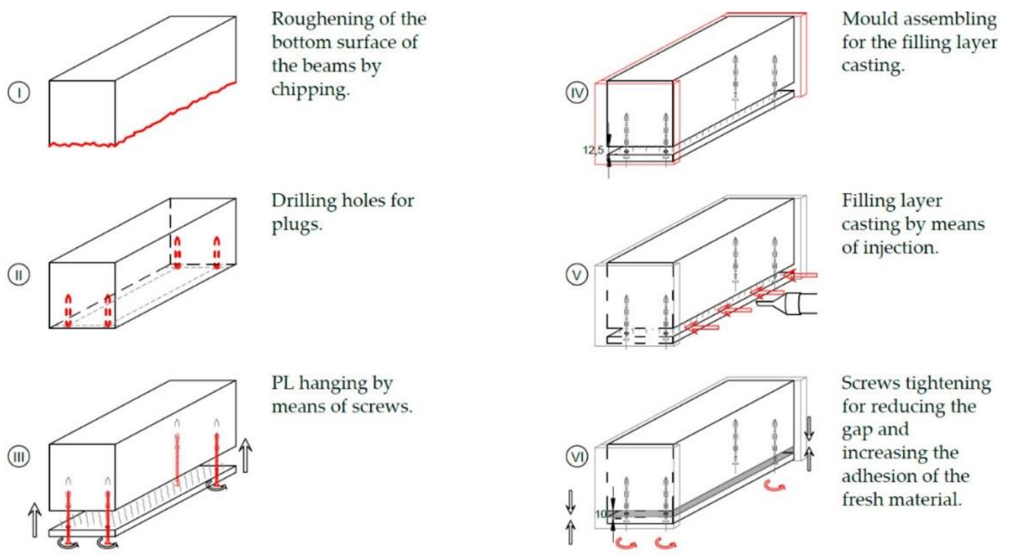

Fig. 31: Assembly procedure used in Test Campaign 1

Table 10: Details of the specimens tested in the Test Campaign 1.

Symbol	Layer Type	Bonding Surface	UHP-FRCC Series	Number of Samples
B_II_T0	None	[-]	[-]	4
B_II_T2_FA0_N20	Type 2	SF – d = 20 mm(1)	FA0	4
B_II_T2_FA0_N40	Type 2	SF – d = 40 mm(2)	FA0	4
B_II_T2_FA20_N20	Type 2	SF – d = 20 mm(1)	FA20	4
B_II_T2_FA20_N40	Type 2	SF – d = 40 mm(2)	FA20	4
B_II_T2_FA70_N20	Type 2	SF – d = 20 mm(1)	FA70	4
B_II_T2_FA70_N40	Type 2	SF – d = 40 mm(2)	FA70	4
B_II_T5_FA0	Type 5	Chipping only	FA0	4
B_II_T5_FA20	Type 5	Chipping only	FA20	4
B_II_T5_FA70	Type 5	Chipping only	FA70	4
B_II_T0	None	[-]	[-]	4

(1) Steel fibers spaced 20 mm apart; (2) Steel fibers spaced 40 mm apart

In all the tests, five strain gauges were glued in the composite beams, one on the lower surface and four on a beam side face. As shown in Fig. 30a, the strain gauges [SG-3], [SG-4] and [SG-5], closer to the bottom, are 90 mm long, and the rest are 60 mm long. By means of these instruments, the strain profile is measured in the cross-section of the constant moment zone, and the debonding phenomena among the layers are also detected. In addition, the measure of the curvature is carried out with the following equation:

$$\chi = (\varepsilon_5 - \varepsilon_1)/d_0 \quad (1)$$

where ε_5 and ε_1 are the strains measured by the strain gauge [SG-5] (on the bottom side) and strain gauge [SG-1], respectively; and d_0 = 110 mm is the distance between the two gauges.

Fig. 32 shows the average moment–curvature relationships of the strengthened beams, whose strength capacity (i.e., the maximum bending moment) is summarized in the histogram of Fig. 33.

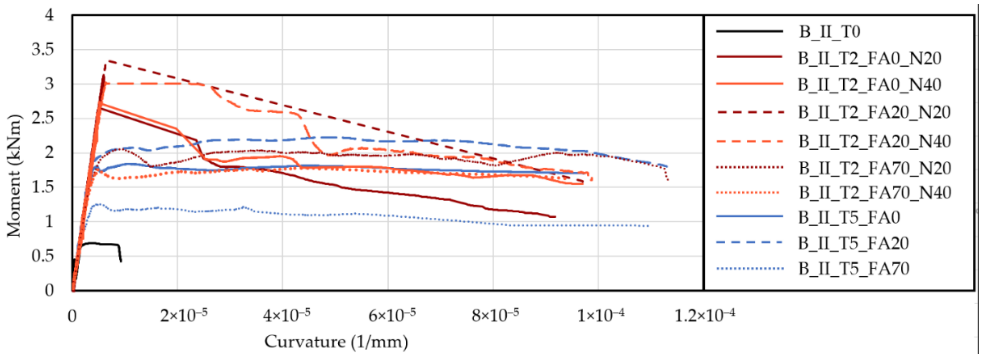

Fig. 32: Bending moment–curvature relationship measured in Test Campaign 1.

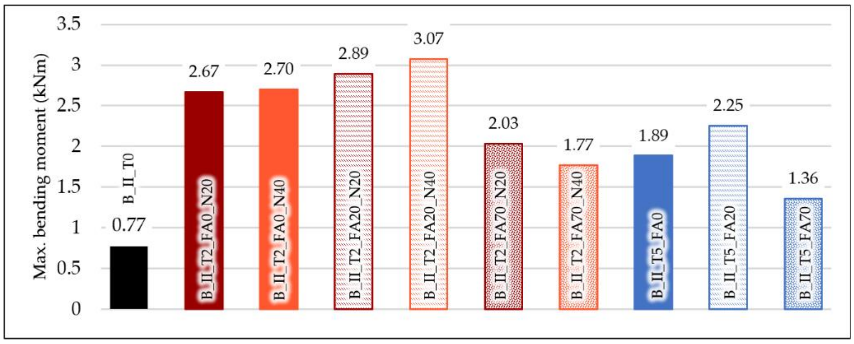

Fig. 33: Average values of the peak bending moment measured in Test Campaign 1.

Compared to the un-reinforced mortar beam (B_II_T0), a substantial increment of strength and toughness are observed when UHP-FRCC layers are used. However, the maximum bending moment of the beams B_II_T2 is higher than that of B_II_T5, regardless of the class of UHP-FRCC. Three collapse modes have been identified by linking the stress–strain relationship of UHP-FRCC (obtained from the tensile tests performed on dumbbell shaped specimens) with the moment curvature–relationship (Fig. 34):

- Failure in the tensile zone is illustrated in the beams B_II_T2_FA70 and B_II_T5 (Fig. 34a). The value of the strain $\varepsilon_{D,y}$ corresponding to the first cracking of the UHP-FRCC substantially coincides with that measured on the bottom of the beam by [SG-5] (see Fig. 30a) when first crack occurs. Afterwards, strain hardening appears both in the stress–strain relationship of the reinforcing layer and in the moment curvature relationship. At the peak of bending moment $M_{B,u}$, the strain gauge [SG-5] measured a value ε_5 equal to $\varepsilon_{D,u}$, which is the strain at the tensile strength $\sigma_{D,u}$ of UHP-FRCC. In other words, the ductile behaviour of these beams strictly depends on the mechanical performances of the precast layer.

- Fig. 34b illustrates the failure due to the crushing of mortar in the compressed zone of the beam. Indeed, during the strain hardening behaviour of the UHP-FRCC layer, the moment–curvature relationship shows a softening branch. The resisting area in compression reduces due to the crushing, whereas in the precast layer, wide cracks are visible. In this case, the bending moment corresponding to the first crack, $M_{B,y}$, coincides with that at the peak $M_{B,u}$. This brittle behaviour, which generally occurs in over-reinforced concrete beams, can be observed in the beams B_II_T2_FA0 and B_II_T2_FA20_N40.
- Shear failure (Fig. 34c) with a sudden drop in resistance. This brittle behaviour is evident in the moment curvature diagram of the beams B_II_T2_FA20_N20, where a diagonal crack appears without crossing the strain gauges of the constant moment zone. As strain localizes in this crack, a reduction of the strain is measured by the gauges before reaching the cracking stress (and strain) in the reinforcing UHP-FRCC layer.

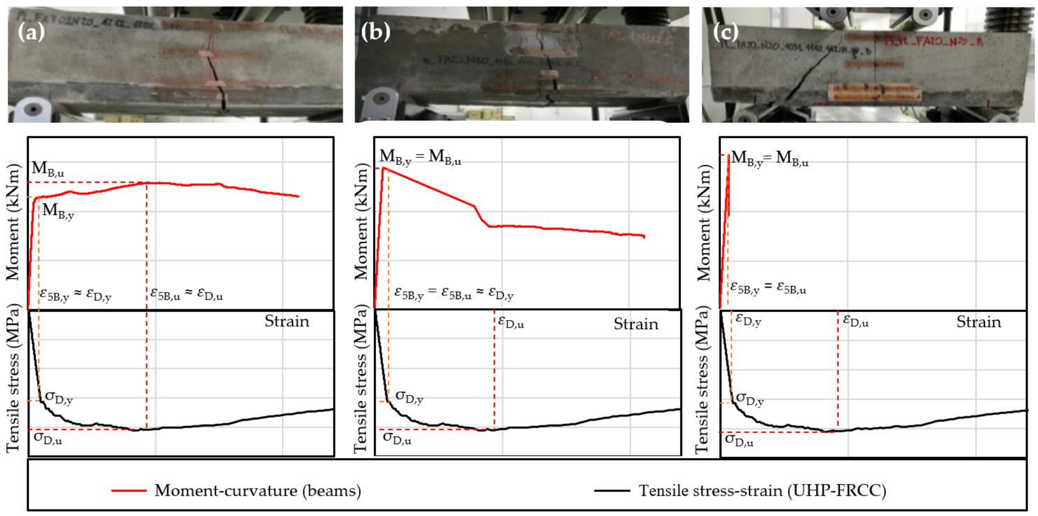

Fig. 34: Failures of the beams in the test campaign 1: (**a**) failure in tension; (**b**) crushing in the compression zone; (**c**) shear failure.

Accordingly, the use of high-strength precast layers (such as FA0 and FA20) leads to the brittle behaviour of the composite beams, either due to the crushing of mortar in the compression zone, or to the shear failure, especially in the presence of height bond strength (i.e., N20 series). Concerning the content of fly ash, low percentages of this industrial waste (i.e., FA20) produce an increment of strength in the UHP-FRCC layer, but a brittle behaviour of the composite beam. On the contrary, the load-bearing capacity of the layer significantly reduces if the substitution rate of cement as fly ash increases (i.e., FA70), but the composite beams show a greater ductility.

To check the effectiveness of the three different types of bond between the reinforcing layer and the existing structure (see Fig. 30), the strain profiles of the composite beams B_II_T2_N20, B_II_T2_N40, and B_II_T5 are illustrated in Fig. 35.

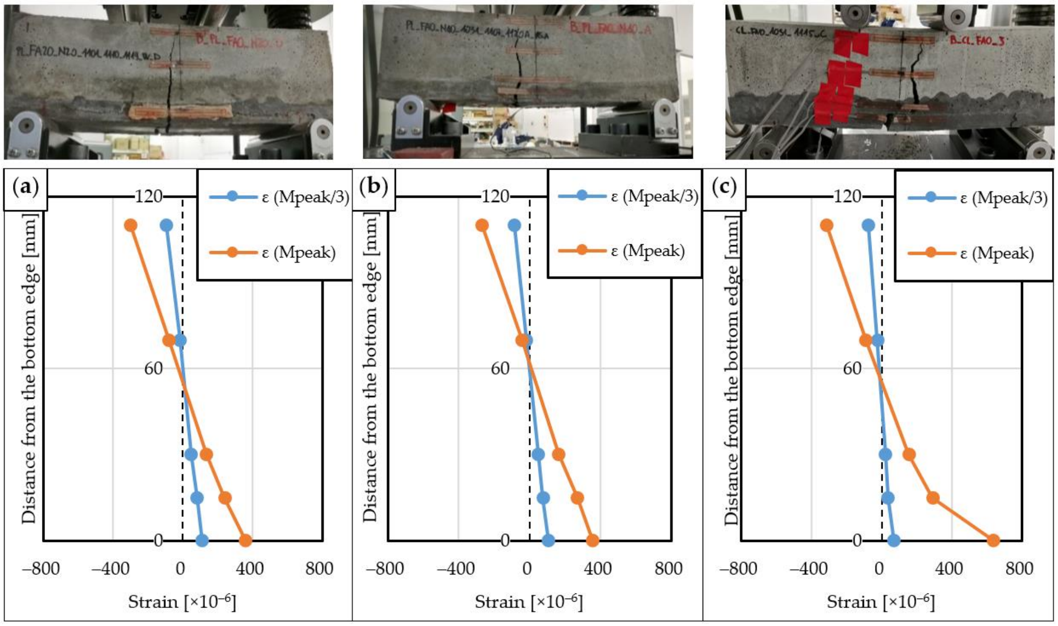

Fig. 35: Strain profiles measured in the beams: (**a**) B_II_T2_N20, (**b**) B_II_T2_N40, and (**c**) B_II_T5.

Fig. 35 shows that, except for the beam B_II_T5, plane sections remain plane up to the maximum bending moment. As a matter of fact, Fig. 35c shows delamination between the mortar and the strengthening layer, which is prevented by combining screws and nails on the surface of the precast UHP-FRCC layer (i.e., Type 2-N20 and Type 2-N40 in Fig. 30b,c).

4.3.4 Test Campaign 2

The second test campaign was carried out on larger scale specimens consisting of longer beams (length = 850 mm) having a cross-section with a high of 150 mm, a width of 125 mm thick and reinforced with longitudinal bars and stirrups (Fig. 36). As illustrated in Fig. 37, one of them was not strengthened (Fig. 37a), one was reinforced with a 3 cm thick layer of UHP-FRCC cast on the bottom face (Fig. 37b), whereas the rest of the beam were reinforced with precast strengthening panels (1 cm thickness as shown in Fig. 37c and d). The latter were fastened to the bottom face by means of dowels, screws and a 2 cm thick filling layer, following the same procedure as in Campaign 1 (Fig. 31). Among these, one panel has no nails (PLS), whereas the others have nails on the upper face spaced of 40 mm (PL N40). The panels with nails spaced of 20 mm were not made consistently with the results obtained in Campaign 1, which showed no significant difference between the Type 2 - N20 and Type 2 - N40 panels. The PLS panel was made with UHP-FRCC without fly ash (FA0), whereas three PL N40 panels were cast with 0%, 20%, and 70% of cement substituted fly ash, respectively (PL N40 FA0, FA20, FA70).

Fig. 38 shows the setup of the 4-point bending tests (performed by means of a 1000 kN Universal Testing Machine) and the arrangement of the measuring devices. Four strain gauges (90 mm long) were applied on lateral face and glued on the bottom face the specimen to measure the strain profiles of beam cross-section. On the other lateral face, two LVDTs (Linear Variable Differential Transducers) were fastened to measure the average curvature of the constant moment zone.

To measure the curvature, the following equation can be used:

$$\chi = (\varepsilon_2 - \varepsilon_1)/d_0 \tag{2}$$

where ε_2 and ε_1 are the strain measured by the upper and lower LVDT, respectively; and d_0 = 100 mm is the distance between the two displacement devices.

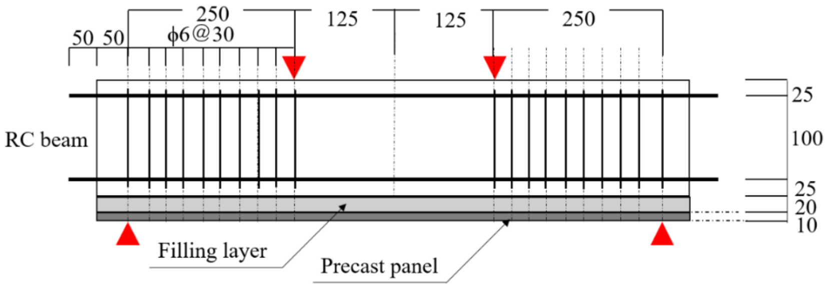

Fig. 36: Arrangement of longitudinal reinforcement and stirrups on the beams reinforced with precast panels.

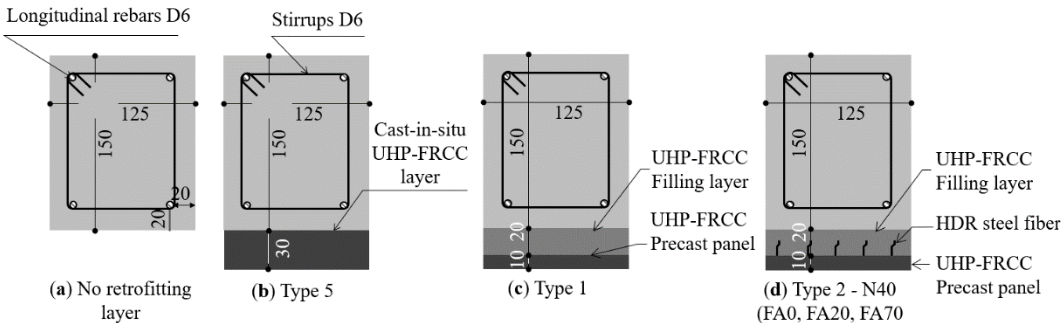

Fig. 37: Specimen tested in Campaign 2: (**a**) RC beam without the retrofitting layer; (**b**) RC beam with a strengthening layer cast-in-situ on the lower surface of beam; (**c**) RC beam strengthened with precast panel without HDR fibres on the upper face, fastened to the beam by means of screws, dowels and filler layer; (**d**) RC beam retrofitted with a precast panel with HDR fibres on the upper face (spaced 40 mm), fastened to the beam by means of screws, dowels and filling layer.

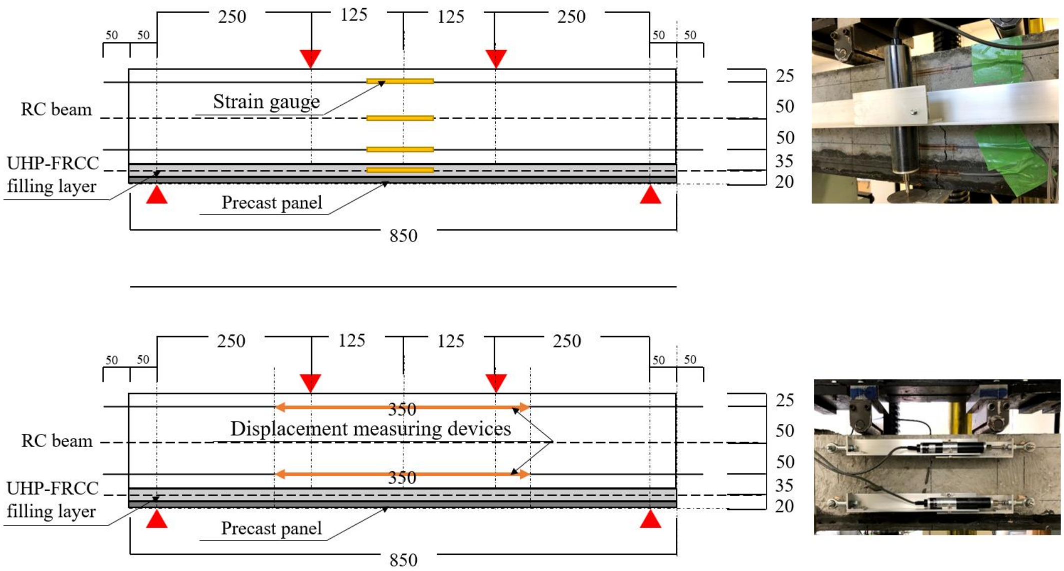

Fig. 38: Position of strain gauges and LVDTs on the beam.

Table 11 summarizes the composite beams investigated in the Campaign 2. In the second experimental campaign only one sample was made for each type of strengthening layer, because the complexity of assembling the steel reinforcement and casting the larger volume of cement did not allow for the production of several specimens, as in the case of smaller samples. On the other hand, the aim of this second part of the research was to investigate the scale effects and the impact of rebar on the behaviour of the strengthened beams. Indeed, the performance of the panels and the influence of the bonding conditions on the overall strength of the retrofitted beam had already been extensively explored in the tests of Campaign 1.

Fig. 39 shows the moment–curvature relationships measured in all the beams, whereas Fig. 40 depicts only the first part of the curves illustrated in Fig. 39. In the curves of Fig. 40, it is possible to observe the points of first cracking, i.e. the points where the moment-curvature diagrams turn to the non-linear behaviour. Moreover, the histogram in Fig. 41 compares the values of bending moment corresponding to first crack (M_{crack}) and to the ultimate load (M_{peak}).

Table 11: Details of the specimens tested in the experimental Campaign 2.

Symbol	**Layer Type**	**Bonding Surface**	**UHP-FRCC Series**	**Number of Samples**
RC	None	[-]	[-]	1
CL_FA0	Type 5	Chipping only	FA0	1
PLS_FA0	Type 1	Chipping only	FA0	1
PL_N40_FA0	Type 2	SF – d = 40 mm(1)	FA0	1
PL_N40_FA20	Type 2	SF – d = 40 mm(1)	FA20	1
PL_N40_FA70	Type 2	SF – d = 40 mm(1)	FA70	1

(1) Steel fibers spaced 40 mm.

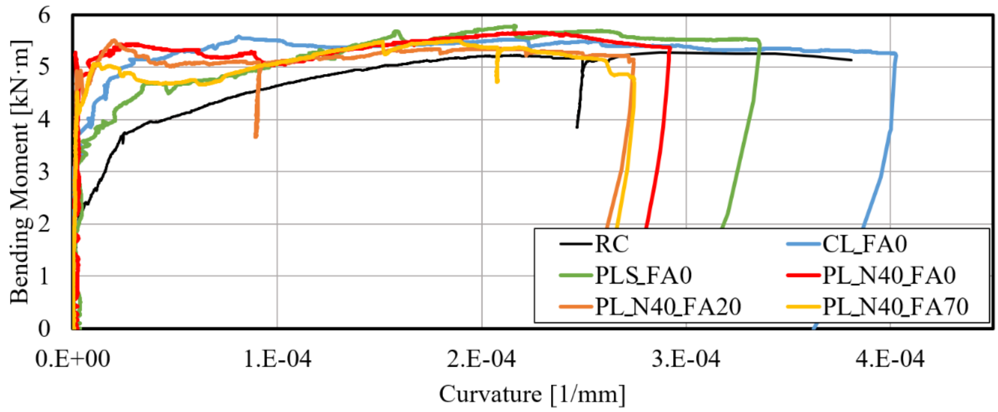

Fig. 39: Moment–curvature relationship measured in the constant moment zone of the beams tested in the experimental campaign 2

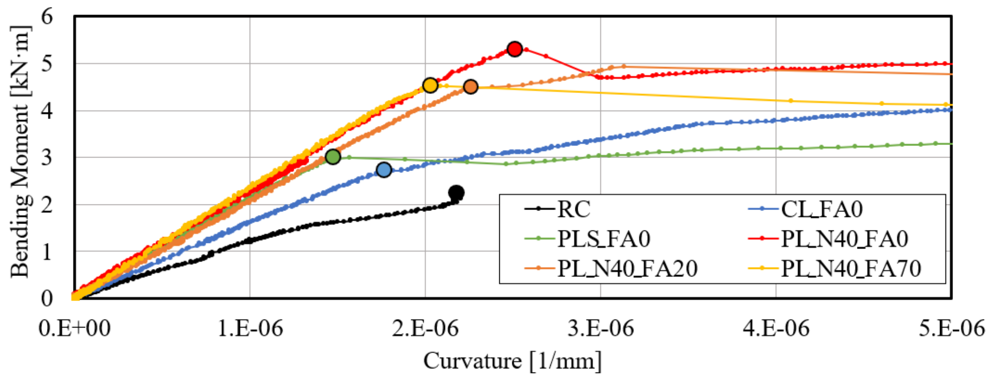

Fig. 40: The first cracking in the beams tested in the Campaign 2.

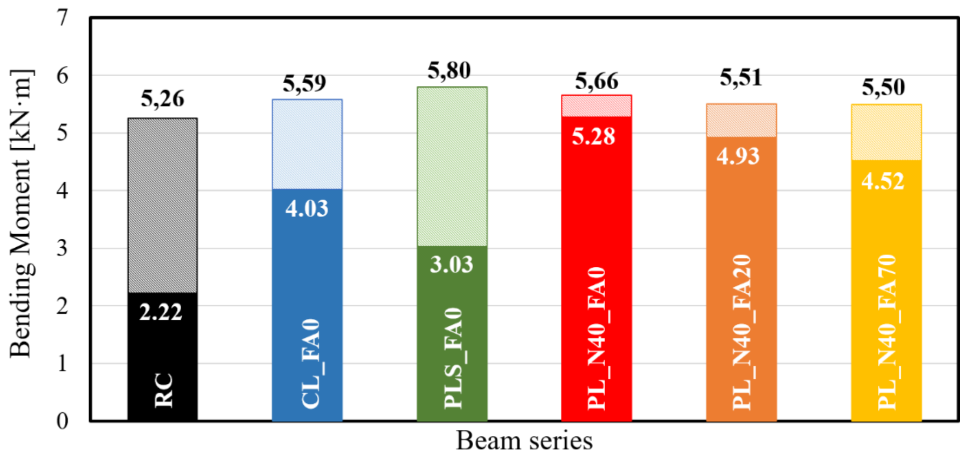

Fig. 41: The first cracking (M_{crack}) and the peak (M_{peak}) of bending moment in the beams tested in the Campaign 2.

Compared to the RC beam, an increase in the bending moment capacity between 4.6% (PL_N40_FA70) and 10.2 % (PLS_FA0) is achieved when a retrofitting layer is used. Among the panels representing the proposed method, PL_N40_FA0 provides the best performances, which is greater than the cast-in-situ layer of the same material without steam curing (CL_FA0). However, the main contribution provided by the precast panels consists of increasing the value of the bending moment that triggers the first crack (M_{crack}). In fact, all beams with N40-series panels show a M_{crack} which is more or less twice that of the beam without any strengthening layer, whereas there is an increase of about 80% and 40% in beams retrofitted with the cast-in-situ layer (CL_FA0) and the precast layer without nails (PLS_FA0), respectively. The contribution provided by the nailed precast panels PL_N40 is interesting because it prevents damage of the structure, because in beams without any reinforcement, or reinforced with a CL or PLS panel, the first crack occurs for lower loads. Furthermore, since strain hardening behaviour is observed after the formation of the first crack, the ductile behaviour of the beam is evident. The sample showing a brittle behaviour is PL_N40_FA20. Indeed, it shows a softening branch due to the loss of strength after the first crack, which is only partially gained back as the strain increases. Nevertheless, all beams collapse in type (a) mode (see Fig. 34), as neither shear failure nor the crushing of the concrete in the compressed zone occur. It can be stated that these brittle collapses are avoided because steel rebar have been implemented into the beam and the reinforcement ratio of UHP-FRCC is smaller than that of in the specimens used in Campaign 1.

To assess the effectiveness of the bonding conditions between the reinforcement layers and the NC beam, the strain profiles in the beams cross-section have been also plotted in this second experimental campaign. Fig. 42 shows the strains measured at the peak of moment (M_{peak}) and at 1/3 of this value ($M_{peak}/3$), as well as the theoretical linear trend of perfect plane sections for $M=M_{peak}$.

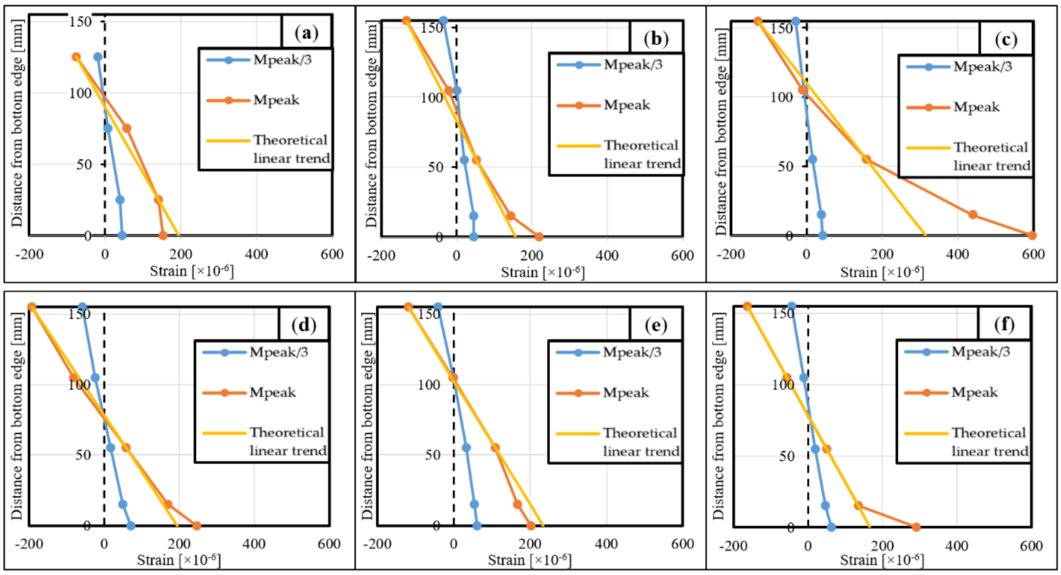

Fig. 42: Strain profiles measured in the beams of Campaign 2: (**a**) RC; (**b**) CL_FA0; (**c**) PLS_FA0; (**d**) PL_N40_FA0; (**e**) PL_N40_FA20; (**f**) PL_N40_FA70.

Fig. 42 shows that, when $M_{peak}/3$ is considered, the strain profiles in the mid-section follow a linear trend in all cases. When the strain measurement is made at the peak moment (M_{peak}), non-linear trends are observed in specimens PLS_FA0 (Fig. 42c) and PL_FA70 (Fig. 42f). In the first sample, a detachment of the panel from the existing beam was observed as depicted in Fig. 43a, because there are no nails and screws with plugs. In the case of beam PL_FA70, no delamination occurred between the precast panel, the filler and the existing beam (Fig. 43b). Therefore, the misalignment of the strain measured by the lower strain gauge is probably due to the breakage or early detachment of the lower strain gauge.

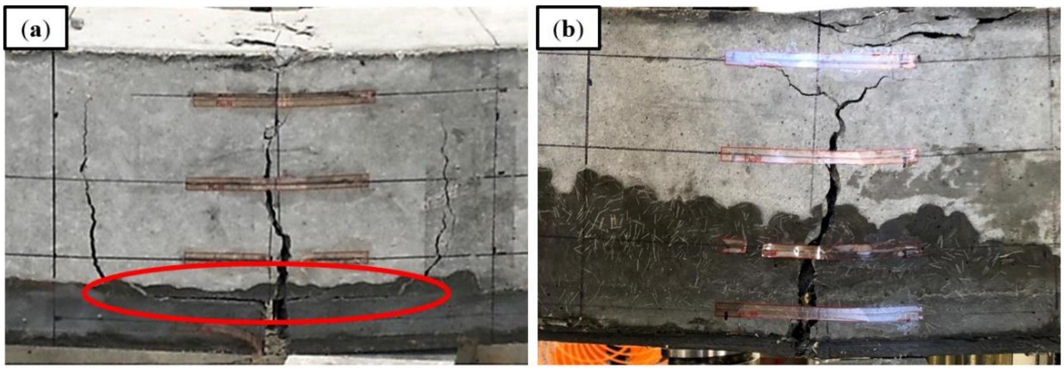

Fig. 43: (**a**) detachment of the precast layer from the existing beam of the PLS_FA0 specimen; (**b**) integrity retention between the beam and strengthening panel after beam failure of the PL_N40_FA70 specimen.

4.3.5 Eco-Mechanical Analysis

A further study regarding both the Test Campaign 1 and 2 is herein performed by evaluating the environmental performances of the beams, through the so-called eco-mechanical analysis [139]. Using the non-dimensional diagram of Fig. 44, a comparative analysis among the beams is carried out in order to select the best reinforcing system, which contemporarily satisfies the environmental and mechanical performances.

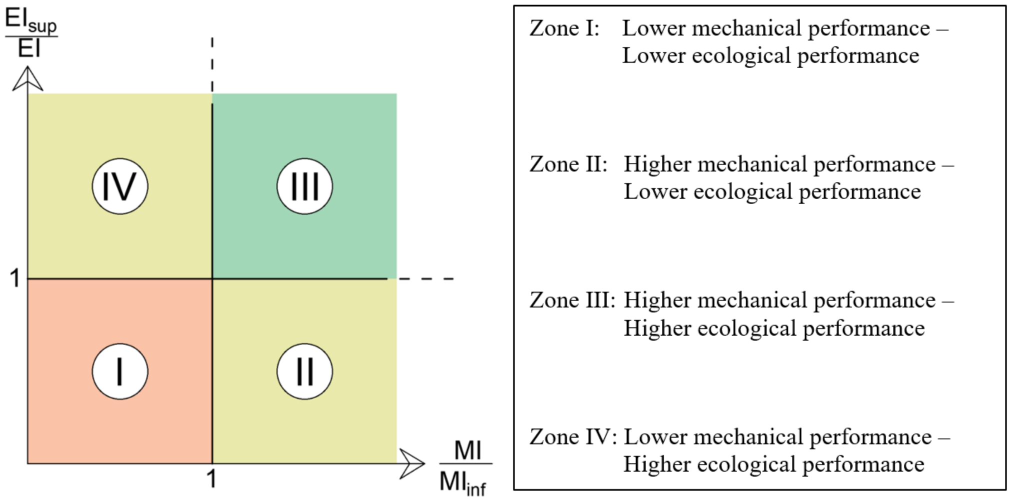

Fig. 44: non-dimensional diagram for evaluating the eco-mechanical performance of concrete manufacts.

The horizontal axis of Fig. 44 reports the ratio between mechanical indexes (MI/MI_{inf}), whereas the vertical axis represents the ratio between ecological indexes (coc). Specifically, MI_{inf} is the lower bound value of mechanical performance, which is the so-called functional unit. In this study, MI_{inf} corresponds to the mechanical performance of the reference series. Similarly, EI_{sup} is the upper bound value of the environmental performance, which corresponds to that of the layers of the reference series. In particular, the environmental impact was computed by multiplying the amount of materials used for each type of layer by the relevant unit carbon footprint, as given by the inventory data issued by the *Japanese Concrete Institute* (JCI) [140]. This computation is consistent with *fib* [141], where only the CO_2 released in the atmosphere is taken into account. The mechanical index, or the functional unit, could be the maximum bending moment of the moment–curvature relationship [142]. On the other hand, according to fib [141], the mechanical index should also consider the overall behaviour of the structure, including the ductility. Therefore, two different parameters are considered herein. The first parameter is the peak of bending moment, whereas the second parameter (i.e., the ductility) is correlated to the work of deformation per unit length (J/m). It is the area D defined by the moment - curvature diagram up to the maximum bending moment, as illustrated in Fig. 45. Accordingly, it vanishes in the case of brittle behaviour (see Fig. 34b,c).

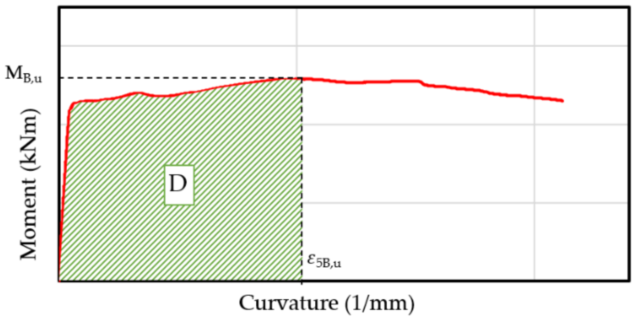

Fig. 45: evaluation of the mechanical indexes.

To calculate MI_{inf} and EI_{sup}, the mechanical and ecological performances of the B_II_T5_FA0 and CL_FA0 samples are considered as the references of Campaign 1 and Campaign 2 respectively. Indeed, they represent the current method of retrofitting the existing RC beams. In these beams, UHP-FRCC does not include any supplementary cementitious materials to reduce the carbon footprint. Table 12 summarizes the values of the parameters of each retrofitting layer used to reinforce the beams of Campaign 1. Similarly, Table 13 refers to the values evaluated in the Campaign 2. The embodied CO_2 computed for Type 2 series of the precast panels also takes into account nails, screws, and the filling layers made with UHP-FRCC without fly ash (FA0). As a result, the environmental indicators are fairly high, even for the Type 2 - FA20 and FA70 series. An additional reduction in CO_2 emission for the FA20 and FA70 series could be achieved by replacing the fly ash also in the filling layer. The results of the Eco-Mechanical analyses are plotted in Fig. 46 and Fig. 47, which refer to the first Test Campaign and second Test Campaign, respectively. In particular, the maximum bending moment (M_{peak}) is the functional unit in Fig. 46a and Fig. 47a, whereas the work of deformation (D) is the functional unit in Fig. 46b and Fig. 47b.

Table 12: Parameters used to compute the environmental and mechanical indexes for the Test Campaign 1.

Parameter	B_II_T2_FA0_		B_II_T2_FA20_		B_II_T2_FA70_		B_II_T5_		
	N20	N40	N20	N40	N20	N40	FA0	FA20	FA70
M_{peak} (KNm)	2.67	2.70	2.89	3.07	2.03	1.77	1.89	2.25	1.36
D.Work (J/m)	0.0	0.0	0.0	0.0	58.9	45.2	125.3	70.2	22.5
CO_2 (kg)	0.9719	0.9634	0.8882	0.8797	0.7045	0.6960	0.9679	0.8006	0.4331

Table 13: Parameters used to compute the environmental and mechanical indexes for the Test Campaign 2.

Parameter	CL_FA0	PLS_FA0	PL_N40_		
			FA0	FA20	FA70
M_{peak} (KNm)	5.59	5.8	5.66	5.51	5.50
D.Work (J/m)	403.2	1075.9	1212.5	97.54	952.3
CO_2 (kg)	9.1344	8.3417	8.3443	7.8333	6.6822

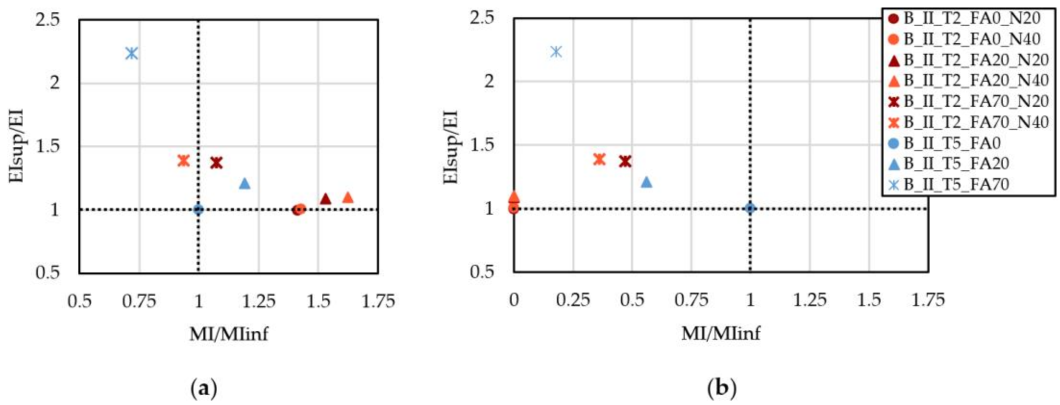

Fig. 46: mechanical and environmental assessment of the precast UHP-FRCC layers of the Campaign 1 by considering (**a**) MI = maximum bending moment and (**b**) MI = work of deformation.

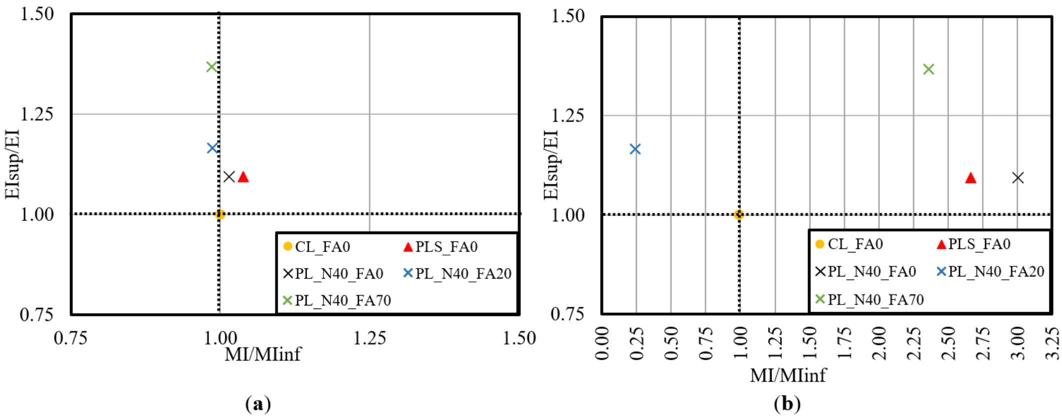

Fig. 47: mechanical and environmental assessment of the precast UHP-FRCC layers of the Campaign 2 by considering (**a**) MI = maximum bending moment and (**b**) MI = work of deformation.

With respect to the Test Campaign 1, Fig. 46a points out the best result, in terms of mechanical performances, achieved by the beam B_II_T2_FA20. On the other hand, B_II_T2_FA70 attains a fair reduction in emissions by preserving, approximately, the same resistance of the beam B_II_T5_FA0. When the work of deformation per unit length is the functional unit (see Fig. 46b), the beams do not show any increment of ductility with respect to the reference beam B_II_T5_FA0. In particular, the beams B_II_T2_FA0_N40 and B_II_T2_FA20_N40 exhibited brittle failures, as shown in Fig. 34b,c. Hence, their mechanical index MI is zero in Fig. 46b. As this brittle behaviour generally affects the over reinforced beams under bending actions, it could be ascribed to the large thickness of the reinforcing UHP-FRCC layer.

As far as the beams of Campaign 2 are concerned (see Fig. 39 and Fig. 41) the precast panels allow reaching approximately the same (or a slightly lower) bending strength as the beam strengthened with Type 5 layer. Therefore, the points identified in Fig. 47a tend to MI/MI_{inf} = 1 in all the cases when M_{peak} is used as the functional unit. Conversely, the values obtained by considering the work of deformation as a functional unit are significantly more spread, pointing out the most effective strengthening method, both from the mechanical and the environmental point of view. The only specimen that falls into Zone IV (according to Fig. 44) is PL_N40_FA20, although further tests are needed to establish whether this result was due to a fault in the specific specimen, rather than a weakness of the panel. All other specimens fall into Zone III, showing both mechanical and environmental improvements with respect to the traditional way of strengthening. The specimen that exhibits the most ductile behaviour is the

PL_N40_FA0, thus the best benefits are provided by Type 2 panel, with 70% cement replaced by fly ash (PL_N40_FA70). More precisely, it provides the lowest carbon footprint, with a 20% reduction in CO_2 emitted, and an increment of the deformation work (almost 2.4 times) compared to the reference specimen (CL_FA0).

4.3.6 Main results

The experimental results previously described lead to the following conclusions:

- the precast panels increase the bending moment capacity of the beams compared to that achieved with the current consolidation method, which consists of a cast-in-situ strengthening layer;
- in RC beams the panels mainly enhance the moment of the first crack, which provides advantages against seismic action because it increases the ductility of the beam and reduces the probability of damage;
- implementing nails embedded in the upper face of precast panels prevents their detachment from the existing beam;
- in mortar beams without reinforcement, over-strengthened retrofitting panels compared to the existing beam lead to brittle collapse. In contrast, no brittle collapse was observed in RC beams;
- in general, the best performing panels are FA0 and FA20, whereas there are no significant differences between the N20 and N40 series. FA70 panels significantly reduce emissions while providing good mechanical performances. Therefore, precast panel sizes and UHP-FRCC mix design should be tailored according to desired mechanical and environmental performances.

4.3.7 Cost and time of production and implementation in real cases

The results obtained in the two experimental campaigns are promising. In fact, it has been proven that, with a robust adhesion between the strengthening panel and the existing beam, the same or higher performances than that obtained with cast-in situ layers can be obtained even by substituting cement with fly ash for reducing the carbon footprint. Once the further research outlined in the previous paragraph has been addressed, it will be necessary to adapt the panels to the specific case they are applied in. For instance, the models could enable one of the parameters of panel thickness, embodied CO_2 and strength of the reinforced beam to be set, returning the other parameters according to design requirements. Moreover, when a significant increase in the beam's resistant capacity is required, the proposed method could be improved by providing additional reinforcements which could be leaned on the strengthening panel hanging from the existing beam to be subsequently incorporated with the filling layer. Hence, the design tools will have to evaluate the thickness of the precast panels and the amount of new rebar to be implemented to increase the ductility of the beam, thereby improving the structural performances also with respect to the seismic action. In the case of beams protruding from the slab, U-shaped retrofitting panels could be produced to cover the side faces of the beam, also increasing the beam's shear strength and ductility. Conversely, as already illustrated in the case of the current retrofitting method in Section 4.2.2, the application of panels in the side faces of beams hidden in the slab is not feasible, unless the slab is demolished.

For what concerns the cost and time for retrofitting the beams using the proposed method, they are related to:

- **the production of the precast panels**: the cost items that form the total panel price include the raw material and the production cost (energy + labour costs). While the cost of raw materials could be easily obtained from market prices, the production cost is difficult to predict. Indeed, it depends on the production method used by the producer (e.g. single and tailored-size panels rather than long panels to be cut on the construction side), the number of workers involved in the production site and their qualifications (ordinary or specialised workers), the local energy cost where the production site is located in. According to the time required for the casting and curing procedure to produce the panels used in the two test campaigns, at least 30 days can be estimated for the production of a panel. This period includes the mixing and casting of cement blend and steam curing (2-3 days) and normal curing (28 days). The production time may vary as the technology and production methods change.
- **Implementation at the construction site**: As well as for the cost of the proposed technology, computing the time of implementation is not trivial. Cooperation should be established with construction companies and producers to determine the optimum method of implementation. The total cost could be the sum of the cost of transportation + cost of demolition of external and internal walls below the beams + cost of demolition of any suspended ceiling + cost of rubble disposal + cost of application of the panel + cost of filling layer + cost due to walls, ceiling and system to be restored + labour costs.

The cost of transportation, depending on the distance between the construction site and the producer, could be easily evaluated. Demolitions, rubble disposal and restoration costs could be lower than that of other retrofitting systems currently used, as a smaller surface of walls, ceilings and systems could be expected to be involved in demolition and restoration because of the low-invasive method proposed. The cost of all the procedures to implement the proposed technique, as well as the number and qualification of labour, cannot be estimated because the method has never been tested on a real case. However, it is worth to be noted that the cost of formwork is avoided if compared with the current method of increasing the cross-section sizes of the beam with cast-in-situ concrete. Accordingly, also the time required for implementation and restoration cannot be estimated until the method of applying the panels to the beams on site is investigated. Nevertheless, taking into account the expected small volume of building components to be demolished and restored and the time required for the injection and curing of the filling layer (28 days, as already experienced during the test campaign) as well as the avoided formworks, a total implementation time of between 40 and 50 days can be assumed.

4.4 Consolidation method for the non-structural elements

There are several school buildings where the flaws of the non-structural components described in Section 4.1.2 occurred. This is due to the fact that until the code rules were issued, structural elements were often neither standardised nor subject to inspection. For instance, a specific section dedicated to non-structural elements in the ARES form (Section. 3.2.1) was only introduced in 2009, which followed the collapse of the suspended ceiling of the Darwin High School in Rivoli. As a result, standards have started to address this issue only recently, thus still needs to be fine-tuned and improved, although significant steps forward have already been observed from NTC 2008 to NTC 2018. In the following sections, the approach adopted by the rules is briefly addressed and its shortcomings assessed. Furthermore, taking into account the FEMA's practical guide, the methods to be adopted in the inspection of non-structural elements, in part already adopted by the MIUR and the Italian standards, are reviewed. Lastly, best practices to provide robustness and safety for new and existing non-structural elements are proposed.

4.4.1 Approach and limitations of the current standards

Regarding Italian school buildings, the definitions and requirements on non-structural elements are provided in Section 7.2.3 of the 2018 Technical Standards for Construction (NTC 2018) [70]. These standards identify two types of non-structural elements:

- elements with stiffness, strength and mass that affect the structural response (such as external walls, internal brick partitions, systems and heavy plant parts);
- elements that do not influence the structural response, but are relevant to the safety of people (such as plant terminals, chandeliers, cornices, decorations, cabinets and bookcases).

The structural rules consists of carrying out checks on the stability of the systems and other non-structural elements with the aim of calculating the strength demand. In this regard, the following distinction is made:

- built-on-site element (e.g. brick partition): the structural designer, besides calculating the resistance demand, must design the element to fulfil such demand, whilst the planning supervisor must verify the correct execution of the work to meet the designer's instructions.
- assembled-on-site element (e.g. plasterboard partition): the performance demands calculated by a structural engineer are submitted to the producer, who must supply elements and connection systems to comply with them. Ensuring the correct assembly of the system is a responsibility of the planning supervisor.

Compared to the previous standards, the NTC 2018 has moved from a purely prescriptive to a more performance-based approach. For instance, section 7.3.6 sets the limits for the displacement of structural elements and inter-storey drift in order to mitigate damage of infill walls and avoid their expulsion. However, there are some limitations. In fact, a displacement that may be acceptable for a brick wall, might not be for a door or window within it. As a matter of fact, a strain of few millimetres can produce either the failure of glass panels in a window, or a window frame overturning due to damage to the fastening system, or a door jamming. With respect to the evaluation of the strength demand for the non-structural element, the NTC 2018 provides a calculation method which allows for the assessment of the horizontal force to be applied to the element. It is proportional to the weight of the element, the maximum acceleration induced into the element during the earthquake and the seismic behaviour factor of the element. In this regard, paragraph 7.2.3 of the "*Circolare applicativa*" [143] provides a detailed procedure to evaluate the acceleration induce into the element, as a function of the floor response spectrum, the ratio between the period of oscillation of the element and that of the structure, and the damping of the element. Table C7.2.I provides the values of the seismic behaviour factor in a set of non-structural elements, generally used school in buildings.

Nevertheless, attention is not devoted to the performance of fastening and joining systems. This is in contrast with the requirements for load-bearing structures, for which calculation and checking methods are also provided for anchorages and connections. Depending on the type and weight of the non-structural element, there are three potential scenarios:

- anchoring of devices and systems with low weight, not excessively flexible, i.e. having stable behaviour during the earthquake. A design is not required for these devices, because the problem choosing the appropriate and safe method of fastening is easily solved when an adequate experience in the field is achieved;
- anchoring of plant systems, prefabricated or assembled-on-site devices with significant weight or prone to oscillations. In this case, the producer is responsible for providing certified fastening systems and materials, which the worker must comply with;
- anchoring of components, elements and devices with significant weight and prone to oscillations without specific standards or certificates, such as anti-overturning measures for brick partitions and infill walls suggested by the standards. In these cases horizontal steel reinforcements to be inserted inside mortar joints and light meshes embedded within the plaster can be used. However, no specifications depending on the supporting structure (e.g. concrete structure, steel profile, brick wall, etc.) are provided on the types and performances of the fastening systems of these reinforcement and meshes [144].

A solution to the lack of the standard, as highlighted in the last point, could be solved by referring to a catalogue of anchoring systems, as in depends on both the resistance demand required by the standards and the support structure. For instance, threaded rods could be prescribed for anchorages to the adjacent RC structures, dowels with resins for anchorages on bricks, and finally welded or bolted anchorages on steel profiles. An issue to be taken into account is also the loss of efficacy of the fastening systems, as a result of damage of the supporting component. For instance, consider a cracked beam or a wall on which a non-structural element is fixed in order to resist to seismic action, as well as prolonged static loads. Such cracking might be negligible from a structural standpoint (fall within the limits provided by NTC 2018), but it could be significant in terms of the anchorage's resistance capacity. For this purpose, the type and arrangement of anchorages, as well as the procedures adopted for their setting, should not represent weak points or preferred crack triggering path of the supporting element.

However, the requirements of Chapter 7.3.6 of NTC 2018 refer to both new and existing buildings when new non-structural elements or systems are installed. On the other hand, there are no requirements or mandatory measures for retrofitting or replacing non-structural elements in existing buildings. Nowadays, the assessment of risks related to the collapse or malfunctioning of non-structural elements in existing schools is entrusted to school managers who, supported by the Risk Prevention and Protection Service, draw up the Risk Assessment Document, according to the Safety Act (Legislative Decree no. 81/2008 as amended by Law no. 215/2021). If potential risks are identified, the school manager reports them to the school owner, which assesses the problem. Only recommendations are listed in Tables C8.7.6.3.I and C8.7.6.3.II of the "*Circolare applicativa*". The first table lists systems, furniture and building components of which estimates a scale of vulnerability and importance, as well as the level of cost for each of them, and suggests also the seismic zones where the evaluation of their retrofitting and the substitution of their fastening devices are recommended. The second table provides solutions to mitigate the risk of gas leaks under seismic actions, such as different types of safety valves and methane gas sensors, detailing the relevant characteristics, benefits, drawbacks and installation requirements. These indications, limited to gas transport systems, which are one of the greatest risk factors when damaged, should also be extended to other types of systems. For instance, a massive water leakage due to a broken plumbing system can cause obstruction of escape routes and functional loss of the building.

Therefore, despite the growing attention devoted to non-structural elements in recent years, the standards still need to be improved, as some aspects are detailed, whereas others are not adequately addressed.

4.4.2 Surveys and checklists on non-structural elements

As mentioned above, a specific section on non-structural elements was included in the ARES sheet when they were updated following the collapse of the suspended ceiling at the Darwin High School. This section is composed of sub-sections corresponding to typical non-structural elements, i.e. suspended ceilings and elements hanging from slabs, vertical and horizontal overhangs, partitions and infill walls, cladding and windows, furniture and equipment, and other systems. The technicians entrusted with filling in the forms has to indicate whether any issues is observed on these elements specifying the severity and suggesting the safety measures to be adopted, alongside the relevant cost. This monitoring approach is effective when carried out on a regular basis, with accurate inspections by specialised staff, and when data collecting and processing is done using smart systems. However, survey programs often are carried out only after negative events or injuries to persons. As a consequence, these surveys could be hasty and inaccurate, with inconsistent judging methods. Besides, data are stored by using managing tools without easy and reliable data processing [145]. FEMA offers useful guidance on how to conduct effective surveys. In particular, it recommends two levels of inspection [106]:

- the first level is carried out by the manager and the school's safety supervisors, who must follow a special training course, using a standardised checklist. The checklist must be accompanied by a floor plan of the building. To each non-structural element must be assigned an ID, indicating its location, consistency (number, area, volume), the degree of risk assessed (both in terms of safety for life and functional loss), and any notes. Surveys carried out by school staff have the advantage of being performed frequently. In other words, damage to both structural and non-structural elements can be identified as soon as it occurs, as well as improperly performed interventions, such as a new shelf that are not fastened or a new door being defective. Depending on the type and risk level either the appropriate measures are adopted or a new inspection level is carried out;
- The second level of survey is carried out by engineers upon request of the school administration (or owner). In this regard, specific procedures are described in ASCE 31/SEI 31-03 and in Chapter 11 of ASCE/SEI 41-06 [146, 147].

Beyond the sheet format and the survey methods to be applied, establishing a standardised method of collecting data using an advanced tool enabling the "one-click" extraction of information and statistics is crucial. BIM and GIS could be examples of information management tools that, alongside those specifically tailored to school buildings [148], allow to:

- have and update detailed information on each building component, including a technical data sheet and maintenance schedule, thereby triggering an alert when an inspection is required;
- query the system punctually, e.g. by clicking on a classroom to obtain a list of non-structural components or identify elements with problems;
- extrapolating general statistics, such as the cost to the owner of several school buildings for renewing a certain type of component (to be used in calls for tenders), or maps identifying schools requiring intervention (useful for planning actions).

4.4.3 Best practice for safety upgrading

As well as for structural elements, substituting or retrofitting the non-structural elements must be in accordance with the seismic risk of the area where the school is located. Furthermore, they must be homogeneous and consistent each other. As an example, it is uncomfortable to know that suspended ceiling is well anchored when heavy steel pipes are only supported by ceiling, or that a shelf is well fastened to a wall prone to overturning. Referring to non-structural components listed in Section 4.1.2, solutions for the risk mitigation are outlined below.

Dropped ceiling: As the suspended ceiling must be lightweight, all heavy ceilings (in Italy, the "*Perret*" type ceilings with brick tiles described in Section 4.1.2 were frequently used) must be removed and replaced by lighter substructure and tiles. Plasterboard ceilings and aluminium substructures are the most common. Waterproof tiles should be used to avoid absorbing water or moisture that would increase their weight and cause damage. Furthermore, the sub-structure should preferably be made of aluminium or a decay-resistant material (e.g. avoiding ordinary steel hangers prone to corrosion). The fastening must be certified and suitable according to the support (e.g. anchoring to a hollow brick must be stronger than anchoring to a RC beam, because the hollow brick is susceptible to detachment). The substructure must be statically indeterminate, i.e. the failure of one or more hangers must not lead to the collapse of the whole ceiling. In addition, the ceiling must be easy to inspect.

External walls and partitions: there are two approaches for mitigating the risk arising from the overturning, or the collapse, of infill walls and partitions:

- using lightweight systems, such as partitions and interior claddings with plasterboard panels and an aluminium sub-structure. These systems are less prone to the inertial force caused by seismic shaking, as well as being more ductile. They can be quickly assembled and allow for greater flexibility of spaces, because they can be easily removed and moved. They also comply with the key circular economy concept of "*design-for-disassembling*", because they can be disassembled and reused elsewhere. The voids between the cladding panels allow for easy housing of systems and pipes, which are barely affected by warping or damage suffered by the wall during the seismic action;

- using the traditional brick walls, because of their greater mechanical and fire resistance performances, as well as their soundproofing properties. In these cases, anti-overturning measures must be implemented. These include the aforementioned horizontal steel bars to be placed in the mortar (Fig. 48a) and light mesh to be embedded within the plaster (Fig. 48b). Both the systems must be fastened to the surrounding structures every 500 mm. In addition to these methods (also suggested by the standards), there are other certified solutions, such as galvanised steel lattice girders or steel and fibreglass wire meshes to be embedded within mortar joints. These products generally are supplied by producers with certified fastening systems.

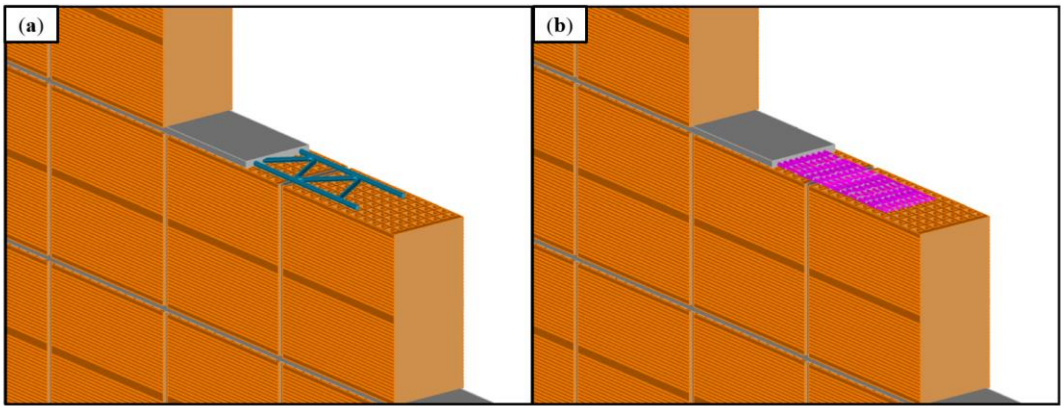

Fig. 48: Anti-overturning measures for external walls and partitions: (**a**) horizontal steel bars; (**b**) light mesh to be embedded within the plaster;

Doorway getting stuck: Door jamming is avoided by keeping the opening movement free from any warping of the door frame. Although this problem is still scarcely addressed in the literature, there are certified door systems designed to avoid jamming in the case of frame crushing [150]. With respect to window glass shattering, it is rather difficult to avoid it. Nevertheless, it is possible to prevent the glass from exploding and injuring people. Two possible solutions can be represented by the used of tempered or laminated glasses, i.e. the glasses used in car windshields, as well as proper containing films that hold the fragmented glass together.

Fig. 49: Emergency System for door opening in critical condition proposed by LF System Italia s.r.l.s. [150].

Cornices, decorative friezes and other protruding elements: All protruding elements that are damaged or not properly fastened must be removed, especially when they are not essential for the school's safety and function. When this is not possible, because they are elements that are distinctive of the building or have an historical value, it is necessary put them on the safe side, e.g. using an anti-fall net. Afterwards, it is necessary to carry out consolidation and repair measures according to the requirements of cultural heritage standards and the directives of any conservation authorities concerned.

Shelving, furniture, equipment and hanging elements or components: The approaches for reducing the risk of overturning or detachment of these elements are rather trivial. As already stated, they consist in fastening the components to supports suitable to withstand the dead load and the seismic inertial force. In the case of shelves and bookcases, devices and objects stored inside must also be properly restrained if they are heavy enough to injure people when dropping. For instance, in Japan it is mandatory to apply retaining bands on each level of shelves and bookcases as a safety measure.

Chapter 5

Sustainability of school buildings[4]

5.1 Introduction

As remarked in section 3.3.2, school building stock are dramatically energy-intensive, due to old buildings and low investment in energy-saving measures. Nevertheless, the sustainability issue affects the entire construction sector, as it requires huge volumes of raw materials and emits large amounts of greenhouse gases. Specifically, it accounts for 36% of global final energy use and 39% of energy-related carbon dioxide emissions, when upstream power generation is included [150]. Also, Construction and Demolition Waste (C&DW) is one of the most important waste streams, accounting for approximately 25% - 30% of all the waste generated in the EU [151]. To solve all these problems related to the construction and management of school buildings, some strategies are necessary. The demolition of old and energy-intensive buildings and the construction of new low-energy (or even passive) building, made with materials with low carbon footprint, is becoming mandatory. In compliance with this strategy, in recent years, concrete has been replaced by more eco-friendly structural materials, such as wood [152], although studies have proved that wooden structures show lower thermal performance than concrete, especially in the long term analyses [153, 154].

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[4] *Part of this chapter has been previously published in:*

O. Mancinelli, A. P. Fantilli and B. Chiaia, "Comparing the environmental performances of new and renovated school buildings" Acta Polytechnica CTU Proceedings, vol. 33, 350-356, 2022.

Nevertheless, the demolition of old constructions, combined with the construction of new buildings, leads to the production of waste and to the consumption of natural resources to produce new materials. On the contrary, rehabilitation, refurbishment and renovation of old buildings make stream waste and the use of new materials limited. Regrettably, in many cases, it is not always economically convenient to attain the same thermal and structural performances of a new building by adapting old building to the current code requirements. To provide guidance tools for policymakers and stakeholders, a large number of studies have been carried out, in which environmental performances of new buildings made of different materials are compared. As a result, in terms of Global Warming Potential (GWP), timber-framed residential buildings tend to have a lower environmental impact than those made with concrete and masonry [155].

Regarding school buildings, studies on structural materials and on envelope systems have shown that concrete and masonry buildings have better thermal performances, because of the heavyweight materials [156]. Although manufacture, construction and demolition of masonry and concrete buildings require larger energy and show higher global warming potential, these buildings exhibit lower annual energy consumption and environmental impact in service, which sometimes makes them more sustainable than those made with timber and steel [153]. Based on these studies on refurbishment and newly constructed buildings, it is still not possible to conclusively determine which of the two alternatives has the best environmental performance over the entire lifespan [157]. Besides, studies using the LCA methodology mainly focus on energy refurbishment when the environmental impacts before and after intervention are compared. Conversely, there are few LCAs on the environmental impact of system reparation [158]. Also, these approaches have to be reviewed, because the system boundaries of LCA are not systematically investigated [159]. The lack of univocal procedures, as well as of the standardised methods of visualising LCA results [160], leads researchers to apply methods and assumption that are appropriate for the single case study, making results difficult to compare. Therefore, further investigations are needed both to fine-tune the assessment criteria and to broaden the assessment scenarios.

5.2 Research significance

Is it more convenient to refurbish an existing school building or to build a new one? To answer to this question, a benchmarking analysis on two case studies is proposed herein:

- School Building #1: a newly timber school building.
- School Building #2: an old school (built in 1960) in which energy efficiency and structural rehabilitation works have been carried out.

5.2.1 The two buildings

The School Building #1 is a 4-storey precast timber construction (one of which is the basement) with a total living area of about 14000 m². This type of building has been chosen for the lower carbon footprint of the wood, as well as for the speed of construction given by the prefabrication. The building components (structures, envelope, internal partitions, etc.) fulfil the structural [161] and energy [162, 163], performances required by the current code rules for new buildings. Besides, this school has been designed by implementing solutions aimed at optimising the use of climatic conditions and solar radiation, such as avoiding windows on the south side of the buildings and providing shading systems and large overhangs. Accordingly, the building consists of a I-shaped main part, where classrooms, offices, laboratories and parking spaces are located. In a separate rectangular block, a sport hall is present.

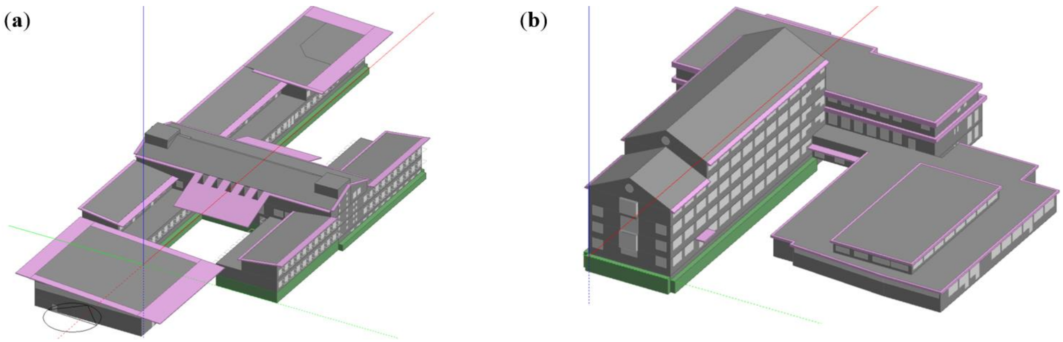

Fig. 50: Models created in DesignBuilder concerning: (a) School Building #1; (b) School Building #2.

The School Building #2 is a 6-storey building (including the basement) with a reinforced concrete frame and a total gross area of about 9900 m². It consists of a multi-storey building, where mainly offices and classrooms are located, a two-storey building containing two sports halls and some classrooms, and a third single-storey block dedicated to teaching laboratories. Refurbishment works, completed in 2013, aimed at complying the structural safety requirements provided by code rules for existing buildings [161]. Specifically, the structure has been strengthened by applying steel cage systems for the columns (with L profiles and transverse plates), introducing steel braces, and strengthening the foundation by means of steel pipe piles. To meet the energy performances [162, 163], an extensive renovation of the building envelope was performed, by providing an insulation layer, installing ventilated facades and by substituting old windows and doors.

Table 14 summarises the main data of the two school buildings. Due to confidentiality reasons, the name of the schools and the place where they are located are undisclosed. However, some boundary conditions are equal as they are in the same city,. These include the climatic conditions, the distance between the material production sites and the construction site, as well as the distance to landfill facilities. The Life Cycle Assessment (LCA) methodology is carried out to assess the environmental performance of both buildings even if the environmental impact of the existing school before the renovation and the embodied carbon of the demolished materials are excluded.

Table 14: Main properties of the two schools analysed herein.

Case study	ID	Gross surface (m^2)	Estimated students (no.)	Type of school	Cost of construction (€)
School Building #1	SB #1	13990	1479	Science high school	23,500,000
School building #2	SB #2	9930	1002	Technical high school	10,600,000

5.3 Methods

The life cycle analysis carried out in this study is based on the JRC Technical Report [151]. The analysis is performed at production level, according to EN 15804 [164], and at building level, in accordance with EN 15978 [165]. In these standards, a modular approach for the definition of system boundaries is adopted, which enables to allocate the greenhouse gas emissions (GHGs) over the entire life of a building, i.e. from cradle-to-grave. It includes the building materials production (Modules A1 to A3), the construction stage (Modules A4 and A5), the use phase (Modules B1 to B7) and the end-of-life phase (Modules C1 to C4).

Lastly, module D considers the possible benefits and loads beyond the system boundary, namely those provided by the recycling, recovery and reuse of materials.

The lifespan of both the buildings investigated herein is assumed to be 50 years, during which the emissions related to maintenance and replacement of components are neglected. Although assuming the same lifespan for both the refurbished and the construction of new building could not take into account the longer life expectancy of new school, it can be considered reasonable within the Italian context. In fact, 50 years is the average life expectancy that Italian building code assumes for ordinary structures [70], and it is also the average time that elapses between major refurbishment works [166].

5.3.1 Life Cycle Assessment

Both the schools have been modelled with Revit software by implementing the stratigraphy and the geometric dimensions of the projects. In this way, the volumes of materials are computed and then used as input data to estimate the embodied CO_2. The unitary impact of each material, expressed in accordance with the climate change indicator, is an input data. The Global Warming Potential (GWP) indicator was selected for the assessment of the environmental impact of the two buildings, because it is consistent with the scope of the study. Moreover, it is the most used indicator in the world, as it provides the result in terms of CO_2 equivalent, which is the main cause of global warming [167].

Basically, there are two types of data sources from which the carbon footprint of materials can be obtained: generic databases (i.e., secondary data) and primary data, so-called Environmental Performance Declarations (EPDs), provided by producers. Results coming from the two categories of data can differ even of 25% for the GWP indicator [168]. This is due to the fact that secondary data are frequently unreliable, because they are based on average local data. Conversely, EPD is nowadays associated to all the new building material. Thus, using product-specific primary data is recommended.

In the current analyses, when no specific design indications were available, the same EPD is associated with the common building components and materials (openings, insulation layers, etc.), in order to make the results of the two buildings comparable.

5.3.2 Thermal analysis

The evaluation of the CO_2 - eq. emitted by the two buildings in 50-years of use (Module B) has been carried out through the thermal analysis of the building, developed within DesignBuilder [169]. These models are shown in Fig. 50.

As a result, the energy for heating and cooling the schools, and that used for Domestic Hot Water (DHW) over the years is computed and converted in terms of CO_2 - eq. by means of suitable conversion factors [170]. Specifically, an electric heat pump system (EER = 1,75) is used to cool both the schools, whereas the needs for heating and DHW are produced by a heating system (COP = 0,85) powered by natural gas. System losses are not assessed. The emissions related to internal furniture, equipment and lighting system are neglected as well, because they are not comparable due to the energy intensive laboratories present in School Building #2 (SB #2). The overall impacts of the two buildings are calculated by summing the contributions from the short-term (i.e. construction phase, in section 5.3.1) and the long-term (i.e., use phase, in section 5.3.2), assessments. To compare the results, two functional units are considered herein:

- CO_2 – eq. emissions per unit of gross building area (kg CO_2 – eq/m^2);
- CO_2 – eq. emissions per student (kg CO_2 – eq/pers.).

5.4 Materials

For both the schools, information on the materials is provided, even if suppliers and the relevant locations are unknown. Thus, EPD was selected for each material, assuming the same producer when a component is present in both the buildings. On the other hand, the embodied carbon of the materials demolished in the School Building #2 has not been taken into account, because the type, the amount, and the percentage of possible recycling, renewal or reuse are not included within the project. Building materials are aggregated into three categories, namely Skin, Space Plan, and Structure, following the layered division (the so called shearing layers) originally suggested by Brand [171], and updated by other authors [172, 173]. Through this approach, the contributions to the overall embodied carbon of the materials given by building envelope (Skin), interior building components (Space Plan), and structure, are computed. As expected, School Building #1 included a quantity of materials larger than that of SB #2. In fact, the existing materials of School Building #2 are not included in the assessment. A significant volume of reinforced concrete has been used in School Building #1 because only concrete provides the strength and durability performances required for some structural elements such as foundations, retaining walls, ground slabs and staircase envelope walls, whilst the rest of the structure is mainly made of wood.

5.5 Results

5.5.1 Comparison of School Building #1 and School Building #2

The histograms of Fig. 51 depict the results in terms of CO_2 - eq., calculated by multiplying the quantities of materials by the unit emissions reported in the corresponding EPDs.

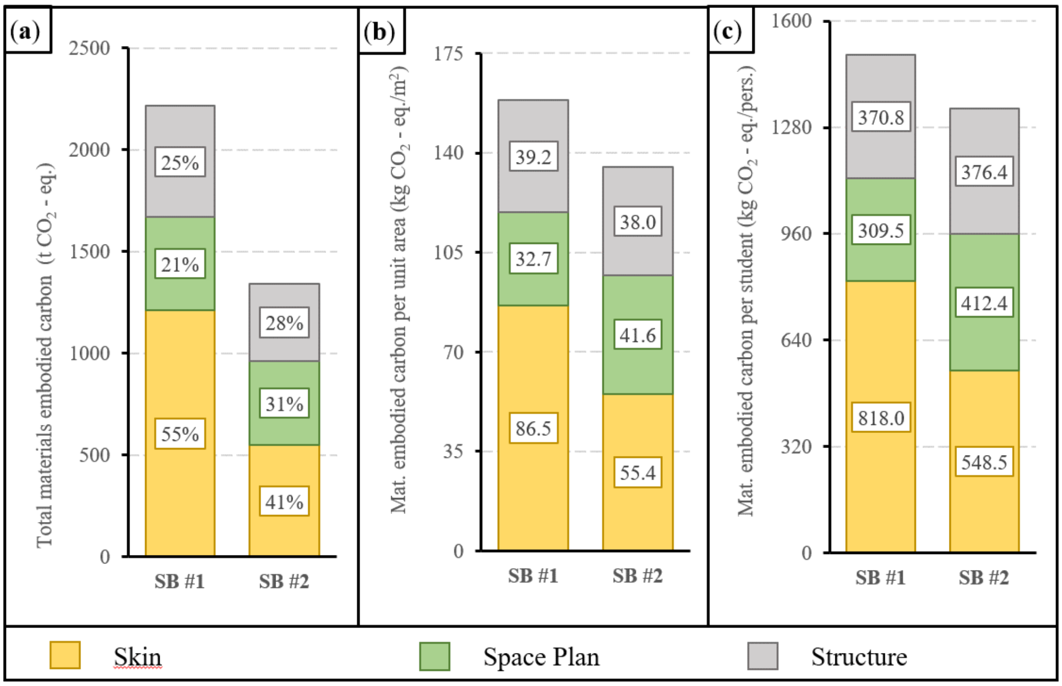

Fig. 51: Embodied carbon of building materials grouped into macro-layers: (**a**) total amount; (**b**) amount per unit of gross building area (u.a.); (**c**) amount per student

In Fig. 51a the total amounts of CO_2 - eq. of the two buildings are reported and divided into the percentages of macro-layers. School Building #1 accounts for the largest embodied carbon of materials, as GHG emissions are 40% higher than in School Building #2. This is due to the greater amount of materials used in new constructions. On the other hand, as shown in Fig. 51b and Fig. 51c, this difference shrinks when the Embodied carbon is referred to the unit of gross floor area and to the number of students, respectively. The incidence of the materials of the building envelope is higher in SB #1 than in SB #2, whereas materials belonging to Space Plan and Structure have a greater impact in SB #2. Indeed, the envelope of both the buildings include expanded polystyrene, glass and aluminium of the windows, which generally have a high environmental impact. On the other hand, the structure and the internal partitions of SB #1, made of wood (which is a biogenic source of carbon storage and highly recyclable), have negative values of CO_2 emissions (modules A1-A3 and D). Therefore, it compensates the CO_2 - eq. of the high-impact materials such as concrete and steel.

In the histograms reported in Fig. 52a, the three types of energy needs (heating, cooling and DHW) are compared. Their incidence as a percentage of the total required energy, computed over one year of use of the two buildings, can also be observed. If on the one hand the heating requirement is the same in both the school (i.e., 76% of the required energy), on the other hand the energy for summer cooling is higher in SB #1 than in SB #2. In fact, the lower thermal inertia of wood has a marginal effect. Due to the large number of labs and sport halls, hot water consumption in SB #2 is larger than in SB #1. However, the most interesting result is the higher overall energy consumption of SB #2, which is about 6% higher than that of SB #1, despite its smaller size and share of students.

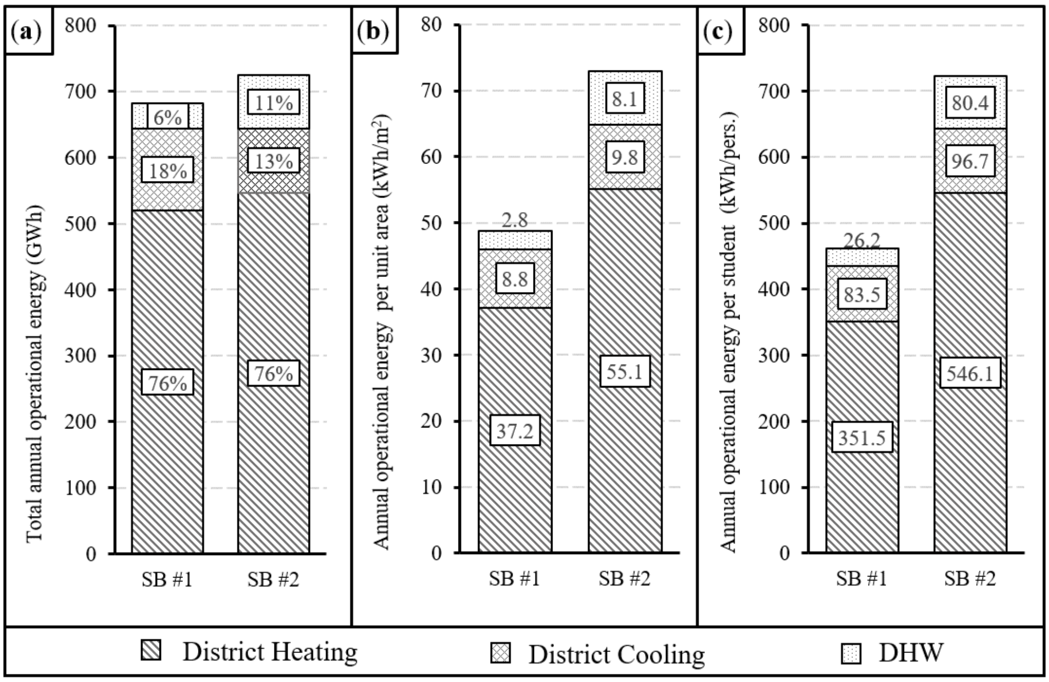

Fig. 52: Comparison of operational energy of buildings grouped by districts: (**a**) total amount; (**b**) amount per unit of gross building area; (**c**) amount per person.

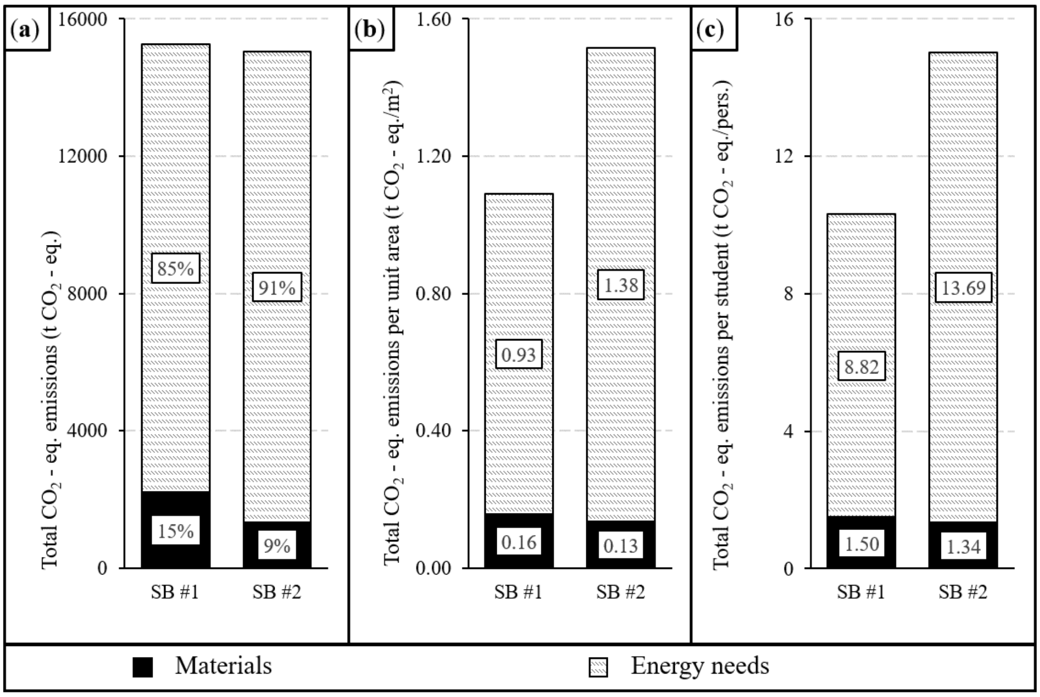

Fig. 53: Comparison of overall CO_2 - eq. emissions marked by sources (material and energy needs): (**a**) total amount; (**b**) amount per unit of gross building area; (**c**) amount per person.

Indeed, this percentage rises to 50% and 57% when related to the gross building area (Fig. 52b) and the number of students (Fig. 52c), respectively. Such result is due to the intrinsic difficulties of making an existing building as efficient as new constructions, because some technical solutions cannot be always put into practice. For instance, in SB #2, some thermal bridges cannot be removed, and it is not possible to modify the orientation of the glazing or to install shielding systems to exploit solar radiation optimally.

Multiplying the energy needs by the proper emission factors, the CO_2 - eq. emissions produced by energy requirements over the 50-year lifespan of the building can be calculated. Fig. 53a summarizes the overall emissions, and the related percentages, due to both materials and energy systems. However, these results are strictly dependent on the assumptions made and boundary conditions, and therefore cannot be generalised. Furthermore:

- As shown, global CO_2 - eq. emissions are strongly affected by winter heating demand (both School Building #1 and School Building #2 are located in an alpine area). Therefore, different results can be obtained if the same analysis are performed in a warmer climate scenario, where the effects of thermal inertia on the thermal behaviour of the building prevails over the transmittance of the building envelope.

- With respect to the embodied carbon of materials, reference has been made to the primary data contained in EPDs. Therefore, the result is strongly influenced by the scenarios assumed by the producers, in particular related to the end of life of the material. For instance, the company producing the timber elements assumed their complete recycling at the end of life.
- A lifespan of 50 years has been assumed for all building components in both schools. Actually, they usually differ according to their function [171, 172, 173], yet it is difficult to reliably estimate their lifespan since it depends on the intrinsic characteristics of the material and the local practices. Besides, emission factors, used to convert operational energy into CO_2 - eq. emissions, are assumed to be constant over the lifespan, yet increasing the use of renewable energy sources and improving efficiency of energy production plants is expected in the future, which might lead to a decrease in the emissions related to energy needs.
- The environmental impact of the excavation of foundations in School Building #1, along with materials demolished in School Building #2 and sent to landfill or to the recycling, renovation, reuse chain was not considered due to lack of information. However, it is reasonable to assume that their influence on the total CO_2 - eq. computation is small. Also, the consumption of undeveloped land, due to the construction of the new school on a vacant lot, is not taken into account.

5.5.2 CO_2 – eq. emission and cost reduction due to the energy refurbishment of the SB#2

With respect to School Building #2, an interesting analysis concerns the long-term benefits of energy refurbishment. For this purpose, in addition to the thermal model of the building after renovation, another model representing the thermal behaviour of School Building #2 before the building intervention was also taken into account. By comparing the two models, a 26% reduction in energy consumption was estimated for the school building. The greenhouse gases and cost for energy needs over the lifetime of the building (50 years) were evaluated by multiplying the energy needs of both the building configurations by the conversion factor in kg CO_2 – eq. [170] and by the unit energy cost, respectively. The unit energy cost (€/kWh) refers to the business use, according to the latest official estimates (September 2021) [174]. The results obtained are shown in Table 15.

Table 15: Energy consumption, total CO_2 - eq. emissions and overall costs for the construction work and energy needs assessed in School Building #2. The end of the construction work and the end of the building's lifespan are used as the reference timeframe.

Parameter	With refurbishment	Without refurbishment
Annual energy needs (GWh)	724,62	984,23
E_0 (t CO_2 – eq.) [1]	1340,02	[-]
E_{50} (t CO_2 – eq.) [2]	15058,69	18573,54
C_0 (million of €) [3]	10,6	[-]
C_{50} (million of €) [4]	17,38	9,20

[1] **E_0** = CO_2 – eq. emissions after the end of works; [2] **E_{50}** = Total CO_2 – eq. emissions at the end of the 50-year building lifespan; [3] **C_0** = Cost of refurbishment; [4] **C_{50}** = Total cost for both refurbishment and energy needs at the end of the 50-year building lifespan.

As can be seen in this Table, the overall amount CO_2 – eq. emissions 50 years after the renovation work are lower than those that would have been achieved without the renovation itself. In other words, improving the building's energy performance offsets the emissions due to the carbon footprint of the new materials used in the renovation within the lifespan period of the school building, as illustrated in Fig. 54.

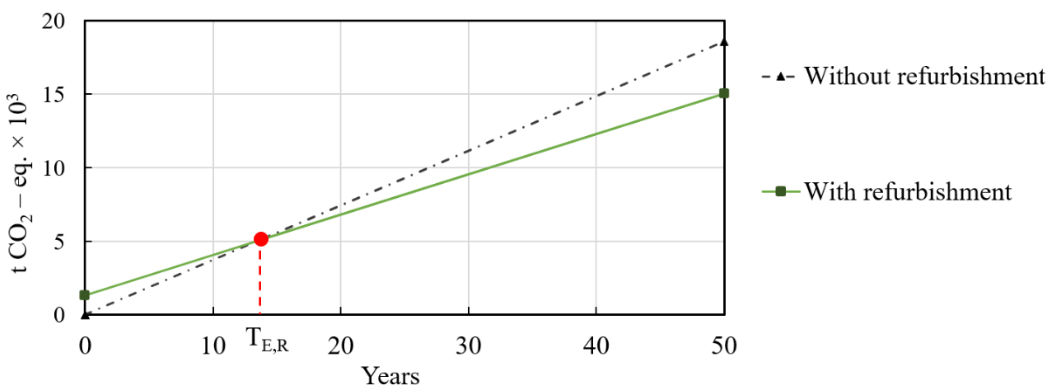

Fig. 54: Comparison of greenhouse gas emission trends during the lifespan of the School Building #2 with and without energy retrofitting.

Fig. 54 shows that the carbon footprint of building materials is recovered after about 14 years ($T_{E,R}$). Therefore, improving the energy performance of the school is an eco-friendly measure over the entire lifespan of the building. On the other hand, the cost savings incurred for energy requirements are not such as to offset the cost of the rehabilitation. In fact, it would take approximately 218 years to recover the whole cost. However, as can be seen from Table 16, the estimated annual energy cost saving is almost 50000 **€**.

Table 16: overview of offsetting emissions and renovation costs due to improved energy performance of School Building #2

Parameter	Emissions of CO_2 – eq.	Costs
Offset period (years)	13,8	218,4
Annual saving	97,08 t CO_2 – eq.	48.538 **€**
Total saving (50 years)	4.854,87 t CO_2 – eq.	2.426.931 **€**

This study shows that, in a specific climatic context, characterized by harsh winters and mild summers, the solution of building a new school with eco-friendly materials, such as wood, should be preferred to the refurbishment of an existing school in reinforced concrete. On the other hand, the renovation of the existing school has proven to be a worthwhile solution in the long term. In fact, the reduction of energy demand led both to the offsetting within the building's lifespan of the embodied CO_2 of the new materials and to substantial savings in energy costs. In addition, the significant rise in energy costs observed since the end of 2021 may emphasise the savings achieved through renovation, shortening the payback time of the retrofitting cost. Thus, the outcomes of the research can be implemented in a decision-making process that compares the benefits and drawbacks of the two alternatives, within the public building sector.

Chapter 6

Conclusions

The ageing of the school building stock, revealed by both the MIUR open data, and the survey carried out with the cooperation of the "*Direzione Regionale Opere Pubbliche*" Office, is mainly due to the typical Italian culture of conservation. In fact, structures in Italy have always been conceived as if they should last forever, without demolition and reconstruction programmes. On the other hand, maintenance and renovation of ageing structures has often been lacking. However, as can be observed from an analysis of the old code rules and durability problems of the structures, many schools were designed and built without considering these problems. When design and construction deficiencies combine with a lack of proper maintenance and retrofitting planning, a high structural risk could be reached. Moreover, along with the well-known risks that can be easily identified by analysing the original projects, or by a visual inspection, also 'hidden' risks can be found. These include risks related to the low mechanical performance of the structural materials, which can only be detected by specific tests on the existing structures, or to elements whose properties or even existence are unknown due to a lack of design information. Therefore, it is necessary to analyse each school building carefully, giving priority to those with the highest risk, depending on the construction techniques and the period of construction.

6.1 Recommendations on design criteria

The historical evolution of both pedagogical models and space utilisation thinking described herein, have shown that school buildings should be designed for flexibility and adaptability. These goals are achieved by adopting lightweight, easily movable and design-for-disassembling construction techniques, such as internal plasterboard partitions. In other words, the school building on the one hand must certainly be safe and robust, yet, on the other hand, it must not be designed to remain unchangeable, but rather must be capable of keeping up with the times. In this regard, design-for-disassembly plays a key role also in the sustainability of a building. Indeed, elements that can be disassembled and reused do not have be disposed in landfill. This is one of the main principles on which the circular economy is based. In general, the sustainable approach should be one of the main principles to be followed when designing new schools, and retrofitting an existing one as well. In section 4.3, the carbon footprint was one of the parameters evaluated, showing that the cement substitution with different percentages of fly ash allows for a reduction in CO_2 emissions while maintaining good mechanical performances. However, ensuring the sustainability of a school building does not mean the irrational replacement of traditional structural materials, which generally have a high environmental impact, with eco-friendly materials. For instance, wood is a material that, despite having a very low carbon footprint and good thermal performance in the winter regime, does not perform well in summer conditions, due to its low thermal inertia. In fact, the analysis carried out within the decision-making process must be extended to the whole building life cycle, using the LCA methodology.

Regarding both the structural types to be used in new constructions and the techniques for strengthening existing structures, there is a plethora of them that have proven to be reliable during earthquakes. Therefore, the challenge facing research is to develop new affordable materials, rather than devising better-performing materials and techniques. This would mean working on more school buildings while using the same economic resources.

Concerning non-structural elements, the main critical issues concern doors and windows, and the fastening systems. The former are particularly prone to even low stresses and strains transferred by the elements they are fitted to, which can compromise their functionality and integrity. Regarding the latter, there are neither specific requirements for the use of certified anchoring systems, nor calculation methods for their design and check. Therefore, they could represent the weakest part of non-structural elements.

6.2 The key role of data collection for future planning

The ARES forms was a worthwhile step toward the introduction of a database providing an overview of the Italian school building stock. However, taking into account the potential represented by current technologies, this system is rather outdated and limited. The shortcomings are related to the data collection and management methods, as well as to the quantity and quality of the data. The data collection system is rudimentary because it does not allow filtering out wrong data, besides well-defined evaluation criteria to obtain consistent and comparable data are not established. Furthermore, the system has only one open-data level that contains general information in the public domain. Technical information resulting from detailed seismic vulnerability analyses or retrofitting interventions is not included. This information is stored by the school owner in electronic files or, as usual practice until a few years ago, in CD or only on paper folders. Therefore, the analysis of projects and data can only be carried out by visiting the archives in person. In other words, there are no shared databases containing such data with instant access. However, the information gathered through surveys on structural elements, non-structural elements and material performances (regarding a vulnerability analyses or structural strengthening interventions) represents a body of knowledge that can be useful for practitioners to carry out more efficient projects. They should be collected in shared databases that allow obtaining reliable data on school structures based on a few information such as the period of construction and location. For this purpose, a smart system for collecting technical information, similar to the materials database proposed in Section 3.4, needs be created.

As BIM methodology is a key solution that is being progressively implemented for new public buildings, it should be introduced also for all new schools and the refurbishment of existing ones. In fact, the primary objective of the BIM is to store very detailed data on any building component, with rapid and multiple ways to extrapolate them. This is extremely useful for maintenance and intervention planning. Along with the BIM, it would be necessary to devise a specific school buildings management system for the rest of the school buildings without a BIM-based project yet, where collecting data from the original project, surveys and vulnerability assessments. This system should be designed to be consistent with the BIM data format to enable data import, comparisons and evaluations.

6.3 Further investigations

Some topics dealt with in this work were not extensively addressed due to their broadness, the limited timeframe and the bureaucratic difficulties faced in obtaining data from the relevant authorities.

Concerning the strength-for-age curves of concrete and steel reinforcement in existing schools, it would be necessary to extend the geographical area of research beyond Piedmont. Furthermore, the investigation scope should be extended to other characteristics of structural elements (such as geometry and quantity of reinforcement). This information could be included in the database described in the previous paragraph to be shared with school building owners and designers. In this way, it would be possible to carry out large-scale vulnerability analyses of school buildings storing from a few input parameters, such as location, size, number of floors and age of construction.

Regarding the newly proposed method for retrofitting existing RC beams by applying precast UHP-FRCC panels, new experimental campaigns should be carried out to address the following issues:

- scale effects by means of full-scale beam testing;
- production of precast panels to be applied to the side faces of the beam, which could increase its shear strength and ductility, whereas panels applied on the upper side would increase the negative moment capacity at the beam nodes (cast-in-situ UHP-FRCC on the extrados of bridge girders to increase resistance against both negative moment and abrasion due to traffic vehicles is currently used [176]). In fact, only an overall strengthening of the beam can significantly improve the performance of the beam against the seismic action, with benefits for the structural behaviour of the whole structure. Further testing is needed to investigate the panels' optimal performance and size and the filling layer's injection methods;
- providing numerical models as design tools to size the panel, tailoring the mix design of the UHP-FRCC and the structural checking of the retrofitting beam according to the required performance in terms of strength, ductility, and carbon footprint.
- application on real beams to investigate the operating procedures for injecting the filler layer and tightening the screws, allowing the technique to be improved and the cost and time of production and implementation to be evaluated.

In general, a win-win solution consists of facilitating the cooperation between Universities and School Building Observatory, and other school authorities as well. It would allow a faster safety target to be achieved through a division of tasks. The authorities could deal with the data gathering, to be shared with the Universities, and with the identification of critical issues in school buildings. Moreover, they should identify the needs and open issues facing the public administration that the University can help to address. The Universities, by means of data processing and experimental research, could suggest new design approaches and monitoring programmes, devise new techniques for structural retrofitting and energy efficiency, along with recommending best practices. Nevertheless, only when the bureaucratic barriers between institutions are removed, a more systematic analysis on the safety and the sustainability of the Italian school building can be performed by using approaches introduced in this work.

References

[1] D..0 Tröhler, «Curriculum history or the educational construction of Europe in the long nineteenth century,» *European Educational Research Journal,* vol. 15, n. 3, pp. 279-297, 2016.

[2] E. Harwood, England's Schools - History, architecture and adaptation, English Heritage, 2010.

[3] L. Baker, «A History of School Design and its Indoor Environmental Standards, 1900 to Today,» National Clearinghouse for Educational Facilities, Washington, 2012.

[4] J. Lancaster, Hints and Directions for Building, Fitting up and Arranging School-Rooms on the British System of Education, 1811.

[5] S. Sauras, "Designing Learning Environment, Humanizing Architecture," 25 03 2020. [Online]. Available: https://www.nonarchitecture.eu/. [Accessed 09 02 2022].

[6] D. Stow, The Training System, 1836.

[7] H. Barnard, School Architecture; or Contributions to the Improvement of School-Houses in the United States, New York: A. S. Barnes & Co., 1850.

[8] Perkins&Will, "Prologue: An Early Education in School Design," 2020. [Online]. Available: https://history.perkinswill.com/prologue/. [Accessed 09 02 2022].

[9] N. Gislason, «Building Paradigms: Major Transformations in School Architecture (1798-2009),» *The Alberta Journal of Educational Research,* vol. 55, n. 2, pp. 230-248, 2009.

[10] T. Hille, Modern Schools: A Century of Design for Education, Hoboken: Wiley, 2011.

[11] J. R. Boice, History and evaluation of the School Construction Systems Development Project, 1961-1967, Menlo Park, Calif: Building Systems Information Clearinghouse, Educational Facilities Laboratories., 1971.

[12] J. Marks, «A History of Educational Facilities Laboratories (EFL),» National Clearinghouse for Educational Facilities, 2009.

[13] "RIBApix," 2022. [Online]. Available: https://www.ribapix.com/Essendon-Primary-School-Hertfordshire-the-hall_RIBA44178#. [Accessed 09 02 2022].

[14] DES, "Children and their Primary Schools. A Report of the Central Advisory Council for Education (England)," Her Majesty's Stationery Office, London, 1967.

[15] T. Beck, «An Australian Study of School Environments,» *Australian Journal of Education,* vol. 24, n. 1, pp. 1-12, 1980.

[16] G. Rodwell, «Open-plan school architecture: a continuation of a tradition of bureaucratically imposed innovation in Australian state schools,» *Education Research and Perspectives,* vol. 25, n. 2, p. 99–119, 1998.

[17] B. Cleveland e K. Woodman, «Learning from past experiences: school building design in the 1970s and today,» *Learning Spaces: The transformation of educational spaces for the 21st century,* vol. 1, pp. 58-67, 2009.

[18] J. C. Thompson, «Cooperative learning in computer-supported classes,» 2005.

[19] Z. Klein e Y. Eshel, «The Open Classroom in Cross-Cultural Perspective: A Research Note,» *Sociology of Education,* vol. 53, n. 2, p. 114–121, 1980.

[20] M. Brogden, «Plowden and Primary School Buildings: A Story of Innovation without Change,» in *FORUM: for promoting 3-19 comprehensive education*, 2007.

[21] GAO, «School Facilities: Condition of America's Schools,» Washington, DC, 1995.

[22] M. Apte, A. Hodgson, D. Shendell, D. Dibartolomeo, T. Hochi, S. Kumar, S. Lee, S. Liff, L. Rainer, R. Schmidt, D. Sullivan, R. Diamond e W. Fisk, «Energy and Indoor Environmental Quality in Relocatable Classrooms,» Ernest Orlando Lawrence Berkeley National Laboratory, Boston, MA, 2002.

[23] A. Taylor, Linking Architecture and Education: Sustainable Design of Learning Environments, University of New Mexico Press, 2009.

[24] L. Kensler e C. Uline, Leadership for Green Schools: Sustainability for Our Children, Our Communities, and Our Planet, Routledge: New York, 2016.

[25] S. Pennisi, «L'edilizia scolastica: evoluzione di una tipologia attraverso un secolo di storia,» in *In Storia dell'Ingegneria. Atti del 4° Convegno Nazionale*, 2012.

[26] M. Sole, Manuale di Edilizia Scolastica, Roma: La Nuova Italia Scientifica, 1995.

[27] V. Viola, «L'edilizia scolastica in Italia ai tempi del Fascismo,» *História da Educação,* vol. 23, 2019.

[28] V. Viola, «The school house". History and evolution of the urban and rural school building in Italy during the fascism,» in *Espacios y patrimonio histórico-educativo*, Erein, 2016, pp. 377-389.

[29] V. Viola, «Gli spazi della memoria scolastica. Prime riflessioni sugli edifici scolastici come luoghi di apprendimento e monumenti attraverso un excursus storico dall'Unità d'Italia alla caduta del fascismo.,» in *La Práctica Educativa. Historia, Memoria y Patrimonio*, Salamanca, FahrenHouse, 2018, pp. 251-264.

[30] E. Cassandri, «L'evoluzione dell'edificio della scuola primaria italiana, tra architettura e pedagogia",» Politecnico di Milano, Milano, 2015.

[31] P. Carbonara, «Edifici per l'istruzione,» 1947.

[32] R. Iovino, F. Fascia e G. P. Lignola, Edilizia scolastica. Riqualificazione funzionale ed energetica, messa in sicurezza, adeguamento antisismico, Palermo: Flaccovio Dario, 2014.

[33] Ministero per i lavori pubblici e Ministero per la pubblica istruzione, «D.M. 18 dicembre 1975 - Norme tecniche aggiornate relative all'edilizia scolastica, ivi compresi gli indici di funzionalità didattica, edilizia ed urbanistica, da osservarsi nella esecuzione di opere di edilizia scolastica,» 1975.

[34] V. Lupo, "L'idea di scuola: didattica e tipologia," 26 09 2021. [Online]. Available: https://web.uniroma1.it/archiscuole/sites/default/files/lupoII.pdf.

[35] P. Fumo, "I padiglioni scolastici nei parchi: trasformazioni tecnologiche e nuove tipologie," 2011. [Online]. Available: https://web.uniroma1.it/archiscuole/sites/default/files/fumoII.pdf. [Accessed

26 12 2021].

[36] A. Selda, M. S. Reyhan e C. K. Nimet, «A Different Perspective on Education: Montessori and Montessori School Architecture,» *Procedia - Social and Behavioral Sciences,* vol. 46, pp. 1866-1871, 2012.

[37] R. De Jesus, Design Guidelines for Montessori Schools, Milwaukee, WI: Center for Architecture and Urban Planning Research, University of Wisconsin--Milwaukee, 1987.

[38] V. Lupo, "L'idea di scuola. didattica e tipologia," Osservatorio sul moderno a Roma, Roma, 2005.

[39] R. Gulli, «L'edilizia scolastica prefabbricata in Italia. La sperimentazione degli anni '60,» in *2° Convegno Nazionale di Storia dell'Ingegneria*, Napoli, 2008.

[40] BPIE, «Europe's Building Under the Microscope - A country-by-country review of the energy performance of buildings,» Buildings Performance Institute Europe,, Brussel, 2011.

[41] M. Economidou, V. Todeschi, P. Bertoldi, D. D'Agostino, P. Zangheri e L. Castellazzi, «Review of 50 years of EU energy efficiency policies for buildings,» *Energy and Buildings,* vol. 225, pp. 1-20, 2020.

[42] A. Kleidouhakis, «Earthquake Resistant Design in Italy, Greece and Turkey – an investigation of Performance-Based Seismic Design,» The University of Greenwich - School of: Architecture, Design and Construction, 2010.

[43] A. Smolentseva, J. Huisman e I. Froumin, «Transformation of Higher Education Institutional Landscape in Post-Soviet Countries: From Soviet Model to Where?,» in *25 Years of Transformations of Higher Education Systems in Post-Soviet Countries*, J. Huisman, A. Smolentseva e I. Froumin,

A cura di, Palgrave Macmillan, Cham, 2018, pp. 1-43.

[44] M. Šojdrová, "Report on the European Education Area: a shared holistic approach (2020/2243(INI))," European Parliament - Committee on Culture and Education, 2021.

[45] N. Kawano, S. Muramaki e K. Kawaguchi, «Elementary School Planning in Japan: A Historical Study for Regulations,» *Civil Engineering and Architecture,* vol. 6, n. 5, pp. 234-241, 2018.

[46] A. Suppasri, E. Mas, I. Charvet, R. Gunasekera, K. Imai, Y. Fukutani, Y. Abe e F. Imamura, «Building damage characteristics based on surveyed data and fragility curves of the 2011 Great East Japan tsunami,» *Natural Hazards,* vol. 66, n. 2, pp. 319-341, 2013.

[47] R. Shears, «Tears as 34 surviving pupils (out of 108) return to school decimated by Japanese tsunami,» *Mail Online,* 30 03 2011.

[48] T. Izumi, R. Shaw, R. Djalante, M. Ishiwatari e T. Komino, «Disaster risk reduction and innovations,» *Progress in Disaster Science,* vol. 2, 2019.

[49] M. R. Guarini, P. Morano e F. Sica, «Historical School Buildings. A Multi-Criteria,» *Sustainability,* vol. 12, n. 3, pp. 1-24, 2020.

[50] A. Rigolon, «European Design Types for 21st Century Schools: An Overview,» *CELE Exchange, Centrefor Effective Learning Environments,* vol. 2010/3, 2010.

[51] M. Kržan, S. Gostič, S. Cattari e V. Bosiljkov, «Acquiring reference parameters of masonry for the structural performance analysis of historical buildings,» *Bulletin of Earthquake Engineering,* vol. 13, p. 203–236, 2015.

[52] G. Menichini, E. Del Monte, M. Orlando e A. Vignoli, «Out-of-plane capacity of cladding panel-to-structure connections in one-story R/C precast structures,» *Bulletin of Earthquake Engineering,* vol. 18, p. 6849–6882,

2020.

[53] A. Belleri, E. Brunesi, R. Nascimbene e P. Riva, «Seismic Performance of Precast Industrial Facilities Following Major Earthquakes in the Italian Territory,» *Journal of Performance of Constructed Facilities,* vol. 29, n. 5, 2015.

[54] L. Liberatore, L. Sorrentino, D. Liberatore e L. D. Decanini, «Failure of industrial structures induced by the Emilia (Italy) 2012 earthquakes,» *Engineering Failure Analysis,* vol. 34, pp. 629-647, 2013.

[55] F. Minghini e N. Tullini, «Seismic Retrofitting Solutions for Precast RC Industrial Buildings Struck by the 2012 Earthquakes in Northern Italy,» *Frontiers in Built Environment,* vol. 7, 2021.

[56] Ministero dei lavori pubblici, Circolare 29 Gennaio 1967 n. 425 - Standards Residenziali, Rome, Italy, 1967.

[57] Italian Ministry of Public Education and Italian Ministry of Works, "Norme tecniche aggiornate relative all'edilizia scolastica, ivi compresi gli indici di funzionalità didattica, edilizia ed urbanistica, da osservarsi nella esecuzione di opere di edilizia scolastica.," 1975.

[58] Ministero dell'Interno, D.M. 26/08/1992 - Norme di prevenzione incendi per l'edilizia scolastica, Rome, Italy, 1992.

[59] Legge 11 Gennaio 1996 n. 23 - Norme per l'edilizia scolastica, Rome, Italy: Gazzetta Ufficiale della Repubblica Italiana, 1996.

[60] Ministro dell'istruzione, Ministro delle infrastrutture e dei trasporti e Ministro dell'ambiente, Decreto Interministeriale 11 aprile 2013 - Linee Guida, Rome, Italy, 2013.

[61] Legge 107 del 31 Luglio 2015 - Riforma del sistema nazionale di istruzione e formazione e delega per il riordino delle disposizioni legislative

vigenti, Rome, Italy: Gazzetta Ufficiale della Repubblica Italiana, 2015.

[62] Ministero dell'Ambiente e della tutela del territorio e del mare, 2017, Rome, Italy, D.M. 11 Ottobre 2017 - Criteri ambientali minimi per l'affidamento di servizi di progettazione e lavori per la nuova costruzione, ristrutturazione e manutenzione di edifici pubblici.

[63] Legge 5 Novembre 1971, n. 1086 - Norme per la disciplina delle opere di conglomerato cementizio armato, normale e precompresso ed a struttura metallica., Rome, Italy: Gazzetta Ufficiale della Repubblica Italiana, 1971.

[64] Legge 2 febbraio 1974, n. 64 - Provvedimenti per le costruzioni con particolari prescrizioni per le zone sismiche, Rome, Italy: Gazzetta Ufficiale della Repubblica Italiana, 1974.

[65] M. d. L. Pubblici, D.M. 9 Gennaio 1996 - Norme tecniche per il calcolo, l'esecuzione ed il collaudo delle strutture in cemento armato, normale e precompresse per le strutture metalliche., Rome, Italy, 1996.

[66] Ministero dei Lavori Pubblici e Ministero dell'Interno, D.M. 16 Gennaio 1996 - Norme tecniche per le costruzioni in zone sismiche., Rome, Italy, 1996.

[67] Presidente del Consiglio dei Ministri, O.P.C.M. 20 Marzo 2003, n. 3274 - Primi elementi in materia di criteri generali per la classificazione sismica del territorio nazionale e di normative tecniche per le costruzioni in zona sismica., Rome, Italy, 2003.

[68] Ministero delle Infrastrutture e dei Trasporti, D.M. 14 gennaio 2008 - Norme Tecniche per le costruzioni, Rome, Italy, 2008.

[69] Ministero delle Infrastrutture e dei Trasporti, Circolare 2 Febbraio 2009, n. 617 - Istruzioni per l'applicazione delle «Nuove norme tecniche per le costruzioni» di cui al decreto ministeriale 14 gennaio 2008., Rome, Italy,

2009.

[70] Ministero delle Infrastrutture e dei trasporti, DM 17/01/2018 - Norme tecniche per le costruzioni, Roma: Gazzetta Ufficiale della Repubblica Italiana, 2018.

[71] Ministero delle Infrastrutture e dei Trasporti, Circolare 21 gennaio 2019 n. 7 - Istruzioni per l'applicazione dell'«Aggiornamento delle "Norme tecniche per le costruzioni"» di cui al decreto ministeriale 17 gennaio 2018, Rome, Italy, 2019.

[72] E. C. f. Standardization, EN 1990 - EN 1999 - Eurocode series, Brussels, 1990 - 1999.

[73] MIUR, "Portale Unico dei Dati della Scuola," 2020. [Online]. Available: https://dati.istruzione.it/opendata/opendata/catalogo/elements1/?area=Edilizia%20Scolastica. [Accessed 06 03 2022].

[74] MIUR, "Portale Unico dei Dati della Scuola," 01 09 2021. [Online]. Available: https://dati.istruzione.it/espscu/index.html?area=anagScu. [Accessed 12 03 2022].

[75] F. Angelli, «Rapporto sull'edilizia scolastica,» Editori Laterza, 2019.

[76] Cittadinanzattiva, "XIX Rpporto - Osservatorio civico sulla sicurezza a scuola," 22 09 2021.

[77] M. Costanzo, «Almanacco del giorno: 31 ottobre 2002. L'Italia piange gli 'Angeli di San Giuliano',» *La Nazione,* 30 10 2021.

[78] O. Giustetti, «Crollo al liceo Darwin, maxirisarcimento alla mamma di Vito per lo "strazio eccezionale",» *la Repubblica,* 17 11 2018.

[79] M. Citterio e G. Fasano, «Report RSE/2009/165 - Indagine sui consumi degli edifici pubblici (direzionale e scuole) e potenzialità degli interventi di

efficienza energetica,» ENEA, 2009.

[80] Legambiente, “Ecosistema Scuola - XX Rapporto sulla qualità dell’edilizia scolastica e dei servizi,” 2021.

[81] F. P. Glasser, J. Marchand and E. Samson, “Durability of concrete — Degradation phenomena involving detrimental chemical reactions,” *Cement and Concrete Research,* vol. 38, no. 2, pp. 226-246, 2008.

[82] M. Mosoarca e G. Victor, «Structural safety of historical buildings made of reinforced concrete, from Banat region – Romania,» *Journal of Cultural Heritage,* vol. 14, n. 3, pp. e29-e34, 2013.

[83] L. Berto, R. Vitaliani, A. Saetta e P. Simioni, «Seismic assessment of existing RC structures affected by degradation phenomena,» *Structural Safety,* vol. 31, n. 4, pp. 284-297, 2009.

[84] S. Kolias e C. Georgiou, «The effect of paste volume and of water content on the strength and water absorption of concrete,» *Cement and Concrete Composites,* vol. 27, n. 2, pp. 211-216, 2005.

[85] H. M. A. Mahzuz, M. M. H. Bhuiyan e N. J. Oshin, «Influence of delayed casting on compressive strength of concrete: an experimental study,» *SN Applied Sciences,* vol. 2, n. 3, 2020.

[86] K. Gkatzogias, G. Tsionis, E. Romano, P. Negro, D. Pohoryles, D. Bournas e L. Sousa, «Integrated techniques for the seismic strengthening and energy efficiency of existing buildings: Pilot Project Workshop 16 19 November 2020,» Publications Office of the European Union, Luxembourg, 2021.

[87] F. Moccia, X. Kubski, . R. F. Miguel e A. Mutton, «The influence of casting position and disturbance induced by reinforcement on the structural concrete strength,» *Structural Concrete,* vol. 22, n. S1, pp. E655-E682, 2020.

[88] A. P. Fantilli, B. Frigo e B. Chiaia, «Retrofit and conservation of historical concrete buildings in Turin (Italy),» in *Le Vie dei Mercanti - XIII Forum Internazionale di Studi*, Aversa, Capri, 2015.

[89] A. P. Fantilli e B. Chiaia, «The strength of concrete in historical bridges,» in *Bridge Maintenance, Safety, Management, Life-Cycle Sustainability and Innovations*, Taylor & Francis Group, 2021, p. 7.

[90] A. . P. Fantilli e B. Chiaia, «Estimation of Steel Rebar Strength in Existing Concrete Bridges,» *ACI Symposium Publication,* vol. 342, pp. 98-113, 2020.

[91] European Committee for Standardization, «Part 3: Assessment and retrofitting of buildings,» in *BS EN 1998-3:2005 - Design of structures for earthquake resistance*, Brussels, 2005.

[92] ACI Commitee 214, ACI 214.4R-03 - Guide for Obtaining Cores and Interpret-ing Compressive Strength Results, American Concrete Institute, 2005.

[93] W. E. Highfield, W. G. Peacock e S. Van Zandt, «Mitigation Planning: Why Hazard Exposure, Structural Vulnerability, and Social Vulnerability Matter.,» *Journal of Planning Education and Research,* vol. 34, n. 3, p. 287–300, 2014.

[94] D. Dominguez-Santos, . P. Ballesteros-Pérez e D. Mora-Melia, «Structural Resistance of Reinforced Concrete Buildings in Areas of Moderate Seismicity and Assessment of Strategies for Structural Improvement,» *buildings,* vol. 7, n. 89, pp. 1-17, 2017.

[95] M. Collepardi , S. Collepardi e R. Troli, Il nuovo calcestruzzo, Tintoretto, 2014.

[96] A. Carpinteri e F. Montagnoli, «Scaling and fractality in fatigue crack growth: Implications to Paris' law and Wöhler's curve,» *Procedia Structural*

Integrity, vol. 14, pp. 957-963, 2019.

[97] O. C. Celik e B. R. Ellingwood, «Seismic fragilities for non-ductile reinforced concrete frames – Role of aleatoric and epistemic uncertainties,» *Structural Safety,* vol. 32, n. 1, pp. 1-12, 2010.

[98] J. C. L. Chiang e L. W. Lim, «Behaviour and Strengthening Methods of Tall Slender Columns in Earthquake Conditions,» *Key Engineering Materials,* vol. 879, pp. 221-231, 2021.

[99] E.-K. M. Ahmed e D. A. Hany, «Improved confinement of reinforced concrete columns,» *Ain Shams Engineering Journal,* vol. 7, n. 2, pp. 717-728, 2016.

[100] M. Bosco, E. M. Marino e P. P. Rossi, «An analytical method for the evaluation of the in-plan irregularity of non-regularly asymmetric buildings,» *Bulletin of Earthquake Engineering,* vol. 11, pp. 1423-1445, 2013.

[101] P. Rajeev e S. Tesfamariam, «Effects of vertical irregularities and construction quality in seismic fragilities for reinforced concrete buildings,» *International Journal of Earthquake and Impact Engineering,* vol. 2, n. 1, pp. 1-31, 2017.

[102] F. Hejazi, S. Jilani, J. Noorzaei, C. Y. Chieng, M. S. Jaafar e A. A. Abang, «Effect of Soft Story on Structural Response of High Rise Buildings,» *Materials Science and Engineering,* vol. 17, 2011.

[103] A. Pavese, I. Lanese e R. Nascimbene, «Seismic Vulnerability Assessment of an Infilled Reinforced Concrete Frame Structure Designed for Gravity Loads,» *Journal of Earthquake Engineering,* vol. 21, n. 2, pp. 267-289, 2017.

[104] M. Pauletta, C. Di Marco, G. Frappa, M. Miani, G. Campione e G. Russo, «Seismic behavior of exterior RC beam-column joints without code-specified ties in the joint core,» *Engineering Structures,* vol. 228, p. 111542,

2021.

[105] F. López-Almansa, D. Domínguez e A. Benavent-Climent, «Vulnerability analysis of RC buildings with wide beams located in moderate seismicity regions,» *Engineering Structures,* vol. 46, pp. 687-702, 2013.

[106] FEMA, «Reducing the Risks of Nonstructural Earthquake Damage – A Practical Guide,» Washington, D.C., 2011.

[107] V. Santarpia, «Liceo Darwin, condanne definitive per il crollo che uccise Vito Scafidi,» *Corriere della Sera,* 03 02 2015.

[108] G. P. Cimellaro, I. P. Christovasilis, A. M. Reinhorn, A. Destefano e T. Kirova, «L`Aquila Earthquake of April 6th, 2009 in Italy: Rebuilding a Resilient City to Multiple Hazard,» University at Buffalo, Buffalo, NY, USA, 2010.

[109] F. Di Trapani, G. Macaluso, L. Cavaleri e M. Papia, «Masonry infills and RC frames interaction: Literature overview and state of the art of macromodeling approach,» *European Journal of Environmental and Civil Engineering,* vol. 19, n. 9, pp. 1-37, 2015.

[110] K. Himoto, «Comparative Analysis of Post-Earthquake Fires in Japan from 1995 to 2017,» *Fire Technology,* vol. 55doi, p. 935–961, 2019.

[111] A. Tarabia e H. Albakry, «Strengthening of RC columns by steel angles and strips,» *Alexandria Engineering Journal,* vol. 53, n. 3, pp. 615-626, 2014.

[112] J. Ramírez, J. Bárcena, J. Urreta e J. Sanchez, «Efficiency of short steel jackets for strengthening square section concrete columns,» *Construction and Building Materials,* vol. 11, pp. 345-352, 1997.

[113] S. Sorace, G. Terenzi e E. Fuso, «Local Retrofit of Reinforced Concrete Structures by the ACM System,» *Buildings,* vol. 11, p. 575, 2021.

[114] A. T. Bassam, A. N. Mohammed, S. Samir e A. Mohammed, «Repairing and Strengthening of Damaged RC Columns Using Thin Concrete Jacketing,» *Advances in Civil Engineering,* vol. 2019, pp. 1-16, 2019.

[115] E. Ugur, T. A.Tugru e S. Ramadan, «Behavior of jacketed columns,» *ACI Structural Journal,* vol. 90, n. 3, pp. 288-293, 1993.

[116] D. J. Oehlers, N. T. Nguyen e M. A. Bradford, «Retrofitting by adhesive bonding steel plates to the sides of R.C. beams. Part 1: Debonding of plates due to flexure,» *Structural Engineering & Mechanics,* vol. 9, n. 5, pp. 1-32, 2000.

[117] R. Kalfat, R. Al-Mahaidi e S. T. Smith, «Anchorage Devices Used to Improve the Performance of Reinforced Concrete Beams Retrofitted with FRP Composites: State-of-the-Art Review,» *Journal of Composites for Construction,* vol. 17, n. 1, pp. 14-33, 2013.

[118] H. A. Safarizki, S. Kristiawan e A. Basuki, «Evaluation of the Use of Steel Bracing to Improve Seismic Performance of Reinforced Concrete Building,» in *The 2nd International Conference on Rehabilitation and Maintenance in Civil Engineering*, 2013.

[119] M. Engindeniz, L. F. Kahn e A.-H. Zureick, «Repair and Strengthening of Reinforced Concrete Beam-Column Joints: State of the Art,» *ACI Structural Journal,* vol. 102, n. 2, 2005.

[120] R. P. Dhakal e R. C. Fenwick, «Detailing of Plastic Hinges in Seismic Design of Concrete Structures,» *Aci Structural Journal,* vol. 135, n. 6, pp. 740-749, 2008.

[121] G. P. Warn e K. L. Ryan, «A Review of Seismic Isolation for Buildings: Historical Development and Research Needs,» *Buildings,* vol. 2, n. 3, pp.

300-325, 2012.

[122] N. Torunbalci, «Seismic isolation and energy dissipating systems in earthquake resistant design,» in *13th World Conference on Earthquake Engineering*, Vancouver, B.C., Canada, 2004.

[123] L. Jianchun, L. Yancheng, L. Weihua e S. Bijan, «Development of adaptive seismic isolators for ultimate seismic protection of civil structures,» *Sensors and Smart Structures Technologies for Civil, Mechanical, and Aerospace Systems,* vol. 8692, 2013.

[124] N. Kovaleva e Y. Rutman, «Estimation of efficiency of damping parameters in seismic insulation systems,» *Magazine of Civil Engineering,* vol. 27, n. 1, pp. 37-43, 2012.

[125] M. Banfi, «Application of Seismic Isolation System in an Existing Structure: Importance to Protect Schools,» in *15th WCEE*, Lisboa, Portugal, 2012.

[126] F. Devincenzi, «Conclusi i lavori di miglioramento sismico alla scuola Corazza,» *Parmapress 24,* 26 September 2017.

[127] B. Tayeh, B. Bakar, M. Johari e Y. Voo , «Utilization of ultra-high performance fibre concrete (Uhpfc) For rehabilitation - A review,» *Procedia Engineering,* vol. 54, pp. 525-538, 2013.

[128] A. Fantilli, L. Paternesi Meloni, T. Nishiwaki e G. Igarashi, «Tailoring Confining Jacket for Concrete Column Using Ultra High Performance-Fiber Reinforced Cementitious Composites (UHP-FRCC) with High Volume Fly Ash (HVFA),» *Materials,* vol. 12, 2019.

[129] A. Gholampour, R. Hassanli, J. E. Mills, T. Vincent e M. Kunieda, «Experimental investigation of the performance of concrete columns strengthened with fiber reinforced concrete jacket,» *Construction and*

Building Materials, vol. 194, pp. 51-61, 2019.

[130] G. Ruano, F. Isla, R. Pedraza, D. Sfer e B. Luccioni, «Shear retrofitting of reinforced concrete beams with steel fiber reinforced concrete,» *Construction and Building Materials,* vol. 54, pp. 646-658, 2014.

[131] G. Martinola, A. Meda, G. A. Plizzari e Z. Rinaldi, «Strengthening and repair of RC beams with fiber reinforced concrete,» *Cement and Concrete Composites,* vol. 32, n. 9, pp. 731-739, 2010.

[132] P. Soares Junior, P. S. Maciel, R. R. Barreto, J. T. Silva Neto, E. C. Siqueira Corrêa e A. C. Bezerra, «Thin Slabs Made of High-Performance Steel Fibre-Reinforced Cementitious Composite: Mechanical Behaviour, Statistical Analysis and Microstructural Investigation,» *Materials,* vol. 12, n. 20, 2019.

[133] P. Jongvivatsakul, L. V. H. Bui, T. Koyekaewphring, A. Kunawisarut, N. Hemstapat e B. Stitmannaithum, «Using Steel Fiber-Reinforced Concrete Precast Panels for Strengthening in Shear of Beams: An Experimental and Analytical Investigation,» *Advances in Civil Engineering,* vol. 2019, 2019.

[134] E. N. B. S. Júlio, F. A. B. Branco e V. D. Silva, «Concrete-to-concrete bond strength: influence of an epoxy-based bonding agent on a roughened substrate surface,» *Magazine of Concrete Research,* vol. 57, n. 8, pp. 463-468, 2005.

[135] N. Malek, K. Muhamad, M. Zahid, N. Hamiruddin, N. Zainol, N. Yahya, N. Bahaman e N. Ramli, «Evaluation of Bond Strength Between Normal Concrete and High Performance Fiber Reinforced Concrete (HPFRC),» in *MATEC Web of Conferences 195*, Solo Baru, Indonesia, 11–12 July 2018.

[136] G. Habert e N. Roussel, «Study of two concrete mix-design strategies to reach carbon mitigation objectives,» *Cement and Concrete Composites,* vol. 31, n. 6, pp. 397-402, 2009.

[137] E. Wille, S. El-Tawil e A. Naaman, «Properties of strain hardening ultra high performance fiber reinforced concrete (UHP-FRC) under direct tensile loading,» *Cement and Concrete Composites,* vol. 48, pp. 53-66, 2014.

[138] J. S. o. C. Engineers, «Recommendations for Design and Construction of High Performance Fiber Reinforced Cement Composites with Multiple Fine Cracks (HPFRCC),» 2008.

[139] A. Lampropoulos, S. Paschalis, O. Tsioulou e S. Dritsos, «Strengthening of reinforced concrete beams using ultra high performance fibre reinforced concrete (UHPFRC),» *Engineering Structures,* vol. 106, pp. 370-384, 2016.

[140] A. Fantilli e B. Chiaia, «The Work of Fracture in the Eco-Mechanical Performances of Structural Concrete,» *Journal of Advanced Concrete Technology,* vol. 11, n. 10, pp. 282-290, 2013.

[141] J. C. Institute, A Textbook of Environmental Aspects for Concrete, Outline of Lci Data of Cement, Tokyo, Japan: JCI, 2013.

[142] I. F. f. S. Concrete, FIB Bulletin No. 67: Guidelines for Green Concrete Structures, Lausanne, Switzerland: FIB Bulletin, 2012.

[143] D. Flower e J. Sanjayan, «Green house gas emissions due to concrete manufacture,» *The International Journal of Life Cycle Assessment,* vol. 12, n. 5, p. 282, 2007.

[144] Consiglio Superiore dei Lavori Pubblici, Circolare 21 Gennaio 2019 n.7 - Istruzioni per l'applicazione dell'«Aggiornamento delle "Norme per le costruzioni"» di cui al decreto ministeriale 17, Roma: Ministero delle Infrastrutture e dei Trasporti, 2019.

[145] C. Pierini, «Elementi costruttivi non strutturali negli edifici in C.A. in zona sismica - Cosa cambia con l’entrata in vigore delle NTC 2018?,» *ingenio,* 2018.

[146] F. Patti, «Edilizia scolastica: i soldi ci sono ma l'anagrafe non è affidabile,» *Linkiesta,* 10 September 2016.

[147] American Society of Civil Engineers, Seismic Evaluation of Existing Buildings, SEI/ASCE 31-03, Reston, Virginia, USA: Engineers, American Society of Civil & Structural Engineering Institute, 2003.

[148] American Society of Civil Engineers, «Chapter 11: Architectural, Mechanical, and Eletrical Components,» in *Seismic Rehabilitation of Existing Buildings*, Reston, Virginia, USA, Engineers, American Society of Civil & Structural Engineering Institute, 2007, pp. 313-351.

[149] G. M. Di Giuda, V. Villa e P. Piantanida, «BIM and Energy Efficient Retrofitting in School Buildings,» *Energy Procedia,* pp. 1045-1050, 2015.

[150] L. S. Italia, «Sistema Quake Saver,» LF System Italia, [Online]. Available: http://www.lfsystemitalia.it/sistemaquakesaver.php. [Consultato il giorno 08 06 2022].

[151] T. Abergel, B. Dean e J. Dulac, «Towards a zero-emission, efficient, and resilient buildings and construction sector. Global Status Report 2017,» International Energy Agency (IEA), 2017.

[152] H. Dos Santos Gervasio e S. Dimova, «Model for Life Cycle Assessment (LCA) of buildings,» Publications Office of the European Union, 2018.

[153] S. Santi, F. Pierobon, G. Corradini, R. Cavalli e M. Zanetti, «Massive wood material for sustainable building design: the Massiv–Holz–Mauer wall system,» *Journal of Wood Science,* vol. 62, pp. 416-428, 2016.

[154] O. S. Alshamrani, K. Galal e S. Alkass, «Integrated LCAeLEED sustainability assessment model for structureand envelope systems of school buildings,» *Building and Environment,* vol. 80, pp. 61-70, 2014.

[155] B. Chiaia, A. Fantilli e O. Mancinelli, «Comparison between the environmental performance of buildings made of reinforced concrete and timber,» *Architecture Heritage and Design,* vol. 4, pp. 73-82, 2019.

[156] B. Soust-Verdaguer, C. Llatas e L. Moya, «omparative BIM-based Life Cycle Assessment of Uruguayan timber and concrete-masonry single-family houses in design stage,» *Journal of Cleaner Production,* vol. 277, p. 121958, 2020.

[157] D. Santucci, D. Voukia e M. Voukia, «Embodied energy as a key for designing low carbon schools,» in *PLEA 2016 Los Angeles - 36th International Conference on Passive and Low Energy Architecture*, Los Angelesm CA, USA, 2016.

[158] Y. Schwartz, R. Raslan e D. Mumovic, «The life cycle carbon footprint of refurbished and new buildings – A systematic review of case studies,» *Renewable and Sustainable Energy Reviews,* vol. 81, n. 1, pp. 231-241, 2018.

[159] A. Vilches, A. Garcia-Martinez e B. Sanchez-Montañes, «Life cycle assessment (LCA) of building refurbishment: A literature review,» *Energy and Buildings,* vol. 135, pp. 286-301, 2017.

[160] W. Pan, K. Li e Y. Teng, «Rethinking system boundaries of the life cycle carbon emissions of buildings,» *Renewable and Sustainable Energy Reviews,* vol. 90, pp. 379-390, 2018.

[161] H. Alexander, K. Benedek, R. Martin, S.-V. Bernardette, H. W. Aoife, L. Sebastien, G. Alina e H. Guillaume, «Review of visualising LCA results in the design process of buildings,» *Building and Environment,* vol. 190, p. 107530, 2021.

[162] E. C. f. S. (CEN), «005 EN 1992-1-1 – Eurocode 2: Design of Concrete Structures,» in *Part 1-1: General Rules and Rules for Buildings*, Brussel,

2005.

[163] The European Parliament and the Council of the European Union, Directive (EU) 2018/844 of the European Parliament and of the Council of 30 May 2018 amending Directive 2010/31/EU on the energy performance of buildings and Directive 2012/27/EU on energy efficiency, Brussel: Off. J. Europ. Union, 2018.

[164] Ministero dello Sviluppo Economico, D.M. 26 Giugno 2015 - Applicazione delle metodologie di calcolo delle prestazioni energetiche e definizione delle prescrizioni e dei requisiti minimi degli edifici, Rome, Italy, 2015.

[165] The British Standards Institution, BS EN 15804:2012 +A1:2013. Sustainability of construction works - Environmental product declarations – Core rules for the product category of construction products, 2014.

[166] The British Standards Institution, BS EN 15978:2011. Sustainability of construction works – Assessment of environmental performance of buildings – Calculation method, 2011.

[167] CRESME, «Città, mercato e rigenerazione 2012 – Analisi di contesto per una nuova politica urbana,» CRESME, Rome, Italy, 2012.

[168] J. Hansen, J. D., L. A., L. S., L. P., R. D. e R. G., «Climate impact of increasing atmospheric carbon dioxide,» *Science,* vol. 213, n. 10.1126/science.213.4511.957, pp. 957--966, 1981.

[169] L. Sébastien, H. Guillaume, P. Bruno e C. Jacques, «Comparison of generic and product-specific Life Cycle Assessment databases: application to construction materials used in building LCA studies,» *The International Journal of Life Cycle Assessment,* vol. 20, p. 1473–1490, 2015.

[170] «DesignBuilder SBEM Training Manual version 6.1.0,» [Online]. Available:

http://oz.berkeley.edu/users/breiman/Using_random_forests_v4.0.pdf. [Consultato il giorno 06 02 2021].

[171] Istituto Superiore per le Protezione e la Ricerca Ambientale, «Fattori di emissione atmosferica di gas a effetto serra nel settore elettrico nazionale e nei principali Paesi Europei,» Rome, Italy, 2019.

[172] S. Brand, How Buildings Learn: What Happens After They're Built, Penguin Books, 1995.

[173] E. Iacovidou e P. Purnell, «Mining the physical infrastructure: Opportunities, barriers and interventions in promoting structural components reuse,» *Science of The Total Environment,* vol. 557–558, pp. 791-807, 2016.

[174] R. Castro e P. Pasanen, «How to design buildings with Life Cycle Assessment by accounting for the material flows in refurbishment,» *IOP Conference Series: Earth and Environmental Science,* vol. 225, 2019.

[175] «Italy electricity prices,» GlobalPetrolPrices, September 2021. [Online]. Available: https://www.globalpetrolprices.com/Italy/electricity_prices/. [Consultato il giorno 10 June 2022].

[176] E. Brühwiler, M. Bastien-Masse, H. Mühlberg, B. Houriet, B. Fleury, S. Cuennet, P. Schär, F. Boudry e M. Maurer, «Strengthening the Chillon viaducts deck slabs with reinforced UHPFRC,» *IABSE Symposium Report,* vol. 105, n. 24, pp. 1-8, 2015.

[177] C. R. Starring e J. O. Knauss, «The Michigan Search for Educational Standards,» vol. 2, Michigan Historical Commission, 1969, pp. 10-11, 39-44, 175-188.