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Article

Construction Failures and Innovative Retrofitting

Nicola Augenti [1], Antonio Nanni [1,2,*] and Fulvio Parisi [1]

[1] Department of Structures for Engineering and Architecture, University of Naples Federico II, via Claudio 21, 80125 Naples, Italy; E-Mails: augenti@unina.it (N.A.); fulvio.parisi@unina.it (F.P.)

[2] Department of Civil, Architecture & Environmental Engineering, University of Miami, 1251 Memorial Drive, Coral Gables 33146-0630, FL, USA

* Author to whom correspondence should be addressed; E-Mail: nanni@miami.edu; Tel.: +1-305-284-3461; Fax: +1-305-284-3492.

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Abstract: The aim of this paper is twofold: (a) to briefly describe the damage caused to historical, residential and industrial buildings by the May 2012 seismic events in the Emilia Romagna region of Italy; and (b) to summarize novel repair and rehabilitation technologies that can be available to practitioners to fix damaged structures or to upgrade undamaged ones. Field inspections after the Emilia Romagna earthquakes showed that seismic vulnerability in that region was primarily due to the lack of seismic detailing in modern buildings, and the presence of heavy nonstructural masses in historical buildings. The novel retrofit technologies discussed in this paper are based on the use of non-conventional or relatively recent material systems where the reinforcement is in the form of continuous or chopped fibers.

Keywords: earthquake; failures; fibers; repair; retrofitting

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1. Introduction

Strong earthquakes around the world have recently demonstrated the need for structural retrofitting in the case of developed countries with a large cultural heritage built inventory not designed to withstand seismic actions. The complex nature of urban networks can rapidly increase seismic risk as a result of the interaction between several components of the built environment and society. Therefore, in the last decade, earthquake effects have highlighted the growing importance of disaster resilience, the ability of a community to respond to natural or man-made disasters, and to recover a target performance level in an effective and rapid way. Japan and New Zealand responded optimally to destructive earthquakes, which occurred in Tohoku and Christchurch in 2011. In Italy, even if the 1997 Umbria-Marche and the 2009 L'Aquila earthquakes showed a high fragility level of the building stock [1,2], short- and long-term earthquake risk mitigation strategies developed in the last few years did not provide a satisfactory safety level to ordinary, historical, and industrial structures. The achievement of a satisfactory disaster resilience level is complicated by the morphology of the Italian territory, which is composed of many sub-parallel faults. When seismic energy is released by a fault, adjacent faults are triggered, and interact each other, causing an earthquake swarm that includes a number of mainshocks, each followed by hundreds of aftershocks with lower magnitudes. In these conditions, it is difficult to achieve safety protection operations and temporary retrofit interventions. This situation has been highlighted once again by the earthquake swarm that struck the Emilia Romagna region, northern Italy, from 16 May to 26 June 2012. The swarm included seven events with magnitude equal to or greater than 5.0. The mainshocks occurred on 20 (02:03:52 UTC) and 29 May (07:00:03 UTC) 2012, causing: seven and 19 casualties, respectively, 350 injuries, approximately 14,000 homeless people, and heavy damage to industrial and historical buildings. The affected area was not included in the national list of seismic zones until 2003 [3]. Furthermore, the new Italian Building Code (ItBC) [4], which includes a site-dependent hazard assessment procedure and performance-based seismic design/assessment methods, became mandatory on 1 July 2009. As a result, the majority of existing structures were designed for gravity loads only.

In this paper, damage to industrial facilities, housing buildings, and historical constructions induced by the 2012 Emilia Romagna earthquakes is discussed. Recurrent vulnerability factors of structures located in the affected area are identified. The description gives room for the presentation of retrofitting techniques that could be implemented to reduce seismic fragility of existing structures and to quickly recover an acceptable functionality level in Emilia Romagna, one of the most important industrial regions of Italy.

2. Construction Failures Due to the 2012 Emilia Romagna Earthquakes

2.1. Earthquake Characteristics and Overview on Losses

The 20 May 2012 earthquake occurred approximately 15 km west of Ferrara; it was characterized by local magnitude $M_L = 5.9$, moment magnitude $M_w = 6.11$ and focal depth $D = 6.3$ km. The epicenter was located at Finale Emilia (44.89° N, 11.23° E) in the Modena province (Figure 1). A reverse faulting mechanism with maximum compression along the N-S direction (dip 46.45°, strike 103.28°, rake 93.87°) generated the earthquake [5].

Figure 1. Location of the affected area.

Both the National and Northern Italy Strong Motion Networks recorded the seismic event. 25 permanent recording stations of the national network are located in Emilia Romagna and 15 mobile stations were installed after the 20 May mainshock. The MRN station located at Mirandola (epicentral distance equal to 13.4 km) recorded horizontal (PGA) and vertical peak ground accelerations of 0.26*g* and 0.31*g,* respectively. PGA reduced to 0.05 *g* at 39.7 km from the epicenter [6]. According to the ItBC [4], PGA at bedrock corresponding to a design earthquake with a 475 year return period (*i.e.*, 10% probability of exceedance in 50 years) is between 0.125 and 0.15 *g* in the epicentral area, so it can be classified as a medium seismicity zone compared to the rest of the Italian territory. A team from the Italian Civil Protection Department (ICPD) [7] visited the affected area soon after the mainshock. A maximum Mercalli-Cancani-Sieberg (MCS) macro-seismic intensity [8] I_o = 7 MCS was assigned to Mirandola and San Felice sul Panaro, while macro-seismic intensities between 6 and 7 MCS were found at Finale Emilia and adjacent villages. The Italian Institute for Electromagnetic Survey of the Environment, in cooperation with the Italian Institute of Geophysics and Volcanology (INGV), processed radar images provided by the COSMO-SkyMed satellite of the Italian Space Agency. The computations provided estimates of the ground surface deformations induced by the earthquake, showing uplift displacements of up to 150 mm. It is emphasized that the same technique estimated a peak ground displacement equal to 250 mm in the case of the 2009 L'Aquila, Italy, earthquake [2].

The 29 May 2012 earthquake took place with an epicenter between Mirandola and Cavezzo (44.85° N, 11.09° E) in the Modena province, 15 km northwest of the epicenter of the 20 May event. That earthquake caused further damage to the built environment and the extension of the affected area to the east side of the Modena province. The mainshock was characterized by M_L = 5.8, M_w = 5.96 and D = 10.2 km [7]. Damage accumulation induced a macroseismic intensity increase between one and two MCS levels, resulting in: I_o = 6–7 MCS at Reggiolo; I_o = 7 MCS at Moglia, Novi di Modena and Concordia sulla Secchia; I_o = 7–8 MCS at Rovereto, a fraction of Novi di Modena. Fire brigades carried out approximately 63,000 rapid usability inspections until 27 July 2012. Teams of experts from several universities, regional institutions, and professional associations performed 39,899 usability inspections through AeDES forms provided by the ICPD (AeDES is the Italian acronym of "Agibilità e Danno nell'Emergenza Sismica", which means "usability and damage after seismic emergency"). By 2 August 2012, such inspections provided the following statistics on buildings located in the affected localities: 36% fit for use, 17.3% temporary unfit for use, 4.5% partially fit for use, 0.6% temporary unfit for use and to be inspected again, 35.5% unfit for use, and 5.5% unfit for use due to external risk.

The area affected by the 20–29 May 2012, earthquakes is located in the Emilian river Po valley, between Modena and Ferrara, and is characterized by unconsolidated soil layers which can induce local amplifications of earthquake ground motion. The Emilian river Po valley is a rather flat alluvial plane which has experienced many floods from several tributary rivers of the Po and Reno rivers in the past. The ground is mainly composed of alternated fine sand and clay layers that caused soil liquefaction phenomena and the formation of sand volcanoes (Figure 2a) in a number of villages (such as Mirabello, Sant'Agostino, Cavezzo and San Felice sul Panaro) during the 2012 Emilia Romagna earthquakes. Soil liquefaction at San Carlo, a fraction of Sant'Agostino, caused surface fractures with a maximum width of 1.05 m and a vertical relative displacement of up to 1.90 m (Figure 2b). Liquefaction-induced differential settlements and lateral spreading of ground were a common cause of collapse in buildings.

Figure 2. Soil liquefaction effects: (**a**) Sand volcano in a cemetery; (**b**) Surface fracture.

(**a**) (**b**)

The 2012 Emilia Romagna earthquakes caused heavy damage to industrial facilities, residential dwellings, and cultural heritage buildings. Most of damage to the built inventory affected unreinforced masonry (URM) buildings, whereas reinforced concrete (RC) buildings were destroyed only in a few cases. Partial collapses and heavy damage to recently constructed URM and RC buildings were found.

2.2. Rural Buildings

About 100 rural clay brick masonry buildings consisting of dwellings and haylofts were strongly damaged or collapsed (Figure 3a,b). Many rural buildings suffered out-of-plane wall failures, tilting of masonry pillars, or a pancake-type progressive collapse of wooden floors (which had significant in-plane flexibility and no connections to masonry walls). Damage to rural buildings had a psychological impact on inhabitants, but did not cause significant economic losses.

Figure 3. Collapse of rural masonry buildings: (**a**) Corner mechanism; (**b**) Out-of-plane mechanism of wall.

(**a**)

(**b**)

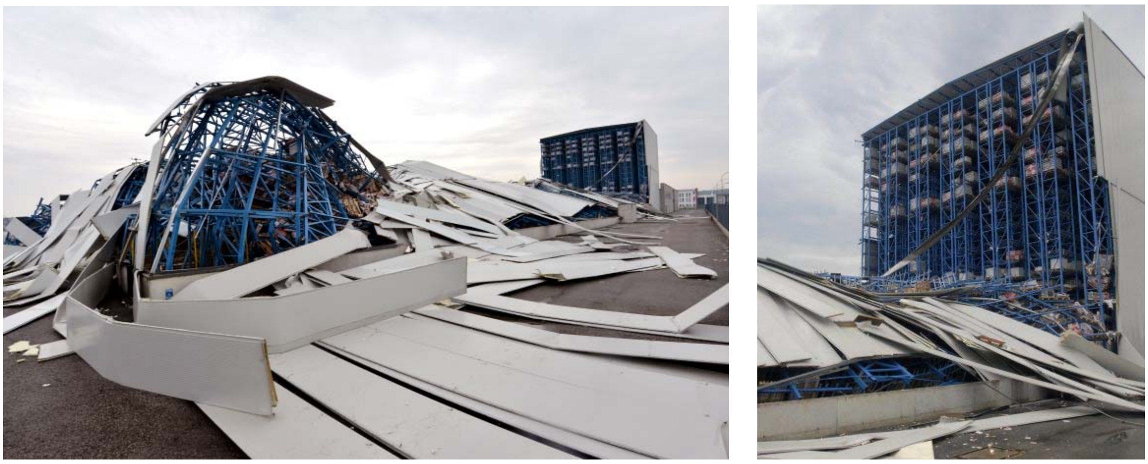
2.3. Industrial Facilities

Apart from a few cases where silos and other industrial facilities collapsed (Figure 4), casualties and most of the total economic loss were caused by the catastrophic failure of buildings. The majority of the industrial buildings located in the affected area have structural systems made of precast RC elements (which are widely used in Italy [9]), precast, prestressed concrete (PC) elements, and steel frames. A smaller number of industrial buildings are also made of masonry walls, hybrid masonry-RC systems, and timber frames.

Figure 4. Collapse of steel silo due to overturning.

Figure 5a,b shows a pottery warehouse in Sant'Agostino which collapsed during the 20 May earthquake, resulting in two casualties. That facility was used to store boxes of ceramic tiles, and was composed of 12 rows of shelves, each of them consisting of two parallel multi-level parallel steel frames along the longitudinal direction of the building plan [10]. The frames were transversally connected to each other at the roof and foundation levels by means of a truss system; no braces were used in the longitudinal direction. The warehouse was enclosed by sandwich insulation panels. The presence of heavy nonstructural masses especially at upper levels induced a severe dynamic demand on the steel frames and hence the collapse of approximately 2/3 of the building. Figure 6a,b shows a partially collapsed building in Medolla with a storage steel structure which experienced large lateral deformations due to the massive stored contents. The lateral deformation of the structure was not prevented by the small size of steel elements, nor by X-braces (Figure 6b).

Figure 5. Pottery warehouse: (**a**) Totally collapsed section; (**b**) Undamaged section.



(**a**) (**b**)

Figure 6. Storage building with internal braced steel scaffolding system: (**a**) Global view; (**b**) Failed connection of steel brace.

(**a**) (**b**)

Many industrial RC buildings were designed for gravity loads only. Roof beams were only partially restrained from relative movement over the column tops by dry friction beam-column joints or neoprene pads. Since no mechanical connections (e.g., shear keys, bolted steel plates) between roof beams and columns were in place, roof beams could slip off the supports and fall down under earthquake excitation. In other cases, RC beams resting on U- or L-shaped seats at the top of columns twisted and badly damaged the support. It is also emphasized that roof systems of industrial buildings often did not have sufficient in-plane stiffness to distribute horizontal seismic actions among lateral load-resisting vertical elements (namely, columns or walls). As a result, many buildings had long periods of vibration and were probably subjected to higher acceleration amplifications induced by local site conditions. In some cases plastic hinges formed at the base of columns, or the ejection of concrete cover and buckling of longitudinal bars was observed.

Overturning of façade panels was a typical mechanism observed during field inspections. Figure 7a shows that this type of damage was suffered by an industrial building located in Cavezzo. Although the weight of such panels was significant (Figure 7b), they were poorly connected to the edge roof beam through a few steel bars (Figure 8a) and to the base of the structure through U-shaped seats of RC ground beams (Figure 8b).

Figure 7. External view of precast reinforced concrete (RC) building with tilted façade panels: (**a**) Damaged façade; (**b**) Detail of heavy façade panels.

(**a**) (**b**)

Figure 8. Connections between façade panels and RC structure: (**a**) Panel-to-edge beam connections; (**b**) Panel-to-ground beam connections.

(**a**) (**b**)

Figure 9 illustrates a damaged U-shaped seat on the top of a column that was built up to provide dry friction supports to the roof beam. The lack of bracing systems contributed to the uncoupled response of roof and vertical structural elements, which included the torsional rotation of roof beams and the resulting flexural damage to flanges of their U-shaped seats. Many similar cases of collapse can be found in post-earthquake field observation reports [5,10,11].

Figure 9. U-shaped seat damaged by twisted roof beam.

Some industrial masonry buildings were also destroyed in 2012 Emilia Romagna earthquakes. Figure 10a,b shows a three-story industrial building located close to Massa Finalese that partially collapsed during the 20 May 2012 earthquake. The structure was composed of load-bearing clay brick masonry walls with openings, one-way RC joist slabs with tiles, and a pitched wooden roof.

Figure 10. Industrial masonry building partially collapsed: (**a**) Global view; (**b**) Detail of collapsed story.

(**a**) (**b**)

2.4. Residential RC Buildings

Residential RC buildings in the affected region are typically masonry infilled RC frames with the infrequent inclusion of RC shear walls at the location of stairwells and elevators. A few buildings experienced global collapse as result of soft storey mechanisms at the ground floor. Figure 11 shows a RC building with pilotis in Cavezzo, where the whole ground floor disappeared and also a part of upper stories collapsed. Similar collapse modes were observed at L'Aquila, Italy [2].

Figure 11. RC building with pilotis at the ground floor.

The 2012 Emilia Romagna earthquakes confirmed the influence of masonry infills on the seismic response of RC framed buildings. Partially infilled frames suffered shear collapse of columns as shown in Figure 12a–d. Earthquake damage due to infill wall-RC frame interaction can be seen in Figure 13a,b, which demonstrate the clear formation of a strut resisting mechanism inside the masonry infill and the resulting cracking of an adjacent RC column.

Figure 12. (**a**) Corner view of damaged building; (**b**) Partially infilled RC frame at the ground floor; (**c**) Shear failure of corner column; (**d**) Shear failure of peripheral column.

(**a**) (**b**)

(**c**) (**d**)

Figure 13. (**a**) Damage to infill masonry walls; (**b**) Diagonal crack propagation from infill wall to RC column.

(**a**) (**b**)

2.5. Residential Masonry Buildings

Most modern URM dwellings suffered slight damage (I_o = 1–2 MCS). Higher damage levels (I_o = 3–4 MCS) were detected in some buildings, which exhibited large cracks, whereas a lower number of URM buildings reached partial or total collapse. Soil liquefaction induced extensive crack patterns within URM dwellings, as a result of settlement-induced differential displacements, uplift, or lateral spreading. This type of damage was mainly observed in San Carlo and Mirabello. Figure 14 shows the impressive collapse of a newly constructed URM building located in Rovereto (fraction of Novi di Modena), which had a portico at the ground floor. That collapse phenomenon clearly demonstrated that a masonry structure does not behave monolithically under a differential settlement caused by a liquefied ground movement, particularly if the foundation system does not have sufficient flexural stiffness. Figure 15 illustrates a residential masonry building in Rovereto that probably collapsed because of its structural deficiencies. This hypothesis is motivated by the fact that similar adjacent buildings did not suffer heavy damage.

Figure 14. Liquefaction settlement-induced collapse of masonry building.

Figure 15. Global collapse of masonry building.

An earthquake-resistant URM building should have masonry interlocking between orthogonal walls, an effective connection between floors and walls through RC tie beams or steel ties, and floor systems with sufficient in-plane stiffness able to distribute horizontal seismic actions among walls. Both old and modern URM buildings located in the affected region did not have one or more of these earthquake-resistant measures, so they suffered local collapses (Figure 16a,b).

Figure 16. Out-of-plane collapse of peripheral load-bearing masonry walls at the level of thrusting pitched roofs: (**a**) Building in Cavezzo; (**b**) Building in Mirandola.

(**a**) (**b**)

If local collapse mechanisms are prevented, a global "box-type" seismic response takes place. The distribution of horizontal seismic actions among walls induces the in-plane lateral loading of individual masonry walls, which typically suffer a concentration of damage in macroelements between openings [12]. Pier and spandrel panels, namely, the vertical and horizontal macroelements, are usually defined in the macroelement mesh of masonry walls with openings. When such macroelements are subjected to in-plane lateral loading, shear or flexural cracking can occur. Figure 17 shows diagonal shear cracks in spandrel panels observed in modern masonry buildings located in Mirandola. Significantly different damage patterns were detected in the case of masonry walls with irregular layout of openings (Figure 18a,b), which often develop a lower seismic capacity compared to their regular counterparts [13].

Figure 17. Diagonal shear cracks in spandrel panel.

Figure 18. Damage to irregular masonry walls with openings: (**a**) wall irregularity due to different number of openings between two stories; (**b**) wall irregularity due to different opening heights at the same story.

(**a**)

(**b**)

Several two- and three-story URM dwellings located in Rovereto were seriously damaged by the May 2012, earthquake and partially collapsed. These buildings were constructed in the last decade and their structure was composed of load-bearing hollow brick masonry walls. Figure 19a,b show large cracks in piers and the collapse of external brick masonry leaf, respectively. Figure 20 illustrates a strange failure mode of building corners. URM dwellings located in the same area suffered just slight damage, indicating that adjacent houses with heavy damage could have been built with poor quality materials and/or incorrect construction procedures.

Figure 19. Heavy damage to hollow brick masonry piers: (**a**) Large sub-vertical cracks; (**b**) Ejection of external leaf and diagonal shear cracks in hollow brick masonry core.

(**a**) (**b**)

Figure 20. Corner failure of hollow brick masonry building.

2.6. Historical Masonry Construction

Many historical buildings, including churches, bell and clock towers, castles, and palaces, were partially or completely destroyed, as also reported after other past earthquakes, such as the 2009 L'Aquila earthquake [2]. The 2012 Emilia Romagna earthquakes confirmed the high fragility level of (a) bell and clock towers which are prone to suffer torsional cracks; and (b) tympanums of churches and masonry buildings without RC tie beams or steel ties which are prone to suffer out-of-plane failure mechanisms (Figure 21a,b); such as vaults and domes which often suffer horizontal shear cracks under large relative displacements of load-bearing walls.

Figure 21. Out-of-plane corner mechanisms of brick masonry building units located in the historical center of San Felice sul Panaro: (**a**) Collapse of one story; (**b**) Collapse of two stories.

(**a**) (**b**)

The 29 May 2012 earthquake caused the almost total collapse of the Church of San Francesco in Mirandola, which was partially destroyed by the 20 May earthquake. In the same town, the second main shock induced further damage to the cathedral and partial collapse of buildings located in the historical center. The collapse of single building floors in Cavezzo increased the macroseismic intensity level to 7 MCS.

The Colleggiata di Santa Maria Maggiore, a baroque church with high cultural value located in Pieve di Cento (Figure 22a), was significantly damaged by the 29 May 2012 earthquake. That church is the oldest one in the area and its apse was built in the second half of the 14th century whereas almost the rest of the building was reconstructed between 1702 and 1710. The church has a single nave with lateral altars. The bell tower was built up in 1487 with Romanesque-Renaissance style and is 48 m high; actually, the tower was constructed on the former structure, which partially collapsed, and the bells were produced in 1809 in Reggio Emilia. The 29 May earthquake caused the partial collapse of the dome (Figure 22b), a crack at the top of the bell tower, cracks on the main façade and other walls, and heavy damage to artwork.

The Cathedral of Santa Maria Maggiore suffered cumulative damage during the 20 and 29 May 2012 earthquakes. The church has a three-nave basilica-type cross plan with a bell tower to the right of the apse (Figure 23a). Decanini *et al.* [5] carried out field inspections after the 20 May earthquake, identifying some iron ties that connect the main façade to the orthogonal walls (*i.e.*, the longitudinal walls of the naves). Iron ties were also identified in the bell tower below the lower gable. The main façade was built in the 19th century, whereas the bell tower was constructed for the first time in 1449 and then was reconstructed in the 17th century. The 20 May earthquake caused some cracks in the bell tower and the loss of the upper left spire of the main façade (Figure 23b) in which out-of-plane collapse mechanism was effectively prevented by iron ties and connection to orthogonal walls. The 29 May earthquake caused damage propagation. The yielding of iron ties of the main façade caused the out-of-plane collapse of the tympanum (Figure 23c); furthermore, the upper right spire collapsed and the bell tower suffered heavier damage (Figure 23d).

Figure 22. Church Colleggiata di Santa Maria Maggiore: (**a**) Main façade before the earthquake; (**b**) Partially collapsed dome.

(**a**) (**b**)

Figure 23. Cathedral of Santa Maria Maggiore in Mirandola: (**a**) Overall view before the 2012 Emilia Romagna earthquakes; (**b**) Main façade after the 20 May 2012 earthquake; (**c**) Overall view after the 29 May 2012 earthquake; (**d**) Large diagonal cracks in the bell tower.

(**a**) (**b**)

(**c**) (**d**)

3. Innovative Retrofitting Techniques

The fairly comprehensive summary of seismic damage presented in the previous section, on different classes of building structures, was intended to illustrate the vulnerability of the building stock in a specific region of the world with its specific culture and traditions. Different sets of questions emerge from this analysis, among them: (a) what can be done to mitigate the vulnerability of existing buildings, damaged or undamaged, by a seismic event; and (b) how can design and construction practices for future buildings be altered to minimize vulnerability. The approach for responding to these questions is not simply technical, but requires a deep appreciation of the socio-economic situation of the affected area. Rather than attempting to holistically address such complex scenario, the authors briefly discuss some of the innovative retrofitting techniques that could be used in the mitigation efforts in Emilia Romagna or elsewhere.

Important characteristics of innovative repair technologies, in addition to structural efficacy and affordable cost are: durability, ease and speed of installation, reduced disturbance to substrate, lightweight, and reversibility.

3.1. Fiber-Reinforced Polymer Technologies

The last two decades have seen the coming of age of composite materials (also known as fiber-reinforced polymer or FRP composites) for the retrofitting of existing structures [14,15]. Italy has been at the forefront in the adoption of the FRP technology from both the practical point of view (*i.e.*, wide variety and number of applications) as well as its inclusion in design and construction specifications [16]. FRP technologies include externally bonded fiber sheets installed by manual lay-up and pultruded FRP plates, adhesively bonded. The former method can provide shear and flexural strengthening in addition to column confinement (Figure 24a), whereas the latter method is primarily used for flexural strengthening (Figure 24b). Pre-tensioning of the FRP prior to bonding has been attempted to increase the efficiency of the strengthening of FRP (Figure 24c), however, commercially available systems have not yet completely overcome the challenges presented by anchoring and installation.

Figure 24. Externally bonded Fiber-reinforced polymer (FRP) installations: (**a**) Manual lay-up for column confinement; (**b**) Bonded plate for flexural upgrade; (**c**) Plate pre-tensioning system.

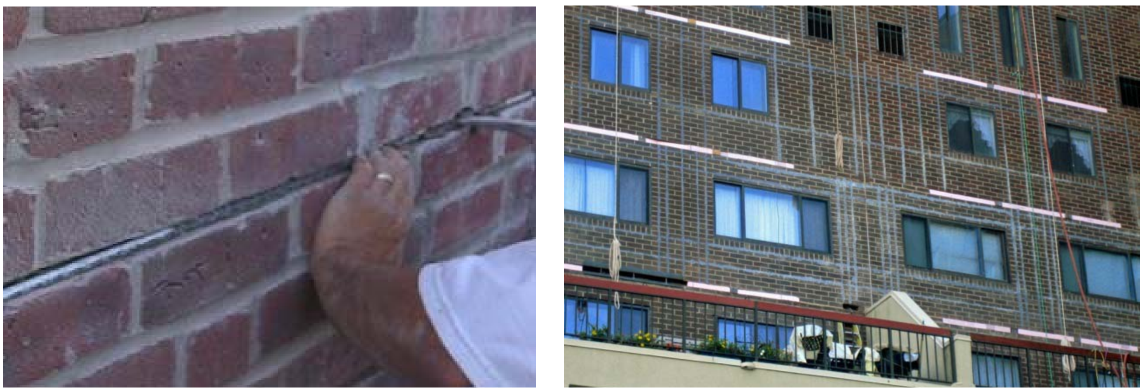
(**a**) (**b**) (**c**)

The FRP technology, known as near-surface-mounted (NSM), FRP reinforcement (in the case of concrete and timber members), or structural repointing (in the case of masonry), is perhaps less known and utilized, even though it may have an aesthetic advantage over externally bonded FRP systems. In the case of concrete strengthening, an FRP bar is embedded in a pre-cut groove then filled with an epoxy resin (Figure 25), whereas in the case of in-plane or out-of-plane masonry wall strengthening, the existing mortar in the bedding joint is typically removed, the FRP bar is inserted between masonry units, and the cavity is filled with resin or mortar depending on the type of application and its design requirements (Figure 26).

Figure 25. Near-surface-mounted (NSM) FRP installation on negative moment regions: (**a**) Exposed concrete slab in a parking garage; (**b**) Concrete below the asphalt overlay in the cantilevered portion of a bridge deck.

(**a**) (**b**)

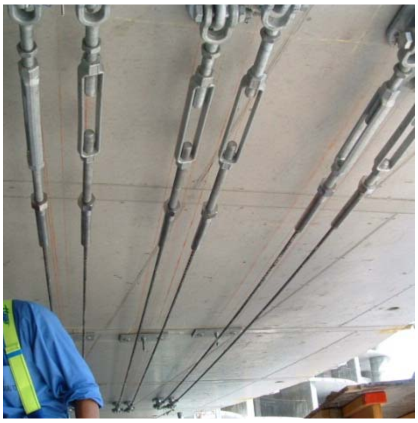
Figure 26. Structural FRP repointing in clay masonry walls: (**a**) Installation of an individual bar in an open joint; (**b**) View of the façade after completion of the repointing.



(**a**) (**b**)

Successful application of FRP bars for external post-tensioning has also been implemented (Figure 27) even though the complexity of developing efficient and inexpensive anchors acts as a deterrent to extensive deployment as for the case of pre-tensioning of externally bonded FRP.

Figure 27. External post-tensioning with carbon FRP tendons.



An FRP nail is a fiber strand that is fully pre-impregnated with resin and cured for approximately half of its length, with the other half left dry (not impregnated). The cured (rigid) portion of the nail is then inserted in a predrilled hole filled with additional fresh resin, while the dry fibers are opened up in a fan-like fashion and bonded to the substrate similarly to manual lay-up (Figure 28a). FRP nails have been successfully used to create reinforcement continuity at connecting members and also to improve the anchorage of externally bonded sheet. Similarly, NSM FRP bars have been successfully used to create continuity and bending moment transfer in particular (Figure 28b). For the case shown in the figure, a quasi-vertical hole is drilled into the foundation while the concrete wall is grooved so that, after installation, the FRP bar is fully encased.

Figure 28. Reinforcement continuity at column/wall to footing connection: (**a**) FRP nail; (**b**) NSM FRP bar.

(**a**)

(**b**)

3.2. Fiber-Reinforced Cementitious Matrix (FRCM) Technologies

Fiber (dry or pre-impregnated) fabrics are being used in conjunction with cementitious (hydraulic or non-hydraulic) mortars for externally strengthening concrete and masonry members. If compared to externally bonded FRP installed by manual lay-up, this technology uses the open mesh fabric to replace the continuous fiber plies as reinforcement, and the cementitious material to replace the organic resin as matrix. This class of material systems has been deployed under different acronyms with the most common ones being TRC for textile reinforced concrete [17] or FRCM for fiber-reinforced cementitious matrix [18].

The challenge and appeal of FRCM systems is in the use of a matrix that is more compatible with the concrete or masonry substrate, but is at the same time brittle and with a strain to failure much lower than that of the fabric reinforcement. From an innovation standpoint, the commercial availability of nano-materials as modifiers of the mortar matrix is the new research frontier. Of particular interest is the use of FRCM to strengthen masonry structures [19,20] (Figure 29).

Figure 29. Masonry surface: (**a**) Before; and (**b**) during strengthening.

(**a**) (**b**)

3.3. FRC Technologies

Randomly distributed fibers can be added to a mortar or concrete to create what is known as FRC (fiber-reinforced concrete). This is the oldest and most investigated among the composite technologies for construction. It has proven very effective in applications such as slabs-on-grade and tunnel linings, but new developments make it particularly attractive, and viable, for the repair and rehabilitation of concrete or masonry when the delivery method is shotcrete. FRC shotcrete may be a superior technology if the substrate surface, after removal of loose or deteriorate portions, is particularly rough such as in the case when the reinforcement is fully exposed (Figure 30a). Another suitable application is in the case of basement walls, either constructed with inadequate building materials such as stone or rubble, or that have suffered extensive damage (Figure 30b,c).

Figure 30. Shotcrete repair: (**a**) Concrete wall in a factory; (**b**) Clay masonry walls in a basement before repair; (**c**) After repair.

(**a**) (**b**)

Figure 30. *Cont.*

(**c**)

4. Conclusions

The 2012 Emilia Romagna earthquakes have highlighted the vulnerability of not only historical structures, but also of modern URM construction and industrial precast RC buildings. The latter structures in particular were designed to resist only gravity loads without any detailing provision for wind and earthquake resistance. As a result, precast RC industrial buildings suffered heavy damage, resulting in casualties and significant economic losses. Such structures had the following deficiencies: (a) absent/ineffective secondary beam-primary beam and beam-column connections, which caused partially constrained systems and hence the loss of support; (b) absent/ineffective connections between façade precast panels and structural elements, which induced out-of-plane collapse mechanisms; and (c) heavy nonstructural masses, which caused severe seismic demands on structures.

The Emilia Romagna earthquakes also confirmed how vulnerable some of the typical elements of historical and modern masonry structures are, such as bell and clock towers, tympanums and vaults of churches, irregular walls with openings, and load-bearing walls without RC tie beams or steel ties at floor levels.

The observations on damage and deficiencies of existing buildings indicate that some retrofit interventions could be carried out using FRP, FRCM and FRC technologies, which have demonstrated effectiveness and adaptability to repair situations.

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