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Emilia Earthquake: the Seismic Performance of Precast RC Buildings

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On 20th and 29th May 2012 two earthquakes of magnitude 5.9 and 5.8 (M_W) occurred in the Emilia region (Northern Italy), one of the most developed industrial centers of the country. A complete photographic report collected in the epicentral zone shows the seismic vulnerability of precast structures, the damage to which is mainly caused by connection systems. Indeed, the main recorded damage is either the loss of support of structural horizontal elements, due to the failure of friction beam-to-column and roof-to-beam connections, or the collapse of the cladding panels, due to the failure of the panel-to-structure connections. The damage can be explained by the intensity of the recorded seismic event and by the exclusion of the epicentral region from the seismic areas recognized by the Italian building code up to 2003. Simple considerations related to the recorded acceleration spectra allow motivating the extensive damage due to the loss of support.

INTRODUCTION

On 20th May 2012 at 02:03:52 a.m. UTC, a 5.9 moment magnitude M_W earthquake occurred in Emilia region (Northern Italy), causing 7 casualties, about 50 injured and 5000 homeless people. The epicenter of the earthquake was located at Finale Emilia (Modena, Northern Italy). A series of after-shocks occurred in the area on the following days until a second main shock of 5.8 moment magnitude struck the same zones on 29th May, 2012, with an epicenter located at Medolla (Modena, Northern Italy), 20 km west from Finale Emilia. It occurred at 09:00:03 a.m. (local time), when the daily activities were starting again, and caused further 20 casualties, about 350 injured and raised the number of homeless from 5000

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to 15000. Besides the loss in human lives, significant damage was mainly recorded in historical masonry buildings and precast industrial buildings.

The earthquake is surely the most reliable test the structures may be subjected to, in order to evaluate their seismic vulnerability. This is the reason why after a strong motion a series of interesting studies are carried out to examine the structural behavior of different building typologies under seismic actions and to test the validity of seismic codes in force.

In Sezen et al. (2000) and in Saatcioglu et al. (2001) the damage to buildings, bridges, industrial facilities and lifeline infrastructures affected by the 1999 Izmit earthquake (Turkey) are studied. The main precast structural typologies in Turkey have overall geometry similar to those used in Italy, but provide pin connections typically composed of steel dowels. The failure of different precast structures is presented; the main reasons of the exhibited poor performance are: (a) the inadequate beam-to-column connections, (b) the lack of transverse reinforcement in the column and beam corbels close to the beam-to-column connections, (c) the inadequate confinement provided at the base of the columns and (d) the interaction with partial height masonry infills.

L'Aquila earthquake (Central Italy) in 2009 caused loss in human lives and widespread damage to the buildings and infrastructures, motivating several scientific studies. In Toniolo and Colombo (2012) the behavior of precast concrete structures affected by the 2009 L'Aquila earthquake is discussed. Besides presenting some structural damage and beam-to-column connections failure, the study focuses on the influence that cladding panels have on the seismic performance of precast structures. The study is motivated by the collapse of several heavy precast panels due to the inadequacy of the panel-to-structure connections. For this reason, possible alternative solutions of cladding-to-panel connections are indicated and their influence on the seismic behavior of one-storey precast buildings is presented.

The 20th and 29th May Emilia earthquakes caused damage mainly to industrial precast structures with a huge economic loss: it has been roughly estimated that the direct economic damage amounts to about 1 billion euros, while the induced economic damage, e.g. the loss due to the industrial production interruption, amounts to about 5 billion euros. The large economic loss compared to the intensity of the event is basically due to the conjunction of two factors:

- the high percentage of industrial precast buildings in the struck area;

- the vulnerability of the mentioned precast buildings.

This paper focuses on the behavior exhibited by precast structures in the municipalities hit by the earthquakes. A description of the typical Italian precast structures is provided, as well as an account of the evolution of Italian building code for precast buildings. A photographic documentation collected in the first days after the mainshocks is presented in order to describe the damage to, and the seismic performance of, the precast structural typology. Furthermore an attempt to identify the main causes of the damage is provided through the analysis of the recorded accelerograms.

PRECAST STRUCTURES DESCRIPTION AND DESIGN CONSIDERATIONS

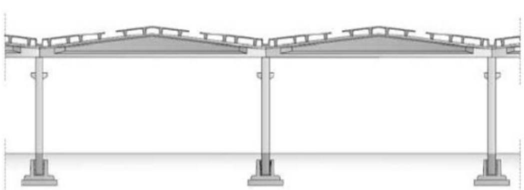
Since the end of the Second World War, precast structures have been widely used in Italy due to the several advantages of serial production of structural elements. Precast elements, produced in factories, are characterized by a more precise control, a better quality and a faster construction time than the cast-in-situ RC elements. In Italy, precast structures are mainly used in the industrial field, where buildings require wide space, i.e. large bays, and very regular plants, e.g. square or rectangular shape. Precast buildings can be classified according to different variables: the structural typology, the number of stories and the roof type. Three main structural typologies can be distinguished: panel structures, column structures and mixed structures. Depending on the number of stories, precast structures can be single-story "industrial" buildings (Figure 1a) and multi-story buildings (Figure 1b). Referring to the roof type, roof elements supported by beams with variable section (Figure 2a), continuous plane roof (Figure 2b), discontinuous plane roof (Figure 3a) and shed roof (Figure 3b) can be found.

In Italy the most common precast buildings are column structures: they consist of socket footing foundations in which precast columns are placed and fixed in-situ by cement mortar; the columns support pre-stressed precast beams that can have different shapes. The most frequent beam cross sections are "T" or "I" section for beams with variable section, and "Y", "H", "L" or rectangular section for plane beams. Reticular beams are also used, especially for very large spans. The main beams support roof elements: in multi-story buildings a cast in-situ slab is provided to cover corrugated elements of intermediate decks; in single-story buildings, instead, a concrete slab is rarely used. Continuous or discontinuous roof elements solutions can be defined: in the first case, tiles are put side by side (Figure 2), in the second case tiles are spaced and alternated by light elements like translucent sheets (Figure 3a) or sandwich panels. An alternative solution is represented by a shed roof: it can be built using reticular beams or discontinuous beams, known in Italy as “knee beams” (Figure 3b), or using inclined beams supported at two different levels. Precast structures are generally completed by precast panels placed along the perimeter that can be inserted between columns or placed externally to the main structure. Infill systems can provide different solutions: horizontal precast panels connected to columns, vertical precast panels attached to horizontal beams and mixed solution including horizontal and vertical panels are all used. A more detailed list of precast structures typologies is provided by Bonfanti et al. (2008).

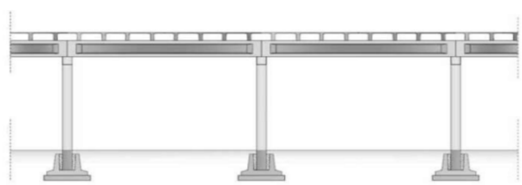
(a)

(b)

Figure 1. Examples of (a) single-story and (b) multi-story precast buildings

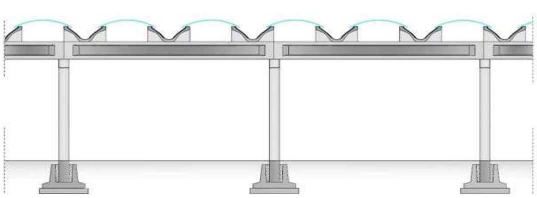


(a)

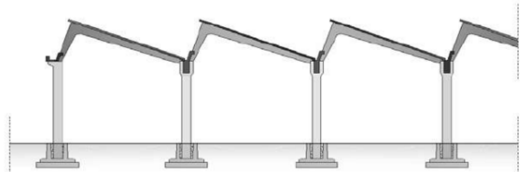


(b)

Figure 2. (a) Double slope roof with corrugated tiles and (b) continuous plane roof (Bonfanti et al. 2008).



(a)



(b)

Figure 3. (a) Discontinuous plane roof and (b) shed roof (Bonfanti et al. 2008).

The most crucial aspect of precast structures regards the connections between structural elements. The connections are made in-situ and executed in order to reflect the calculation model assumed in the design phase. Typical connections include:

1. floor or roof adjacent elements connection;
2. roof element-to-beam connection;
3. beam-to-column connection;
4. column-to-foundation connection;
5. cladding panel-to-structural element connection.

The roof adjacent elements connections are generally made of steel angles and plates welded or bolted in order to ensure the slab continuity (Figure 2b).

The roof element-to-beam connections can be provided in different ways. The most common connection type provides a neoprene pad at the interface between the beam and the roof element, resulting in a friction connection. Another solution consists of steel angles bolted both to the roof element and to the beam defining a fixed connection (Figure 4a). A fixed connection is also given by the presence of a dowel, inserted in the roof element and in the beam.

A beam-to-column connection can be a friction connection or a dowel connection. The former type is very common in existing precast structures and generally consists of neoprene pad at the beam-to-column interface without providing any mechanical connectivity. It relies on friction for absorbing resisting forces. In the latter type a steel dowel is inserted inside the column and anchored in predefined vertical holes in the beam (Figure 4b); the connection requires a final grout casting. This solution defines a hinged support in the longitudinal direction of the beam.

The most common column-to-foundation connection is the socket foundation (Figure 4c). This typology is characterized by a RC hollow core body in which the column is inserted. Concrete or special mortar is poured to fill the gap between the column and the hollow core body of the socket foundation. The socket foundation is generally modeled as a rigid connection, due to the study performed by Osanai et al (1996), in which it is concluded that the connection is rigid if the column embedment depth is larger than 1.5 times the depth of the column cross section.

Connections between cladding panels and structural elements (Figure 4d) can provide different solutions, based on steel connectors such as channel bars, fasteners, angles, brackets, etc.

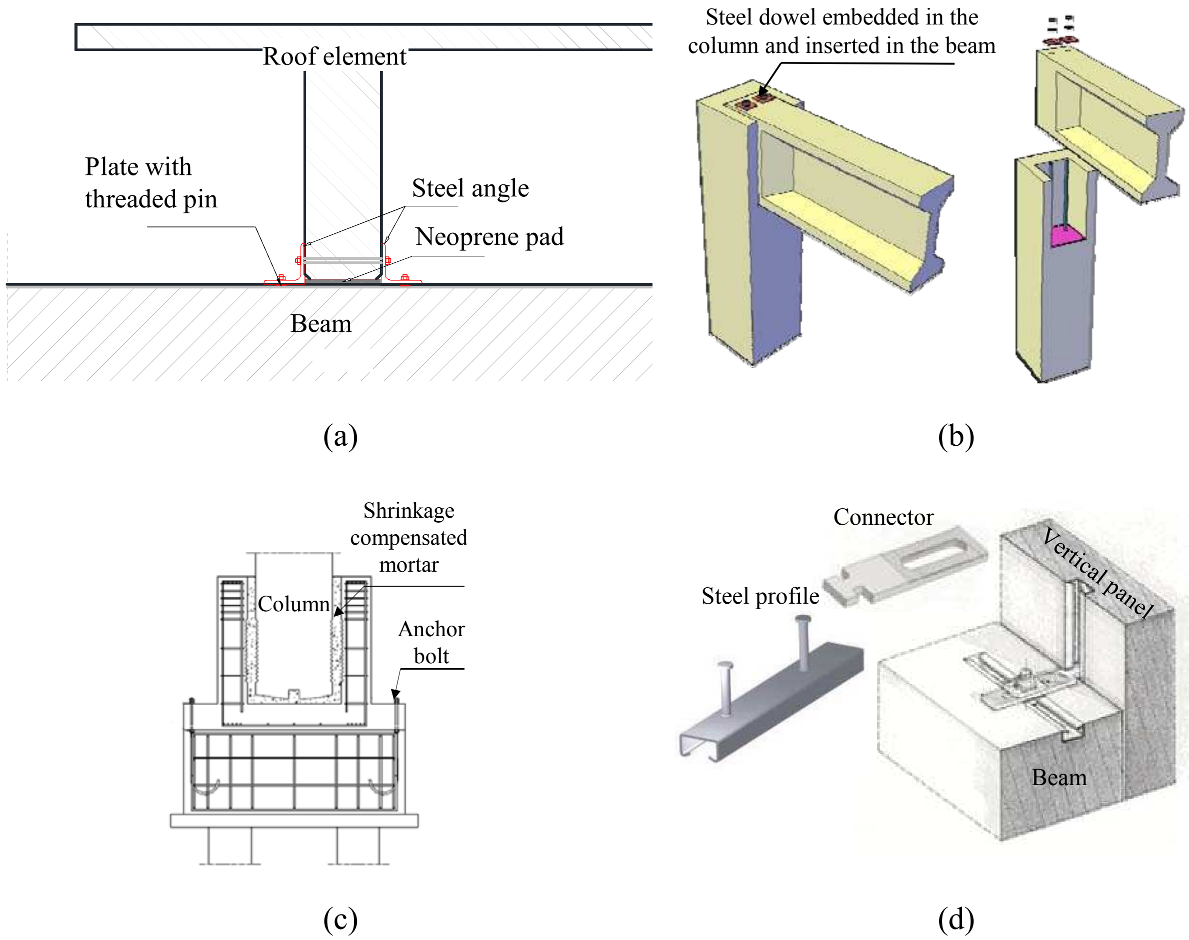


Figure 4. Examples of connections in precast structures: (a) pin roof element-to-beam connection; (b) dowel beam-to-column connection; (c) socket column-to-foundation connection; (d) vertical panel-to-beam connection.

A detailed list of connections in precast structures used both in Italy and Europe is provided by Mandelli et al. (2007).

The area struck by the Emilia earthquakes is characterized by a high density of precast structures. Indeed, referring to 2001 data of Italian National Institute of Statistic (Istituto Nazionale di Statistica, ISTAT), the percentage of commercial, industrial, transport, communication, office and hotel buildings in the whole Italy is 3.65%, which, with a good approximation, are precast structures. Considering the area hit by the seismic events, e.g. Medolla, Mirandola and San Felice sul Panaro, this percentage increases up to 9%; this illustrates both the high incidence of precast buildings and the large influence of the vulnerability of this structural typology on the global seismic risk of the area.

In order to give an idea of the vulnerability of precast concrete buildings, a brief overview of the code evolution is given in the following, focusing the attention on the code provisions regulating the design of elements and connections in precast structures (Table 1).

Legge 1684 (1962) and its integration (Legge 1224, 1964) only specify the horizontal actions to consider in seismic zones in Italy without any particular requirement for precast structures. A noteworthy code is published in 1965 (Circ. M. LL.PP. n.1422 1965), that forbids the use of horizontal joints without mechanical devices if the ratio T/N was larger than 0.35, where T is the maximum value of the shear force, N is the expected axial compression force and, implicitly, 0.35 is the friction coefficient of the connection.

In 1974, the code (Legge 64 1974) introduces specific indications for the seismic design of structures. However, concerning precast structures, the code gives only a few general indications and these are for load-bearing precast panels structures.

The first specific regulations for precast structures are in the DM 3/12/1987, that already point out the role of the connections, considering also the transition phases of the construction. The requirements for the structural elements and for the connections design are still limited; it is forbidden in seismic zones to use beam–column connections that transfer horizontal forces by friction alone. The only prescriptive provision is given for the width of the beam-to-column support: “For the beams, the end support must be not smaller than $8cm + l / 300$, where l is the clear beam span in centimeters”.

More detailed suggestions on precast structures are given in OPCM 3274 (2003). According to the Italian government, the application of this code is compulsory only in the case of infrastructure and strategic buildings. Multi-story framed structures and single-story structures with isostatic columns are taken into consideration, according to the number of stories and the capability of the connections in transferring bending moments. A specific behavior factor, i.e. 5.0 and 3.75 respectively, is assigned to the two structural typologies. Moreover it is recognized the significant influence of the connections on the static and dynamic behavior of the whole structure. In the case of framed structures, the codes distinguished three possible conditions:

a. connections located well outside critical regions not affecting the energy dissipation capacity of the structure;
b. connections located within critical regions but adequately over-designed with respect to the rest of the structure, so that in the seismic design situation they remain elastic while inelastic response occurs in other critical regions;
c. connections located within critical regions properly designed in terms of strength, ductility and quantity of energy to dissipate.

For single-story structures with isostatic columns, the beam-column connections may be fixed or free to slide horizontally. The connections must transfer the seismic design horizontal forces, without taking into account the friction strength. For the fixed connection the capacity design approach is considered, i.e. its strength must be larger than the horizontal force that produces the ultimate resistant bending moment at the base of the column.

In Europe the precast concrete structures are regulated by the EC8 (CEN 2003), which underlines the importance of the connections. It is required that friction resistance should be neglected in evaluating the resistance of a connection both for the beam-to-column connections and for the primary seismic elements-to-diaphragm horizontal joints. However, it should be underlined that the EC8 is not compulsory in Italy. Concerning the structural typologies, the following systems are considered for precast concrete structure: (a) frame structures, (b) wall structures, (c) dual structures (mixed precast frames and precast or monolithic walls), (d) wall panel structures (cross wall structures) and (e) cell structures (precast monolithic room cell systems). The behavior factor for one-story framed systems ranges from a maximum of 4.95 to a minimum of 1.65 that corresponds to connections not regulated by the code.

The current Italian code (DM 14/01/2008) gives more attention to precast structures than do the past Italian codes. It takes the main framework of OPCM 3431, adopting some provisions of EC8. Concerning the precast column systems, the two structural categories defined in OPCM 3431 are provided, i.e. framed structures and isostatic column structures: the former include structures with continuous or hinged joints, the latter concern one-story buildings with beams hinged at one side and with a sliding support at the other one. Furthermore, the connections have to transfer the horizontal forces under the design seismic load without taking into account the friction strength; this last rule also applies to roof-to-beam connections. The code forces a reduction of 50% of the behavior factor, if some of the specific requirements concerning the connections are not followed.

Table 1. Italian building code evolution: title, acronym, presence of requirements on precast structures and on connections between structural elements, compulsoriness and relationships between the most important codes for precast structures.

Code	Acronym	Precast structures requirements	Friction connection forbidden	Compulsoriness
Legge 25 novembre 1962, n. 1684	Legge 1684	No	-	Yes
Legge 5 novembre 1964, n. 1224	Legge 1224	No	-	Yes, integrates Legge 1684
Circolare del Ministero dei Lavori Pubblici n.1422 del 6 febbraio 1965	Circ. M. LL.PP. n.1422	No	Yes, if T/N > 0.35	Yes, integrates Legge 1224
Legge 2 febbraio 1974, n. 64	Legge 64	Yes	-	Yes, replaces previous codes
Decreto Ministeriale del 3/12/1987	DM 3/12/1987	Yes	In seismic zone	Yes, integrates Legge 64
Ordinanza del Presidente del Consiglio dei Ministri n. 3274 del 30/3/2003	OPCM 3274	Yes	Yes	Yes, only for infrastructures and strategic buildings
Eurocode 8	EC8	Yes	Yes	No
Decreto Ministeriale del 14/01/2008	DM 14/01/2008	Yes	Yes	Yes, integrates Legge 64 and replaces previous integrations

PRECAST STRUCTURES DAMAGE OBSERVATION IN EMILIA REGION

The commercial and industrial precast structures are the structural typology that suffered the most damage during the Emilia seismic events. Indeed, a direct inspection of the epicentral industrial zones in the days after the two mainshocks highlighted that more than a half of the existing precast structures exhibited significant damage. Moreover, the collapse of many non-structural components (Magliulo et al. 2012b, Magliulo et al. 2012c), such as internal partitions, ceilings and high-rack steel structures is recorded. The high-rack steel structures are widely used in industries in order to store various kinds of goods and, in presence of seismic events, they can be subjected to significant horizontal loads, as addressed by Kilar et al. (2011).

In this section the structural and non-structural damages, that occurred in precast structures during the Emilia earthquakes, are presented by a photographic documentation (Ercolino et al. 2012a, Ercolino et al. 2012b).

DAMAGE TO CONNECTIONS BETWEEN STRUCTURAL COMPONENTS

Most of the damaged precast buildings provides friction connections between horizontal elements (beams and roof elements) or between horizontal (beam) and vertical (columns) members. The lack of connection devices is the main cause of damage in precast structures, in which the low strength given by friction mechanism causes the loss of support of both roof elements from beams and beams from columns. The consequences are disastrous: Figure 5a shows the loss of support of the roof elements from the main beam due to the use of friction connections and a very limited support width. Figure 5b shows the loss of support of a beam from the column and the consequent collapse of the roof elements, causing the failure of the whole structure.

(a) (b)

Figure 5. (a) Roof elements collapse due to the loss of support from main beam. (b) Loss of support of beam from column.

The lack of mechanical devices causes the loss of support of the beams from the columns also in Figure 6a. In other cases the loss of support causes the change of the beam restraints, that let the beam act as a cantilever, and make it collapse under the weight of the roof elements (Figure 6b). The vulnerability recorded in precast structures is certainly larger than the vulnerability exhibited by similar precast structures in Turkey after 1999 Kocaeli earthquake (Saatcioglu et al. 2001, Sezen et al. 2000); the main reason is the common presence of connections relying on friction in Emilia region, contrasting with the doweled connections used in Turkey.

(a) (b)

Figure 6. (a) Loss of support of beam from column. (b) Collapse of main beam due to the loss of support on column.

Some precast structures show the failure of the connections even in cases in which pin beam-to-column connections are used, due to the inadequacy of the connection design. Figure 7 represents a significant example of unsuitable design of the connection. The strength of the pin connection is evaluated in correspondence to the failure of the dowel; instead in Figure 7a, the spalling of concrete cover occurs before the yielding of the dowel, due to the small size of the cover and to the lack of dense stirrups close to the supporting zones. Consequently, it causes the collapse of the beam and roof elements which are supported by the beam (Figure 7b).

(a) (b)

Figure 7. (a) Pinned beam-to-column connection failure and (b) consequent loss of support of the beam from column.

The above presented damage highlights the low robustness of the investigated precast structures under seismic actions: in most cases the collapse of only few (even one) connections can cause the collapse of the whole structure and, consequently, the loss of both life and inventory.

COLUMNS DAMAGE

In Italian precast existing structures, columns are generally precast elements connected at the bottom to a socket foundation and at the top by a horizontally sliding or fixed support to the beams. Therefore the columns can be assumed to act as cantilevers fixed at the base. In presence of strong earthquakes, precast columns show:

- loss of verticality due to a rotation in the foundation element (Figure 8a) caused by a possible inadequate column-to-foundation connection, even if this cause is not easily ascertainable unless a direct inspection of foundation is made;
- plastic hinge development at the column base: Figure 8b shows an incipient plastic hinge evidenced by extensive cracks at the base, and Figure 8c indicates a case of longitudinal bar buckling due to the visible lack of a proper stirrup spacing in the critical zone of the column.
- shear failure due to the interaction with traditional masonry infill systems (Figure 9).

(a) (b) (c)

Figure 8. Damage in columns: (a) column loss of verticality due to rotation in the foundation element; (b) cracking of the base section in a column; (c) plastic hinge at the bottom of the column and buckling of a longitudinal bar at the base.

Figure 9. Shear collapse of column due to the interaction with infill masonry panel.

INFILL PRECAST PANEL COLLAPSE

Precast buildings infill systems in Emilia region are mostly constituted by precast cladding panels. Horizontal (Figure 10a) and vertical (Figure 10b) panels collapse is the most frequent damage in precast buildings.

The causes of collapse can be attributed to:

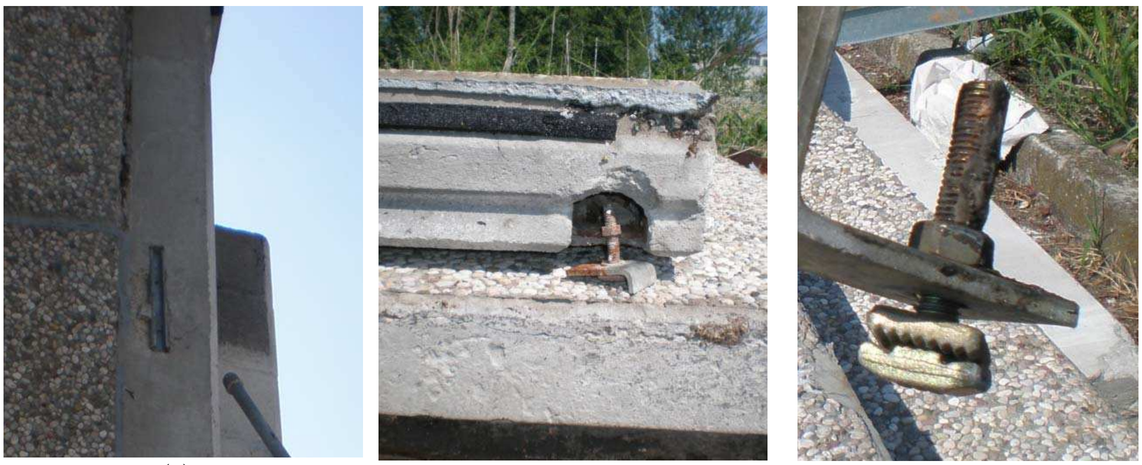
a) the lack of seismic design in cladding panel-to-structural element connection devices;
b) the pounding of roof elements, columns or other precast panels;
c) the panel-to-structure interaction that causes additional lateral forces in the connection devices, not considered during the design process.

(a)

(b)

Figure 10. (a) Collapse of horizontal precast panels. (b) Collapse of vertical precast panels.

Figure 11 shows the collapse of a horizontal panel-to-column connection due to the failure of the anchor channel embedded in the column (Figure 11a) and the shear failure of the steel angle plate that joints the panel to the structure (Figure 11b); details of the hammer head screw can be observed in Figure 11c. Figure 12 shows the collapse of a vertical panel-to-beam connection: in this case a particular connection device is used, i.e. a steel profile is embedded in the beam (highlighted in Figure 12b) and some hammer head elements are welded to the profile and inserted into the anchor channel of the vertical precast panels (Figure 12a). Under the seismic action the screw-to-profile welding fails and causes the collapse of the panels.



(a) (b) (c)

Figure 11. Details of a connection device at the top of horizontal panel: (a) anchor channel embedded in the column, (b) steel angle plate and (c) hammer head screw out of the anchor channel.

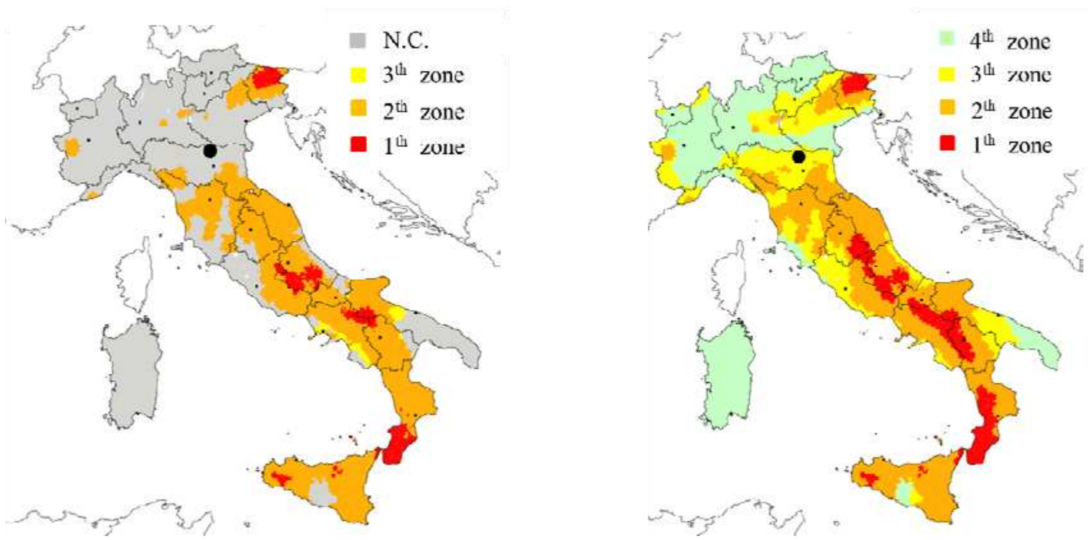
(a) (b)

Figure 12. Collapse of vertical precast panel connection: (a) the anchor channel embedded in the panel and the failed hammer head element; (b) profile located in the beam which the hammer head elements are welded to.

SEISMIC ACTION AND CONSIDERATIONS CONCERNING DAMAGE DUE TO THE LOSS OF SUPPORT

In order to understand the damage recorded after the Emilia earthquakes, a description of the Italian seismic zones is presented.

The definition of seismic zones in Italy started in 1909 following the Reggio Calabria and Messina Earthquake in 1908 that causes about 80.000 casualties. The regions in Southern Italy that suffered from this earthquake were defined seismic zones. Since then, the map has been refreshed enlarging the zones defined as "seismic" after each significant Italian earthquake. The Emilia region that was struck by the recent earthquakes (black dot in Figure 13) was still out of the seismic zones in the 1984 map (Figure 13a). Finally, in 2003 the whole Italian territory was classified as seismic (Figure 13b), distinguishing four seismic zones: zone 1, 2, 3 and 4, corresponding to design peak ground acceleration at the bedrock equal to 0.35g, 0.25g, 0.15g and 0.05g, respectively. The region close to the epicenter of Emilia earthquakes was inserted in the 3rd zone.



(a) (b)

Figure 13. Seismic zone classification in Italy (a) in 1984 and (b) in 2003; the black dot indicates the Emilia earthquake epicentral zone (INGV 2012).

Hence, it is expected that all structural typologies in Emilia region, designed up to 2003, do not take into account seismic design at all, increasing the seismic vulnerability of structures built in that region. In particular, precast structures built up to 2003 typically provide beam-to-column friction connections because friction connections were forbidden only in seismic zones since 1987 (Table 1).

Lastly, the current Italian code (DM 14/01/2008) defines hazard parameters continuously for the whole national territory, without distinguishing different seismic zones. In particular, for Mirandola (Modena, Italy) the PGA with a 10% probability of exceedance in 50 years is equal to 0.140g.

The acceleration time histories recorded (Figure 14) by the station MRN of the Italian National Accelerometric Network yields a maximum acceleration equal to 0.264g and 0.261g for the N-S and E-W components, respectively; the spectral ordinates reach values up to 1g (Figure 15). It should be noted that the recorded accelerograms include seismic site effects; indeed, MRN station is placed on a “C” class soil site (shear wave velocity ranging from 180m/s to 360m/s), based on geological data, and T1 category according to EC8 (flat surface), as reported in the *Italian Accelerometric Archive* (Luzi et al. 2008).

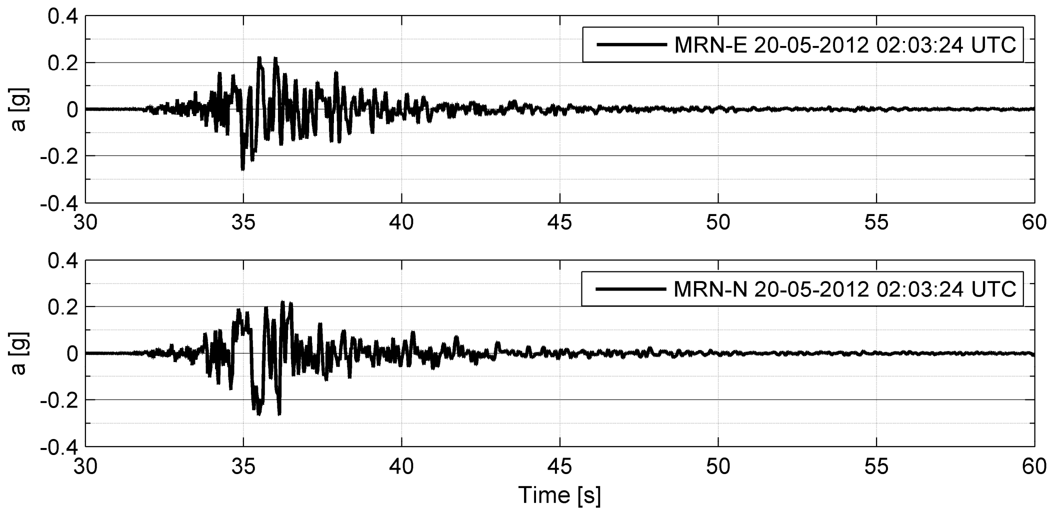


Figure 14. Accelerograms recorded in the station of Mirandola (Modena, Italy) (the origin of time is set at 20-05-2012 02:03:24 UTC).

In Figure 15 the recorded spectra are compared with the design spectra in the epicentral zone for return periods equal to 475 and 2475 years (C soil and T1 surface). The comparison demonstrates the rarity of the event, according to the actual Italian seismic hazard maps and the historical data they are based on; the NS component spectrum is generally included between the two considered design spectra for low period range, i.e. before 0.6sec, and it exceeds the spectrum with the higher return period for high period range, i.e. beyond 0.6sec.

In order to establish the spectral accelerations in the precast structures during the investigated Emilia seismic events, two period ranges can be distinguished in the spectrum of Figure 15, according to the extensive parametric study provided by Magliulo et al. (2013) on single-story precast structures designed according to the current Italian code in low-to-high seismic zones. The bare precast structures exhibit an elastic fundamental period ranging from 0.54sec to 1.45sec, while infilled precast structures range from 0.09sec to 0.40sec, due to the presence of cladding panels. No significant difference between the spectral ordinates for bare and infilled structures for NS-component is evidenced; on the contrary, in the case of EW component, the 0.09sec-0.40sec range provides larger spectral ordinates.

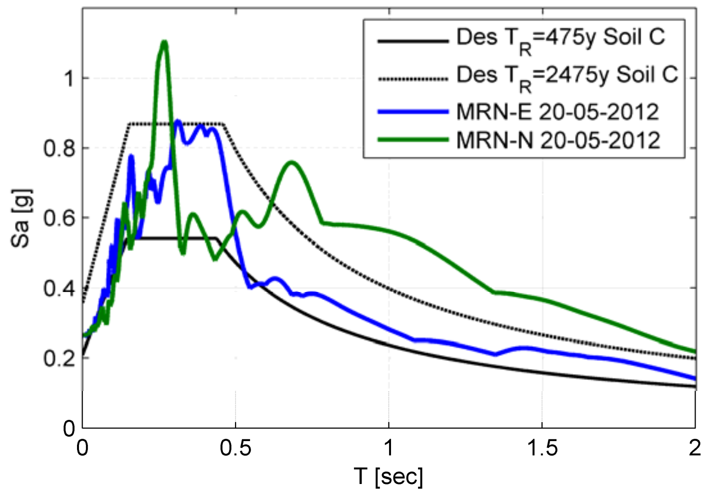


Figure 15. Elastic response spectra recorded on 20 May 2012 in Mirandola NS (green) and EW component (blue) compared to elastic response spectra for return period equal to 475y (black) and 2475y (dashed black) provided by Italian building code (DM 14/01/2008) for soil class C. A damping ratio equal to 5% is assumed.

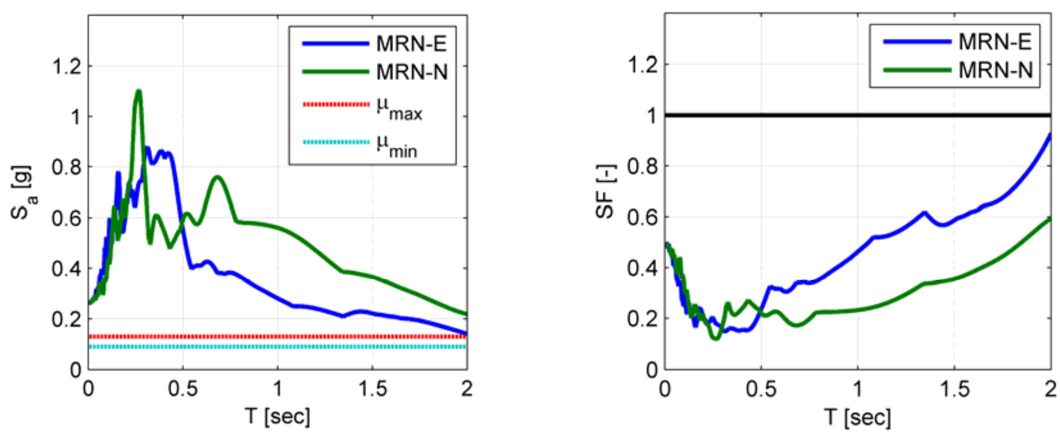
In the previous section it has been highlighted that loss of support has been the main cause of collapse in precast structures in Emilia region. This can be deduced also upon simple considerations on the recorded spectra (Figure 15). Assuming that the rigid diaphragm is not ensured, as commonly found in Emilia region precast buildings, the total seismic force F_{tot} is divided among the columns using a criterion based on influence area, i.e. proportionally to the ratio between the dead loads W_i acting on the column and the total weight of the structure W_{tot}. Considering that the participating mass ratio is 100% for the translational modes, the seismic force V_{Ed} acting on a connection can be evaluated as follows:

$$V_{Ed} = F_{tot} \cdot \frac{W_i}{W_{tot}} = W_i \cdot S_a\left(T_1\right)/g \tag{1}$$

The strength of a friction connection V_{Rd} can be evaluated multiplying the vertical force acting on the connection and the friction coefficient μ. Based on these considerations, the loss of support mechanism is immediately checked comparing the friction coefficient with the acceleration spectral ordinates in g, as shown in Figure 16. Indeed, a safety factor SF can be evaluated and plotted (Figure 16b) versus the fundamental period for the recorded spectra.

$$V_{Rd} = \mu \cdot W_i \Rightarrow SF = V_{Rd} / V_{Ed} = {}^{\mu}\!\big/_{S_a\left(T_1\right)/g} \tag{2}$$

According to the experimental studies conducted by Magliulo et al. (2011) on neoprene-to-concrete connections, the friction coefficient varies in the range 0.09 ÷ 0.13 for compressive stress varying between 1.7 MPa and 5.3 MPa. In Figure 16a these limits are compared to the recorded spectral ordinates. Figure 16b shows the safety factor SF, evaluated considering μ equal to 0.13. The safety factor SF is much below 1 for a wide range of periods and confirms the vulnerability recorded in friction connection of precast structures in Emilia region.



(a) (b)

Figure 16. (a) Acceleration spectral ordinates recorded in Mirandola compared to the friction coefficient upper and lower bounds evaluated by Magliulo et al. (2011); (b) loss of support safety factor plotted versus fundamental periods for the recorded accelerograms, assuming μ=0.13.

It should be noted that the simple considerations above presented neglect both the vertical component of the seismic action and the bi-directionality of the input motion. Obviously, if the two phenomena had been taken into account, lower safety factors would have been found. Even in case larger friction coefficients had been considered, e.g. Caltrans (1994) suggests a coefficient ranging from 0.2 to 0.4 in case of neoprene/concrete interface for bridge applications, the loss of support would not have been avoided for a wide range of structural periods.

The use of an unreduced elastic spectrum for the evaluation of the force acting on beam-to-column friction connections may be questioned, since precast structures may dissipate energy inelastically. However, inelastic action in the concrete elements will not occur if the frictional strength of the connection is lower than the plastic shear, i.e. the force that causes the formation of the plastic hinge at the column base. Indeed, in this case no plastic sources are exploited and, hence, the unreduced elastic spectrum must be used for the evaluation of the seismic actions.

It is concluded that, if the shear failure of the connection comes before the flexural hinging in the column, precast structures with neoprene-concrete friction connections will exhibit a loss of support of their horizontal elements under the recorded seismic excitation. Magliulo et al. (2008) anticipated this evidence, demonstrating that precast structures with friction connections suffer from loss of support due to the sliding of the beam from the column. This statement is based on nonlinear dynamic analyses, performed on space models subjected to the three components of an earthquake (Magliulo and Ramasco 2007, Magliulo et al. 2007, Magliulo et al. 2012a, Maddaloni et al. 2012) typical of an Italian medium seismicity zone.

The mainshock occurred on the 29th May 2012 is well recorded, due to the installation of temporary seismic stations around the epicentral area. The considerations above presented for the 20th May mainshock can be extended to the 29th May mainshock as well, based upon the horizontal acceleration spectra recorded in the area close to the epicenter (Figure 17).

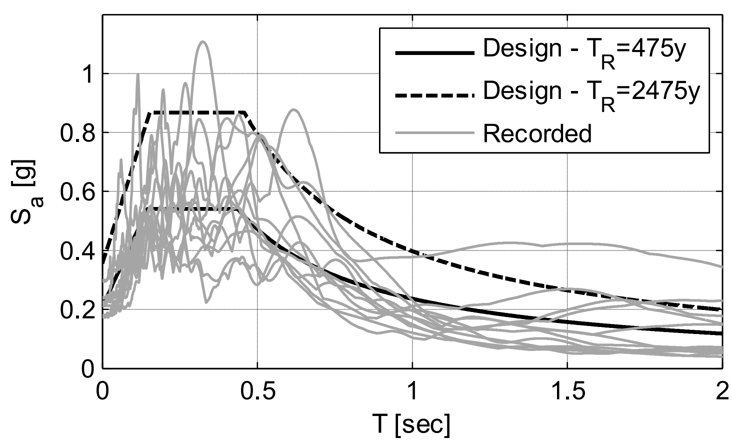


Figure 17. Elastic response spectra recorded on 29 May 2012 in Cento (CNT), Finale Emilia (FIN0), Moglia (MOG0), Mirandola (MRN), San Felice sul Panaro (SAN0) and San Martino Spino (SMS0) compared to elastic response spectra for Mirandola for return period equal to 475y (black) and 2475y (dashed black) provided by Italian building code (DM 14/01/2008) for soil class C.

CONSIDERATIONS CONCERNING USE OF FRICTION CONNECTIONS

Precast structures hit by the Emilia earthquake were designed according to different codes, depending on the construction time. Most of the precast structures in Emilia were designed without taking into account seismic forces, based on the above mentioned considerations on the seismic hazard map evolution in Italy; however, horizontal forces, i.e. wind and crane actions, were also considered.

Since the wind horizontal forces imply lateral loads on the connections, the use of friction connections may be questioned. For this reason, a parametric study is carried out in order to justify a similar widespread design choice. The study assumes a one-story precast building located close to the epicenter as a benchmark structure. Some geometrical parameters are considered, resulting in 48 case-studies, i.e. the column height H (7m, 9m, 11m, 15m), the number of the longitudinal bays (4, 6, 8, 10), the width of the two transverse bays B (15m, 19m, 25m).

In the parametric study, the horizontal shear demand in the connections caused by the wind actions is evaluated according to different past Italian codes and compared to the friction strength. In particular, the wind action is evaluated according to CNR Instructions (CNR-UNI 10012 1967) and DM 16/1/1996, as shown in Table 2. The NTC 2008 (DM 14/01/2008) is not taken into account because Emilia region has been a seismic zone since 2003 and, according to the current code, friction connections are forbidden in seismic areas.

Table 2. Evaluation of the wind equivalent forces according to past Italian building codes (CNR-UNI 1967 and DM 1996).

CNR 1967	**p**	N/m^2	wind velocity pressure	$p = c \cdot k \cdot q$
	c	[-]	external exposure and shape coefficient	0.8
	k	[-]	slenderness coefficient	$f(H/(2 \cdot B))$
	q	N/m^2	wind kinetic pressure	600
D.M.1996	**p**	N/m^2	wind velocity pressure	$p = q_{ref} \cdot c_e \cdot c_p \cdot c_d$
	$\mathbf{q_{ref}}$	N/m^2	kinetic pressure	$q_{ref} = v^2_{ref}/1.6$
	$\mathbf{v_{ref}}$	m/sec	wind speed	25
	$\mathbf{c_e}$	[-]	external exposure coefficient	$f(H)$
	$\mathbf{c_p}$	[-]	shape factor (upstream facades)	0.8
	$\mathbf{c_d}$	[-]	dynamic factor	1.0

The ratios between the design shear demand in beam-to-column connection induced by wind and the connection friction strength is evaluated for the different case studies (Figure 18). In particular, the shear demand is evaluated according to CNR Instructions and DM 16/1/1996 and the shear strength is calculated according to friction coefficient equal to 0.35, 0.13 and 0.09. It is found that, if the friction coefficient c = 0.35, recommended by the mentioned Italian code (Circ. M. LL.PP. n.1422 1965), is used, the shear demand will be always much smaller than the capacity. This outcome justifies the use of friction connections in existing structures. Vice versa, if the experimental coefficients proposed by Magliulo et al. (2011) are considered (c = 0.13 ÷ 0.09), the capacity decreases and, in 25% cases, it can be exceeded by the shear demand.

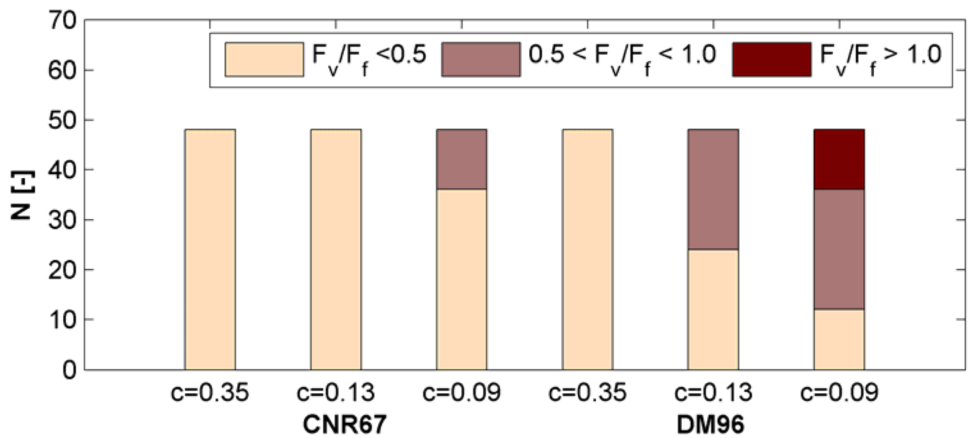


Figure 18. Ratios between the design shear demand F_v in beam-to-column connection induced by wind, evaluated according to CNR 1967 (CNR67) and DM 1996 (DM96), and the connection friction strength F_f, evaluated according friction coefficient c equal to 0.35, 0.13 and 0.09, for the different case studies.

It can be concluded that an unrealistic high friction coefficient for the evaluation of the shear capacity of the connections in the past Italian codes allowed the use of friction connections.

CONCLUSIONS

The 20th and 29th May Emilia earthquakes caused damage mainly to industrial precast structures with a huge economic loss, because of both the high percentage and the vulnerability of the precast buildings in the area. From the study of the precast structures, the review of the past code design provisions and the recorded structural damage, the following conclusions can be drawn.

- A direct inspection of the industrial zones shows that at least half of the industrial precast structures exhibits significant damage and a large number of people suffered death, injury and loss of property. If the first main shock had occurred during the workday, the overall balance would have been even more disastrous.
- The damage to precast structures were caused mainly by inadequate connection systems: the main recorded failures are the loss of support of structural horizontal

elements due to the sliding of friction connections and the collapse of the cladding panels due to the failure of the panel-to-structure connections.

The damage can be explained by two main reasons: (a) the rarity of the event and (b) the exclusion of the epicentral region from the code-recognized seismic areas, which implied that friction connections were acceptable up to 2003.

The poor performance exhibited by the precast structures with friction connections reinforces the belief that connections relying on gravity-only load paths are not acceptable in seismic areas.

- Based on simple considerations on the recorded spectra, it is confirmed that precast structures providing neoprene-concrete friction connections should be expected to suffer from loss of support of their horizontal elements under the recorded seismic excitation.
- The vulnerability of friction beam-to-column connections is also due to the high friction coefficient (c = 0.35) suggested by past Italian codes for the evaluation of the friction strength. If experimental values (Magliulo et al. 2011) had been taken into account (c = 0.13 – 0.09), the use of friction connections would have been limited.

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