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WHY DID MANY PRECAST RC BUILDINGS COLLAPSE DURING THE 2012 EMILIA EARTHQUAKES?

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ABSTRACT

On 20th and 29th May 2012 two earthquakes of magnitude 5.9 and 5.8 (MW) occurred in the Emilia region (Northern Italy), causing 27 casualties, about 400 injured and 15000 homeless people, heavy structural damage to precast and masonry buildings. A complete photographic report collected in the epicentral zone and reported in this paper shows the seismic vulnerability of precast structures.

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. Simple considerations related to the recorded acceleration spectra allow motivating the extensive damage due to the loss of support, beside of the exclusion of the epicentral region from the seismic areas recognized by the Italian building code up to 2003.

INTRODUCTION

On 20th May 2012 at 02:03:52 a.m. UTC, a 5.9 moment magnitude MW earthquake occurred in Emilia region (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 and caused further 20 casualties, about 350 injured and raised the number of homeless from 5000 to 15000. Besides the loss in human lives, the 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 evolution of Italian building code for precast buildings is provided and 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. The post-earthquake activity related to the retrofitting of

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precast structures is also briefly discussed. Same examples of beam-to-column connection improvement are provided in order to avoid the loss of support as well as some retrofitting techniques for panel-to-structure connections.

PRECAST STRUCTURES DESIGN IN ITALY

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. In Italy, precast structures are mainly used in the industrial field, where buildings require wide space and very regular plants.

The most common precast buildings in the area hit by the earthquakes 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 which in turn support roof elements. Most of the damaged precast buildings provides friction connections between horizontal elements (beams and roof elements) or between horizontal (beam) and vertical (columns) members.

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 n. 1684 (1962) and its integration (Legge n. 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 n. 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 (1987), that already point out the role of the connections. The requirements for the structural elements and for the connections design are still limited; it is forbidden in seismic zones to use beam–to-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 n. 3274 (2003), but 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:

- connections located well outside critical regions not affecting the energy dissipation capacity of the structure;
- 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;
- 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 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: frame structures, wall structures, dual structures (mixed precast frames and precast or monolithic walls), wall panel structures and 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, 2008) gives more attention to precast structures than do the past Italian codes. It takes the main framework of OPCM n. 3431 (2005), adopting some provisions of EC8. Concerning the precast column systems, the two structural categories defined in OPCM n. 3431 (2005) 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 infrastractures 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 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.

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 to the main structure and the collapse of many nonstructural components such as internal partitions, ceilings and cladding panels.

In this section the structural and nonstructural damages, that occurred in precast structures during the Emilia earthquakes, are presented by a photographic documentation (Ercolino et al., 2012a; Ercolino et al., 2012b). More details are provided in Magliulo et al. (2013)

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 Fig. 1a 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. Fig. 1b presents 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 1. (a) Roof elements collapse due to the loss of support from main beam. (b) Loss of support of beam from column.

Some precast structures evidence 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. In Fig.2a, 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 (Fig. 2b).

(a) (b)

Figure 2. (a) Dowel beam-to-column connection failure and (b) consequent loss of support of the beam from column.

In presence of strong stresses produced by the earthquakes, precast columns show:

- loss of verticality due to a rotation in the foundation element (Fig. 3a) 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: Fig. 3b shows an incipient plastic hinge evidenced by extensive cracks at the base, and 0c 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 (Fig. 3d).

(a) (b) (c) (d)

Figure 3. 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; (d) shear collapse of column due to the interaction with infill masonry panel

Precast buildings infill systems in Emilia region are mostly constituted by precast cladding panels. Horizontal (Fig. 4a) and vertical (Fig. 4b) panels collapse is the most frequent damage in precast buildings. The causes of collapse can be attributed to several causes like the lack of seismic design in cladding panel-to-structural element connection devices, the pounding of roof elements, columns or other precast panels, besides the panel-to-structure interaction that causes additional lateral forces in the connection devices, not considered during the design process.

(a) (b)

Figure 4. (a) Collapse of horizontal precast panels; (b) collapse of vertical precast panels.

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

In order to understand and motivate the damage recorded after the Emilia earthquakes, a description of the Italian seismic zones is necessary. 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. Since then, the map has been refreshed enlarging the zones defined as “seismic” after each significant Italian earthquake. The Emilia region (black dot in Fig. 5) was still out of the seismic zones in the 1984 map (Fig. 5a). Finally, in 2003 the whole Italian territory was classified as seismic (Fig. 5b), 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. However, this zonation becomes compulsory only on 23th October 2005.

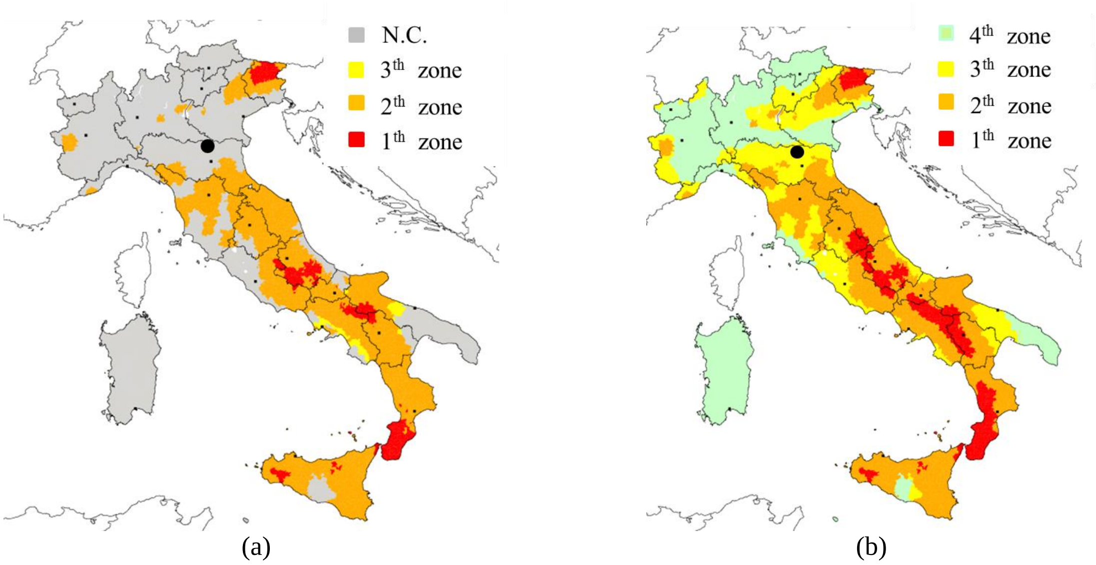


Figure 5. Seismic zone classification in Italy (a) in 1984 and (b) in 2003; the black dot indicates the Emilia earthquake epicentral zone (INGV 2012).

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).

The acceleration time histories recorded (Fig. 6) 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 (Fig. 7). 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).

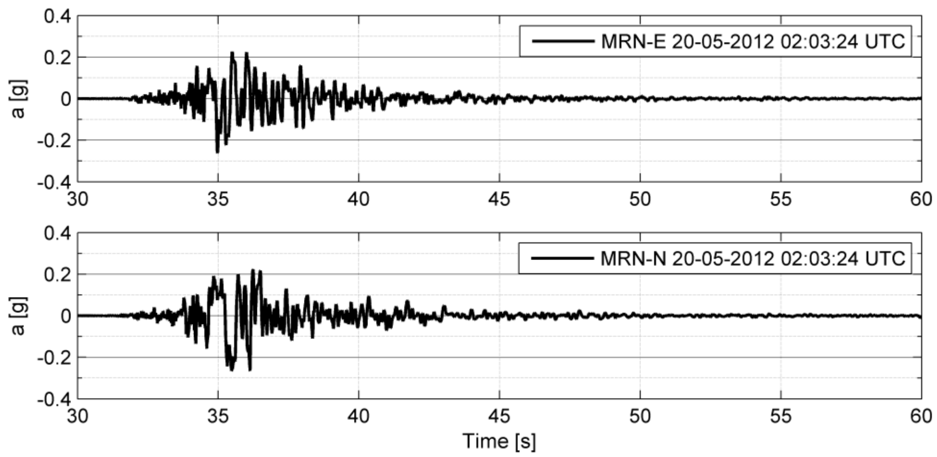


Figure 6. Accelerograms recorded in the station of Mirandola (Modena, Italy) (the origin of time is set at 20-05-2012 02:03:24 UTC).

In Fig. 7 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.

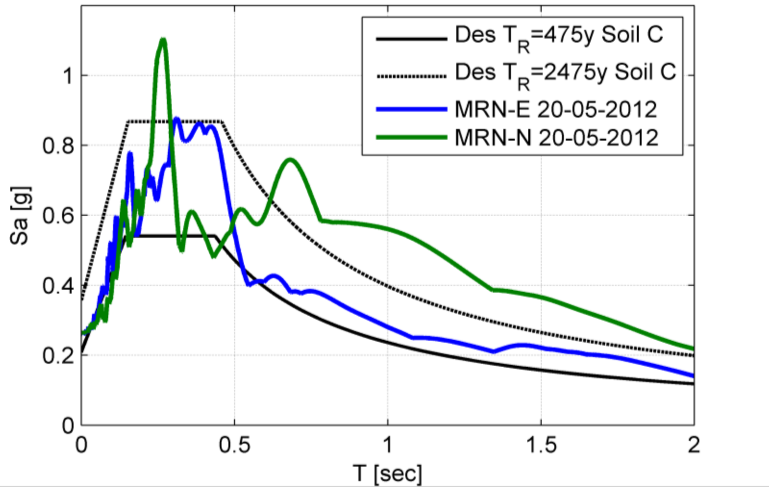


Figure 7. Elastic response spectra recorded on 20 May 2012 in Mirandola NS (green) and EW component (blue) compared to elastic response spectra for return period equal to 475y (black) and 2475y (dashed black) provided by Italian building code (DM 14/01/2008) for soil class C. A damping ratio equal to 5% is assumed.

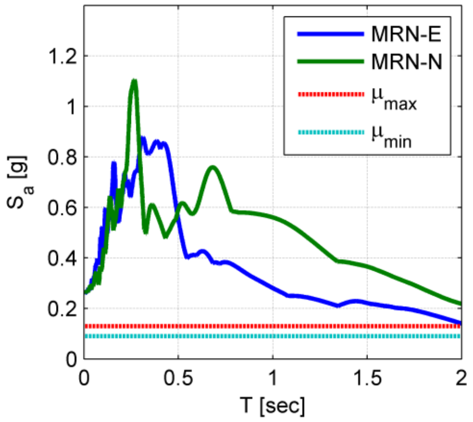
In the previous section it has been highlighted that loss of support has been the main cause of collapse in precast structures in Emilia region. This can be deduced also upon simple considerations on the recorded spectra (Fig. 7). Assuming that the rigid diaphragm is not ensured, as commonly found in 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 in Eq. (1):

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

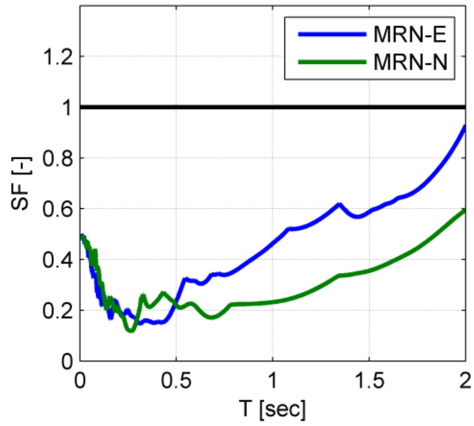
The strength of a friction connection V_{Rd} can be evaluated multiplying the vertical force acting on the connection and the friction coefficient μ. Based on these considerations, the loss of support mechanism is immediately checked comparing the friction coefficient with the acceleration spectral ordinates in g, as reported in Eq (2) and shown in Fig. 8. Indeed, a safety factor *SF* can be evaluated and plotted (Fig. 8b) 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 Fig. 8a these limits are compared to the recorded spectral ordinates. Fig. 8b 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 8. (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 (Maddaloni et al., 2012; Magliulo et al., 2007; Magliulo et al., 2012; Magliulo and Ramasco, 2007) typical of an Italian medium seismicity zone.

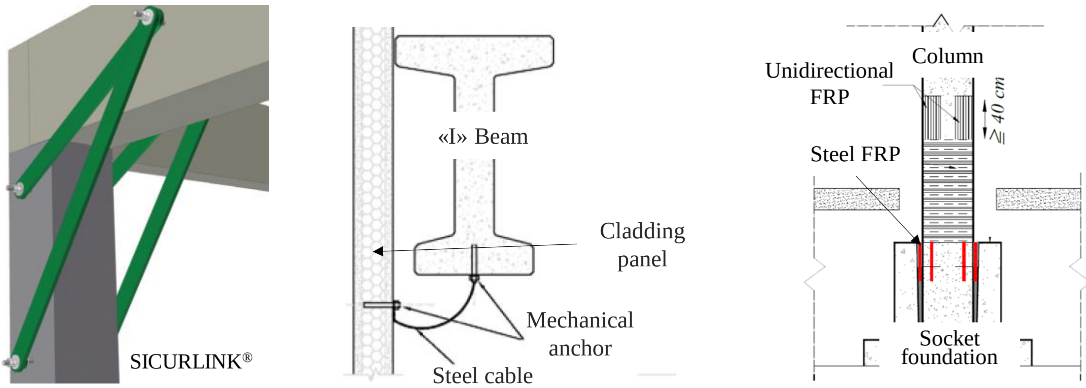
POST EARTHQUAKE RETROFITTING OF PRECAST STRUCTURES

The several collapse evidenced after the 20th and 29th May 2012 Emilia earthquake point out the high vulnerability of precast structures designed with no seismic actions. This aspect is quite significant both for life safe and the economic and social impact due to the large number of industrial building in the hit area. Legge n. 122 (2012), related to the emergency due to the earthquake in Emilia region, outlines the regulatory framework for the retrofitting of the existing structures. It tries to accomplish both the need to protect the human life and the need to reduce the economic and social impact on industries.

For precast structure it is required to follow a procedure, that provides two main phases:

- the first phase, in which the elimination of the most important structural deficiencies must be guaranteed;
- the second phase, in which a series of extensive actions must be provided in order to achieve a given performance level.

The common deficiencies, that must be solved during the first phase, are: (a) the lack of mechanical connection devices between vertical and horizontal elements and between horizontal elements; (b) the presence of cladding panels not adequately connected to the main structure; (c) the presence of not braced storage-rack structures that may involve the main structures in their failure. Some instruction on retrofitting techniques aimed to support the technicians' work are provided in "Linee di indirizzo per interventi locali e globali su edifici industriali monopiano non progettati con criteri antisismici" (Gruppo di Lavoro Agibilità Sismica dei Capannoni Industriali (in Italian), 2012). The Fig.9 shows some solutions: the beam-to-column connections must avoid the loss of support of the beam and should not induce additional bending moment at the column top; the panel-to-structure connection devices should avoid the panel detachment and its overturning. Moreover, some suggestions are provided in order to increase the column strength and stiffness.



(a) (b) (c)

Figure 9. Retrofitting techniques for (a) beam to column connection, (b) cladding panel-to-structure connections and (c) precast column

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.
- 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.
- 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.

ACKNOWLEDGEMENT

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