The Emilia-Romagna earthquake: Damages to precast/prestressed reinforced concrete factories

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– Immediately after the seismic event (started on May 20th, 2012) which struck the Padana Flat area between the municipalities of Ferrara, Modena, Reggio Emilia, Bologna (Emilia-Romagna Region), Mantova (Lombardia Region) and Rovigo (Veneto Region), an ENEA team of experts (Maurizio Indirli, Bruno Carpani, Elena Candigliota, Alessandra Gugliandolo, Francesco Immordino, Giuseppe Marghella, Anna Marzo, Giuseppe Nigliaccio, Alessandro Poggianti, Maria-Anna Segreto) supported the Italian Civil Defense, in order to perform prompt investigations regarding the safety evaluation of different typologies of structures (bridges, industrial factories, residential houses, etc.) made by various kinds of materials (masonry, reinforced concrete, precast/prestressed reinforced concrete, mixed). General information is given in several preliminary reports [among them, Decanini et al., 2012). In particular, this article is devoted to the behaviour analysis of precast/prestressed reinforced concrete (p/p. r.c.) construction, widely used in the affected area since the second half of the last century, trying to deepen some crucial aspects of the matter.

Keywords: precast structures, factories constructions, seismic damages, Emilia Romagna earthquake.

1. A brief summary of damage in p/p. R.c. Structures after previous major earthquakes

Since several decades, p/p. r.c. structures represent a widely used typology for industrial facilities and other kinds of commercial destination all over the world. Although most of them (if well-designed and well-detailed) gave a good performance in case of major earthquakes, they showed a very sensible response when lacks in seismic codes or construction features were evident. Therefore, a short summary of cases with insufficient behaviour is set out hereafter.

For example, after the January 17th, 1994 Northridge earthquake [Los Angeles, California, USA, M_W 6.7, see EERI, 1994], several parking lots suffered widespread damage or collapse. Among others, a typical example was the Cigna Garage, a multi-storey assemblage of precast and in situ cast elements (Fig. 1), located at a distance of approximately 5.5 km from the epicentre (the closest recording station measured peak values of 0.47g, horizontal, and 0.30g, vertical). The building connections failed, due to loss of bearing of supporting elements [Bonacina, Indirli & Negro, 1994]. The Northridge earthquake clearly demonstrated the deficiencies in pre-1971 (i.e. the watershed date of the M_W 6.6 San Fernando seismic event, which struck California in the same year) designed r.c. (including p/p.) structures. A large-scale revision of the code standards was carried out for various types of buildings in 1973, 1975, and after.

The January 17th, 1995 Great Hanshin-Awaji earthquake [Kobe, Japan, M_W 6.9] represented another impressive lesson for seismic engineers [Various Authors, 1995]. Due to the strong ground motion amplification on soft soils, the extensive ground failures (caused by settlement and liquefaction), and the fires after earthquake, the damage to industry resulted very heavy. In the framework of this scenario, p/p. r.c. structures, located in the more affected area, performed "remarkably well" [Muguruma et al., 1995]. In fact, they were newer, high quality, regular shaped construction, designed according to the 1981 Japanese revision of the code requirements (large for r.c. buildings, more limited for steel ones) dating back to 1924, date of the Great Kanto seismic event. On the other hand,

Fig. 1. *Cigna Garage damaged by the 1994 Northridge earthquake (Bonacina, Indirli & Negro, 1994).*
Garage Cigna danneggiato dal terremoto di Northridge del 1994 (Bonacina, Indirli e Negro, 1994).

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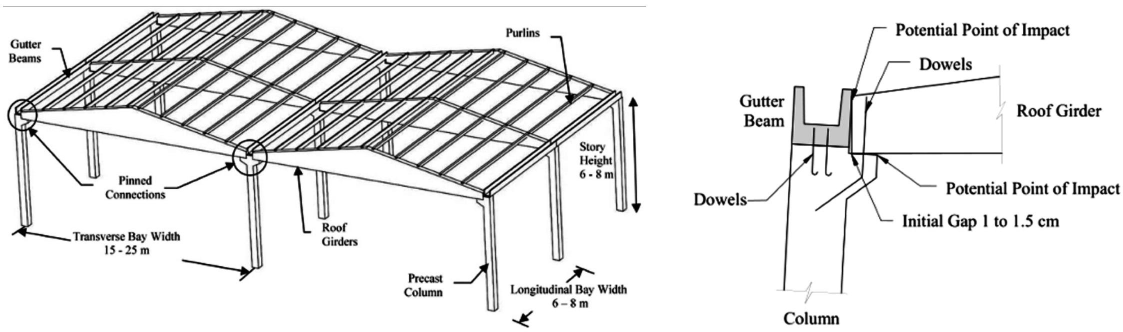

Fig. 2. *A Turkish typical configuration of one storey precast construction near Kocaeli and details (Senel and Kayhan, 2010).*
Tipica configurazione Turca di costruzione prefabbricata ad un piano presso Kocaeli e dettagli (Senel e Kayhan, 2010).

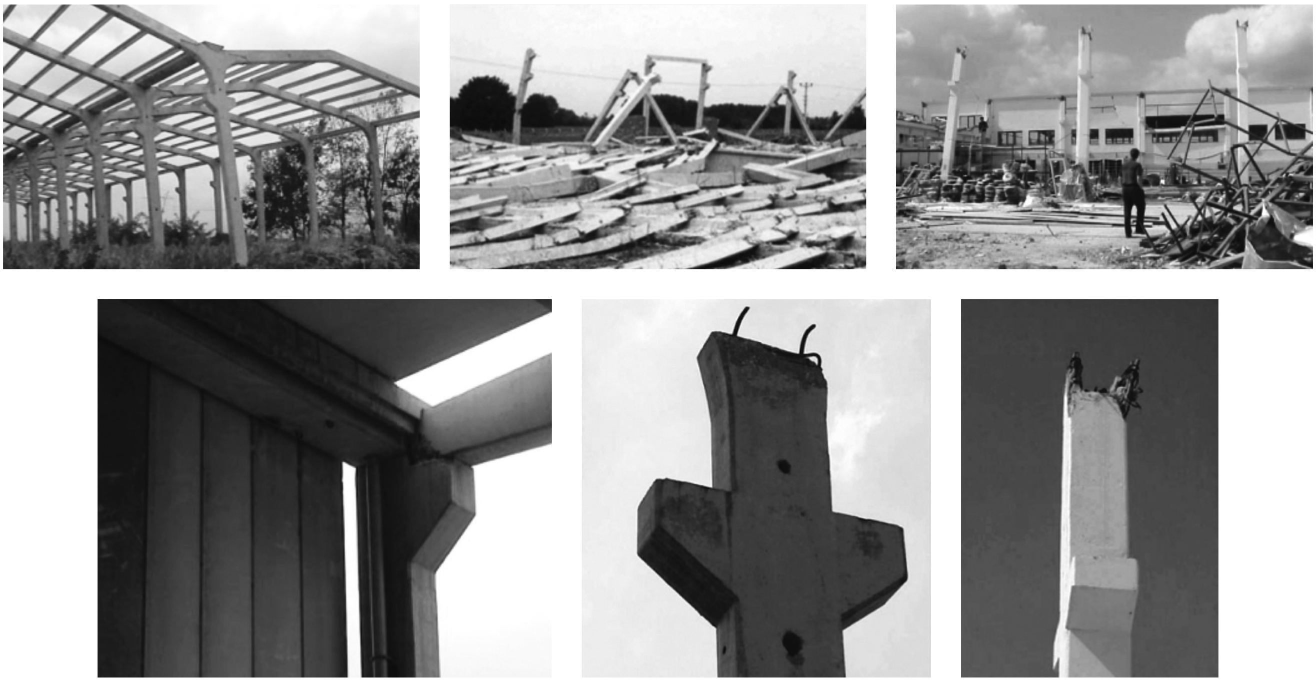

Fig. 3. *P/p. r.c.structures, with different stage of construction and seismic behaviour located in the epicentral area of the Kocaeli earthquake (above) and details of the connections (below) (Saatcioglu et al., 2001).*
Costruzioni prefabbricate in c.a. con diverse fasi di costruzione e comportamento sismico localizzate nell'area epicentrale del terremoto di Kocaeli (sopra) e dettagli delle connessioni (sotto) (Saatcioglu et al., 2001).

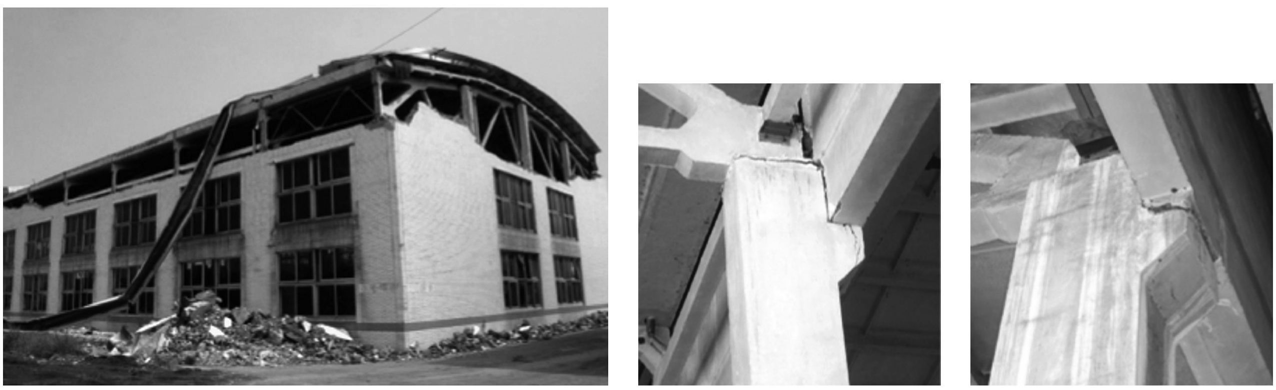

Fig. 4. *Damage and partial collapse of the Xiting Package Ltd. Factory complex in Mianzhu City, Wenchuan, China, (Zhaoa et al., 2009).*
Danno e parziale collasso del complesso industriale Xiting Package Ltd in Mianzhu City, Wenchuan, Cina (Zhaoa et al., 2009).

collapse and extensive damage was concentrated in pre-1981 r.c. and steel stock.

Some typical one-storey industrial pre-cast structures [Fig. 2, see Senel and Kayhan, 2010], located in the

Fig. 5. *P/p. r.c. structures in L'Aquila lack of adequate anchoring led to collapse L'Aquila (Myiamoto 2009).*
Strutture prefabbricate a L»Aquila; mancanza di adeguate connessioni hanno comportato il crollo (Myiamoto, 2009).

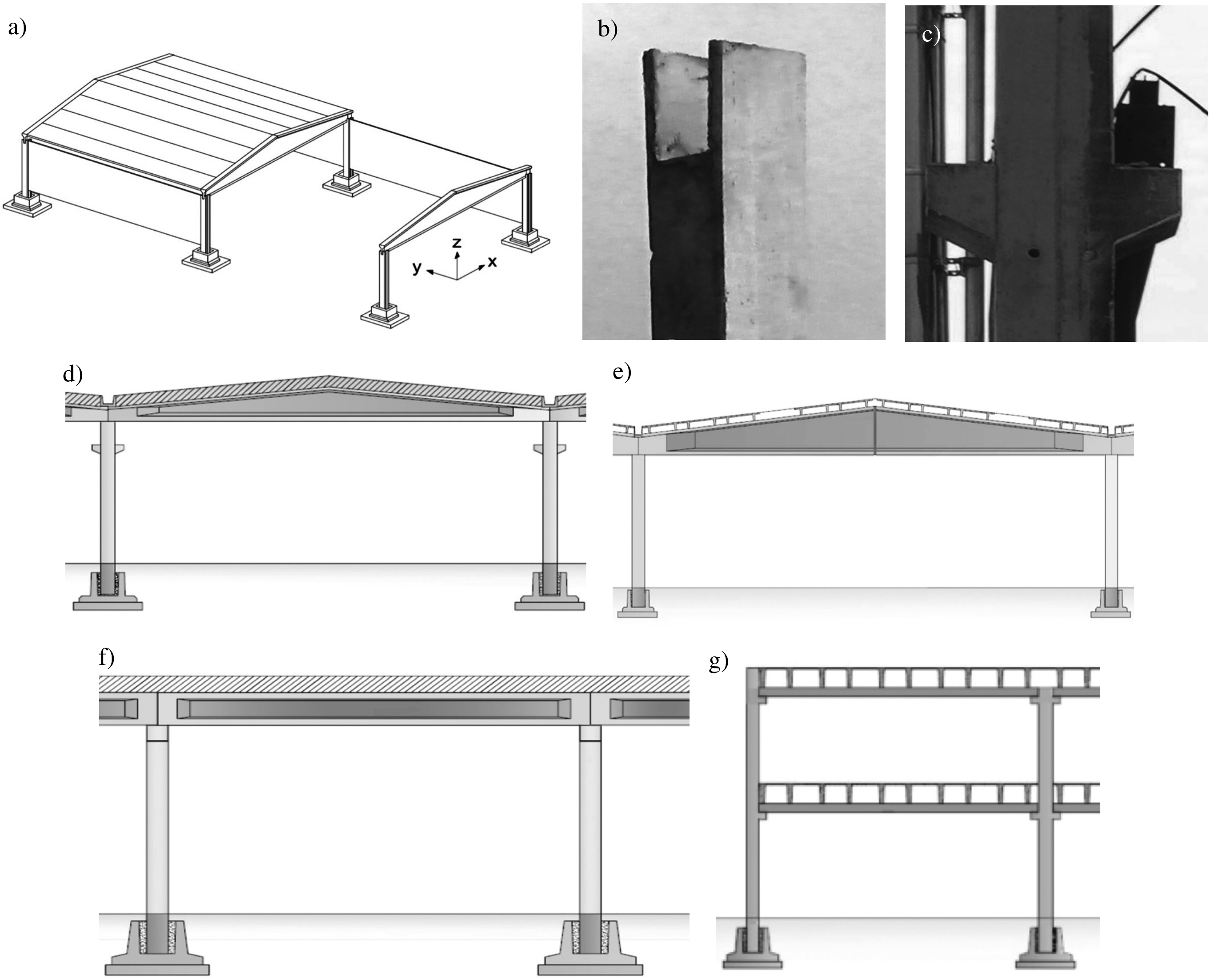

Fig. 6. *Typical resistant elements connections (a) and structural typologies (b-g) in the p/p. r.c industrial factories of Emilia-Romagna.*
Tipiche connessioni tra elementi resistenti (a) e tipologie strutturali (b-g) per edifici industriali prefabbricati dell'Emilia Romagna.

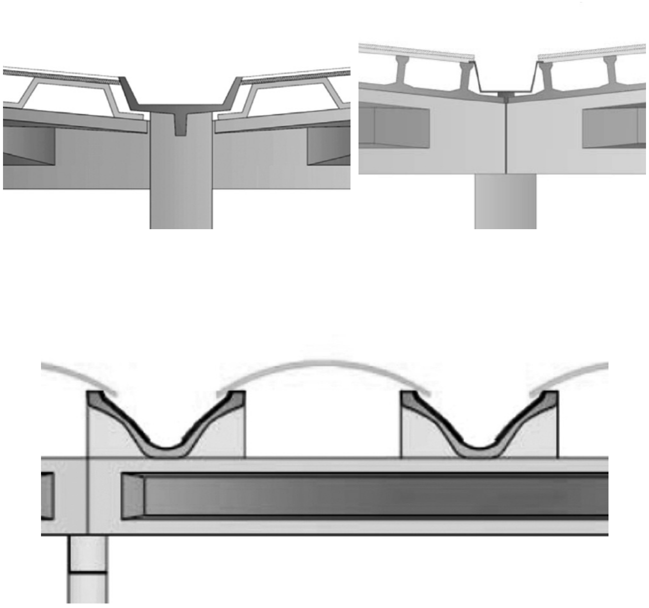

Fig. 7. *Covering typologies.*
Tipologie di coperture.

epicentral area of the August 17th, 1999 Kokaeli earthquake [Turkey, M_W 7.4, see EERI, 2000], didn't show good results, while others remained undamaged [Fig. 3, Saatcioglu et al., 2001]. The collapse was mainly due to poor design, detailing, and construction, leading to the lack of diaphragm action caused by the inadequate connections (pinned or dowel, simple to realize by the prefabricators) between columns and beams. In addition, buildings under construction were susceptible to collapse when the roof girders rotated off their supports [Posada and Wood, 2002; Doneux et al., 2006].

Also after the May 12nd, 2008 Wenchuan earthquake (China, M_W 7.9), industrial buildings and facilities, in the area near the fault rupture, were severely affected, as shown by Fig. 4 [Zhaoa et al., 2009].

The April 6th, 2009 Abruzzo earthquake (Italy, M_W 6.3) again seriously affected some p/p. r.c. buildings, generally unanchored or inadequately braced; taking into account the relatively new vintage and the existing seismic classification of the area before the event, the level of damage was surprising [Fig. 5, Myiamoto, 2009].

2. **P/p. r.c. structures affected by the emilia-romagna seismic event**

2.1. *Description*

P/p. r.c. structures, located in Emilia-Romagna, are often built up with a precast system, which structural elements are made of prestressed concrete [Assobeton, 2012; Reluis, 2012], in simple or multi-storey buildings. The vertical structures consist in squared column fixed to the foundation (Fig. 6a), with various connection systems for the beam location, i.e. mainly: two forks at the top (Fig. 6b) or properly shaped horizontal supports (Fig. 6c). For the single-storey configuration, the horizontal structures are composed of: one- (Fig. 6d) or two- (Fig. 6e) segment symmetrically sloped beams; plain covering (Fig. 6f); for the multi-storey configuration, the horizontal intermediate floor is generally realised with alveolar panels or tiles completed with r.c. cast (Fig. 6g). The upper covering system is realized with tiles of different shape, also made with prestressed concrete (Fig. 7). Beam-tiles and beam-column connections are simply friction contacts. This type of constraint has been accepted for a non-seismic zone [Decree, 1987; Circular, 1989], according to the rules in force at the time of the construction of most of the existing factories (before 2003, as mentioned in the following section 4). Therefore, many current structures are not adequate to support seismic actions.

Another common typology of factory structure, surveyed during the post seismic investigation, is made of mixed materials (both masonry and prestressed concrete vertical structures). The prestressed concrete columns are located in the central part of the building, while the masonry walls run along all the perimeter. The latter are made of solid bricks in regular configuration. The covering structure is often vaulted and made of concrete-hollow brick.

2.2. *Seismic damage*

The current section summarizes the damages observed in the structures showed above, mainly due to the following reasons:

- loss of the support;
- collapse of cladding elements;
- column damage.

2.2.1. *Collapse due to loss of the support*

The more common collapse of p/p. r.c. single-storey buildings occurred because of the inadequacy of the supports of the horizontal structural elements (tiles and beams). In particular, the tiles fell down from the beams and the beams from the columns, due to the insufficient contact constraint. In fact, the horizontal forces have been transmitted thanks to friction only (tiles/beams simply supported by beams/columns respectively). In other cases, the presence of a mechanical connection, realised by a metallic pin, has been, in any case, unable to absorb the horizontal forces due to the seismic excitation, collapsing likewise (Fig. 8).

A noteworthy case of undamaged factory, located close to the collapsed structure shown by Fig. 8a, has been investigated by our team. It shows a similar construction system, but a significant difference in the constraint details, due to the shape of the covering tiles, realising a tile/beam connection, effective in the seismic direction as a tying system (Fig. 9). This fact demonstrates that a simple technology detail preserves the entire building.

Fig. 8. *Collapse due to the lack of supports: a) the breakdown of the upper fork; b) loss of support.*
Crolli per assenza di supporto: a) La rottura del collegamento superiore; b) perdita di sostegno.

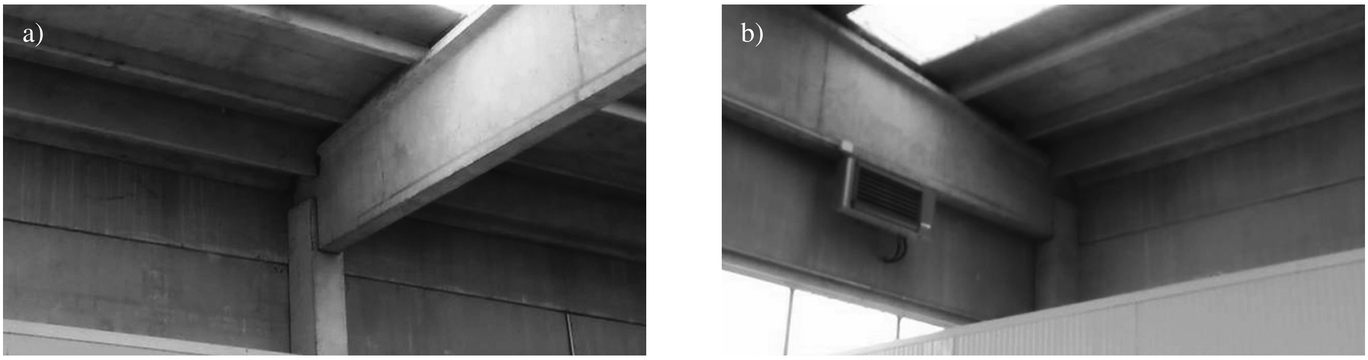

Fig. 9. *Undamaged structure: a) covering structure to beam connection; b) corner configuration.*
Struttura intatta: a) connesione trave copertura; b) configurazione d'angolo.

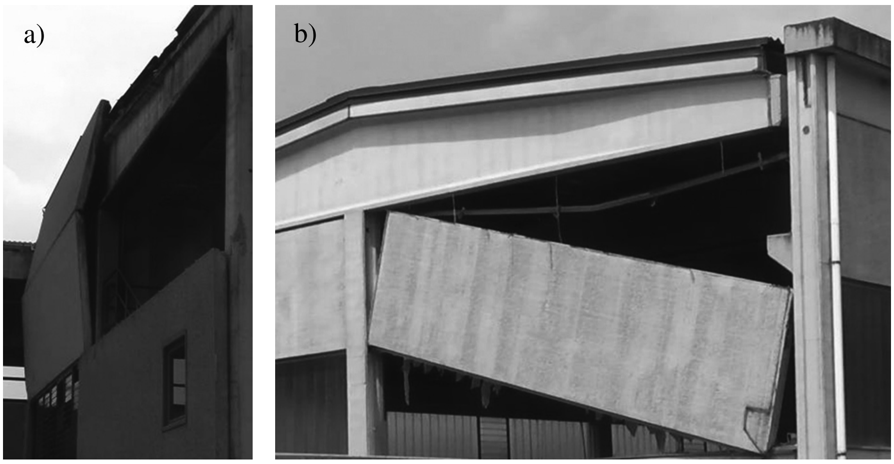

Fig. 10. *Precast cladding: a) inadequate support; b) lack of the support.*
Facciate prefabbricate: a) supporto inadeguato; b) mancanza di supporto.

Fig. 11. *Solid brick cladding damage.*
Danni a facciate in mattoni pieni.

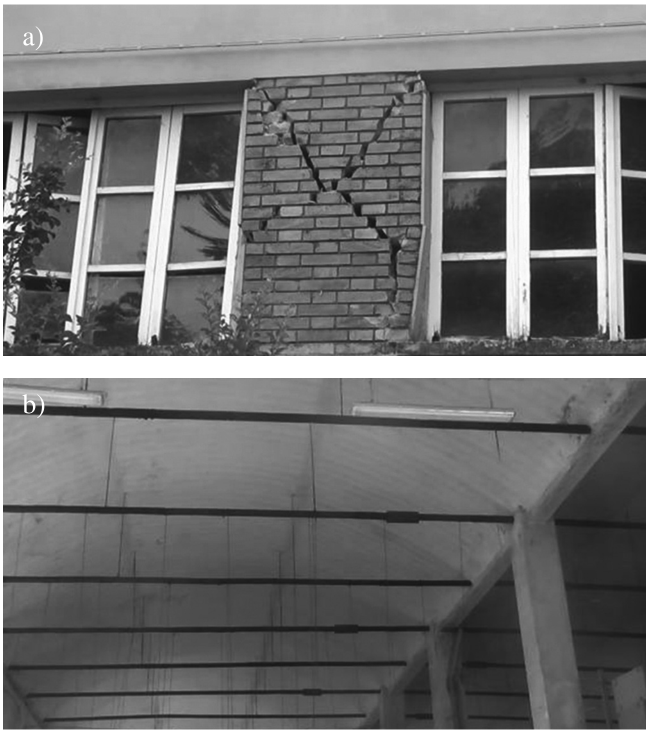

Fig. 12. *Mixed masonry-prestressed structures: a) reduced resistance piers; b) presence of steel ties.*
Combinazione muratura-struttura prefabbricata: a) maschi di ridotta resistenza; b) presenza di catene.

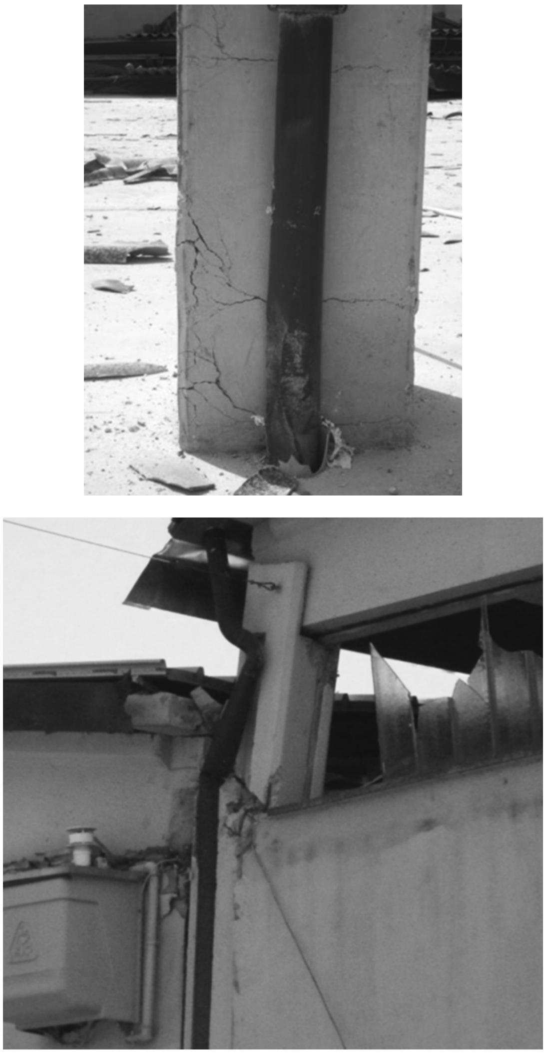

Fig. 13. *Column damages.*
Danni a colonne.

2.2.2. *Collapse of the cladding elements*

The p/p. r.c. cladding system is generally realized with r.c. precast (horizontal or vertical) panels. The connection types between panels and structural elements (beams or columns) are several. Among the damaged factories, the most common system is done with angle-steel or r.c. supports, the latter being realised by appropriate shaped columns or beams. The main cause of collapse is due to the inappropriate features of supports to absorb horizontal forces (Fig. 10). In addition, differential displacement between adjacent columns and hammering of covering versus columns caused the collapse of the cladding panels, because of the lack of the support.

Cladding made of solid brick is observed in the case of more recent simple-storey structures. In this case, the cladding suffered vast damages such as cracks (due to in-plane mechanism) or overturning (due to out-of-plane ones, Fig. 11).

Finally, mixed masonry-prestressed r.c. structures underwent damages due to the opening location (windows and main entrance), which significantly reduced the resistance of the masonry piers (Fig. 12a). In the observed case, it is worth noticing that the presence of steel ties significantly enhanced the response against the seismic effects, thanks to the increase of the box behaviour (Fig. 12b).

2.2.3. *Column damages*

Foundations are usually shallow and include precast r.c. isolation footing, in which the columns are framed. The upper end of the columns is generally unrestrained, as already said. Therefore, the structural scheme is represented by a cantilever element. Consequently, a plastic hinge often has formed at the base of the columns, when the seismic event occurred (Fig. 13a). In other cases, the column collapse has been due to the support failure (Fig. 13b).

3. **Italian codes for p/p. R.c. Structures**

3.1. *First regulations*

Some years after the Italian law containing the first measures for construction in seismic areas [Law, 1974], a Technical Code for p/p. r.c. structures was provided through a specific Decree [1987] and the following Circular [1989]. These regulations provided criteria and instructions for design, realisation, and verification, in addition to appropriate procedures for structural check and maintenance. With reference to the most important problems evidenced by this kind of buildings during the Emilia-Romagna seismic crisis, the main prescriptions are listed hereafter.

Supports

– the supports should be designed according to the worst conditions, due to the combination of production and assembling tolerances; they should not be less resistant than the connected elements, evaluating thermal variation, structural deformability and creep phenomena;

– for floor/covering elements, a minimum support length equal to 5 cm should be guaranteed; in case of non-continue supports, the length should be doubled;

– for beams, the minimum support length should be 8 cm + L/300 (where L is the clean beam length); it means that the support length should be 15 cm, at least, for a beam span of 20 m;

– in seismic zones, friction supports (i.e. if the transmission of horizontal loads is entrusted only to friction) are not allowed; they are permitted only if the capacity to transmit horizontal actions is not taken into account (i.e. out of seismic areas, wind non considered); the support should allow relative displacements according to seismic prescriptions.

Beams

– the forks which bind the beams should be verified for a working supplementary moment equal to *VxL/300*, where *V* is the beam constraint reaction and *L* its span;

– the minimum thickness of the forks should be 5 cm; in addition, it should be thicker not less than 5 times the diameter of the inside pre-stressed cables;

– if post-stressed cables are present, the thickness should be at least 5 cm greater than the scabbard diameter;

– web and stiffening should be reinforced at both the sides.

A following Decree [1992] spoke about the calculation procedures, through the allowable-stress method, with reference to prestressed elements only.

3.2. *Current regulations devoted to p/p. r.c. structures*

Current regulation for design, realization and test of p/p. r.c. structures are contained in the Italian Rules [NTC, 2008 and Circular n. 617, 2009] or in the European one [Eurocode 2, 1992].

The main significant instruction provided by this codes are following summarized, with reference to the structural elements or detail discussed in the previous paragraph. Of course, the current regulation contains more instructions, but the only aim of this section is to underline the codification evolution: thus, for the complete details about all the structural elements, please see the codes [NTC or EC 2].

Connections

– damage due to cyclic deformation in plastic field should be considered; the strength design of the connections should be reduced in order to taken into account of seismic stress, if it is required;

– in case of panel-bearing structures both r.c. and welded joints effectiveness should be experimentally tested;

– beam-column connections should guarantee the matching of both vertical and horizontal displacements and the stress should be transferred by means of mechanical devices;

– it is consented the realization of a sliding support at one end of the beam, but the sliding surface should be more extended then the seismic displacement required;

– in case of connection between non monolithic parts ad hoc design criteria should be considered (three types are possible);

– Column-horizontal elements connection should be fix (rigid or elastic) when the columns are isostatic elements;

In general, the connections between both structural and completion elements should be guarantee the transmission of both static and seismic stress in the safe side, therefore simply friction connections are not permitted.

Beams

– The beam width should be grater then 20cm, while for small beams (namely "flat beams") it should not exceed the column width, increased on both sides by the half height of the same beam;

– The ratio between transversal dimension of the beam (width/height) should not exceed 0.25;

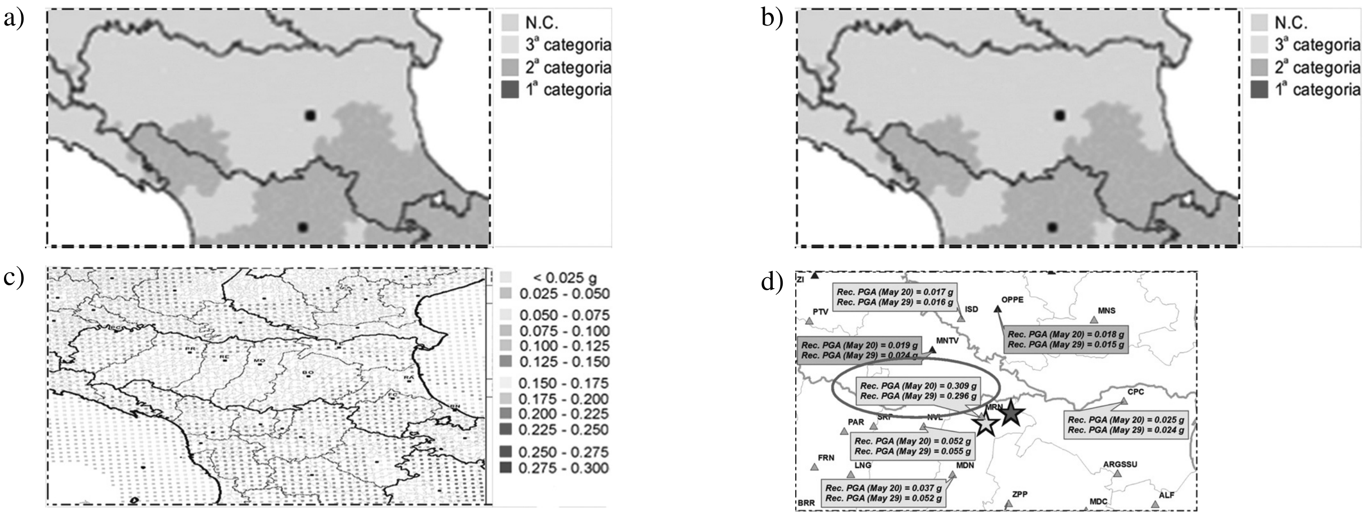

Fig. 14. *Evolution of seismic classification in Emilia-Romagna: a) Decree 1984; b) OPCM 3274, 2003; c) OPCM 3519, 2006; d) PGA related to the Emilia-Romagna seismic events.*
Evoluzione della classificazione sismica in Emilia Romagna: a) Decreto 1984; b) OPCM 3274, 2003; c) OPCM 3519, 2006; d) PGA associate agli eventi sismici dell'Emilia Romagna.

– Axes of both beam and columns supported (namely "in false") should be aligned; in addition the beams should have at least two supports (columns or shell); the shell can not be supported on beam or plate "in false";

– The critic zones are extended to 1 or 1,5 times the beam height for slow ductility class or high ductility class, respectively. In case of beam which supports "in false" columns the critic zone should be extended for two times of the beam high, starting from the beam-column node in all cases.

Columns

– The column transversal section should be grater then 25cm, while for small beams it should not exceed the column width, increased on both sides by the half height of the same beam.

In addition, the codes provide same limitation about reinforcement, which are evaluated case by case.

3.3. *Seismic classification*

Until 2003, most of the Emilia-Romagna Region (including the affected area) was not classified as a seismic zone [Decree, 1984, see Fig. 14a], except a few municipalities located in the Apennines, although part of its territory had already been mentioned among "high seismic risk areas" [OMI 2788, 1998]. But after the tragedy occurred during the 2002 Molise-Puglia earthquake, when the collapse of the primary school in San Giuliano di Puglia killed 27 kids and their teacher [Indirli et al., 2004], the Italian seismic hazard classification (together with the seismic code) was finally updated [OPCM 3274, 2003] and most of Emilia-Romagna included in 3rd category; only the Forlì province ad a little part of the Reggio Emilia district were classified in 2nd category (Fig. 14b). Subsequently, the OPCM 3519 (2006) defined the seismic hazard map (Fig. 14c). It is worth noticing that the horizontal Peak Ground Acceleration (PGA) foreseen by the updated seismic hazard map [OPCM 3515, 2006] in the affected area of Emilia-Romagna is between 0.150-0.175g, while the maximum recorded PGA in Mirandola (Modena) is about 0.30g (Fig. 14d). Furthermore, significant vertical seismic actions has been also found; finally, the local amplification of seismic effects due to soft soil should be taken into account, foreseeing accurate microzoning campaigns.

3.4. *NDSHA versus PSHA*

Most of the seismic zonation adopted worldwide by the current regulations (including Italy), either on a national or a regional scale (resulting from Global Seismic Hazard Assessment Program-GSHAP maps), have been defined according to a well-established, but obsolete, conventional probabilistic approach (PSHA, Probabilistic Seismic Hazard Assessment), which describes hazard in terms of a single parameter, the peak ground acceleration (PGA). The probabilistic PGA estimates are basically affected by the following limitations: a) strongly dependent on the available observations, unavoidably incomplete due to the long time scales involved; b) not considering adequately the source and site effects, since they resort to convolutive techniques, which cannot be applied when dealing with complex geological structures; c) time-independent, being based on the assumption of random occurrence of earthquakes [Panza et al., 2011; Zuccolo et al., 2011; and references therein). Lessons learnt from recent destructive earthquakes, including L'Aquila (2009), Haiti (2010), Chile (2010) and Japan (2011), provided new opportunities to revise and improve the seismic hazard assessment. In fact, the neo-deterministic approach (Neo-Deterministic Seismic Hazard Assessment-NDSHA [Peresan et al., 2011; and references therein) permits to integrate the available information provided by the most updated

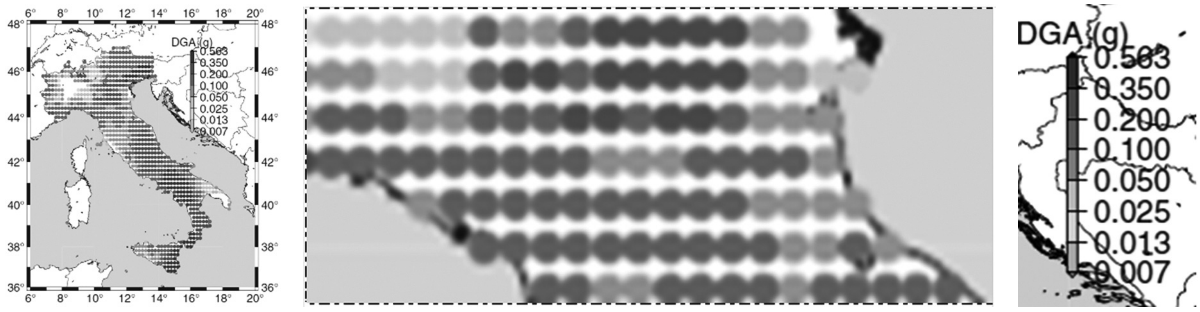

Fig. 15. *NDSHA Map for the Emilia-Romagna area.*
NDSHA mappa per la regione Emilia Romagna.

Fig. 16. *Deficiency due to inadequate connections among structural elements (DPC, 2012).*
Problemi conseguenti connessioni tra elementi strutturali non adeguate (DPC, 2012).

seismological, geological, geophysical and geotechnical databases for the site of interest, as well as advanced physical modelling techniques. NDSHA means earthquake scenario-based methods, where attenuation relations, and other similarly questionable assumptions about local site responses, are not allowed in, but realistic synthetic time series are used. NDSHA provides strong ground motion parameters based on the seismic waves propagation modelling at different scales – regional, national and metropolitan – accounting for a wide set of possible seismic sources and for the available information about structural models. The scenario-based methodology relies on observable data being complemented by physical modelling techniques, which can be submitted to a formalized validation process.

The PSHA practical limits for adequate structural design and, in general, for seismic risk mitigation, are clearly outlined by the comparative analysis of PSHA and NDSHA estimates, performed for the Italian territory [Zuccolo et al., 2011]. NDSHA provides values larger than those given by PSHA in high-seismicity areas and in areas identified as prone to large earthquakes, while lower values are provided in low-seismicity areas. In addition, PSHA expected ground shaking estimated with 10% probability of being exceeded in 50 years (associated with a return period of 475 years) appears severely underestimated (by about a factor 2) with respect to NDSHA, particularly for the PGA largest values. These observations point out one of the PSHA basic limits, particularly severe as far as building codes are concerned, that is the overly dependency of ground shaking on earthquakes recurrence (i.e. on the probability threshold selected for the maps).

For the Emilia-Romagna Region, the NDSHA studies provided values of acceleration closer (with respect to PSHA) to those recorded in May-June 2012 (between 0.200-0.350g, see Fig. 15).

3.5. *Final remarks*

With respect to the seismic zonation and the building code in force until 2003 in the Emilia-Romagna Region, the following remarks can be underlined:

– the area was not classified as a seismic hazard zone until 2003, e.g. the design of relatively recent p/p. r.c. construction ignored any prescription regarding antiseismic protection; therefore, only vertical actions have been considered, neglecting horizontal ones; as a consequence, most of the surveyed factories showed the ineffective behaviour of several components, in particular of the simple friction supports, causing most of the damage/collapse;

– although the seismic zonation was updated in 2003, the Probabilistic Seismic Hazard Assessment (PSHA) estimates, on which the seismic hazard map is based, provided lower PGA values than the recorded


Fig. 17. *Deficiency due to inadequate connections between cladding panels and structural elements (DPC, 2012).*
Problemi legati a connessioni tra elementi strutturali e pannelli di facciata non adeguate (DPC, 2012).


Fig. 18. *Deficiency due to shelf units without bracing.*
Crollo di scaffalature non controventate.

ones for the reference event (10% probability of being exceeded in 50 years, associated with a return period of 475 years), contrary to the NDSHA approach.

4. **Seismic upgrading of the existing p/p. R.c. Structures**

4.1. *Technical and legislative context*

After the seismic events in Emilia-Romagna and their effects on factories, a Guidelines draft [DPC, 2012] has been providing some suggestions for professional engineers, suitable to evaluate quickly seismic safety and set up reasonable antiseismic strengthening interventions for simple-storey p/p. r.c. construction, taking into account both the safeguard of human life and the recovery of serviceability conditions, saving machinery and equipment. The process should be planned in two phases, as mentioned in Decree (2012) and also reported by the current code NTC (2008):

– re-establishment of safety conditions (certified by a qualified **practitioner** through a preliminary technical report), eliminating the main local drawbacks, i.e. lack of structural connections between vertical and horizontal elements, presence of precast infills not sufficiently anchored to the bearing structures, presence of shelves supporting heavy materials that can cause damage/collapse of the entire construction;

– global antiseismic improvement, reaching a level equal at least to the 60% of a new construction, through an appropriate project by a structural designer, taking into account the real level of the seismic hazard and carrying out a deep in situ investigation (geometric survey, mechanical characteristics of materials, construction details, etc.), in order to achieve an adequate knowledge level of the building.

4.2. *Main local drawbacks*

As written before, the main local drawbacks considered by the Guidelines draft [DPC, 2012] are the following:

– deficiency due to inadequate connections among structural elements (Fig. 16);

– deficiency due to inadequate connections between cladding panels and structural elements (Fig. 17);

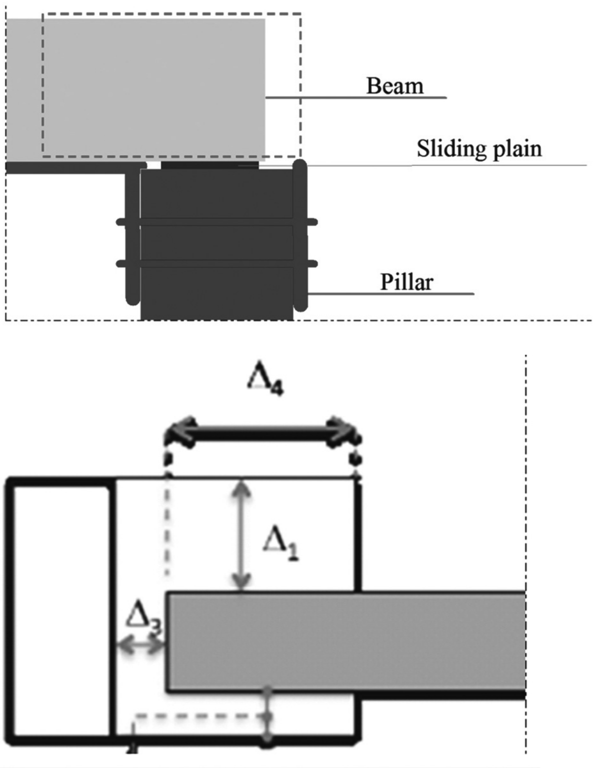

Fig. 19. *Sliding constraint; a) a practicable solution; b) sliding surface.*
Vincolo a scivolamento; a) una soluzione praticabile; b) superficie di scivolamento.

– deficiency due to shelf units without bracing (Fig. 18).

4.2.1. *Deficiency due to inadequate connections among structural elements*

All the structural elements (beams, columns, floors and foundations) should guarantee a box behaviour under the horizontal forces due to the earthquake. Friction connection is not allowed. The efficiency of precast linking system should be evaluated, while the resistant mechanism of the in situ casted r.c. connection systems involves both concrete and steel bars mechanical features. In the latter case, only the shear strength of the concrete can be considered, if the information about steel bars is absent. According to the dimension of the support, specific interventions should be realized in case of absent or inadequate shear resistant systems, aiming at the reduction of both beam-covering and beam-column relative displacements (see Fig. 6a). At this purpose, the realization of steel connection systems can be useful to upgrade or retrofit the support perform in case of both sliding and fix constraints, thanks to their easy and speed in execution. Sliding connections should be designed taken into account the main seismic displacement (Fig. 19). On the other case, the fix constraints should not lead additional significant moments to the connected parts (beam and column) because they are not designed to perform this stress. Therefore, a hinge constraint was also suggested at the column-beam connection (Fig. 20a,c). The consolidation of the column forks can be realized by means of steel elements, similarly, where it is required (Fig. 20b). The decrease of the permanent load can be useful in case of upgrading of the shear connection strength. Covering structures can be suspended to an *ad hoc* system. In addition, the reduction of relative displacements in x direction (column-beam, see again Fig. 6a) should be also realized. In any case, it is necessary to carry out shear connection elements. Finally, vertical connection between structural elements should be realized by considering the results of in situ inspections, if the vertical seismic component can produce relevant effects.

4.2.2. *Deficiency due to inadequate connections between cladding panels and structural elements*

The cladding panels, located internally or externally to the frame, are assemblages of juxtaposed single bi-dimensional elements. The whole configuration can be made by series of vertical (top- and bottom-restrained along their short side to the beams) or horizontal (right- and left-restrained along their short side to the columns) units. In special cases, the panels may be made by means of blocks. There are two different panel configurations: in a first layout, they are located in the vertical plane of the columns, interacting directly with them; in a second arrangement, they take place along the vertical plane external to the columns, interacting with the main structure through the anchorages. The linking elements are generally steel-made if the panels

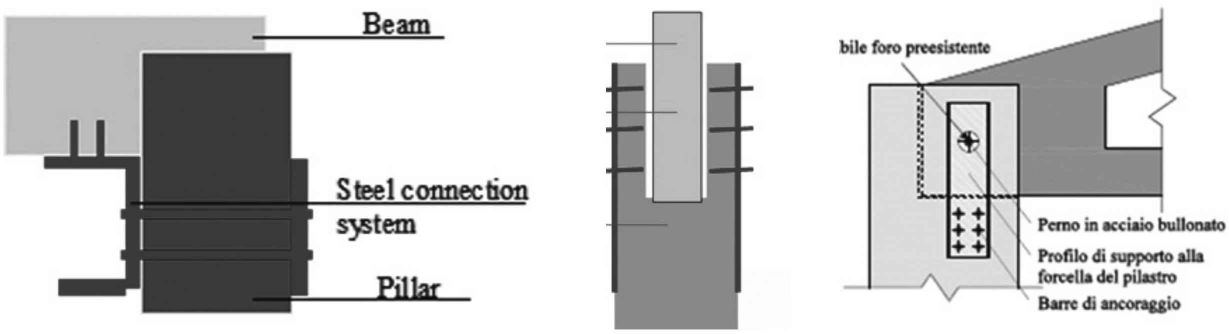

Fig. 20. *Fix constraint; a) a practicable solution; b) forks consolidation system; c) solution suggested by DPC 2012.*
Giunto fisso; a) una soluzione praticabile; b) rinforzo della forcella; c) soluzione suggerita da DPC 2012.

are external to the frame, while the panels located inside the frame are generally connected by r.c. elements. There are several technological solutions for both the types.

In presence of seismic actions, any type of anchorage may be subjected to two different effects: the inertial action parallel or orthogonal to the panel plane; and the stresses arising from the deformability of the main structure, usually much greater than the panel deformability in its plane. The Decree (2012) recommends the structural engineer to focus his attention on the various typologies of connections, installed in the p/p. r.c. structures present in the affected area, because most of them have been designed by considering only vertical forces (or very low horizontal components). Moreover, these mechanical links often show an evident lack of flexibility, being connected to very rigid panels; subsequently, they are not able to sustain the displacements imposed by the earthquake to the building, resulting in brittle collapse. On the contrary, excessively resistant connections may originate local or global breakage. Therefore, the anchoring elements should have suitable resistance to out-of-plane action induced by the earthquake, sufficient shear strength in relation with the demand due to in-plane panel/structure interaction, and finally allow relative displacements between frame and shell. Plastic mechanisms can occur only if overturning mechanisms are avoided.

The most suitable interventions, related to the anchoring systems, are the following:

– increase of the number of the anchorage units, or substitution of the existing ones with more ductile new elements; in both the cases, the connection system resistance should be lower than the resistance of the connected elements (panels and structural parts);

– increase of the system bending capability, also introducing additional devices;

– limitation of the interaction between bearing structure and cladding panels.

An appropriate solution can be the removal of the band windows, contributing significantly to the reduction of the cladding vulnerability.

4.2.3. *Deficiency due to shelf units without bracing*

The Guidelines draft [DPC, 2012] considers also the risk due to the presence of metallic shelves, storing heavy materials, because they could lead to damage/ collapse of structural parts under the seismic excitation.

In general, shelf units are made of steel cold formed profiles. The connection types are several and usually very weak. After a preliminary evaluation of damages induced by the seismic event on the whole structure and those caused indirectly by shelf elements, it is possible to assess the following requirements:

– the shelves should be unconnected from the structural elements, otherwise an appropriate certificate show that the structure is adequate to absorb the actions applied from it;

– the installation should be unconnected from the shelves;

– all the loaded planes should be equipped of fall protections;

– out-of-plane displacement should be lower than 1/100 of the height.

In addition should be controlled the base of each shelf and the maintenance conditions.

The reduction of the carried load to the 60% of the total one is the first action that should be done. Then, a structural analisys can be performed in order to evaluate the safety level according to the current rules, such as NTC [2008], the EC 3 [1993] and the UNI EN 15512 [2009].

5. **Conclusion**

This article tried to provide a quick *excursus* regarding the precast/prestressed reinforced concrete construction, summarising the response after major earthquakes, with a special focus on its behaviour after the recent Emilia-Romagna (Italy) seismic event.

In general, it can be concluded that this structural typology is very sensible to earthquake excitation, and any drawback in design, execution, and maintenance is always heavily paid in terms of damage or collapse (considering both the protection of human life and the serviceability of industrial sectors). About building codes, in particular in Italy, a special effort should be devoted to produce a homogeneous and coherent document, with particular attention to regulations for seismic improvement/retrofitting, starting from the guidelines draft recently produced, which needs a crucial implementation effort, with the participation of all the scientific community.

Finally, the evaluation of the seismic input should be carried out very carefully, taking into account updated procedures for seismic hazard assessment.

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RIASSUNTO ESTESO

Il sisma in Emilia Romagna: danni agli edifici industriali con struttura prefabbricata in cemento armato precompresso

Marzo, G. Marghella, M. Indirli

Introduzione

Dopo l'evento sismico del 20 maggio 2012 verificatosi nell'area della Pianura Padana e che ha interessato le città di Ferrara, Modena, Reggio Emilia e Bologna in Emilia Romagna, squadre di esperti dell'ENEA Bologna si sono recate nelle località colpite al fine di condurre le verifiche di agibilità (compilazione delle schede AeDES), a supporto della Protezione Civile. Nel corso dei sopralluoghi sono stati ispezionati ponti, edifici civili in c.a. e muratura, edifici industriali e i tipici casolari di campagna in muratura pertinenze dei fondi agricoli. Il presente articolo però focalizza l'attenzione sugli opifici prefabbricati in cemento armato precompresso (c.a.p.) il cui danneggiamento ha colpito il cuore produttivo dell'Emilia Romagna, fermando molte delle attività industriali. La urgente necessità di far ripartire tutte le attività produttive in sicurezza ha spinto ad agire rapidamente su questo tipo di attività mediante l'emanazione di un decreto apposito (DM n. 74 del 06/06/12) e mediante la pubblicazione di Linee Guida da parte del Reluis (2012).

Nei paragrafi che seguono saranno descritte le principali carenze strutturali degli edifici industriali evidenziatesi nel corso dei sopralluoghi. Inoltre, un excursus dell'evoluzione delle norme sugli edifici prefabbricati porrà in evidenza il gap esistente tra l'evoluzione della sismicità dei siti e le norme in materia di adeguamento sismico.

Le strutture industriali prefabricate in c.a.p. colpite dal sisma in Emilia Romagna

Gli edifici industriali in Emilia Romagna sono prevalentemente prefabbricati in c.a.p. a uno o due piani. La struttura orizzontale è costituita da travi a concio unico o a due conci, a doppia pendenza se di copertura, oppure orizzontali se di piano. La sezione trasversale delle travi è ad I. I solai sono realizzati con tegoli con varie configurazione trasversali (ad omega, a doppio T, a cupola ecc.) generalmente montati a secco sulle travi. I pilastri sono a sezione quadrata incastrati al piede nei plinti "a bicchiere" e con l'estremità superiore sagomata a forcella per l'alloggiamento della trave. Il collegamento trave-pilastro non presenta alcun vincolo meccanico oppure presenta un piccolo perno utile solo al posizionamento in fase di realizzazione. Pertanto, il vincolo trave-pilastro e tegolo-trave funziona semplicemente per attrito. Inoltre, le forcelle terminali dei pilastri hanno uno spessore dai 7 ai 10cm.

Dai sopralluoghi eseguiti è risultato evidente che i crolli sono avvenuti principalmente per perdita dell'appoggio da parte dei tegoli di copertura e delle travi. Infatti, essendo i tegoli non vincolati alle travi e queste ultime non vincolate ai pilastri, lo scuotimento sismico ha causato un movimento relativo tra gli elementi a contatto con spostamenti non congruenti tra di essi ed i conseguenti crolli. Sia i tegoli, sia le travi crollati risultano inoltre pressoché integri, avendo riportato solo danni dovuti all'impatto con gli elementi sottostanti. Un'altra tipologia di danno rilevata è la rottura delle forcelle causate dal ribaltamento delle pesanti travi di copertura, che nel crollo hanno distrutto la struttura sottostante. I pilastri hanno subito danni sia in testa (rottura delle forcelle) sia al piede a causa del martellamento da parte delle travi che hanno applicato una forza concentrata orizzontale in sommità per la quali essi non erano stati dimensionati. Per quanto riguarda i pannelli prefabbricati di tamponatura sono state evidenziate rotazioni e perdite di appoggio, nonché ribaltamenti per rottura dei sistemi di ancoraggio, evidentemente non sufficienti a sopportare spinte orizzontali. Laddove i tamponamenti erano stati realizzati in muratura si è verificato il meccanismo di ribaltamento delle facciate, separate dalla struttura portante.

Evoluzione della normative relative alle strutture prefabricate in c.a.p.

La prima norma relative alle strutture prefabbricate è il D.M. 03/12/87 sulle "Norme Tecniche per la progettazione, esecuzione e collaudo delle costruzioni prefabbricate" seguito dalla Circolare sulle Istruzioni del 16/03/89. Con riferimento agli appoggi detta norma stabiliva quanto di seguito sintetizzato:

– Gli appoggi devono essere progettati con riferimento alle peggiori condizioni (montaggio, tollerante e resistenza degli elementi collegati);

– Per gli elementi di copertura la lunghezza di appoggio minima dovrà essere di 5cm per appoggi completati in opera e dovrà essere raddoppiata in caso di appoggi definitivi;

– La lunghezza minima di appoggio delle travi dovrà essere 8 cm + L/300 (con L luce della trave); per una luce di 20 m la lunghezza minima di appoggio risultava circa 15cm;

– In zona sismica non sono consentiti appoggi che funzionano solo per attrito;

– In caso di appoggio scorrevole la dimensione dovrà essere tale da contenere con adeguato margine gli spostamenti dovuti al sisma;

– Le forcelle dovranno avere dimensione minima di 5cm oppure, in presenza di cavi post-tesi, dovranno avere dimensione almeno 5cm oltre il diametro del cavo.

Pertanto, finché l'Emilia Romagna era zona non sismica, potevano essere realizzate strutture con appoggi di dimensioni limitate e con vincolo di semplice attrito. Solo nel 2003 con l'OPCM 3274 tutta la regione è stata dichiarata sismica e successivamente con l'OPCM 3519 del 11/05/06 sono state definiti le mappe di pericolosità sismica secondo le quali nelle zone interessate dal sisma del 20 e 29 maggio 2012 la PGA attesa varia tra 0,125g a 0,175g. Si ritiene a questo punto evidenziare che nella zona di Mirandola è stata misurata una PGA pari a circa 0,30g. Tale valore era stato previsto negli studi sperimentali del prof. Panza, applicando il metodo neo-deterministico.

Dalle indagini eseguite è risultato che la maggior parte dei capannoni crollati erano stati costruiti dagli anni '60 al 2000, pertanto progettati secondo le sole azioni statiche verticali. Purtroppo, anche dopo la dichiarazione di sismicità della zona, non è stato adottato alcun provvedimento che obbligasse ad eseguire l'adeguamento sismico delle strutture. Solo in seguito al sisma dello scorso mese di maggio il DM n. 74 del 06/06/2012 ha prescritto interventi in merito. Dal 1987 al 2005 per la progettazione e la realizzazione degli edifici prefabbricati erano disponibili solo norme europee (Eurocodice 2, 1992). Il Testo Unico del 2005, successivamente sostituito dalle Norme Tecniche sulle Costruzioni (2008) e relativa Circolare n. 617 (2009) hanno poi fornito indicazioni dettagliate e specifiche per la progettazione, esecuzione e collaudo delle strutture prefabbricate. Finalmente il Decreto n. 74 (2012) prescrive ai proprietari degli opifici di procedere ad una messa in sicurezza provvisoria per riprendere le attività e ad un adeguamento sismico definitivo pari almeno al 60% delle capacità di una struttura nuova, per rendere sicure le strutture produttive. Inoltre snellisce l'iter di autorizzazione sismica dei progetti da parte degli enti preposti. In seguito a detto decreto il RELUIS ha provveduto a pubblicare linee guida per gli interventi d'urgenza. Tali Linee Guida non sono prescrittive né cogenti, ma vogliono solo fornire indicazioni di massima sulle tipologie di intervento, senza volersi sostituire ai professionisti che, sotto la propria responsabilità, sono tenuti a progettare gli interventi contestualizzandoli allo specifico caso e tenendo conto anche delle modalità di esecuzione. Quest'ultima infatti dovrà avvenire in assoluta garanzia della sicurezza del lavoratore. Gli interventi, che riguardano principalmente i collegamenti tra gli elementi strutturali, dovranno mirare a realizzare nodi fissi o scorrevoli che tengano conto delle azioni orizzontali dovute al sisma. Non dovranno introdurre momenti aggiuntivi su elementi strutturali che non erano stati dimensionati per tali sollecitazioni e dovranno garantire la trasmissione delle azioni sismiche, senza produrre spostamenti relativi tra le parti collegate tali da provocare crolli. Dovranno inoltre integrare o sostituire i collegamenti tra i pannelli di tamponamento e la struttura per evitarne il distacco. Infine si prescrivono anche interventi relativi alle parti non strutturali, ma parimenti importanti quali le scaffalature che dovranno essere autoportanti (per azioni statiche e sismiche), indipendenti dalle strutture principali e dotate di sistemi che impediscono la caduta della merce in essi stoccata.