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THE EMILIA EARTHQUAKES: REPORT AND ANALYSIS ON THE BEHAVIOR OF PRECAST INDUSTRIAL BUILDINGS FROM A FIELD MISSION

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Abstract. *A series of earthquakes, the highest of magnitude M_w 5.9, hit a portion of the Po Valley in Northern Italy, which was only recently classified as seismic. The paper reports the findings and the lessons learnt from a preliminary field survey which was conducted immediately after the second event. As a result of the economic attitude of the affected area, and possibly of the characteristics of the event, an unprecedented number of industrial precast buildings were affected, resulting into most of the casualties as well as in large economic losses. Whereas most of the damaged and collapsed buildings were designed for gravity loads only, evidence of poor behavior of some precast buildings designed according to seismic provisions were discovered. The paper provides a description of the performance of precast buildings, highlighting the deficiencies that led to their poor behavior as well as some preliminary recommendations.*

1 INTRODUCTION

There are many evidences about the behavior of precast structures during past earthquakes such as the 1976 Friuli (Italy) Earthquake [1], the 1977 Vrancea Earthquake [2], the 1979 Montenegro Earthquake [3]. More recently, experience has been gained in more modern structures after the Northridge Earthquake [4] and the Kocaeli Earthquake [5,6].

The main causes associated to the damage of the precast structures in these earthquakes were failure of connections, insufficient ductility of the columns, insufficient stiffness of the roof or slab system, being failure of the connections the main factor leading to most of the collapses. However, existing knowledge is rather incomplete and controversial. In fact, in most past earthquake events there is evidence of excellent behavior of precast structures as well as reports of catastrophic collapses, which does not come as a surprise, since performance depends upon the specific structural system, the type of connections, the adequacy of the design and the quality of construction.

Restricting the focus to precast frames, the typology which is most commonly used in Europe, evidences of very good structural behavior go hand in hand with reports of collapses, as in the already mentioned cases of Friuli and Kocaeli Earthquakes. For this kind of structures, recent evidence after the 2009 L'Aquila Earthquake [7] seems to demonstrate that the behavior of such structures is satisfactory, whereas some problems exist with the non-structural components connections, in particular with the heavy cladding elements as was reported by [8].

The present paper follows a technical mission to the area affected by the Emilia earthquakes. The economic activities of the area, in which a large number of small-size industrial facilities were concentrated, and possibly the specific characteristics of the earthquakes, resulted into an unprecedented number of precast buildings being affected by the earthquakes, with a large percentage of them being damaged or destroyed. The lessons which can be learnt from this experience are of much importance because of the coexistence of modern seismically designed buildings and of, still recent, non-seismically designed ones. The consequences are profound both for the risk which is represented by some non-seismically designed precast structures and for the importance of carefully designing the connections in modern buildings.

2 SEISMIC EVENT

The On 20 May 2012, at 02 h 09 min (UTC) – 04h 09min (local time), a 5.9 M_w magnitude (as estimated by the National Institute of Vulcanology and Geophysics of Italy, INGV) earthquake occurred at Finale Emilia, Province of Modena in Northern Italy, at a depth of 6.3 km. The main shock was followed by two very strong aftershocks, the strongest of which (M_L= 5.8) occurred on May 29 at 07h 00 min (UTC) – 09h 00min (local time) at Medolla (Province of Modena). The authors conducted their survey in the affected region immediately after the first aftershock. A last strong aftershock of 5.1 M_w magnitude took place on June 3 at 19h 20 min (UTC) - 17h 20 min (local time) at Novi di Modena. The whole area affected by the earthquakes, including all epicentres, is approximately 60 km (East-West) x 30 km (North-South). The areas mostly affected by the earthquakes comprise the municipalities of San Felice sul Panaro, Sant'Agostino, San Carlo, Finale Emilia, Mirandola, Medolla, Cavezzo, Concordia sulla Secchia and Novi di Modena , with a toll of 23 casualties, 400 injured and a total of approximately 20,000 homeless. The USGS PAGER System estimated that the economic losses were in the range of 1% of the national GDP. According to results provided by the Italian Civil Protection, for a total of 6,700 structures inspected 37% were habitable, 17% temporary uninhabitable but habitable following emergency measures, 6% partially uninhabitable, 2% temporary uninhabitable and to be reviewed in more detail, 33% uninhabitable and 5% uninhabitable due to external factors (such as near-to-collapse neighboring buildings).

A series of accelerometer stations of the RAIS (Strong Motion Network of Northern Italy) and RAN (National Strong Motion Network) networks were installed in the affected area, some already existing, and others installed following the first event. Figure 1 presents the time histories of the E–W, N–S and vertical components of the acceleration recorded at the Mirandola (MRN) station. The maximum recorded peak ground acceleration (PGA) is in the order of 0.30g (Fig. 1).

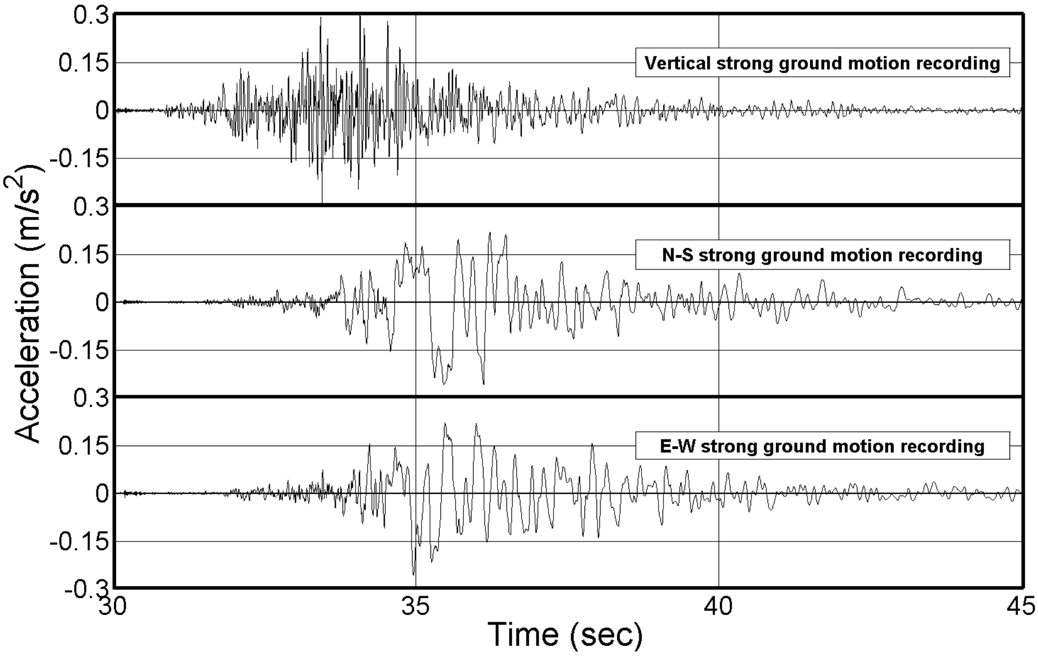


Figure 1: Vertical, N-S and E-W strong ground motion recordings at Mirandola station, 20 May 2012, 04:09 (UTC) ML= 5.9. (Source: Italian Civil Protection).

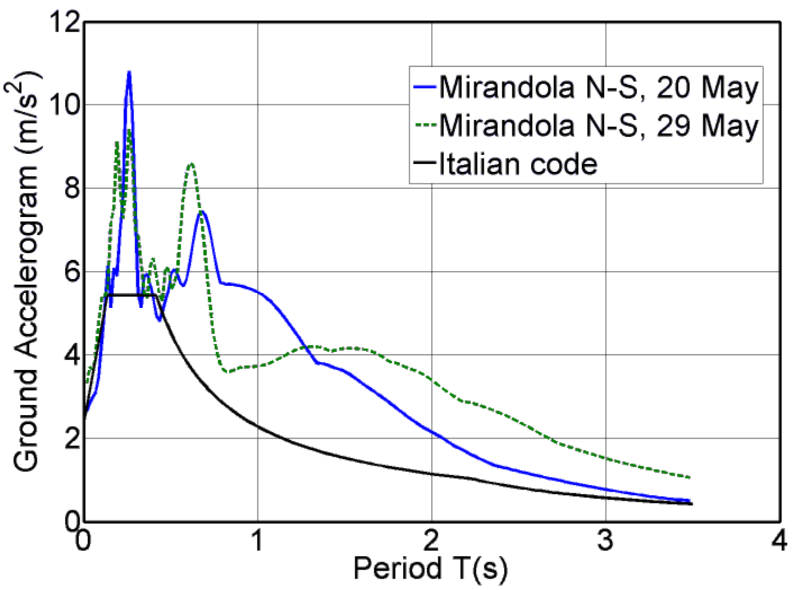


Figure 2: Spectra of the N-S component of the two strongest records compared to the Italian code spectrum.

Figure 2 plots the spectra of the N-S component of the two strongest records at the Mirandola station as compared with the current Italian code, both for ground type B and 5% viscous damping. It can be observed that the spectra from the recorded ground motions are consistently larger than the 475 years spectrum currently specified by the Italian norms. As it can also be observed, beyond the peak value of acceleration, this occurred for low periods; only the N–S component of the response spectra in both earthquakes shows high accelerations for higher periods. In particular, for periods in the range of 0.7-1.8 sec, the spectral acceleration of the N-S component of the 29 May earthquake is approximately equal to half of its peak spectral value. This low frequency content of the N-S component may have added to the high levels of damage experienced by structures with high periods of vibration, such as the relatively flexible precast industrial buildings. It should be noted here that the fundamental period of a typical single-storey industrial precast concrete building, calculated following a benchmark design study among Italy, Greece, Slovenia and Turkey, ranges between 0.8-1.4 s [9]. Finally, it should be also pointed out that the second event was more damaging because of its higher energy content in low periods of vibration.

3 NORMATIVE PROVISIONS FOR THE AFFECTED AREA

A new seismic zoning of Italy was issued in 2003, which practically established a fourth seismic zone for all regions that before 2003 were considered as non-seismic. Figure 3 presents the Italian seismic zoning map for design before and after 2003, and shows that the area affected by the earthquakes was not classified as seismic before 2003. It should be considered that the seismic zoning issued in 2003 was waived during a grace period of 18 months, and in Italy the norms applied in construction are those in force at the time in which the permission is filed at the local authorities and the permission might have been extended for years. As a result, it is very difficult to assess whether seismic provisions were taken into account with the sole estimation of the period of construction.

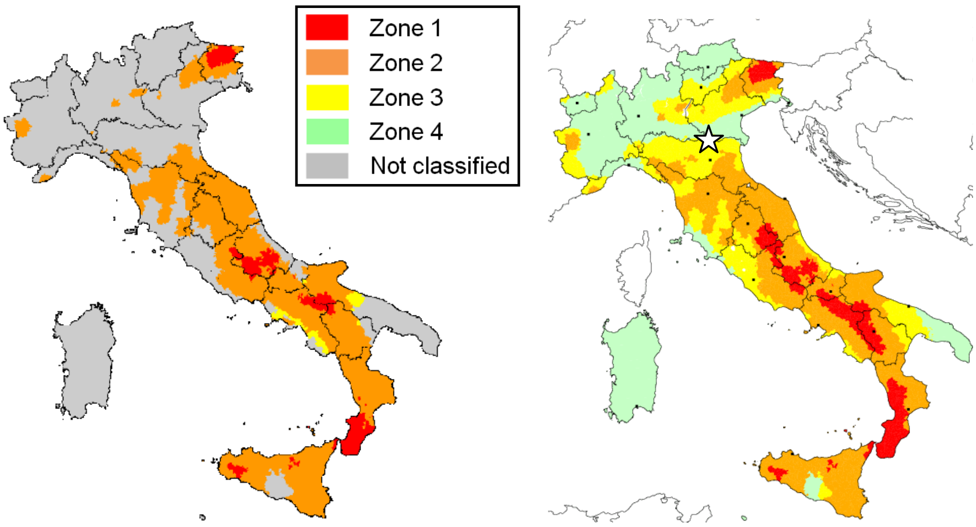


Figure 3: Italian seismic zoning map for design: (a) Before 2003. (b) After 2003.

The current seismic design code of Italy, based on Eurocode 8 and approved in 2008 with its application waived for 18 months, divides the entire country into four regions, assigning peak ground accelerations of 0.05, 0.15, 0.25 and 0.35 g for regions 4, 3, 2 and 1, respectively (Fig. 3b). The area most affected by the earthquakes is presently classified as Zone 3 (Fig. 3b), corresponding to 0.15g PGA for a return period of 475 years. The maximum acceleration recorded at Mirandola (0.30 g) corresponds to twice of the design acceleration given by the current codes (0.15g). These earthquakes revealed that the most vulnerable structures in the area were mainly the precast concrete industrial buildings, particularly those constructed without seismic provisions, and some historical unreinforced masonry buildings (i.e. churches, towers). Slight damage to motor-way bridge structures, to a new *pilotis* building and effects of liquefaction in the town of San Carlo were also observed. The focus of the paper on the performance of precast industrial buildings recognizes the important effect that the Pianura Padana-Emiliana earthquakes had on this type of structures, with no precedent in other earthquakes in Italy and Europe.

4 DAMAGE REVIEW

4.1 General

Most of the precast industrial buildings in the affected area were designed for gravity loads only and were characterized by lack of beam-column joints capable of transferring the seismic forces down to the foundation, insufficient seating and isolated column foundations. A small number of buildings were designed and constructed during the last 5-7 years, presumably conforming to the updated seismic zoning of the area. The findings of the field mission indicate that approximately three quarters of the precast concrete industrial buildings designed with non-seismic provisions in the affected area presented damage and detachment of the exterior cladding elements, with one quarter of the total presenting partial or total collapse of the roof, mainly due to the loss of seating of the main girder. Apart from one building that presented partial collapse, all precast industrial buildings designed with seismic provisions – based on the architecture of the buildings and discussions of the reconnaissance team with the owners and inhabitants of the affected area – presented no damage to the structural elements. The damage on non-structural elements, which typically comprised the detachment of cladding panels from the main structure due to insufficient capacity of the connections, were not significantly reduced in the buildings designed with seismic provisions.

4.2 Structural damage

The weak link in the vast majority of the industrial buildings visited was the absence of mechanical connections between roof-girders and columns. The most common typology corresponded to double-slope beams, simply supported over special openings (forks) at the top of columns as shown in Fig. 4. This typology was used in Italy during the 60s and 70s for agricultural buildings, now being replaced by the more common flat-roof systems. In this connection typology, the capacity of transferring lateral loads depends entirely on the static coefficient of friction between the supporting surfaces of beam and column and on the length of the beam seating when the friction forces are exceeded by the earthquake force demands.

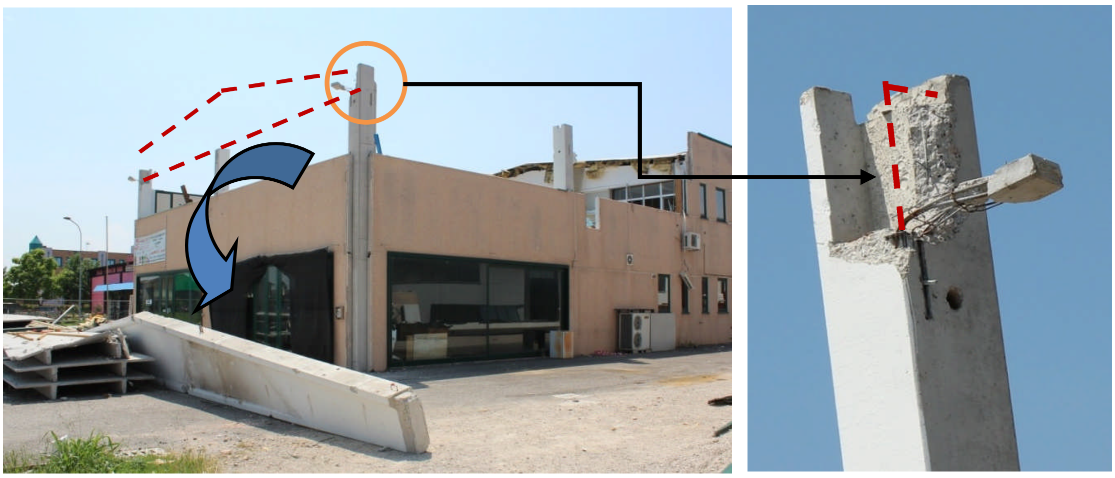
The collapse of most of the precast buildings was due to unseating of the transverse girders from the columns' forks. The seating loss was in the majority of the cases observed in the central column, where the seating length of the girders was rather limited and the relative displacement between the column and girder exceeded the available width, as shown in Fig. 5. In some cases, the collapse of the girders took place in the out-of-plane direction of the girder, after failure at the base of the forks. Figure 6a illustrates the out-of-plane collapse of a double-slope precast beam after unseating, following failure of the lateral restraints of the fork at the seat pocket (the original location of the beam is shown in red dashed lines). Figure 6b presents a detail of the failure of the lateral restraint, showing with red dashed lines the original undamaged geometry of the restrain at the fork.

(a) (b)

Figure 4: (a) Seating of double slope precast beams showing the fork at the top of the column: seating at an end column. (b) Seating at an intermediate column of a "T" section beam (with dashed lines indicating the section of the beam at the seat pocket).

(a) (b)

Figure 5: (a) Loss of beam seating from the central column and associated collapse of a double slope precast beam (the blue curved arrow shows direction of collapse and the orange arrow shows the intermediate beam); (b) Longer seating of the end beam at the external column (blue arrow).

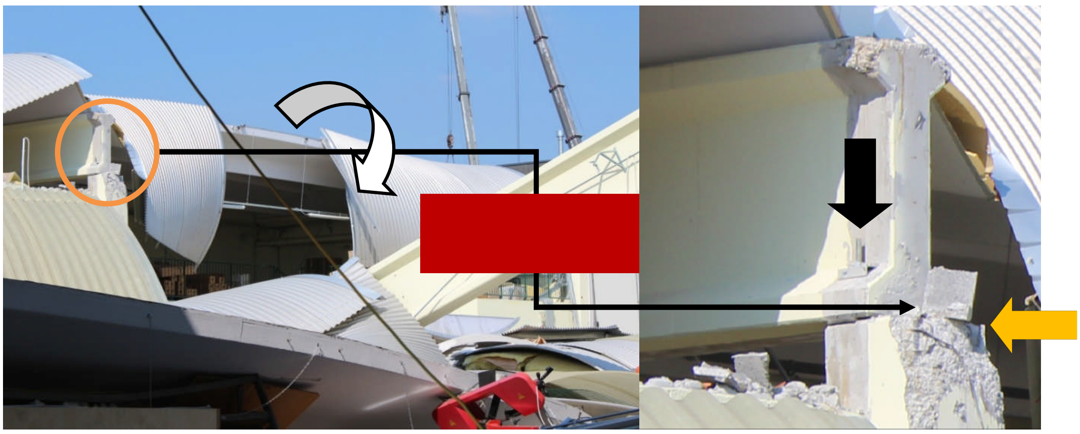


(a) (b)

Figure 6: (a) Out-of-plane collapse of a double slope precast beam after unseating following failure of the lateral restrain of the fork at the seat pocket; (b) Detail of the shear failure of the fork.

The main feature of precast industrial buildings constructed with seismic provisions lies in the type of beam-column connections. When horizontal forces are taken into consideration in design, the most common connection system for the construction of single-storey industrial buildings in Europe comprises hinged beam-column connections by means of dowel bars (shear connectors). This type of connection is able to transfer shear and axial forces resulting from the seismic actions. Practically, the horizontal beam-column connection is established by means of vertical steel dowels (typically one or two) which are protruding from the column into special beam sleeves. This pinned beam-column connections are constructed by seating the beams on the column capitals and by holding the beam ends in place by the use of these vertical steel dowels. In general, recently constructed precast concrete buildings, including a high-rise precast parking structure, which most probably incorporated steel dowels in the beam-column joints, exhibited apparent good performance. It should be noted that, due to safety measures in force immediately after the main shocks, no inspection near or inside these buildings was possible.

It is worth mentioning that a building completed in 2010 showed partial collapse, in spite of having been designed (according to information provided by the owner) with pinned beam-column connections following the new Italian construction standards. Visual inspection revealed failure at the top of one of the central column beam-column connections. This failure was followed by loss of the girder seating and its subsequent collapse (Fig. 7a). The rather limited distance of the dowels from the edge of the column and the limited amount of transverse reinforcement might have resulted into the formation of a shear crack across the concrete cover (Fig.7b), followed by the loss of the dowels anchorage and consequently the loss of the girder seating. This failure reveals the absence of specific provisions for detailing the beam-to-column connections in the current Italian construction standards and the as well as in the Eurocodes [10].



(a) (b)

Figure 7: Loss of girder seating in a recently constructed industrial building after failure of the pinned beam-column connection: (a) General overview of the collapsed beam (red dashed lines); (b) Detail showing failure of the column support at the beam-column pinned connection. Orange arrow shows damaged area at the top of the column, while red arrow shows an undamaged pinned connection.

4.3 Non-structural damage

Most of the inspected buildings – designed with or without seismic provisions – presented failure of the connections of the cladding elements due to their insufficient displacement capacity that led to overturning of the cladding elements. The panel connections were designed to transfer the vertical (self weight) load of the panel, as well as any out-of-plane loading, to the main elements of the precast structure (beams and columns). For small drifts of the structure, the connections do not provide any in-plane stiffness interaction with the panels. However, during the earthquake the precast buildings might have been subjected to excessive interstorey drifts, as well as high out-of-plane inertial lateral forces, for which these connections were not designed for.

Current design practice for precast industrial buildings is based on a bare frame model, where the peripheral cladding panels enter only as masses, without any stiffness contribution. In addition, some designers introduce only the inertial mass contribution of the walls orthogonal to the plane of the walls. The panels are then connected to the structure with fastenings devices which are dimensioned by means of a local calculation, with anchorage forces orthogonal to the plane of the panels computed based on their mass and design spectral acceleration. The connecting devices are expected to allow for all other relative deformations. However, when the free relative deformation capacity of the connection is exceeded, the panels become an integral part of the resisting system, conditioning its seismic response. The high stiffness of this resisting system leads to much higher forces than those calculated from the frame model. These forces are related to the global mass of the floors and are primarily resisted in the plane of the walls. Furthermore, the seismic force reduction considered in precast structures relies on the energy dissipation resulting from the formation of plastic hinges at the columns bases. Due to the large flexibility of precast structures, very large drifts of the columns are typically needed to activate the energy dissipation mechanism assumed in design. However, the capacity of the connections between the cladding elements and the structure is typically exhausted well before such large drifts can develop.

Figure 8a illustrates the in-plane detachment of an exterior cladding element after failure of the connections (Fig. 8b) with the main structure. The design of the claddings connections proved to be insufficient also in the orthogonal direction. The out-of-plane inertial effects of the panel led to the development of high out-of-plane lateral forces that induced failure in the panel-to-frame and panel-to-panel connections.

(a) (b)

Figure 8: (a) Cladding panel collapse after failure of the connections with the main structure (orange arrows showing the location of connections of the collapsed exterior cladding); (b) Detail showing failure of the mechanical connection.

5 STRENGTHENING MEASURES

5.1 Overall damage and lesson learnt

The high damage and collapse of the precast industrial buildings – designed for gravity loads only –was due mainly to the absence of mechanical connections between beams and columns. Simply supported beam-column connections resulted into a large number of collapses induced by unseating of the transverse girders and longitudinal beams. The loss of seating mainly occurred in the central columns, where the seating length of the girders was shorter. In other cases the collapse of girders took place in the out of plane direction – of the girder –, after failure at the base of the forks. Recently constructed industrial buildings, which most probably incorporated steel dowels in the beam-column joints, exhibited a better seismic performance. However, the partial collapse of a building completed in 2010, in spite of having been designed with pinned beam-column connections following current Italian construction standards, brought in light the lack of specific provisions for the detailing of beam-to-column connections (including the Eurocodes).

The detachment of the cladding panels from the main structure due to the insufficient capacity of the connections was practically not improved in the newly constructed buildings. The panel connections were designed to transfer the vertical load of the panel, as well as any out-of-plane loading, to the precast structure, allowing for free deformations along the other directions. There were two types of loading situations for which the panel was not designed for: i) excessive drifts that exhausted the fastenings sliding capacity and led to the development of high forces and fracture of the fastenings; and ii) high out-of-plane inertial forces that led to the fastenings failure.

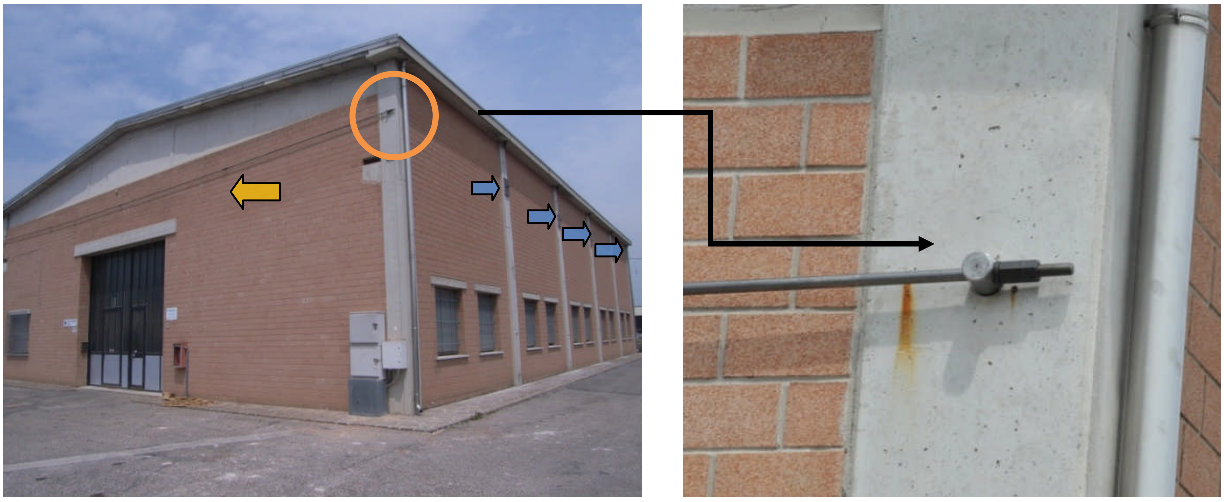
After the second main shock it was decided by the local and national authorities that the tagging of precast industrial buildings for usability would have been made by civil engineers being called upon by building owners. For most of the precast buildings existing in the affected area (i.e., designed for gravity loads only) a tagging procedure solely based on damage proved in general to be inadequate, as a number of industrial buildings that survived with no damage the first main shock did collapse during the second one. However, simple procedures for vulnerability assessment do not exist for this class of structures. Even if undamaged, precast buildings designed with no seismic provisions are highly vulnerable and may experience high levels of damage in the event of future aftershocks similar to the main event. Moreover, there is a high potential of indirect economical losses due to the interruption of the economic activities associated to the prefabricated industrial buildings. There is a risk of relocation of activities to areas not affected by the earthquake, which would badly impact on the economy of the affected area.

On the basis of the current situation, a series of provisional recommendations concerning the structural performance and safety of new precast buildings and the retrofit of existing ones in seismic regions are presented, regarding: (i) development of guidelines for retrofitting precast buildings designed with non-seismic provisions; (ii) development of guidelines for the rational design of connections of precast buildings in seismic regions; and (iii) improvement of the design of the connections of panels.

5.2 Retrofitting proposal for simply supported precast beams

In response to the risk associated with the seismic response of industrial precast buildings an ordinance comprising general guidelines for vulnerability assessment and intervention was issued by the national authorities [11]. This draft document calls for the need of adopting interventions to reduce the relative displacements between column heads and supporting beams by means of mechanically connecting the beams with the columns. At present there are not experimentally validated strengthening techniques for the seismic connection of beams simply supported to the column forks, although various retrofitting proposals for this typology of industrial buildings have been proposed.

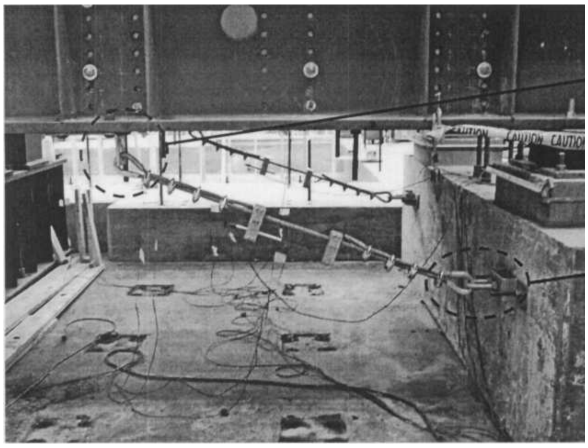
During the field visit the team inspected a precast industrial building that had been retrofitted following the first event (after information gathered from the building owner). The building corresponded to the typology with simply supported beams on columns designed for gravity loads only. The retrofit consisted in a series of steel ties in line and below the main girders, as shown in Fig. 9. The steel ties were anchored at the top of the columns in the transverse direction and might have prevented opening of the columns and loss of seating of the girder during the 29 May event.



(a) (b)

Figure 9: (a) Steel ties anchorage system installed after the main event to stabilize the structure: orange arrow showing the external tie, blue arrows showing the internal ties. (b) Detail of the anchorage at the external tie.

A proposal for the strengthening of existing industrial building consists in adopting a scheme which is used for the seismic retrofit of simply supported bridges: to reduce the likelihood of collapse due to unseating, cable restrainers are used between the girders and the piers/abutments of the bridge. Cable restrainers were first used in the United States after the collapse of several multi-frame bridges that collapsed due to unseating during the 1971 San Fernando earthquake in California [12]. The performance of bridges in the 1989 Loma Prieta and 1994 Northridge earthquakes showed that cable restrainers were effective in limiting damage [13-15]. DesRoches et al. 2003 [16] evaluated the force-displacement behavior of a cable restrainer assembly, used for the seismic retrofit of simply supported bridges. The cable restrainers were connected to the bridge pier using steel bent plates, angles, and undercut anchors embedded in the concrete as specified by typical bridge retrofit plans. The test results demonstrated that this strengthening configuration was effective in cases where seat widths were very small and relative displacements needed to be limited. The results of this study might form a good basis for future research on the use of cable restrainers for the retrofit of industrial buildings with beams simply supported at the top of columns (Fig. 10). The restrainers in such a system would be anchored at the column and beam ends, where the bending moment is minimum, with the advantage that their installation would not significantly disturb the functioning of the building.



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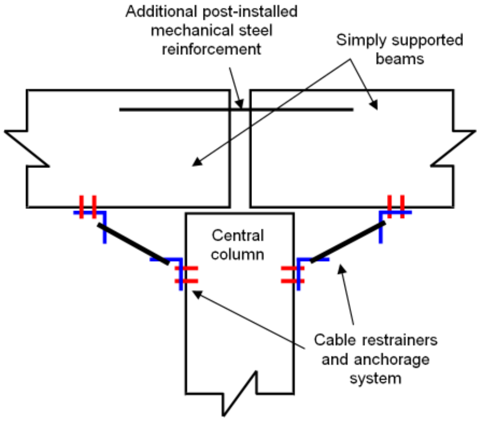


(b)

Figure 10: (a) Side view of a full-scale bridge model with seismic cable restrainers (DesRoches et al. 2003); (b) General concept of a retrofit solution for industrial buildings with beams simply supported on the column top comprising seismic cable restrainers and possibly additional steel reinforcement.

5.3 Seismic design of pinned beam-column connections

The partial collapse of the industrial building with pinned beam-column connections completed in 2010 and described in Section 4.2 brought in light the lack of specific provisions for the detailing of beam-to-column connections by the Italian construction standards and the Eurocodes. To address such issues related to the seismic design of precast concrete structures a large amount of pre-normative research [17, 18] for the development and maintenance of the Eurocodes was carried out at the European Laboratory for Structural Assessment (ELSA) of the Joint Research Centre (JRC) of the European Commission at Ispra (Italy). To this end the research project SAFECAST (Performance of innovative mechanical connections in precast buildings structures under seismic conditions, Grant agreement n° 218417-2), financed by the Seventh Framework Programme of the European Commission, was recently undertaken to fill the gap in the knowledge of the seismic behavior of the mechanical connections used in precast concrete structures. A set of guidelines for the design of connections of precast structures in seismic areas was finally delivered in the framework of SAFECAST [19].

The structural capacity of pinned beam-column connections, which represent the most common connection system in construction practice in Europe was investigated by the National Technical University of Athens [20] and the University of Ljubljana [21].

Based on the work of Psycharis and Mouzakis 2012 [20], a rational procedure for the seismic design and proper detailing of pinned beam-to column connections was proposed. Following the concept of Eurocode 8, the connections are overdesigned with respect to the strength of beam and column. The prevailing energy dissipation mechanism of the structure relies on the formation of plastic hinges within the critical regions of the columns, with the connections remaining elastic. The design of the columns is based on a prescribed force reduction factor q, whereas the design of the connections follows the capacity design rule: the design shear force E_d for the connection is obtained assuming that the ultimate flexural resistance is developed at the base of the column, calculated by multiplying its flexural resistance M_{Rd} by the overstrength factor γ_{Rd}. Verification for the shear resistance of the connection R_d is made, namely by satisfying the inequality $E_d \leq R_d$. On the basis of the experimental results obtained by Psycharis and Mouzakis 2012 the following formula for the shear resistance of the connection was proposed:

$$R_d = \frac{C_0}{\gamma_{Rd}} \cdot n \cdot D^2 \cdot \sqrt{f_{cd} \cdot f_{yd} \cdot}\ , \qquad \text{for } d/D > 6 \tag{1a}$$

$$R_d = \frac{C_0}{\gamma_{Rd}} \cdot (0.25d/D - 0.50) \cdot n \cdot D^2 \cdot \sqrt{f_{cd} \cdot f_{yd}}\ , \qquad \text{for } 4 \le d/D \le 6 \tag{1b}$$

where $C_0 \approx 0.90 \div 1.10$ is a coefficient that depends on the expected joint rotations. For large joint rotations (flexible columns) a value of around 0.90–0.95 is suggested, while for small joint rotations (stiff columns and walls) it may be increased up to the maximum value of 1.10, for practically zero joint rotations. f_{ck} and f_{sy} are the characteristic strengths of concrete and steel (units in MPa), with their design values $f_{cd} = f_{ck}/\gamma_c$ and $f_{yd} = f_{sy}/\gamma_s$, where γ_c and γ_s are the partial safety factors for concrete and steel. The recommended values for these coefficients in Eurocodes 2 and 8 are: γ_c = 1.50 and γ_s = 1.15. D is a diameter of the dowel, d the distance from the centre of the dowel to the face of the column (units in mm), and n is the number of dowels. The suggested general safety factor γ_{Rd} is equal to 1.30 as proposed by fib 2008 [22].

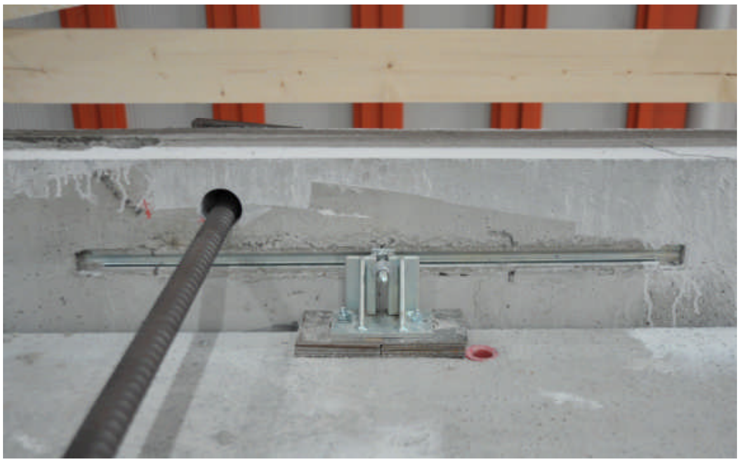
In addition, a series of specific provisions for the detailing of hinged beam-column joints were proposed by Psycharis and Mouzakis 2012:

- It was suggested to use a sufficient cover of the dowels (d/D ≥ 6), otherwise spalling might occur, which much decrease their resistance.
- The presence of horizontal hooks in front of the dowels was found to be very important for the hinged joint's seismic response.
- The use of high strength grout inside the sleeves increases the resistance of the connection and improves its cyclic response by decreasing pinching and increasing ductility.
- For flexible columns, large rotations can occur at the joints, which reduces shear strength and increase damage to the connection, which increases with repeated cycles.

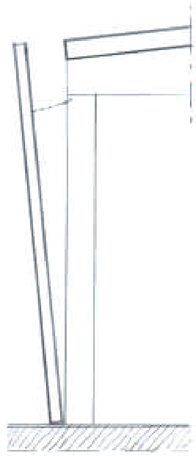
Besides the tests on individual beam-column joints, within the SAFECAST project a series of pseudodynamic tests on a full-scale three-storey precast concrete building, comprising pinned beam-column connections, were carried out at the ELSA Laboratory. On the basis of these tests it was shown that in the case of multi-storey buildings with hinged beam-to-column connections, due to the participation of the higher modes, there is no reduction of the inter- storey forces when the structure enters into the nonlinear regime, as one would expect as a consequence of ductility. This resulted into large (i.e., much larger than when taking into account the q factor) forces in the beam-column connections. Therefore, the large magnification of storey forces should be considered in determining the capacity of the pinned connections. A possible conservative simplification could be to multiply the design storey forces in all stories by q [21, 23-24] (Fischinger et al. 2012, Negro et al. 2012, Bournas et al. 2012), even though less conservative approaches have also been developed [21].

5.4 Cladding panels design

The detachment of cladding panels from the main structure due to insufficient capacity of their connections with the main structure demonstrated their high vulnerability. Classifying precast claddings as non-structural elements because they are not expected to contribute to the strength of the building is indeed misleading, since they may provide a stiffness contribution at large drift, inducing failure of their connections resulting in the fall of panels up to 10 tons of weight. In order to ensure the standard design approach in which the claddings do not contribute to seismic response, stringent design criteria for the design of the connections should be enforced: connections devices should be able to provide the required strength in the vertical and out-of-plane directions while accommodating large relative deformations in all other directions (Figure 11a). A viable option would be to prescribe adequate fail-safe restraints, so that the panels would not fall even in case the relative displacement capacity of the connection is exceeded (Figure 11b). Another possibility would be to design the claddings and their connections as being an integral part of the structure, by adequately taking into account their strength and stiffness in the design model. These are the objectives of the recently activated EU-funded project SAFECLADDING (Grant Agreement no. 314122).



(a)



(b)

Figure 11: (a) Panel connection device that provides resistance in the vertical and out-of-plane directions and accommodates large relative deformations in the lateral direction. (b) Fail-safe restraints to prevent panel falling.

6 CONCLUSIONS

The present paper follows a technical mission to the area affected by the earthquake focusing mainly on the performance of precast industrial buildings. From the findings of the field reconnaissance mission, the following set of conclusions and recommendations may be drawn:

- Most of the damage experienced by precast industrial buildings in the affected area was observed in buildings designed according to the seismic zoning in force until 2003, which classified the area as non-seismic, corresponding to a design for gravity loads only, with beam-to-column joints not capable of transferring horizontal loads and isolated column foundations.
- Approximately 75% of the precast industrial buildings designed with no seismic provisions presented damage and detachment of the exterior cladding, with 25% presenting partial or total collapse of the roof and girders.
- The weak link in the majority of industrial buildings designed with no seismic provisions only was the absence of a mechanical connection between beams and columns. This resulted in a large number of collapses induced by unseating of the main beams. The loss of seating occurred mainly in the central columns, where the seating length of the girders was rather limited. In other cases the collapse of girders took place in the out of plane direction – of the girder – following failure of the lateral restrains at the top of the column.
- Industrial buildings designed under the seismic zoning in force at the time of the earthquake and corresponding to 0.15g PGA (475 year return period) exhibited better seismic

performance. However, the partial collapse of a building with pinned beam-column connections completed in 2010 and designed following the Italian construction standards brought in light the lack of specific provisions for the detailing of beam-to-column connections.

- The detachment of cladding panels from the main structure was practically not improved in the newly constructed buildings, due to insufficient capacity of the connections between the panels and the structure to accommodate in-plane displacements and resist the out-of-plane inertial forces of the panels.
- It is recommended to develop guidelines for the seismic retrofit of precast buildings designed with non-seismic provisions, in particular for the beam-to-column connections and for the connections of the cladding panels with the structure. Such guidelines will be of very much use in similar areas in Europe that are upgrading their seismic zoning, classifying as seismic areas that have been historically considered as non-seismic.

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