DAMAGES AND COLLAPSES IN INDUSTRIAL PRECAST BUILDINGS AFTER THE EMILIA EARTHQUAKE

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SUMMARY – The present paper describes the collapses in precast industrial buildings caused by the 2012 Emilia earthquake. This kind of buildings, not designed with seismic criteria, has been found very vulnerable for several reasons, such as the absence of connecting systems between the various precast monolithic elements, the inadequacy of connection systems of external precast walls to the bearing elements (columns and beams), the interaction between non-structural walls and structural elements, and the inadequacy of the foundations.

SOMMARIO – La presente memoria descrive i danni e crolli osservati in edifici industriali prefabbricati dopo il terremoto dell'Emilia. Questo tipo di costruzioni, non progettate con criteri antisismici, si è rivelato particolarmente vulnerabile per diverse ragioni, come ad esempio l'assenza di sistemi di collegamento tra i vari elementi prefabbricati, l'inadeguatezza dei sistemi di collegamento tra pannelli prefabbricati esterni ed elementi portanti (pilastri e travi), l'interazione tra tamponamenti e pilastri ed, in alcuni casi, l'insufficienza delle fondazioni.

Key words: industrial buildings, precast technology, connection systems, earthquake.

1. **Introduction**

A series of strong earthquakes struck the Emilia region, in Northern Italy, in May 2012. Two main earthquake sequences can be identified, with mainshocks featuring similar energy: the first mainshock, with local magnitude M_L = 5.9 (latitude = 44.89, longitude = 11.23), struck on 20th May and the second mainshock, M_L = 5.8 (latitude = 44.85, longitude = 11.09) on 29th May [Dolce *et al.,* 2012a,b]. The peek ground horizontal-accelerations recorded

during those two earthquakes were PGA_h = 303.30 cm/s^2 (epicentral distance R_{epi} = 13.4 km) and PGA_h = 240 cm/s^2 (epicentral distance R_{epi} = 16.0 km), respectively. Furthermore, it is interesting to notice that vertical peek ground accelerations up to 900 cm/s^2 were recorded close to the epicentre.

The region was not covered by seismic design requirements until 2005–2006 therefore most of the buildings hit by the earthquake were designed without seismic rules. The mainshock on 20th May caused the collapse of several precast buildings in the industrial areas of S. Agostino, Bondeno, Finale Emilia, S. Felice sul Panaro while the earthquake on 29th May was particularly severe for industrial buildings in Mirandola, Cavezzo and Medolla. The first estimates indicate that, in some industrial areas, almost 70% of precast buildings collapsed or were severely damaged. Issues and collapses related to precast buildings were reported by many authors after past earthquakes [Ghosh and Cleland, 2012; Iverson and Hawkins, 1994; Khare *et al.*, 2011, Muguruma *et al.*, 1995, Posada and Wood, 2002, Toniolo and Colombo, 2012].

The main causes of the aforementioned collapses were related to the vulnerabilities of precast buildings not designed with seismic criteria. In particular, the lack of connecting systems between various precast monolithic-elements was the main cause of most collapses, even if the large displacements that caused the fall of precast beams from the column supports were in some cases induced by other phenomena. For instance, in many cases, the interaction with non-structural walls, such as masonry infills or secondary reinforced concrete elements, had strong negative effects and caused partial collapses of buildings. Furthermore, several failures involved reinforced concrete precast cladding panels, because of the inadequate anchorages on the bearing elements (columns and beams). Finally, some very large collapses were also due to the inadequacy of the foundations.

The present paper collects and describes some typical failure modes that were observed by the authors during field surveys. In most cases, it was possible to identify the reasons of the collapses, in relation with the usual design criteria for non-seismic zones adopted in the region. The loss of support of roof elements from beams and of beams from column supports, as well as the failures of cladding panels connections, were the most recurrent and evident effects of the earthquake. In many cases, other causes, such as the interaction with irregular infills panels or the rotation of the foundations, contributed to the failures.

2. **Code prescriptions in the region for design of reinforced concrete structures**

The deficiencies in the seismic response of existing precast structures were, in general, due to substantial deficiencies in the design, such as incorrect structural details and lack of a

correct seismic design philosophy. In fact, following the codes at the construction time, precast concrete portal frames were generally designed to support gravity loads only. Those structures have been found inadequate to support the horizontal loads due to the earthquake.

Before 2005, the design of precast elements followed the requirements of the Italian National building codes, i.e. the DM 9.1.1996 and the DMs 16.1.1996 [Italian Ministry of Public Works, 1996a,b,c], and Technical Guidelines, such as the CNR 10025 [CRN, 1984]. The DM 16.1.1996 [Italian Ministry of Public Works, 1996b] provided the general load definitions and the load combination rules with exception of earthquake loads. The DM 9.1.1996 (Building Code Requirements for Reinforced Concrete and Steel) [Italian Ministry of Public Works, 1996a] provided the general proportioning and detailing procedures for reinforced concrete structures in non-seismic regions, and the DM 16.1.1996 [Italian Ministry of Public Works, 1996c] contained seismic design rules but it was applied only to the Italian municipalities classified as seismic which did not include those interested by the May 2012 earthquakes.

The first seismic hazard map for Italy was prepared after the Messina earthquake (1908). In the following years, several updates were implemented, typically after the occurrence of a strong earthquake. An important improvement was achieved in 1996 [Italian Ministry of Public Works, 1996c] when four seismic zones, corresponding to different seismic hazard levels, were identified in Italy: the first was characterized by the highest hazard while the fourth had no seismic hazard. No seismic design was required for this latter zone. The inter-zone boundaries followed administrative boundaries. The Emilia region was mostly classified as no seismic zone, with the exception of some upland areas.

Geometry and detailing requirements for reinforced concrete components in non-seismic areas were essentially related to the vertical safety conditions, including dead and live vertical-loads and wind load combinations; minimum dimensions were specified for columns (250 mm x 250 mm) and no limits were imposed to the dimension of beams with respect to columns. Minimum reinforcement ratios and sizes were set for columns longitudinal bars (0.3% of the concrete cross section, with minimum 4 bars of 12 mm) and for shear reinforcement (minimum stirrup diameter of 6 mm and maximum stirrup spacing of 250 mm). Connections between beams and columns were essentially based on friction or, in a few cases, contained small steel connectors mainly designed for ease of the construction phases. It is interesting to notice that the CNR 10025 Technical Guideline [CNR, 1984] contained several design rules for seismic areas, some of which similar to modern design criteria, but in the no-hazard zone those design rules were substantially never adopted.

The seismic hazard map was updated in 2003 and 2005 and the seismic code was revised in 2003, 2005, and 2008 (OPCM 3274 [Prime Minister Decree, 2003], OPCM 3432

[Prime Minister Decree, 2005], and DM 14.1.2008 [Italian Ministry of Public Works, 2008]). The major modification to the maps was a site–specific probabilistic seismic hazard assessment that lead to an increase in the number of municipalities in seismic areas. Almost the entire Emilia region is presently classified as a low or medium-hazard zone. With reference to a 475 years return period, peek ground acceleration spans from 0.14g to 0.17g on rock (about 0.22g - 0.26g on soft soils, that are similar to those in the zone hits by the earthquake).

3. **Types of precast buildings in the region**

The region struck by the earthquake mainshocks is one of the most productive areas in Italy. The industrial areas started to be built in the seventies, and have been under expansion until now. Almost all the buildings have a single-storey reinforced concrete structure.

The typical layout of a single-storey industrial building is normally composed of a series of basic portal frames. Each frame has two columns with moment-fixed connections at the foundations and a pin-joined roof beam. Being the region not covered by seismic code requirements until a few years ago, the beam-column connections are typically friction-based supports without any connection device. In some cases, neoprene pads are placed at the column-beam and column-base interfaces. No shear walls are present. Building stability and wind–load support are based on the cantilever behaviour of the columns.

Since different precast technologies were identified in the buildings investigated, in order to simplify the presentation, the reinforced concrete precast buildings will be roughly divided into two main categories:

- Type 1: old precast buildings (built in the 70's and in the 80's);
- Type 2: new precast buildings (built after the 80's).

Most of the Type 1 industrial buildings have precast beams with spans around 12 – 20 m and distance between frames is about 6 – 10 m (Figure 3.1).



Figure 3.1. *Precast building with sloped beams with I-shaped cross section (SI beams) and masonry infills between beams and columns.*

The precast beams have a sloped pane (SI beams), and the columns are quite slender, with square cross-sections with edges about 30-40 cm long. No beam-column connections are present, and often both main and secondary beams have friction-based supports only on the columns, without neoprene pads at the interface. The beams typically have either no or little restraints against the out-of-plane movements, especially when both transverse and longitudinal beams are supported at the top column. Most of these buildings have masonry infills, along both short and long walls.

Type 2 precast buildings typically adopt I-shaped beams with straight profiles. Flat precast elements are placed on load carrying beams in the long direction of the building (Figure 3.2). In this case, the span between the frames is larger, about 10-12 m. The roof is built with different kinds of prestressed elements: TT-shaped beams, beams with a constant section with a V-shaped symmetric or asymmetric profiles, and "wing contour" roofing elements. In the latter case, between the structural elements there are curved tiles made of glass or transparent polycarbonate for lighting purposes. When the latter solution is adopted, quite commonly in the last 15 years in large industrial buildings (spans longer than 20 m), the roof plan is of course highly deformable. Finally, two different kinds of reinforced concrete curtain walls can be present, either made of horizontal or vertical cladding panels. (Figures 3.2, 3.3).



Figure 3.2. *Precast building with an intermediate portal frame, long roof prestressed beam elements and external horizontal cladding panels*.

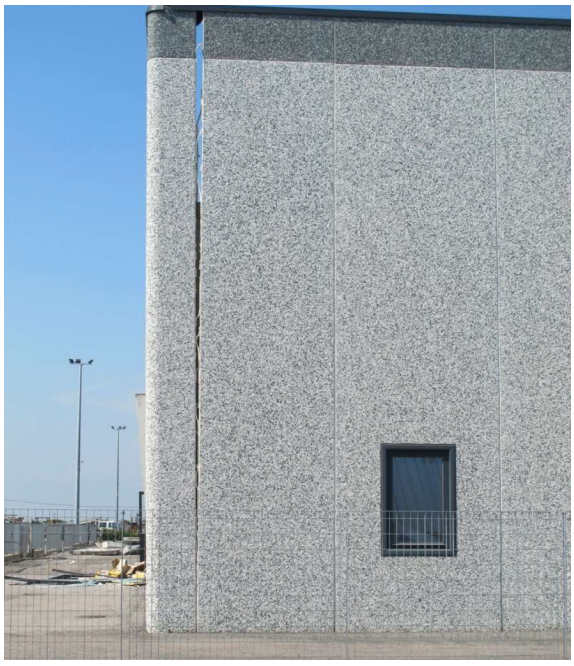


Figure 3.3. *Precast building with vertical cladding panels.*

4. **Damages and collapses in Type 1 industrial buildings**

The reinforced concrete precast industrial buildings built in the 70's and 80's, classified as Type 1 in the present paper, are very flexible under horizontal loadings. Therefore, the effect of the in-plane stiffness of the curtain walls is very important.

Considering the behaviour in the transverse direction of the building (i.e. in parallel to the main frames), when the curtain wall was regular, i.e. the entire wall is masonry made, its

strength was usually sufficient to resist the horizontal seismic action and no significant damage occurred to the buildings. Most buildings of this kind were in fact only slightly damaged by the earthquake (Figure 4.1). In general, very few buildings with regular masonry panels have collapsed. In the building of Figure 4.2, the high deformability of the roof system was probably the cause of collapse, not being sufficient to effectively transfer the horizontal seismic force to the external masonry walls.

On the contrary, when strip-windows were present just under the precast beam, the interaction of the wall with the precast columns often caused critical damages, up to the failure of the beam falling from the column support. This was probably the most frequent cause of failure in some industrial areas (e.g. San Felice sul Panaro and Mirandola) hit by the earthquake (Figures 4.3). In most cases the beams fell from the support on the column with the effective span reduced by the masonry panel (short column effect). In fact, in this case, most of the total horizontal force is transferred to short, and therefore stiffer column, and the ratio between the horizontal force and the vertical force overcomes the friction between beam and column, leading to the fall of the beam not restrained by mechanical connectors. Typically the failure involved only the external portal frame, and in a few cases also the following one (Figure 4.3*e*).



Figure 4.1: *Precast building with regular masonry external panels slightly damaged by the earthquake.*



Figure 4.2. *Collapsed precast building with regular masonry external panels*.



(*a*)



(*b*)



(*c*)



(*d*)



(*e*)

Figure 4.3. *Collapses of the first precast beam from the column support in the case of irregular masonry external panels (presence of a strip-window below the precast beam).*

The interaction of the portal frame elements with non-structural facade elements often caused other kinds of severe damages or even collapses. Figure 4.4 reports a collapse that was mainly caused by a reinforced concrete facade column at the mid–span of the precast beam of the external portal frame (the column of the left of Figure 4.4). In fact the beam was originally designed neglecting the non–structural facade column and therefore considering two supports only (i.e. positive bending–moments only), but the facade column created a statically redundant condition (three–point support). The beam then lost the side support on the right structural column during the ground–motion, and failed at the cross section with the maximum negative bending moment.

Irregular panels not fully restrained by the reinforced concrete elements, because of the strip–windows, often collapsed or were strongly damaged (Figures 4.3*b*, 4.5).

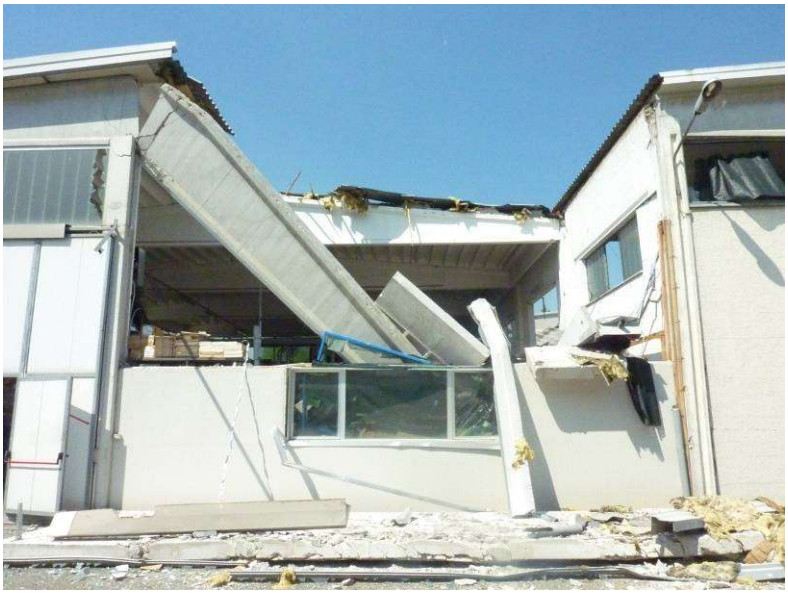


Figure 4.4. *Failure of the external portal frame due to the interaction with non structural façade elements.*



Figure 4.5. *Failure of a masonry external–panel not fully restrained to the reinforced concrete elements, due to the presence of a strip–window below the precast beam.*

5. **Damages and collapses in Type 2 industrial buildings**

More complex and various are the failure mechanisms of the most recent industrial buildings. In some cases, partial failures can be related to the fall of the roof elements, not restrained to the precast beams, even without any evident damage of the vertical columns. Especially in the case of roofs with precast elements alternated with strip–windows, the roof plan deformability and the significant relative displacements between the beams caused the loss of support (Figure 5.1).



(*a*) (*b*)

Figure 5.1. *Falling of precast roof elements, without any evident damage of the vertical columns.*



Figure 5.2. Falling of the roof elements related with the plan irregularity of the building.

In the case of the multi-storey building of Figure 5.2, the loss of support to precast roof elements was due to the irregularity of the building: with a mechanism similar to those depicted in Figures 5.3; the greater lateral stiffness of the right part of the building with respect to the left part, caused the transfer of a large horizontal force, which overcame the friction over the beam supports at the building depth change.

Many collapses were related to the failure of some intermediate columns, and in these cases very large portions of the building were involved. In the case of Figure 5.3, a significant damage was registered in intermediate columns after the first mainshock, and the building fully collapsed during the following earthquakes. Intermediate columns had smaller cross–sections than the external columns, being probably the former designed only to carry vertical loadings, and the latter also to carry the lateral wind forces. Since window strips separated the roof elements, then the roof floor was fully deformable and the horizontal force acting on intermediate columns was approximately double with respect to the side columns. Being lateral columns stronger than the internal ones, and stiffened by the external vertical cladding walls clamped in the reinforced concrete foundation beam, often no damages were evident from outside (Figure 5.3).

Some complete collapses of industrial buildings are related with base rotations of the vertical columns. Even if a certain identification of the causes would require more information than available to the writers, the complete absence of any cracking along the columns indicates, as the most plausible cause, a rotation of the foundation (Figure 5.4). This conclusion is also supported by the large use, after the ‘90s, of pocket foundations, with the precast column fixed into the pocket with grout or concrete. In some cases, because of the absence of seismic regulations for design in the region, pocket precast elements were simply supported over larger cast-in-situ foundations, with the verification against overturning ruled

by the acting vertical loadings and only the wind action as horizontal force. Moreover, the behaviour of the columns during the earthquake ground–motions was elastic — in fact no cracks were observed — because their cross–sections were, in general, oversized in order to reduce the drift under wind actions. Therefore the bending moment transferred to the foundations overcame the overturning moment of these latter, producing permanent rotations. Cracking in concrete pavements around the columns indicating foundation rotations were clearly found in some not collapsed buildings.



(*a*) (*b*)

Figure 5.3. *Full collapse of an industrial building, with failure of intermediate columns and main beams.*



(*a*) (*b*)

Figure 5.4. *Full collapses of industrial buildings, with evident rotations of the column foundations*.

6. **Failures of external cladding walls**

Several failures of external cladding walls occurred, especially in the cases of horizontal panels (Figures 6.1). A visual detection always showed the lack of appropriate connections

for horizontal forces, being typically the wall elements just supported one over the other, with just a couple of connectors on the external columns. Especially in the top wall elements, the lack of the stabilizing vertical force and the presence of connectors only in the lower part of the panels, did not provide for a sufficient constraint.

The behaviour of vertical cladding walls was better, in general. When clamped in the lower reinforced concrete foundation beam, they also provided a significant stiffness to the building, which did not allow the rotation or damage of the columns (Figure 3.3). Some collapses of vertical panels not clamped in the foundation and with a sandwich structure have also been found (Figure 6.2).

Figure 6.3 shows the collapse of the external cladding horizontal panels of another industrial building. In this case, the collapse was also related to the lateral rotation of the beams carrying the roof elements (Figure 6.3*b*). In the same picture, the failure of the system connecting the external panels with the columns can be clearly seen.



(*a*)



(*b*)

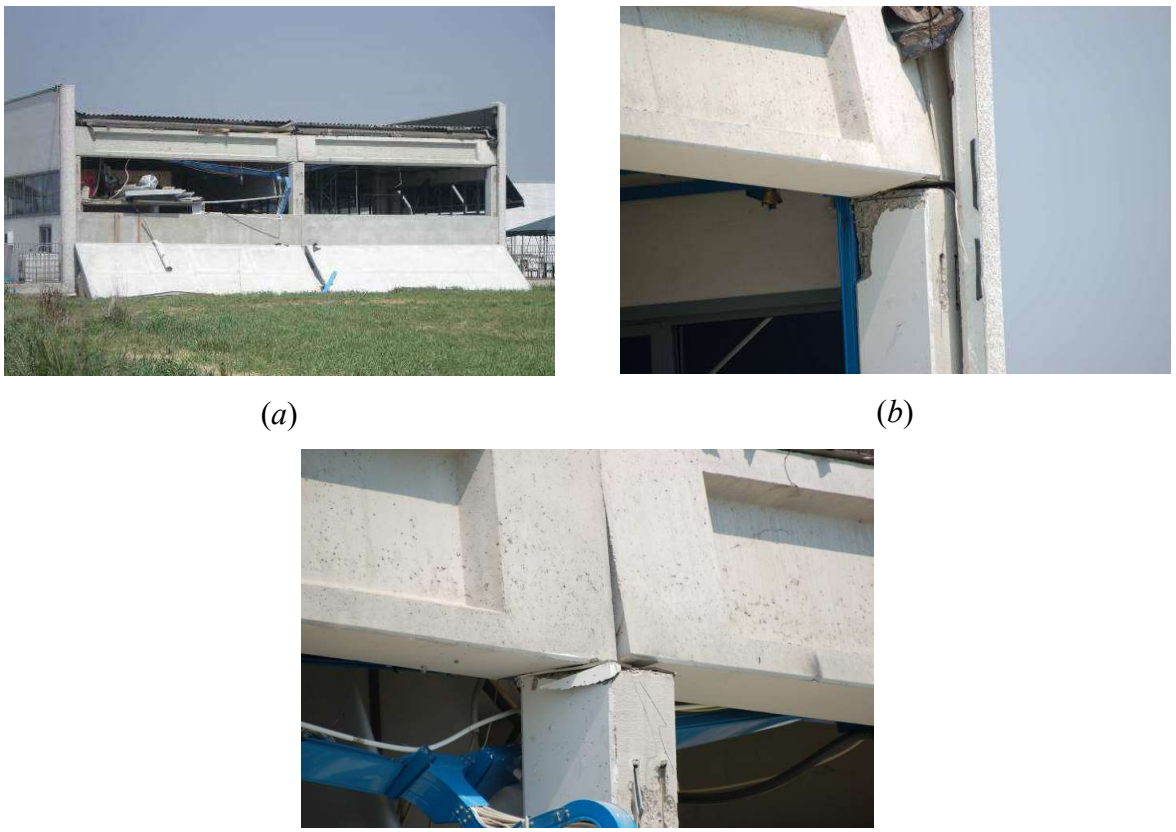


(*c*)

Figure 6.1. *Falling of horizontal cladding panels.*



Figure 6.2. *Full collapse of sandwich vertical cladding panels not clamped in the foundation.*



(*a*) (*b*)

©

Figure 6.3. (*a*) *Collapse of the external cladding panels and* (*b, c*) *details of the rotation of the high rectangular beams that supported the panels. In the pictures, the failure of the connection systems of the cladding panels on the columns can also be seen.*

5. **Conclusions**

In the present paper, several failures and collapses of industrial buildings after the Emilia earthquake mainshocks are documented. The main causes of the failures are related to the

lack of seismic design requirements in the region until a few years ago. A preliminary classification of the main collapses documented can be the following:

- Loss of support of the roof elements from the main beams;
- Loss of support of the main beams from the column supports;
- Loss of lateral stability of high main beams;
- Failures of the cladding panel connections.

In several cases, the following factors contributed to the aforementioned failures: the interaction of portal frames with irregular masonry infill panels; the failure of intermediate columns exhibiting large displacements due to formation of plastic hinges at the base and the absence of a rigid-diaphragm behaviour of the roof; the rotation of the foundations, mainly in the case of pocket precast foundations not anchored to the cast-in-situ foundation.

7. **Acknowledgements**

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SOMMARIO ESTESO IN ITALIANO

Nel maggio 2012 la regione Emilia è stata colpita da due forti eventi sismici, caratterizzati analoga energia: la prima scossa, con magnitudo locale M_L = 5.9 è avvenuta il 20 maggio, e la seconda scossa (M_L = 5.8) il 29 maggio [Dolce et al., 2012a, b]. Le accelerazioni massime orizzontali registrate durante questi due eventi sono state rispettivamente PGA_h = 303,30 cm/s^2 (distanza epicentrale 13,4 km) e PGA_h = 240 cm/s^2 (distanza epicentrale 16.0 km). Nelle zone epicentrali sono state registrate accelerazioni massime verticali fino a 900 cm/s^2. La regione interessata dai terremoti non era storicamente valutata come sismica; infatti, solo dal 2005–2006 un aggiornamento della mappa di pericolosità sismica ha definito la zona come a bassa/media sismicità. Per tali ragioni la maggior parte degli edifici prefabbricati interessati dal sisma sono stati progettati senza alcuna regola sismica.

Il sisma del 20 maggio ha causato il crollo di diversi edifici prefabbricati nelle zone industriali di S. Agostino, Bondeno, Finale Emilia, S. Felice sul Panaro, mentre la scossa del 29 maggio è stata particolarmente severa per gli edifici industriali in Mirandola, Cavezzo e Medolla. Le prime stime indicano che, in alcune aree industriali, quasi il 70% degli edifici prefabbricati ha subito crolli o è stato gravemente danneggiato. Gli edifici prefabbricati non progettati secondo corretti criteri antisismici hanno mostrato anche in passato gravi vulnerabilità.

La struttura tipica dell'edificio industriale è composta da una serie di telai ad un solo piano, caratterizzati da colonne incastrate in fondazione e trave semplicemente appoggiate. Poiché le zone interessate dai terremoti in oggetto erano classificate non sismiche sino al 2005, i collegamenti trave-colonna erano in genere di tipo attritivo senza alcuna connessione di tipo meccanico; solo in alcuni casi è possibile osservare un appoggio in neoprene tra trave e pilastro. In generale non sono presenti pareti di taglio e i meccanismi resistenti ai carichi orizzontali (tipicamente la sola azione del vento) sono basati sul comportamento a mensola delle colonne.

Sulla base della tipologia di danni osservati è conveniente per chiarezza di esposizione identificare due diverse tipologie di prefabbricati: i) Tipo 1: edifici prefabbricati più vecchi (costruiti negli anni '70 e '80); Tipo 2: edifici prefabbricati di più recente realizzazione (costruiti dopo il 1980).

La maggior parte degli edifici di Tipo 1 presenta travi prefabbricate con campate di circa 12 – 20 m. e interassi tra telai di circa 6 - 10 m (Figura 3.1). Le travi prefabbricate sono tipicamente a doppia pendenza e le colonne sono abbastanza snelle, con sezioni quadrate di lato 30 – 40 cm. Le travi non hanno sistemi di ritenuta o irrigidimenti per movimenti fuori dal piano, in particolare quando sia le travi trasversali sia quelle longitudinali sono collegate in

testa alla colonna. La maggior parte della costruzioni di questo gruppo presenta tamponature in muratura.

Nella seconda tipologia di prefabbricati sono tipicamente presenti travi sagomate con sezione filante a I (Figura 3.2) con interasse tra le colonne di circa 10 –12 m. La copertura è composta da diversi tipi di elementi precompressi, tipicamente profili alari o microshed, con interposizione di elementi traslucidi. Questi ultimi rendono il piano di copertura particolarmente deformabile. I pannelli di chiusura verticale sono anch'essi prefabbricati e sono disposti orizzontalmente o verticalmente. (Figure 3.2, 3.3).

La maggior parte degli edifici prefabbricati realizzati negli anni '70 e '80, (Tipo 1) sono stati solo lievemente danneggiati dal terremoto (Figura 4.1), probabilmente grazie alla rigidezza dei pannelli murari, che ha permesso il trasferimento della forza sismica alle pareti in muratura esterne e alle pareti stesse. Nei casi in cui erano presenti finestrature a nastro sotto la trave prefabbricata, l'interazione del muro con le colonne ha spesso causato danni critici, fino al collasso della colonna (Figure 4.2–4.4). L'irregolarità in altezza dei pannelli di tamponamento ha creato talvolta il collasso dei tamponamenti stessi, come mostrato in Figura 4.3b e 4.5.

Più complessi e vari sono i meccanismi di rottura degli edifici industriali più recenti (Tipo 2). Gli effetti più ricorrenti ed evidenti sono inerenti alla perdita di appoggio i) degli elementi di copertura dalle travi e ii) delle travi dai supporti delle colonne, in alcuni casi anche senza alcun danno evidente nelle colonne stesse (Figura 5.1). Più spesso i crolli sono dovuti alla crisi di alcune colonne intermedie, coinvolgendo anche importanti porzioni dell'edificio (Figura 5.2–5.3). Infine, alcuni collassi sono legati a rotazioni delle colonne in corrispondenza delle fondazioni (Figura 5.4), dovute al fatto che l'azione sismica ha prodotto sollecitazioni sugli elementi di fondazione molto maggiori rispetto a quelle attese, valutate tipicamente per il solo effetto dell'azione del vento.

Infine, numerosi crolli hanno riguardato pareti di tamponamento esterne, soprattutto nei casi di pannelli disposti in orizzontale (Figure 6.1). In generale è stata riscontrata l'assenza di adeguate connessioni nei confronti di forze orizzontali, soprattutto con riferimento agli elementi posti in prossimità della copertura. I pannelli disposti in verticale si sono rivelati meno vulnerabili, grazie al vincolo imposto dalla trave di fondazione (Figura 3.3). La figura 6.3 mostra il crollo dei pannelli esterni orizzontali di un edificio industriale nel quale il ribaltamento del pannello è stato anche associato ad una rotazione delle travi laterali in copertura (Figura 6.3b).

Si può quindi concludere che le cause principali dei danni e dei crolli che hanno interessato edifici prefabbricati ad uso industriale sono principalmente da attribuire alle vulnerabilità degli edifici stessi, non progettati con criteri antisismici. In particolare, gran

parte delle crisi sono state causate dalla perdita d'appoggio delle travi prefabbricate dai pilastri e dall'interazione con le pareti non strutturali. Inoltre si sono verificati numerosi crolli che hanno coinvolto i pannelli di tamponamento esterni in cemento armato prefabbricato, a causa di sistemi di ancoraggio sugli elementi portanti (pilastri e travi) insufficienti o inadeguati. Infine, alcuni crolli sono stati dovuti all'inadeguatezza dei sistemi di fondazione.

DIDASCALIE IN ITALIANO

Figura 3.1. Trave a doppia pendenza con tamponamenti in muratura

Figura 3.2. Edificio prefabbricato a telaio con elementi precompressi di copertura e pannelli prefabbricati orizzontali.

Figura 3.3. Edificio prefabbricato con pannelli prefabbricati verticali

Figura 4.1. Edificio prefabbricato con pannelli in muratura, questi leggermente danneggiati dal terremoto.

Figura 4.2. Edificio prefabbricato collassato e con pannelli in muratura esterne regolari.

Figura 4.3. Collasso causato dall'irregolarità in altezza della muratura (presenza di finestrature a nastro)

Figura 4.4. Crisi della travata esterna a causa della interazione con elementi non strutturali di facciata.

Figura 4.5. Crisi fuori piano di pannelli in muratura non solidari con gli elementi in cemento armato, a causa di finestra sotto la trave prefabbricata.

Figura 5.1. Caduta di elementi prefabbricati di copertura, senza alcun danno evidenti delle colonne verticali.

Figura 5.2. Caduta degli elementi di copertura dovuti all'irregolarità dell'edificio.

Figura 5.3. Collasso completo di un edificio industriale, con crisi dei telai intermedi.

Figura 5.4. Collassi completi di edifici industriali, con evidenti rotazioni alla base delle colonne.

Figura 6.1. Caduta di pannelli orizzontali.

Figura 6.2. Collasso completo di pannelli di rivestimento a sandwich verticali non vincolati alla fondazione.

Figura 6.3. (a) crollo dei pannelli di rivestimento esterno e (b, c) dettaglio della rotazione delle travi. Nella seconda immagine, si noti l'inefficienza dei sistemi di connessione dei pannelli alle colonne.