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DAMAGE SUFFERED BY INDUSTRIAL BUILDINGS DURING THE 2012 EMILIA (ITALY) EARTHQUAKES

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ABSTRACT

The seismic sequence that struck the Emilia Region of Northern Italy at the end of May 2012 had a strong impact on a highly industrialised area of recent codified seismicity. Industrial buildings, which have high exposure in terms of human life, building content and importance of production processes, have been severely damaged. In this study, damage observed during field surveys in such buildings is presented. The interpretation of the structural performance is carried out recognising different types of damage, e.g. formation of plastic hinges at the base of the columns, short-column failures, unseating of the beams and collapse of cladding panels. Non-linear dynamic analyses are performed on plane models representing typical configurations of industrial buildings present in the most affected area. The analyses highlight the directionality of damage, the relevance of the vertical component of earthquake excitation, and the significant inelastic rotation induced in the columns. Finally, recommendations are made to improve the seismic behaviour of these types of structures.

INTRODUCTION

At the end of May 2012 a seismic sequence struck the Emilia Region of Northern Italy, with two main events on 20th May (local magnitude M_L 5.9) and on 29th May (M_L 5.8) (INGV, 2012a, 2012b). On 3rd June another event of magnitude M_L 5.1, located to the W of the previous ones (Figure 1) struck the same area (INGV, 2012c). Since the 20th of May a total of 7 events with magnitude $M_L \geq$ 5.0 occurred. The earthquakes were caused by the reactivation of the basal thrust in the central portion of the Ferrara-Romagna arc (Mirandola Earthquake Working Group, 2012). The fault planes lie approximately along the E-W direction (INGV, 2012a).

The strong ground motion was rather severe in the near fault, especially in terms of horizontal displacements and vertical accelerations, and highlighted the presence of a rather long pulse in the N-S components of the near-fault records (Liberatore et al., 2013a). This is probably related to the N-S direction being approximately normal to the causative faults. The maximum intensity, measured on the European Macroseismic Scale EMS98, was as high as 8, considering the cumulative effect of the sequence. Isoseismal lines, traced on the basis of the observation of (Arcoraci et al., 2012), are shown in Figure 1.

Based on historical data, the seismicity of the area may be defined as medium-low. For the epicentral area the present seismic zonation (DM 2008) specifies an expected maximum horizontal

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acceleration approximately equal to 0.15 g on stiff soil, and of 0.22 g on type C soil (for a return period of 475 years).

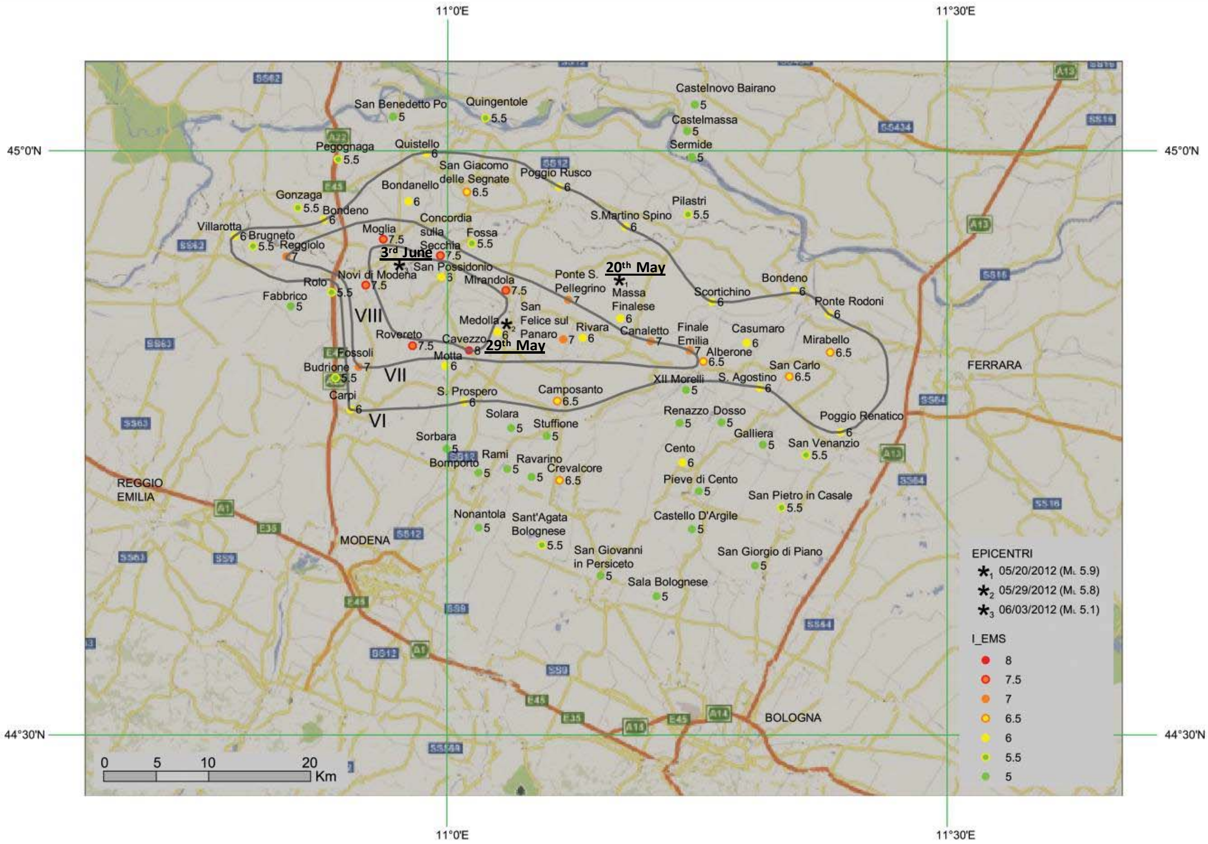


Figure 1 Most affected area and isoseismal lines (after Liberatore et al., 2013a). Macroseismic Intensity refers to the cumulative effects of the events that affected the area. Stars indicate the epicenters of the events

The seismic sequence caused significant damage to industrial buildings (Savoia et al., 2012, Liberatore et al., 2013b), masonry buildings (Bracchi et al., 2012), churches (Sorrentino et al., 2013a), rural buildings (Sorrentino et al., 2013b) and, to a lesser extent, to reinforced concrete (RC) residential buildings (Verderame et al., 2012, Liberatore et al., 2013c) and bridges (Franchin et al., 2012).

There were seven casualties of the main shock of 20th May, of which four were caused by the collapse of industrial plants (Clemente et al., 2012). The 29th May aftershock caused a total of 20 further casualties, most of them being workers involved in the rescue of equipment and goods. Altogether, the seismic sequence had a great impact on manufacturing buildings, which are at high risk in terms of human life, building content and importance of production processes. In this study, damage observed in industrial buildings during field surveys is presented. The interpretation of the structural response is carried out recognising different types of damage related to: column base, short column, column top, shed beam, roof element, cladding / infill panel and steel stand. Finally, non-linear dynamic analyses on plane models representing typical configurations of industrial buildings present in the most affected area are performed with the aim of interpreting the observed structural damage

OBSERVED DAMAGE

Approximately 500 factories in the Modena district have suffered severe structural damage. Most of these buildings were not designed to withstand seismic action, because the zone was not classified as a seismic-prone area until 2003. The severe damage that affected these buildings has been probably the most controversial issue raised by the Emilia earthquakes, because of the high exposure in terms of human life, building content, and the importance of production process continuity (Braga et al., 2014).

Examples of damage that occurred to this type of structure are reported here, based on a survey of more than 30 buildings. The inspection level varied based on the level of interest associated with the damage and the accessibility of the site. In several cases the buildings to be surveyed were suggested by local experts and inspection took place in the presence of the owners.

Most precast RC structures surveyed in the area are single-storey buildings, with pitched roof of limited slope. In older buildings the roof is a clay-block RC diaphragm, more recent buildings present precast roof elements. The roof is usually supported by transverse shed beams; longitudinal beams may or may not be present, depending on the arrangement of the roof elements. Shed beams rest on pocket supports at the top of the columns, supports and beams are not connected with dowels, and usually no rubber pad is present between concrete surfaces. The columns are clamped to the foundation through sleeve footings, not connected to one another. Lateral closure is achieved in older buildings through unreinforced brick- or block-work; recent buildings resort to external precast cladding panels, either horizontal or, less frequently, vertical, which make it possible to fasten the construction process. Most of the buildings examined have been designed according to the 1987 code for precast RC structures (DM 1987), which allowed friction connection at joints outside of seismic-prone zones.

In the following, the description of the structural performance is carried out according to the elements involved, i.e. columns, beams and roof elements, cladding/infill panels, steel stands.

Damage to columns

Columns have suffered different types of damage: i) formation of a plastic hinge at the base, ii) short-column failure, iii) failure at the top.

The activation of a plastic hinge at the base of a column (e.g., Figure 2a) has been observed in more than 40% of the buildings investigated. As a matter of fact, despite the reduced ductility due to axial force, the formation of plastic hinges at the base represents the main source of energy dissipation of these buildings due to the absence of bilateral connection between columns and beams. Rebar buckling has been observed in a few examples (Figure 2b); in these cases the rotation of the columns has been substantial.

Short-column failure has been fairly frequent due to infills (Figure 2c). In other cases it has been originated by adjacent new constructions without an adequate seismic joint or by contiguous halls of different height (Figure 2d). Another short-column source is the presence of sawtooth roofs with inclined beams: the marked change in stiffness at the base of the tooth induced the severe damage shown in Figure 2e.

The top of the columns has been frequently impaired. Two types of damage have been reported: spalling of the concrete directly supporting the beam and failure of the cantilever lateral restraints of the pocket supports (Figure 2f). The first type of damage, which can be related to the very thick fire-protection cover concrete, and to the lack of a rubber interface between the concrete elements, is seldom critical, unless coupled with beam sliding. On the contrary, the second damage mode is frequently associated with beam unseating. The connection between the two lateral restraints and the beam head may reduce the bending moment at the base of the pocket support walls and hamper the unseating of the beam.

The activation of a plastic hinge at the base, the short-column mechanism and the failure of the lateral restraints of the pocket supports are the only failures related to the reinforcement observed during the surveys. The 1987 code for precast structures and the 1986 code for earthquake-resistant structures (DM 1986) did not specify any regulations in terms of design for shear against short-column failure and minimum amount of longitudinal reinforcement. On the contrary, such provisions are present in the current code. Contrarily, the 1987 code prescribed a verification of the pocket support against bending moment, which is not present in the current building code. Clearly, the design of the lateral restraints of the pocket did not avoid the failures represented in Figure 2f.

(a) (b) (c) (d) (e) (f)

Figure 2 Damage to columns: a) plastic hinge at the base, b) buckling of longitudinal rebars, c) short column failure due to infills, d) short column failure due to different height of adjacent halls, e) short column failure at sawtooth roof base, f) failure of the lateral restraint of a pocket beam support

Damage to beams and roof elements

The most severe damage to beams has been their unseating (e.g., Figure 3a), which was observed in almost 30% of the buildings investigated. It is clear that the reliance on friction alone is not sufficient to prevent such damage, especially when a significant vertical component of ground motion can be expected, as investigated in the next section. Because of any lack of redundancy, a catastrophic collapse of the roof has usually followed the failure of one or more shed beams (Figure

3b). A case of out-of-plane rotation of the shed beam is presented in Figure 3c. This type of damage is associated with (unconnected) roof panels, while older clay-block RC slabs avoided such rigid-body movements.

An uncommon damage is shown in Figure 3d. The building has internal shed beams, which suffered limited displacement at beam-column joint, due to column rotation. However, on 20th May the façade suffered a very severe crack of the tympanum, whose beam is completely unreinforced. Due to its high vulnerability, the building front collapsed on 29th May, although most of the surrounding structures survived the new event without additional damage. Therefore, particular care should be taken for the assessment of older buildings, checking for adequate reinforcement in cast-in-place beams, lintels and cantilevers.

Unseating of the roof panels has been observed in a few cases, although less frequently than that of the shed beams. Of course, collapse of the shed beams systematically induced that of the roof panels. Older clay-block RC slabs usually performed better, being able to behave as rigid diaphragms.

(a)

(b)

(c)

(d)

Figure 3 Damage to beams and roof elements: a) unseating of shed beams, b) collapse of the roof, c) out-of-plane rotation of a shed beam, d) failure of the façade tympanum due to the lack of reinforcement in the concrete beam

Damage to cladding/infill panels

Cladding panels have been the most vulnerable elements of industrial buildings: almost two factories out of three suffered such damage. The same behaviour has been observed in other earthquakes where the overall structural performance was much better, such as in L'Aquila (Toniolo and Colombo, 2012) and in Christchurch (Henry and Ingham, 2011).

Based on a survey of the damaged buildings, it is quite difficult to understand if the connections failed due to in-plane or out-of-plane forces. No in-plane displaced configuration has been observed: this seems to suggest a predominance of the out-of-plane failure. However, it is still possible that the fastenings failed as a result of combined in-plane and out-of-plane actions, with the panel finally overturning because of lack of any other lateral restraint.

The high vulnerability of cladding panels is evident in the case shown in Figure 4a. The light connections proved inadequate to restrain the heavy elements. Concrete pullout has been registered in several buildings, as proved by the steel connectors attached to columns or beams. In some other cases it is the channel section embedded in the column that failed, whose brim deformed so much as to allow the connector head to be extracted. On the contrary, the steel connection of Figure 4b, between external panel and channel bar embedded in the column, failed in tension.

Vertical external cladding panels embedded in the flooring have shown a much better performance. In the buildings surveyed the occurrence of damage to the closure elements has been less significant when brick- or block-work infills have been used; nonetheless, in a few cases shear cracks or out-of-plane dislocations have been recorded.

Finally, it seem interesting to mention that a sensitivity to ground motion directionality has been observed in different out-of-plane collapses (Liberatore et al., 2013b).

(a) (b)

Figure 4 Failure of cladding panels: a) failure due to the light connections, b) tensile failure of the steel fasteners (both left and front sides)

Reduction of the vulnerability of cladding elements is one of the most urgent tasks for improving the seismic performance of precast RC buildings. This can be done through the design of the fastenings for appropriate earthquake-related forces. The 1986 seismic code prescribed generically to guarantee external cladding panels against separation from the structure (DM 1986). The performance in this and L'Aquila earthquakes indicates that no effective measures were taken. The current Italian code provides rules to calculate the acceleration to be applied on any cladding panel (DM 2008), but does not specify its distribution among the fastenings. A procedure for estimating the design force in the fastenings is suggested by Liberatore et al. (2013b). It is based on the statics related to the out-of-plane mechanism and assumes that the rotation takes place around the bottom support and that a symmetric top-bottom arrangement of the fastenings is provided.

Damage to steel stands

Damage to steel stands has been also observed. However, it is difficult to estimate the occurrence of this kind of damage because it was not always possible to survey the interior of the buildings.

Given that the cost of the structure is only a fraction of the value at risk, which is also represented by the equipment, the stored goods and the possibility to keep production ongoing, collapse of the storage stands inside the building may have serious consequences. Moreover, and more important, it may endanger the life of the workers.

Different situations are present according to whether the steel stands are independent from the structure or not, in the latter case steel stands may be connected to the structure or be part of the main structure itself. The steel stands in Figure 5a are not connected to the precast RC structure. They have longitudinal braces every four bays. Although the braces are eccentric, since not in the same plane as columns and beams, and although most of the braces buckled, the stands did not suffer a total collapse and were also able to support the roof elements, once the front shed beam failed. This proves that these secondary structures may have a safety reserve with regard to gravity loads.

Probably, the most catastrophic failure surveyed among factories is that of the building in Figure 5b, induced by the May 20th event. In this case the stands are also the structure of the building. The steel trusses are connected to each other only at roof level, where a truss beam linked the stands in the transverse direction; from what is visible in the surviving portion of the structure, the longitudinal braces are inserted in a single bay; at least seven bays without longitudinal braces have been counted. This can be explained by the fact that longitudinal braces reduce the storage capacity of these stands. As a matter of fact the stands are usually 1.0 m deep, while pallets are 1.2 m deep, and goods stored on pallets may even exceed this size. Therefore, longitudinal braces, even if confined to the back face of a stand, are considered a waste of valuable storage space. In this building the longitudinal stiffness and strength are very small. The high flexibility of the structure and the presence of considerable masses are likely to be the causes of collapse. The second-order P-Δ effect due to the horizontal deformations, increased by the heavy masses, triggered the loss of stability and led to the consequent total failure of a significant part of the building.

(a)

(b)

Figure 5 Damage to steel stands: a) damaged steel stands not connected to the main structure, b) collapsed building, in which the steel stands were part of the structure

CASE STUDY

With the aim of interpreting the observed structural damage, non-linear dynamic analyses of a transverse frame have been performed. To consider the most typical situations, two different types of beam-column connections have been investigated: beam pinned to the columns and beam resting on the column tops (Figure 6). Span and height are typical of the buildings surveyed, the spacing between two consecutive frames has been assumed equal to 8 m. The columns have a 450×600 mm^2 cross-section and the beam has a 250×900 mm^2 cross-section. Two hypotheses for the columns longitudinal reinforcement ratio have been adopted: i) 0.3% of the gross cross-section area, with rebars at corners (as surveyed), ii) 1.0% of the gross cross-section area (as the minimum requirement in the current Italian building code, DM 2008), distributed at corners and at the centre of each side. In the case of beam resting on the column tops, a frictional spring is used to model the joint. The coefficient of friction is assumed equal to 0.4 (Tassios and Vintzēleou, 1987). Further details on the models are given in Liberatore et al. (2013b).

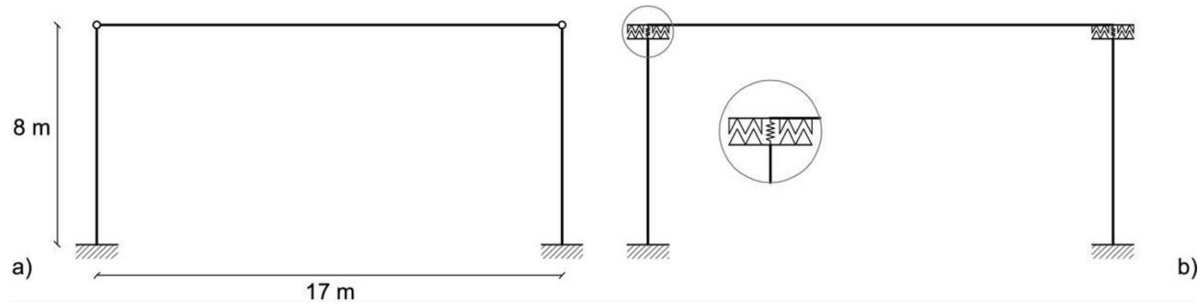


Figure 6 Structural models considered: a) beam pinned to the column; b) beam resting on the column tops, a frictional spring is used to model the joint

The accelerograms recorded in Mirandola on 20th and 29th May have been used as seismic input. With the aim of evaluating the role of the vertical component of the ground motions, the analyses were performed both with the horizontal component alone (either North-South, N, or East-West, E) and with the horizontal components combined with the vertical one (NV, EV). Some Intensity Measures of the accelerograms are reported in Table 1. The vertical peak ground acceleration component is noticeable, especially for the shock of 29th May. Intensity Measures estimated for the N-S components are generally much higher than those of the E-W components, this is particularly the case for velocity-related and displacement-related measures, such as PGV, PGD, IV and ID.

Table 1. Intensity Measures of the accelerograms recorded in Mirandola.

Station/day_ component	PGA	PGV	PGD	Δt_{vh}	IV	ID	I_A	D_S
	g	cm/s	cm	s	cm/s	cm	cm/s	s
Mainshock of May 20th - closest distance to the fault projection = 8 km								
MRN/20_N	0.26	46.5	10.9	1.38	73.8	16.8	86.7	6.0
MRN/20_E	0.26	29.8	9.4	0.88	31.2	8.6	71.3	5.5
MRN/20_V	0.31	5.9	2.2	-	7.0	1.7	47.5	5.8
Mainshock of May 29th - closest distance to the fault projection = 1 km								
MRN/29_N	0.29	40.1	19.8	2.41	78.4	28.6	132.2	7.0
MRN/29_E	0.23	23.6	9.2	1.57	25.3	12.6	78.2	7.5
MRN/29_V	0.87	22.7	5.7	-	21.5	8.4	279.5	5.3
PG = Peak Ground, A = acceleration, V = Velocity, D = Displacement, Δt_{vh} = interval between vertical and horizontal PGA, IV = maximum incremental velocity, ID = maximum incremental displacement, I_A = Arias Intensity, D_S = Significant Duration.								

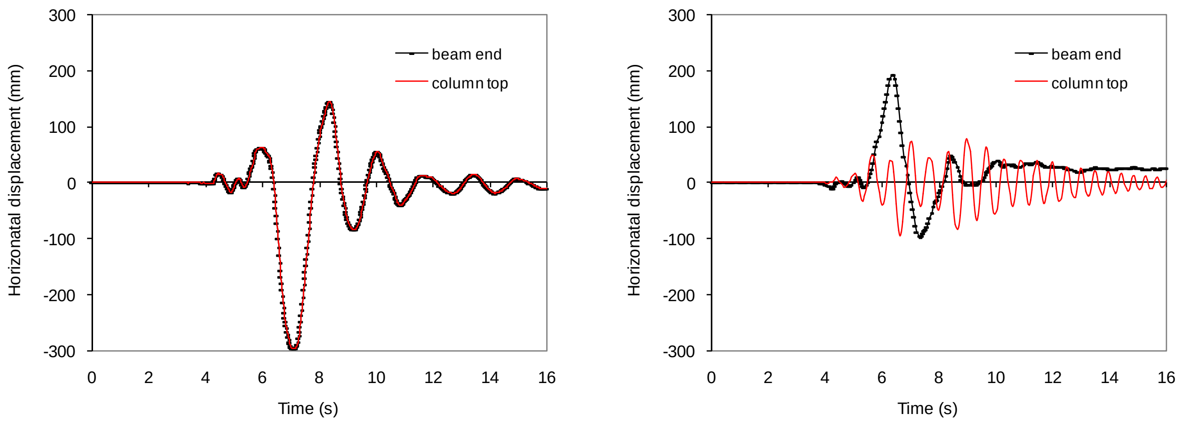
In Table 2 the maximum inter-storey drift (δ_{max}) and the beam end-column top relative horizontal displacement for the frictional spring model (Δd_{max}) are reported. There is a strong influence of the component (N or E) on the response in terms of displacement. The maximum drift induced by the N component is on the average twice of that due to the E component. This is consistent with the observations of damage directionality and with the rocking spectra presented in Sorrentino et al. (2013b). As shown in Liberatore et al. (2013b), the influence of the component is recognizable also with reference to the base shear, although this is reduced because plastic hinges form at the base of the columns in most of the analyses.

The influence of the steel reinforcement ratio on the displacement demand is moderate for the May 20th event and higher for the 29th event. Considering that the column is clamped at the base and has a zero-bending moment at the top, the drift is coincident with the chord rotation. According to the model by Panagiotakos and Fardis (1999) for the rotation capacity, and assuming the axial force induced by gravity loads, the ultimate capacity of the "lightly" reinforced column is equal to 1.20% and that of the "highly" reinforced column to 1.64%. The comparison between the maximum drift estimated from the analyses and the capacity shows that, despite an impact of the increased reinforcement on the capacity, the minimum 1.0% reinforcement of the code is clearly insufficient when considering the N component of the motion.

Table 2. Maximum inter-storey drift (δ_{max}) and beam end-column top relative horizontal displacement (Δd_{max})

Seismic input	Pinned model Steel reinf. = 0.3%	Pinned model Steel reinf. = 1.0%	Frictional spring model Steel reinforcement = 0.3%	
	δ_{max} (%)	δ_{max} (%)	δ_{max} (%)	Δd_{max} (cm)
MRN/20_N	2.1	2.2	2.2	0
MRN/20_E	1.5	1.2	1.4	0
MRN/20_NV	2.2	2.2	1.2	16.1
MRN/20_EV	1.2	1.2	1.0	10.5
MRN/29_N	4.2	2.7	4.0	0
MRN/29_E	1.6	0.9	1.4	0
MRN/29_NV	3.9	2.8	2.6	22.1
MRN/29_EV	1.5	0.9	1.4	11.3

Considering the frictional spring model, no relative horizontal displacements between beam ends and columns top are observed when the vertical component is not considered (Table 2). Contrarily, the presence of the vertical component leads to relative displacement even when considering the May 20th event, especially in the North-South direction. The very large vertical component in Mirandola on 29th May has caused relative displacements up to 220 mm (Table 2, Figure 7b). Such displacement demand is to be compared with the corbel depth, which may often amount to less than 100 mm. This may explain the observed unseating of the beams. Moreover, it should be noted that no out-of-plane response has been considered in these analyses; in some cases out-of-plane rotations induced the failure of the pocket supports and, thus, the unseating.



(a) (b)

Figure 7 Horizontal displacement: a) MRN29_N; b) MRN29_NV

CONCLUSIONS AND RECOMMENDATIONS

The seismic sequence that affected the Emilia region at the end of May and the beginning of June, 2012, caused significant damage to industrial buildings. Such buildings proved to be vulnerable to the large displacement demand and high vertical accelerations, which reduce the friction reaction at supports, as also showed by non-linear dynamic analyses carried out on a fibre model with either pin connection or frictional spring at beam-column joint.

The analysis of the observed performances carried out in this study may serve as a guide to possible strategies aimed at improving the seismic behaviour of these structures. The following recommendations are suggested.

- Foundations should be connected to one another and the collaboration of the building flooring, frequently rather thick, should be taken advantage of. These details will grant synchronous motion and an effective rotation restraint at the base.
- The base of the column should have appropriate stirrup spacing, in order to avoid the buckling of longitudinal bars, and adequate longitudinal reinforcement. Intervention on existing structures with reinforced concrete, steel or fiber reinforced polymers can supply sufficient confinement and

improve ductility. According to the type of intervention, an increase of stiffness and/or strength can also be obtained (Gruppo di Lavoro, 2012).
- Short-column failure should be averted by: appropriate design of the infills, adequate joint between adjacent halls or buildings, avoiding inclined-beam sawtooth roofs, specific design of the column.
- Beams and columns should be connected using mechanical devices to prevent lateral failure of pocket supports and the more dangerous unseating of the beam; as a matter of fact, the numerical analyses proved that it is not safe to rely on friction, as stated also in the Italian code.
- The fastening between roof panel and beam will preclude the unseating of the top element and the rigid-body torsion of the beam; moreover, it will stabilise the columns in the direction normal to the beam.
- The cover concrete should not be too thick, in order to avoid spalling due to beam pounding and a rubber interface should separate concrete elements.
- Particular care should be devoted to the connection between the main structure and cladding panels, in order to control their dynamic interaction with the resisting frames and to avoid their dangerous out-of-plane failure (Liberatore et al., 2013b).
- The vulnerability of steel stands, which seems related to inadequate longitudinal bracing, should be prevented by means of design for earthquake-induced loads. European Codes for both static (CEN 2005, 2009) and earthquake loads (CEN 2004) provide rules and recommendations. Moreover, in-depth recommendations about inspection and possible retrofitting of internal stands are specified in Consiglio Superiore Lavori Pubblici (2012).

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