OPENSEESDAYS

PERFORMANCE OF PRECAST BUILDINGS DURING EMILIA-ROMAGNA EARTHQUAKES: A CASE STUDY

Marianna ERCOLINO*

Ph.D.
Department of Structures for Engineering and Architecture, University of Naples, Italy
Via Claudio 21, 80125, Naples, Italy
marianna.ercolino@unina.it

Gennaro MAGLIULO

Assistant Professor
Department of Structures for Engineering and Architecture, University of Naples, Italy
Via Claudio 21, 80125, Naples, Italy

gmagliul@unina.it

Gaetano MANFREDI

Full Professor
Department of Structures for Engineering and Architecture, University of Naples, Italy
Via Claudio 21, 80125, Naples, Italy

gamanfre@unina.it

Abstract

On May 2012 two earthquakes occurred in Emilia-Romagna region (Italy) causing several damages to existing industrial precast structures. These damages were mainly due to inadequate connection systems and the main recorded failures were the loss of support of structural elements due to the sliding of friction connections. This paper aims at justifying some of these damages by investigating the response of a real industrial building, located in Mirandola (Modena, Italy) and seriously damaged after the second main shock. In this structure the main damages were significant rotations at the columns base and relative displacements in both beam-to-column and roof-to-beam connections. Nonlinear dynamic analyses are performed in the OpenSees program with the accelerograms of the second event and the experienced damage in the structure is justified. The defined frictional element is able to simulate the behavior of both beam-to-column and roof-to-beam connections. Moreover, the nonlinear dynamic analyses demonstrate the damage in the precast columns.

Keywords: precast structure, friction modeling, earthquakes, nonlinear analyses

1. Introduction

Two earthquakes on May 2012 strongly hit RC precast buildings in Emilia-Romagna region. Most of the industrial precast structures exhibited significant damages, causing huge economic losses as well as deaths and injured. The seismic response of these structures underlined some structural deficiencies, as the connection systems vulnerability: the main collapse and damages were caused by the failure of the connections between the structural elements [1] and between structural and nonstructural components [2].

In Magliulo et al. [1] the widespread damage was justified by some simplified considerations about the seismic events and the design of the precast structures. According to the authors, both the rarity of the event and the absence of seismic details in the structures caused a bad response of the precast structures. In Belleri et al. [3] similar conclusions were presented; in this work, the authors described the most common damages as well as the main features of the damaged structures. Some simplified considerations about the seismic input demonstrated the response of the structures as well as the widespread failures of the nonstructural cladding panels.

The huge number of damaged/failed structures gave to the scientific community the opportunity to develop the knowledge of this structures by studying the real response under earthquake. Moreover, the comparison between the real behavior and the numerical results of nonlinear analyses can demonstrate the capability or inadequacy of the adopted structural modeling assumptions.

This paper studies the seismic response of a real precast buildings, damaged after Emilia earthquakes. All the main features of the structure were furnished by technical reports, whereas the damage description was given by an in situ survey after the seismic events. The study justify the main structural damages by means of nonlinear dynamic analyses. The comparison between the analytical results and the real recorded damages allows to verify some modeling assumptions for existing precast one-story structures.

2. The case study and the recorded damages

An existing one-story precast RC building is investigated in order to justify the recorded damages after the earthquake on May 29th 2012 in Emilia-Romagna region (Northern Italy). The structure is located in Mirandola (Modena, Italy) and it was designed and built in 1990. The structure (Figure 1) consists of 6 bays of 20m in X direction (transversal direction) and 5 bays of 10m in Y direction (longitudinal direction). The precast RC columns are connected to the foundation by means of isolated socket foundations and their total height is equal to 7.3 m. The cross section of the columns are reported in Figure 2 for all the typologies, individuated in Figure 1.

In X direction the columns are connected to the prestressed beams by simple friction connections: only a neoprene pad between the two concrete elements is provided with a contact surface of 23cm. In Y direction the columns are connected by girders with a U cross-section, whereas the roof elements are connected to the principal beams by friction connections, i.e. only a neoprene pad is placed between the elements with a contact surface of 13cm.

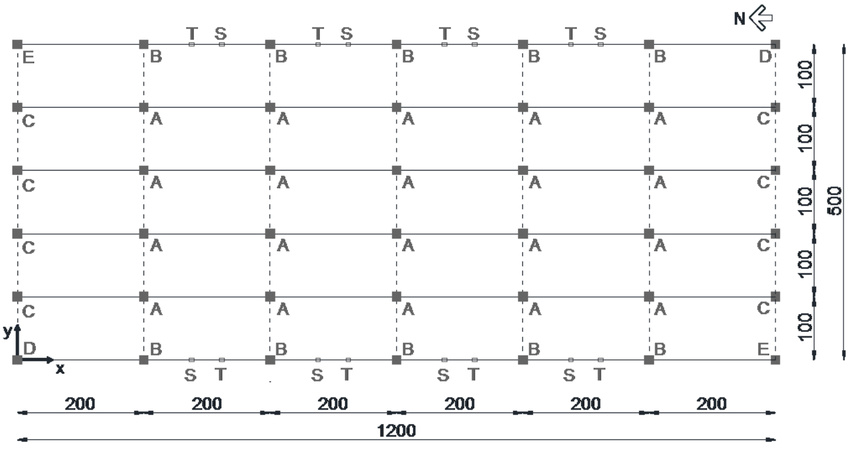
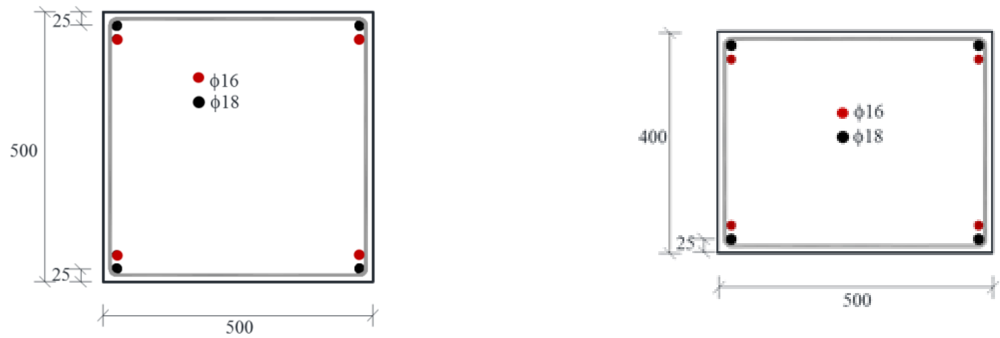


Figure 1 Plan view of the case study



(a) (b)

Figure 2 Cross sections of column for (a) columns B, D and E and for (b) columns A and C

The damage observation in the case study was performed after the seismic event on May 29th before any kinds of retrofitting actions. According to this direct survey, the main recorded damages were: 1) significant damage and rotations of the structural vertical elements in X direction; 2) significant relative displacement between the principal beams and the columns and 3) significant relative displacement between the roof elements and the principal beams.

(a) **(b)** **(c)**

Figure 3 Recorded damage after the 29th May event in: (a) columns; (b) beam-to-column connections and (c) roof-to-beam connections

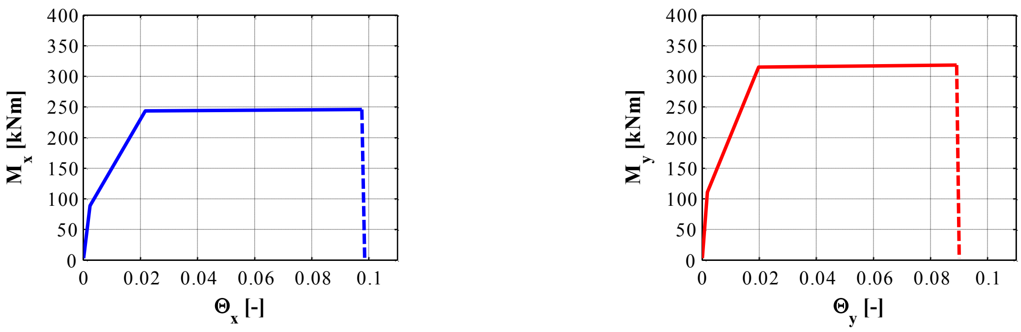
3. Nonlinear model

In order to justify the damage, nonlinear dynamic analyses are performed in the OpenSees program [4] with the accelerations-time histories recorded during the two main seismic events. The numerical model includes all the structural elements of the buildings: the beams in both the directions, the roof elements and the columns. The inelastic behavior of the structural elements (i.e. at the base of the precast columns) is introduced by means of a lumped plasticity model and all the horizontal structural elements (beams, girders and roof elements) are introduced as beam elastic elements. All the masses are distributed along the structural elements and the panels weight is introduced as a distributed mass on the supporting beams or girders.

The dynamic analyses are performed with the accelerations-time histories recorded by the station MRN of the Italian National Accelerometric Network during the two main events of the 29th May 2012.

3.1 Model of the structural elements

The moment-rotation envelope consists of three characteristic points: the cracking, the yielding and the ultimate points. The yielding moment is assumed equal to the yielding moment of the moment-curvature curve and the ultimate moment is defined considered a low hardening (1%) value, assumed for numerical issues. The yielding and ultimate rotations are evaluated according to Fardis and Biskinis [5]. The hysteretic rule follows the indications given in Ibarra et al. [6]. According to the described assumptions, the moment-rotation backbone curves are obtained for the columns of the case study; in Figure 4 the curves of the columns A are reported along both the two horizontal directions.



(a) (b)

Figure 4 Moment-rotation envelope of the column A around (a) the X direction and (b) the Y direction

3.2 Connections modeling

In order to take into account the real location of all the elements, all the eccentricities between the elements are considered. All the connections are defined with the "equalDOF" command between the two linked elements and the assumed degrees of freedom are reported in Table 1. In this table U indicates the translation degree of freedom and R the rotational ones.

Table 1 Fixed degree of freedom in the connection between the structural elements

Connection	**Fixed degree of freedom**					
[-]	Ux	Uy	Uz	Rx	Ry	Rz
Beam – column	NO	YES	NO	YES	NO	NO
Girder – column	YES	YES	YES	NO	YES	NO
Roof – beam	YES	NO	NO	NO	YES	NO

In order to model the frictional connections in the nonlinear model, the “Flat slider bearing” element of the OpenSees library (Figure 5) is introduced between the two linked structural elements. In this command the element is defined as an object between two nodes, i.e. the node on the flat sliding surface (i.e. the column) and the slider (i.e. the beam). The frictional properties are defined by a frictional model, assumed as a Coulomb model. The initial elastic stiffness is defined as the shear stiffness of the neoprene pad. The force-deformation behavior is defined for all the directions with different elastic “UniaxialMaterials”: in the vertical direction the compressive neoprene stiffness is assigned, whereas in the other directions a very flexible material is defined. In order to capture the uplift behavior of the bearing, the user-specified UniaxialMaterial in the axial direction is modified for no-tension behavior.

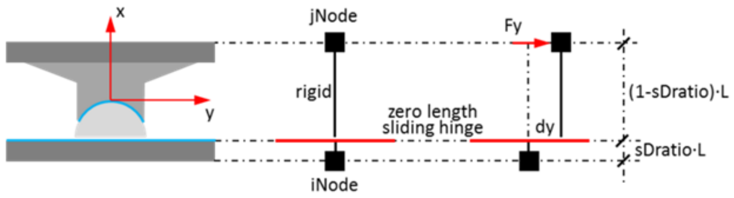
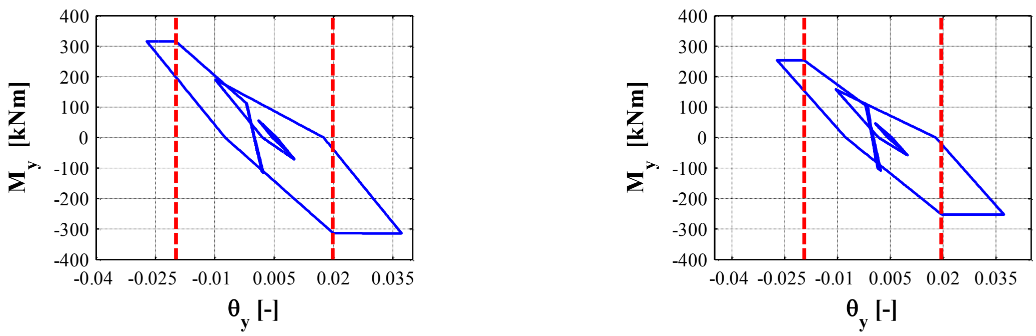


Figure 5 Flat bearing element

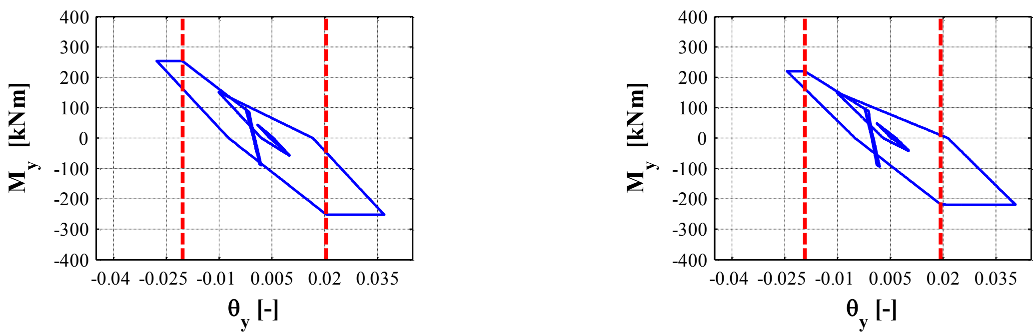
4. Results of dynamic analyses

In the following sections the results of the performed nonlinear analyses are described and discussed. A first result on the seismic response of the considered case study can be drawn in terms of moment-rotation curves, recorded in the analyses with the three components of the 29th May event (Figure 6 and Figure 7). The results demonstrate that inelastic rotations are experienced by the columns in the X direction after the second event (29th May 2012).



(a) **(b)**

Figure 6 Moment-rotation envelope around the Y direction: (a) column A and (b) column B



(a) **(b)**

Figure 7 Moment-rotation envelope around the Y direction: (a) column C and (b) column D

The assessment of the beam-to-column connection behavior is performed by investigating the frictional element response under the seismic excitations. The frictional element envelopes are reported for some typical beam-to-column connections (Figure 8) of the structure in terms of deformation-force curve. The deformation of the element corresponds to the relative displacement between the beam and the column and the force is the shear force in the connection in X direction. The figure shows that only the corner connections experiences significant dislocation of the horizontal element from the column. The described results on the beam-to-column connections do not correspond to the recorded damages, since many beams in the structure showed significant relative displacements with respect to columns after the 29th event.

Concerning the behavior of the roof-to-beam connections, Figure 9 shows the force envelopes of the central roof element for some longitudinal Y bays in the 4th transversal bay. All the reported elements do not experience any relative displacements. Also these results do not correspond with the recorded damages: most of the roof elements had significant relative displacements with respect to the supporting beams and some of them lost the support, failing during the excitation.

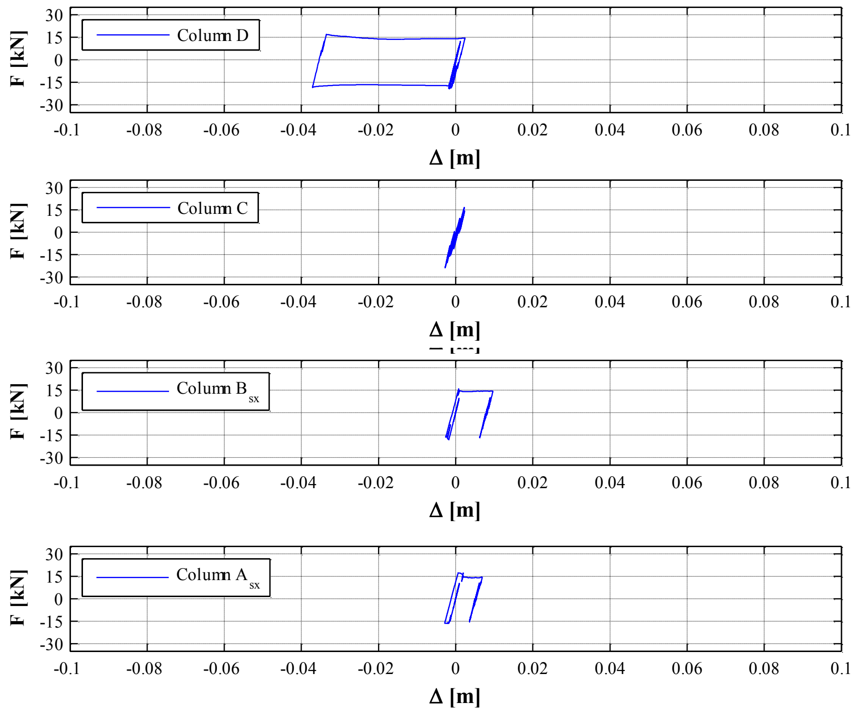


Figure 8 Force-deformation envelopes of the frictional elements (beam-to-column connections) under the horizontal components of the earthquake

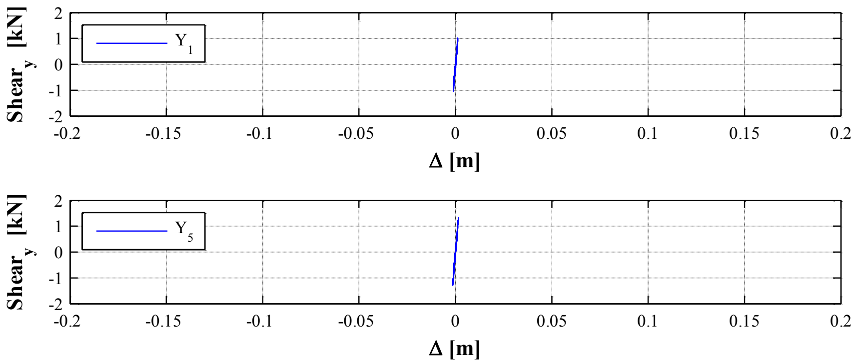


Figure 9 Force-deformation envelopes of the frictional elements (roof-to-beam connections) under the horizontal components of the earthquake

In order to justify the damage, the vertical component of the earthquake is also considered in the nonlinear dynamic analysis on the same structural model. Figure 10 and Figure 11 report the force-deformation envelopes of the frictional elements for the beam-to-column and the roof-to-beam connections, respectively. These results can justify the recorded damages in the connection systems: most of the beams and roof elements experience significant relative displacements.

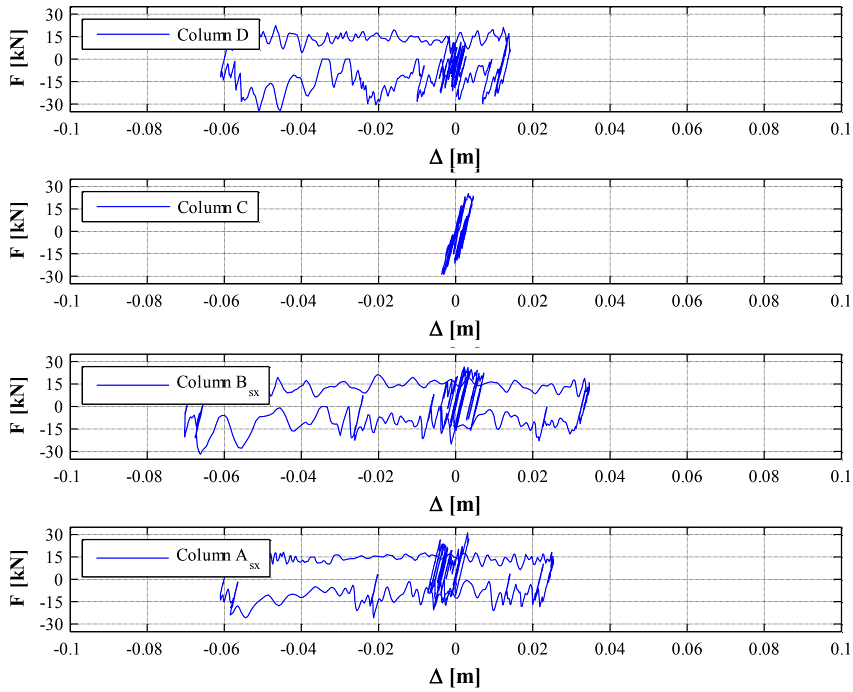


Figure 10 Force-deformation envelopes of the frictional elements (beam-to-column connections) under the three seismic components

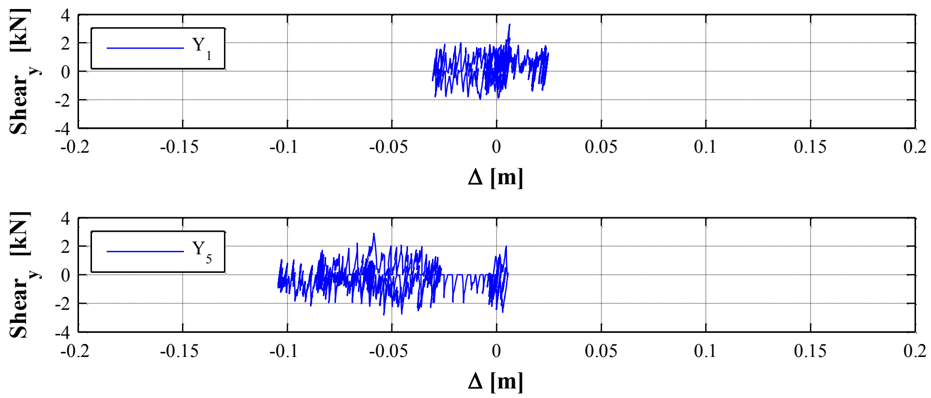


Figure 11 Force-deformation envelopes of the frictional elements (roof-to-beam connections) under the three seismic components

5. Conclusions

The seismic behavior of an existing one-story precast building is investigated by means of nonlinear dynamic analyses. The considered structure is located in the epicentral area of the Emilia earthquakes and it was seriously damaged after the second main shock. All the structural detailed are available and the damages were recorded during an in situ survey after the seismic event. A detailed nonlinear structural model is implemented in the OpenSees program by taking into account both the real layout of the structural elements and the connections systems features.

According to the results of the performed nonlinear dynamic analyses, the following conclusions can be drawn.

- A direct inspection after the 29th May event showed that the most serious damages hit the columns and the connections in the structure: significant damages at the base of the vertical structural elements occurred and significant relative displacements of the horizontal elements were experienced.
- The defined frictional element in the structural model is able to simulate the behavior of both the beam-to-column connections and the roof-to-beam connections under the seismic actions: the displacements of the analyses are quite close to the recorded displacement in the structures after the earthquake.
- The nonlinear dynamic analyses demonstrated the structural damages in the columns.
- The vertical component of the 29th May earthquake can mainly justify the significant damages in the connection systems, i.e. both the relative displacements of the beams and the cases of failure of the roof elements due to the loss of support phenomena.

6. References

[1] MAGLIULO G, ERCOLINO M, PETRONE C, COPPOLA O, MANFREDI G, "Emilia Earthquake: the Seismic Performance of Precast RC Buildings", *Earthquake Spectra*, 30, (2),2014: 891-912.

[2] MAGLIULO G, ERCOLINO M, MANFREDI G, "Influence of cladding panels on the first period of one-story precast buildings", *Bulletin of Earthquake Engineering.* DOI: 10.1007/s10518-014-9657-2,2014

[3] BELLERI A, BRUNESI E, NASCIMBENE R, PAGANI M, RIVA P, "Seismic Performance of Precast Industrial Facilities Following Major Earthquakes in the Italian Territory", *Journal of Performance of Constructed Facilities*, 0, (0),2014: 04014135.

[4] MCKENNA F, FENVES GL, "OpenSees Manual". Pacific Earthquake Engineering Research Center; 2013

[5] FARDIS MN, BISKINIS D, "Deformation capacity of RC members, as controlled by flexure or shear". In: Kabeyasawa T SHeP-beferrcsavhSO, editor. University of Tokyo2003

[6] IBARRA LF, MEDINA RA, KRAWINKLER H, "Hysteretic models that incorporate strength and stiffness deterioration", *Earthquake Engineering & Structural Dynamics*, 34, (12),2005: 1489-511.