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# Influence of modelling assumptions in the expected loss evaluation of a precast industrial building

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**Abstract**

The economic losses arisen in the industrial sector after the Emilia earthquakes in 2012 highlighted the importance of conducting reliable seismic assessment analyses on the existing industrial building stock in order to ascertain both safety and potential losses associated to seismic events. To accomplish to such task, an accurate representation and quantification of the actual vulnerabilities in such buildings is required and reliable structural models need to be adopted.
The paper investigates how various assumptions and levels of sophistication in finite element modelling affect the results in terms of economic losses, herein assumed as the reference decision variable. After the definition of the main seismic vulnerabilities of precast industrial buildings typical of the Italian territory, different types of finite element models are adopted and non-linear time history analyses conducted; in particular, the reinforced concrete fork at the top of the column is modelled in different ways, also considering the seismic retrofit.
Appropriate fragility curves under selected engineering demand parameters are defined and provided within the Performance Based Earthquake Engineering methodology developed at the Pacific Earthquake Engineering Research Center for the assessment of the expected losses under a scenario-based earthquake. The influence of the modelling assumptions in the seismic risk estimate is evaluated. The results indicate that for the considered case study and in the absence of loss of support, simplified single-column models are suitable to estimate the expected losses.



*Keywords:* Precast connections; Industrial buildings; Beam-to-column joint; Performance-based earthquake engineering.

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## 1. Introduction

During the last decade, extensive research was conducted on the effect of earthquakes in terms of damage and economic losses on the affected buildings. Among these studies, the most applied framework is the probabilistic Performance Based Earthquake Engineering method developed at the Pacific Earthquake Engineering Research Center (PEER-PBEE methodology). The PEER-PBEE methodology allows estimating the performance of structures in seismic zones in terms of direct and indirect losses (typically downtime, economic losses, and casualties) and it is subdivided into four main steps: the hazard analysis, the structural analysis, the damage analysis, and the loss analysis [1]. The outcome of the loss analysis is a loss curve, which is obtained by applying the total probability theorem to the combination of all four phases. Through the calculation of such integral, it is possible to obtain the annualized expected annual loss of the considered building or facility. Simplified loss estimations are also available [2-3].

The application of the PEER-PBEE methodology allows evaluating the influence of the rate of recovery costs associated to non-structural elements (or secondary elements) and/or the contents of the building. Especially in the industrial field, the monetary value of the facilities, the activities, and the internal installations could be significant. Let us think about the damages suffered by many companies in the Emilia-Romagna region (Italy) following the seismic sequence in 2012 (Fig. 1). Most of the observed damage is closely related to the lack of seismic provisions, being those structures not designed according to modern seismic codes [4-6]. The main vulnerabilities, which caused both local and global collapses, are related to the inefficiency of the horizontal load transfer mechanism between precast elements and to the displacement and rotation compatibility among structural elements and between structural and non-structural elements [7-10].

a.  b. 

Fig. 1 (a) Damage caused by the collapse of the roof in a RC precast industrial building in Emilia, 2012 (b) failure of the fork at the top of the column due of the out-of-plane stress of the beam.

The structural layout of the precast concrete structures considered in this paper, typical of industrial and commercial buildings in the Italian territory, is made of cantilever columns pin-connected [11, 12] to pre-stressed RC beams spanning in one direction, which support pre-stressed concrete roof elements spanning in the transverse direction. The columns are placed inside cup footings or connected to the foundation by means of mechanical devices or grouted sleeves solutions [13, 14]. The paper considers how different modelling assumptions influence the results in terms of expected Economic Losses (EL); the assessment is carried out following the PEER-PBEE methodology. The vulnerable elements are considered in terms of fragility curves at selected damage states. Various finite element models are considered and non-linear time history analyses are conducted. The EL are calculated through the Performance Assessment Calculation Tool (PACT) freely available as a result of the ATC- 58 project [15].

## 2. PEER-PBEE application to a precast industrial building

The benchmark structure is a precast one-storey RC industrial building (Fig. 2), with structural layout and details typical of the Italian practice before the enforcement of anti-seismic regulations. The facility is composed by precast RC columns placed in socket foundations; the column's height is 8,3m and a RC fork is placed at the column's top to avoid overturning of the main double-tapered beams during erection. The columns' cross section is 0,45x0,45m with 12ϕ14 longitudinal rebars equally spaced along the perimeter. The cross section of each element of the RC fork is 0,12x0,45m with 8ϕ8 longitudinal rebars placed in 2 rows of 4 rebars each at the edges of the long sides. Vertical cladding panels (2,5x9x0,15m) surrounds the whole building. The cladding panels top-connection considered herein is shown in Fig. 2. The roof elements are made by pre-stressed double-T beams. The connections between the structural elements (roof elements, beams and columns) rely only on friction for transferring lateral loads. For all structural elements, the considered concrete cylindrical strength is 60MPa and the steel yield stress 450MPa.

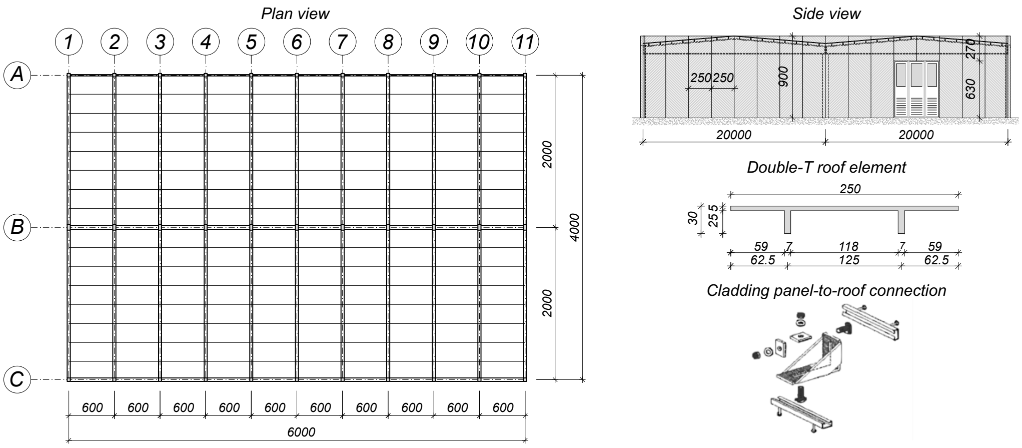


Fig. 2 Geometric characteristics of the considered case study. Measures in centimetres.

The finite element modelling affects the PEER-PBEE results in terms of expected losses. Four models with increasing complexity are considered herein and represented in Fig. 3. The first model considers a single column as a cantilever beam fixed at the base with a tip mass (26338kg) corresponding to the roof tributary mass (total roof mass divided by the number of columns). The non-linearity is modelled by a plastic hinge at the column's base according to Takeda hysteresis [16]. Such model represents the structural performance after retrofitting of the RC forks at the column's top by means of steel profiles, as indicated in Belleri et al. [6]. The second and third models consider the influence of the RC fork. In the second model, one beam element is provided with a plastic hinge at the fork's base according to Takeda hysteresis; the plastic hinge considers the influence of a single fork element, owing to the absence of mechanical connections assuring a bilateral behaviour. The third model considers the presence of both fork's elements. In this model, a vertical beam element is provided to represent the main beam connected at the top of the column. The upper node of the beam is connected to the top of the fork's elements through two compression-only springs with an initial gap of 1cm. An additional model has been defined to catch a closer seismic response of the real structure and the activation of local vulnerabilities; the model considers two portals and one bay and it represents a portion of the central part of the facility. The columns and the RC forks have been modelled as described in the third model. The roof elements and the double-tapered beams are included as beam elements. The cladding panels are modelled as point masses.

The following damageable elements are considered in the EL assessment: column, RC fork at the column's top, roof element, and cladding panel. The damage related to the building content, as machineries and plants, is not considered herein. Table 1 reports the fragility, in terms of Engineering Demand Parameter (EDP), at each Damage

State (DS) and the related repair cost for each damageable element. The values are expressed as median values following a lognormal distribution with dispersion 0.6 for the fragility and 0.4 for the repair cost.

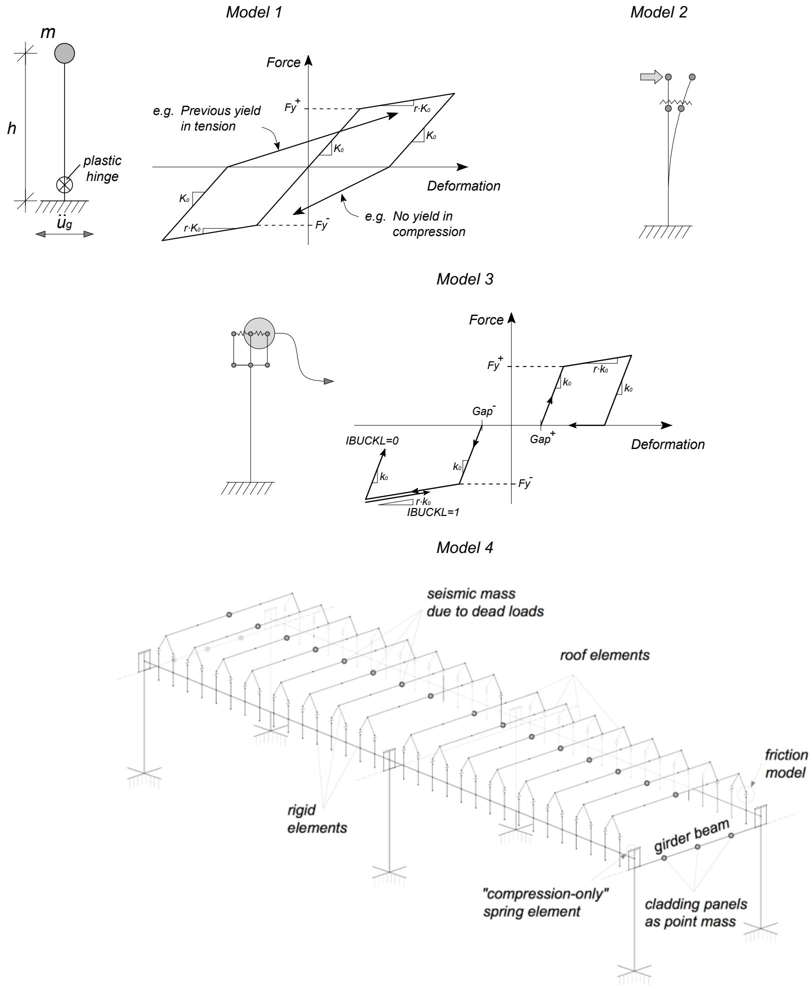


Fig. 3 Considered finite element models and related hysteretic models.

EL are evaluated following the PEER-PBEE methodology under a scenario-based approach. The second main shock of the Emilia's seismic sequence is considered (May 29th, 2012). Non-linear time history analyses have been conducted with the finite element software MidasGen [17] including the three components of the earthquake. The EDP obtained from the analyses have been recorded and included as input value in the PACT software. The results of the probabilistic simulation are represented in Fig. 4. The results show how the cladding panels represent the most vulnerable element of the considered building and how they significantly impact the estimation of the total repair cost. In addition, it is observed how Model 2, RC forks modelled with a single element, highly overestimates the cost related to forks' failure. Model 3 and Model 4 provide very similar results. At a first sight, Model 1 seems accurate enough for EL estimation. This is a consequence of the low value of economic losses associated with RC forks for the considered case study and considered scenario. The same considerations apply in the case of the loss of support of roof elements, which is expected to affect significantly the results; indeed, no loss of support is recorded herein.

Therefore, in the case of a scenario-based analysis, a first estimation of the repair costs can be obtained with Model 1 in the case the EL associated with the RC forks are low, otherwise the more refined Model 3 should be adopted. Further research is required to address the issue of loss of support.

Table 1 Damage states, repair actions, EDP values, and repair costs of the considered damageable groups.

| Damageable element | Damage state (Repair action) | EDP | EDP value | Unit repair cost (€) |
|---|---|---|---|---|
| Column | DS-1: Slight cracking at the base (Injection with epoxy resin) | curvature (rad/m) | 0.00094 | 126 |
| | DS-2: Spalling of the cover at the base (RC jacketing) | | 0.00958 | 286 |
| | DS-3: Collapse of the column (Restore collapsed elements) | | 0.06859 | 50175 |
| RC fork | DS-1: Slight cracking at the base (Injection with epoxy resin + UPN 200 at each side) | curvature (rad/m) | 0.00189 | 325 |
| | DS-2: Spalling of the cover at the base (RC jacketing + UPN 200 at each side) | | 0.03083 | 599 |
| | DS-3: Fork's failure (Restore collapsed elements) | | 0.23378 | 38165 |
| Beam-to-roof element joint | DS-1: Small relative displacement (Restore of sealant in 25% of perimeter + steel wire connection to avoid extra displacements) | Relative displacement (cm) | 3 | 151 |
| | DS-2: Intermediate relative displacement (Restore of sealant in 100% of perimeter + steel wire connection to avoid extra displacements) | | 6 | 200 |
| | DS-3: High relative displacement and loss of the support | | 8 | 1237 |
| Structure-to-cladding panel connection | DS-1: Yielding of the top connections (Restore of sealant in 25% of perimeter + steel brackets to avoid overturning) | Relative displacement (cm) | 1 | 197 |
| | DS-2: Rupture of the connections and fall of the cladding panel (Provision of a new panel) | | 4 | 1963 |

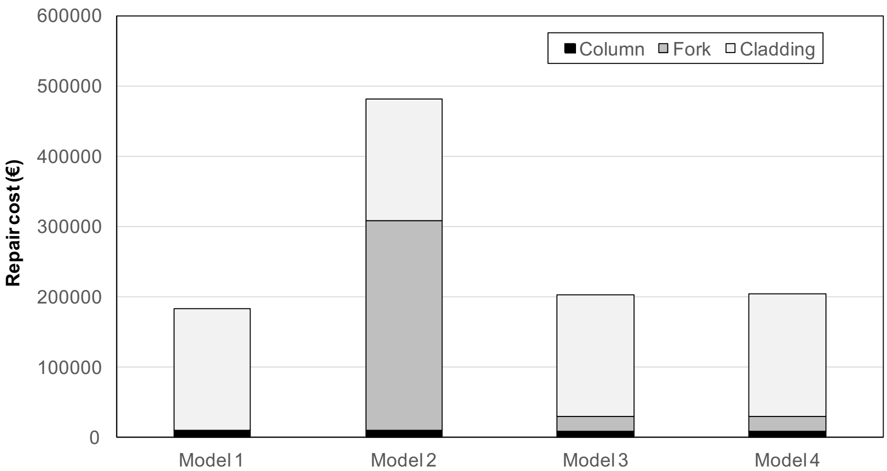


Fig. 4 Expected loss as a function of the considered finite element models.

## 3. Conclusions

The paper investigates the influence of various finite element modelling schemes in respect to the expected losses associated to a scenario-based analysis. A case study resembling a typical Italian precast industrial building is selected.

The structural elements and connections do not consider modern anti-seismic regulations, i.e. the elements have been designed for gravity loads. Four finite element models with increasing complexity are selected. The first model considers a single column without accounting for the local vulnerability associated to the RC fork at the column top. The second and third models consider a single column with the RC fork modelled by one and two non-linear elements respectively. The fourth model represents a subassembly of the central portion of the building made by two portals and one bay. The results of the expected loss estimation show how the cladding panels represent the most vulnerable element of the considered building. In addition, the importance of properly modelling the RC forks is highlighted. The results show how a simple single-column model represents a good starting point for estimating economic losses for a scenario-based assessment, when low damage in the RC forks is expected. For a general time-based assessment, i.e. considering all possible earthquakes and their probability of occurrence, the third model is preferable because suitable to capture the RC forks damage. In such model, each fork's element is modelled with a nonlinear beam and the contact between the main beam and the fork is provided by compression-only springs. No significant differences are obtained between the simple single-column model and the three-dimensional model as long as the damage suffered by the forks is minimal and the collapse of roof-elements due to loss of support is prevented. Further research is required to address the issue of the loss of support of roof elements.

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## References

[1] M. Günay and K. Mosalam, "PEER Performance Based Earthquake Engineering Methodology, Revisited," J. Earthq. Eng., vol. 17, no. 6, pp. 829–858, 2012.
[2] D. P. Welch, T. J. Sullivan, and G. M. Calvi, "Developing Direct Displacement-Based Procedures for Simplified Loss Assessment in Performance-Based Earthquake Engineering," Journal of Earthquake Engineering, vol. 2, pp. 290-322, 2014
[3] D. Cardone, T. Sullivan, G. Gesualdi, and G. Perrone, "Simplified estimation of the expected annual loss of reinforced concrete buildings." Earthq. Eng. Struct. Dyn., doi: 10.1002/eqe.2893, 2017
[4] G. Magliulo, M. Ercolino, C. Petrone, O. Coppola, and G. Manfredi, "The Emilia Earthquake: Seismic Performance of Precast Reinforced Concrete Buildings," Earthq. Spectra, vol. 30, no. 2, pp. 891–912, 2014.
[5] A. Belleri, E. Brunesi, R. Nascimbene, M. Pagani, and P. Riva, "Seismic Performance of Precast Industrial Facilities Following Major Earthquakes in the Italian Territory," J. Perform. Constr. Facil., pp. 1–10, 2014.
[6] A. Belleri, M. Torquati, P. Riva, and R. Nascimbene, "Vulnerability Assessment and Retrofit Solutions of Precast Industrial Structures," Earthq. Struct., vol. 8, no. 3, pp. 801–820, 2015.
[7] E. Brunesi, R. Nascimbene, D. Bolognini, and D. Bellotti, "Experimental Investigation of the Cyclic Response of Reinforced Precast Concrete Framed Structures," PCI J., vol. 60, pp. 57–79, 2015.
[8] R. Scotta, L. De Stefani, and R. Vitaliani, "Passive control of precast building response using cladding panels as dissipative shear walls," Bull. Earthq. Eng., vol. 13, pp. 3527-3552, 2015.
[9] A. Belleri, M. Torquati, A. Marini, and P. Riva, "Horizontal cladding panels: in-plane seismic performance in precast concrete buildings," Bull. Earthq. Eng., vol. 14, no. 4, pp. 1103–1129, 2016.
[10] A. Belleri, M. Torquati, and P. Riva, "Seismic Performance of Ductile Connections Between Precast Beams and Roof Elements," Mag. Concr. Res., vol. 66, no. 11, pp. 553–562, 2014.
[11] G. Magliulo, M. Ercolino, M. Cimmino, V. Capozzi, and G. Manfredi, "FEM Analysis of the Strength of RC Beam-to-column Dowel Connections Under Monotonic Actions," Constr. Build. Mater., vol. 69, pp. 271–284, 2014.
[12] B. Zoubek, M. Fischinger, and T. Isakovic, "Estimation of the cyclic capacity of beam-to-column dowel connections in precast industrial buildings," Bull. Earthq. Eng., pp. 2145–2168, 2015.
[13] A. Belleri and P. Riva, "Seismic Performance and Retrofit of Precast Concrete Grouted Sleeve Connections," PCI J., 2012.
[14] G. Metelli, C. Beschi, and P. Riva, "Cyclic Behaviour of a Column to Foundation Joint for Concrete Precast Structures," Eur. J. Environ. Civ. Eng., vol. 15, no. 9, pp. 1297–1318, 2011
[15] Applied Technology Council – ATC Development of Next Generation Performance-Based Seismic Design Procedures for New and Existing Buildings, ⟨https://www.atcouncil.org/Projects/atc-58-project.html⟩, 2012.
[16] S. Otani, "Hysteresis Models for Reinforced Concrete for Earthquake Response Analysis," J. Fac. Eng., pp. 125–159, 1981.
[17] MidasGEN 2012 (v3.1), MIDAS Information Technologies Co. Ltd.