



THE IMPACT OF STRUCTURAL COMPONENTS ON FRAGILITY CURVES OF SINGLE-STOREY INDUSTRIAL PRECAST STRUCTURES

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ABSTRACT

The impact of structural components on fragility curves of single-storey precast industrial buildings was investigated. Five different structural systems, including older structural systems simulated on the basis of friction connections between columns and beams, were defined. The fragility functions were assessed for the limit state of significant damage and for the collapse of a structure, which occurred due to the collapse of roof or collapse of columns. The limit-state intensities were obtained by the truncated incremental dynamic analysis. It was observed that the implementation of the dowel in beam-to-column connections had beneficial effect on the collapse intensities, whereas the presence of cladding panels did not significantly affect the collapse of the structure. In addition, a sensitivity study was performed in order to investigate the impact of variation of structural parameters on the occurrence of the limit states. Results from this study showed that fragility curves associated with significant damage were mainly influenced by the parameters of cladding panels, whereas properties of the column and the beam primarily affected the collapse fragility curves.

INTRODUCTION

The society became aware of the seismic vulnerability of precast industrial buildings after earthquakes which struck Emilia-Romagna region in 2012, causing 27 casualties and huge losses (Magliulo et al., 2013). Many precast industrial buildings, which were built in last decades, partly or totally collapsed (Figure 1). The reason for such huge losses is at least twofold. Firstly, seismic response of precast systems was not well understood at the time of construction, and secondly, most of the buildings had been designed before 2003, when the affected region was for the first time classified as a seismic-prone area (e.g. Liberatore et al., 2013). It is therefore important to assess vulnerability of existing precast buildings and to provide an insight into the parameters that have the greatest impact on the fragility curves which represent a significant component of the loss assessment procedure.

The recent catastrophic events in Emilia-Romagna region once more revealed an issue of the damage of nonstructural elements, especially cladding panels, which may significantly contribute to the total losses. Even if a structure withstands the shaking of the ground, precast panels may collapse (e.g. Figure 1a), which, in the case of industrial buildings, may lead to an interruption of the manufacturing process. This may cause huge indirect losses. According to Magliulo et al. (2013), indirect losses due to earthquakes in Emilia Romagna region were estimated to about 5 billion euros, whereas the direct losses amounted to about 1 billion euros. Such observations show the necessity to

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simulate the nonstructural damage in order to realistically assess the economic loss due to a certain seismic event.



Figure 1. a) collapse of cladding panels and b) collapse of a precast RC building due to the earthquakes in Emilia Romagna region

Numerous studies dealing with seismic vulnerability of precast structures have been carried out in recent years. Among others, Senel and Kayhan (2009) performed fragility-based damage assessment of existing precast industrial buildings using twelve models, which reflected the variation of stiffness, strength and ductility properties of previously inspected building stock. Two-dimensional single-storey frame models were built and so-called strain-based limit states were defined using pushover analyses. However, seismic performance assessment of the structures was based on nonlinear response history analysis. It was concluded that the ductility and stiffness have greater impact on the probability of damage than the strength. Bolognini et al. (2008) performed seismic risk assessment of precast industrial buildings using randomly generated stock of models that represented four typical types of precast buildings constructed in Northern Italy. Single-storey and multi-storey buildings were analysed according to SP-BELA procedure. Two limit states were defined using simplified pushover analyses, highlighting the chord rotation of the column at first yielding and the collapse of the structure. Additionally, the failure of connection between columns and beams was taken into account. Displacement demand within this study was obtained by the displacement spectrum (Borzi et al. 2008). It was confirmed that the connections are one of the weak points of the analysed structures. Casotto (2013) used a similar methodology as Bolognini et al. (2008). However, the seismic demand was obtained on the basis of response history analysis, whereas the collapse was defined by simulating strength degradation or by the threshold value of inter-storey drift. It was pointed out that the percentage of the beam-column connection failure was significant for the connections based on friction and for buildings having strong columns.

The result of the fragility analysis, which was the goal of the aforementioned studies, is the fragility function, i.e. a function that specifies probability of a designated limit state of a structure as a function of some ground motion intensity measure (Baker, 2014a). Usually peak ground acceleration or spectral acceleration with a specified period and damping are chosen for the intensity measure. An important step of the fragility analysis is represented by simulations of seismic response of a structure in order to obtain a sample of limit-state intensities. Different algorithms can then be used to fit the analytical fragility curve to the sample values of limit-state intensities. In this study, the truncated incremental dynamic analysis is done to obtain the sample of limit-state intensities, and the maximum likelihood method is used to estimate the parameters of the fragility curve. This approach is described elsewhere (Baker, 2014a). Truncated incremental dynamic analysis is performed in the same way as incremental dynamic analysis (Vamvatsikos and Cornell, 2002), however, the maximum value of intensity is specified, above which no further analyses are performed. Maximum likelihood method finds the fragility function parameters such that the resulting distribution has the highest likelihood of having produced the observed data (Baker, 2014a).

A fragility function may be derived for a specific building or a class of buildings. In the case if fragility analysis is performed for the building class, only few parameters of the structure can be used in the fragility analysis. In this case it is reasonable to perform the sensitivity analysis in order to

determine, which parameters have the greatest impact on fragility curves. In this study, simple (so-called local) sensitivity analysis was used, which involves comparisons of the results determined from the base-case model and the results based on the model where one uncertain parameter is set to different values, while simultaneously holding the rest of the other model parameters equal to those of the base-case model (e.g. Celarec and Dolšek, 2013).

It is worth to emphasise that the accuracy of the fragility analysis significantly depends on the capabilities of the structural model, which should, in general, adequately simulate most important failure modes of the precast industrial buildings. Recent studies have been focused on development of simple structural models of the connections in precast structures. Zoubek et al. (2014) examined behaviour of the dowel connections between columns and beams. Failure mechanisms were defined as functions of geometry and material characteristics. The ductile types of failure took place if the shear strength of the dowel was the weak link of the connection. The brittle failure of the connection was attained if the concrete was torn out from the column due to very large strength of the dowel. Magliulo et al. (2010) performed an experimental study on neoprene-concrete friction relationship in order to assess coefficient of friction between these materials, which are commonly used in the case of beam-to-column connections of older precast buildings. In addition, the panel-to-frame connections were addressed by Isaković et al. (2014). The objective of the study was to understand the mechanism of the cyclic behaviour of the typical push-pull connectors, consisting of two channels, linked by hammer-head strap.

The objective of the study presented in this paper, was to select the most important parameters that affect the fragility parameters (i.e. the median limit-state peak ground acceleration and the corresponding dispersion) of the selected types of precast industrial buildings. For this purpose, a simple sensitivity analysis was performed for a single-storey precast building which was designed by Fischinger et al. (2008) and tested in ELSA laboratory (Negro et al., 2007). The following parameters were varied in the sensitivity analysis: the strength of steel and concrete, the span between columns, the column height, the amount of longitudinal reinforcement in columns, the amount of transverse reinforcement in columns, the diameter of the dowel (in the case of connections incorporating a steel dowel – dowel connections), the area of the joint surface between the column and the beam (in the case of friction connections), the presence of the cladding panels, interaction between them and their connection to foundation.

Based on the assumption that roof elements act as a rigid diaphragm simplified two-dimensional models were built consisting of a one-storey one-bay frame. The fragility curves were assessed for the limit state of significant damage (i.e. collapse of the first cladding panel) and collapse of the building, which was defined as a consequence of either collapse of the roof (failure of the beam-column connection) or collapse of the columns. The limit-state peak ground accelerations (limit-state PGAs) were assessed for 30 recorded ground motions. The vertical component of ground motion was used in addition to the horizontal component. The horizontal ground motions were selected in such a way that the median spectra normalized to the peak ground acceleration match the elastic spectrum of Eurocode 8 (CEN, 2005b). All response history analyses within the truncated incremental dynamic analysis (Vamvatsikos and Cornell, 2002) were performed by OpenSees (PEER, 2013). The maximum value of PGA was assessed from the ground motion prediction model according to Lazar and Dolšek (2013). Results are presented in terms of fragility curves. Finally, a discussion is given in order to explain which input parameters cannot be simply ignored in the fragility analysis of the precast reinforced concrete buildings.

ANALYSED STRUCTURAL SYSTEMS OF THE PRECAST BUILDING AND THE PARAMETERS OF THE SENSITIVITY STUDY

Single-storey precast frame structures were analysed in this study. Such structures usually consist of cantilever columns, which are connected to a rigid or semi-rigid roof system in its own plane. In Europe, slender columns and pinned beam-to-column connections are typically used. Such a structural system is very flexible after the failure of the cladding panels. Fortunately, the storey drifts capacity can be as high as 8% (Fischinger et al., 2008) if designed according to current European building code. However, implementation of precast buildings often depends on the construction company. Therefore

many different variants of the precast industrial buildings exist. The main differences can be observed in the implementation of the beam-to-column connection and the non-structural elements, which may affect seismic response of the structure. Therefore it was decided to define five different systems (Table 1) by varying the type of the column-to-beam connection (steel dowel, friction only), the presence of cladding panels and the type of the connection of the cladding panels to the ground. These structural systems will be called herein after the prototype structures I to V.

Table 1. Definition of prototype structures

Prototype structure	Beam-to-column connection	Non-structural elements	Connection of non-structural elements to foundation
I	Incorporating a steel dowel	Precast claddings panels	Unrestrained
II	Incorporating a steel dowel	Precast claddings panels	Welded/bolted
III	Incorporating a steel dowel	w/o cladding panels	/
IV	Friction only	Precast claddings panels	Unrestrained
V	Friction only	w/o cladding panels	/

All prototype structures are variants of the structure designed by Fischinger et al. (2008) and tested at ELSA laboratory (Negro et al., 2007). However, this structure was built to investigate the cyclic behaviour of the columns at the base, whereas the performance of the beam-to-column connection was not the primary interest of the study. Therefore, the type of connection of the prototype structure was changed to be more representative of the as-built structures (e.g. Magliulo et al., 2013). The beam-to-column connection of prototype structures I, II and III consisted of a dowel, while only a neoprene pad at the column-to-beam interface was assumed for prototype structures IV and V (so-called friction connection). Additionally, it was considered that some of the prototype structures (III, V) were without cladding panels, whereas others (I, II, and IV) had vertical concrete cladding panels, which were seated on the foundation beam in a way that rocking of the panels was allowed (I, IV), or connected to the foundation beam by bolts or welds (II). For all vertical cladding panels, so-called push-pull connections to the roof beam were assumed, which allowed for some displacement in the plane of the panels, but prevented the displacement perpendicular to them. In all the prototype structures no interaction was assumed between neighbouring panels.

The prototype structures were defined in order to perform simple sensitivity analysis at the level of the different types of structural systems of the industrial precast buildings. The sensitivity analysis at the level of a given structural system was also performed. Twelve input parameters of the prototype structure I were varied (Table 2). Additionally, importance of the length of the contact between the column and the beam, if the connection between these elements was based on friction, was assessed for the prototype structure IV (Table 2). It should be emphasised that variations of the parameters of the sensitivity analysis are not associated with the standard deviation of the random variables, which is commonly used in a sensitivity analysis, but were assessed according to the engineering judgment in order to capture a large part of all possible precast industrial buildings constructed in areas with different levels of seismic hazard.

Table 2. Parameters of the sensitivity study

Parameter	Label	Prototype structure	Base-case value	Variations
Beam Length	BL	I	8m	6m, 10m
Column Height	CH	I	5m	3m, 4m, 6m, 7m, 8m
Mean compressive strength of the unconfined concrete	F _{cm}	I	55MPa	38MPa, 48MPa
Mean yield stress of the reinforcement	F _{ym}	I	555MPa	270MPa, 316MPa, 408MPa
Total longitudinal reinforcement ratio in the column	As _l	I	1.12%	0.63%, 1.75%, 2.51%
Diameter of the stirrups in the column	SD	I	8mm	6mm, 12mm, 16mm
Distance between the stirrups in the critical region	DS	I	50mm	75mm, 150mm
Number of cladding panels	NP	I	4	1, 6
Strength of the push-pull connectors	SPP	I	7.5kN	5kN, 12.5kN
Ultimate displacement of the push-pull connectors	DPP	I	7cm	5cm, 12cm
Diameter of the dowel	DD	I	16mm	12mm, 20mm, 24mm
Shear interaction strength between the panels	IBP	I	0kN	20kN, 40kN
Column-beam joint surface	CBS	IV	40/40cm	40/20cm

The span between the columns and the column height were varied in order to capture the variations in geometry. It was assumed that the strength of the material can also significantly differ. Since the mean concrete compressive strength of the prototype structure was large, the concrete of quality C30/37 and C40/50 (CEN, 2004a) were assumed for additional variants. In the case of the yield strength of the steel, the median yield strength of steel used for variants of the prototype structures was determined according to Eurocode 8 (CEN, 2005a) by increasing the characteristic strength of steel S235, S275 and S355 by 15%. In addition to the strength of material, the percentage of the longitudinal and the transverse reinforcement was also varied in order to simulate the construction of precast building in regions with different seismic hazard. All other parameters basically relate to the construction technology of such buildings. Note that the strength and ultimate displacement of push-pull connectors were determined according to results provided by Isaković et al. (2014). However, the shear strength of the interaction between the panels was assessed on the basis of adhesion strength of the silicone (Jiesheng et al., 2013), which is commonly used as sealant between precast panels (e.g., Lafarge, 2014). Quite different values of shear strength were assumed due to the uncertainty of the implementation of this connection (e.g., RIBA, 2014).

STRUCTURAL MODEL, LIMIT STATES AND GROUND MOTIONS

A 2D structural model was defined on the basis of a 3D symmetric one-storey prototype structure, consisting of one-bay frames and cladding panels attached to the external frames in the direction of loading and two-bay frames in the perpendicular direction (Figure 2). Note that one half of the 3D structure was accounted for by the 2D model. Therefore the stiffness and mass of a frame of 2D model was multiplied by 1.5. However, characteristics of panel-related parameters, i.e. stiffness and strength of panel-to-beam connections, panel-to-foundation connections and panel-to-panel connections were not modified for the 2D model since the panels were assumed to be attached only to the external frames. Columns were modelled by one-component lumped plasticity elements, whereas beams and cladding panels were based on linear elastic elements with very high stiffness. The connections between elements were simulated by nonlinear springs. Masses of the bearing frames and cladding panels were concentrated at the centres of the beams and panels, respectively (Figure 2).

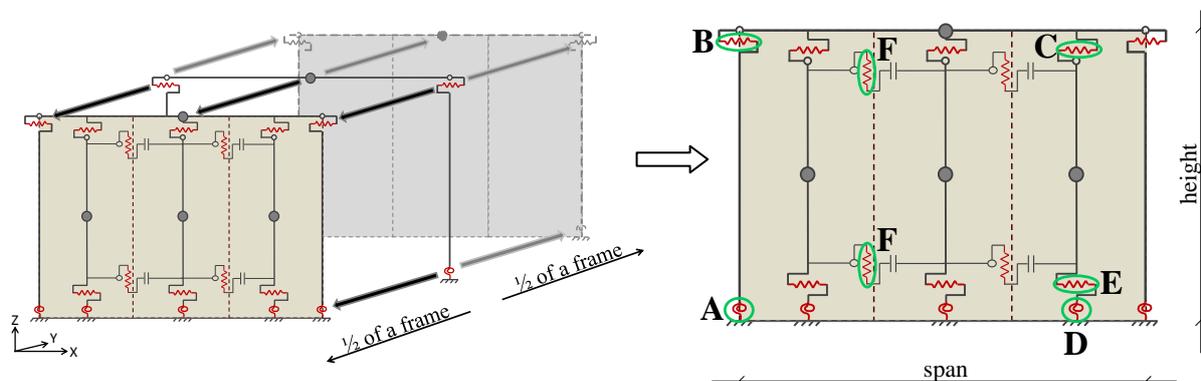


Figure 2. Transition from a three-dimensional to a two-dimensional model

The plastic hinge at the base of the columns (spring A on Figure 2) was defined by the four-linear moment-rotation relationship, taking into account the pre-crack, post-crack and post-yield stiffness, as well as the negative linear post-capping stiffness (Figure 3a). The characteristic rotations were determined according to previous study (Dolšek, 2010), whereas the characteristic moments were based on moment-curvature analysis using zero length fibre section elements (PEER, 2013) and the constant axial force from gravity load. Takeda hysteretic rules were applied in the plastic hinges of the columns.

The pinned connections were assumed between the column and the beam, whereas the dowel was simulated with the shear spring (spring B in Figure 2 and Figure 3b) based on the characteristics proposed by Zoubek et al. (2014). A tri-linear force-displacement relation was defined, representing either ductile (failure of the dowel) or brittle behaviour (failure of the concrete of the columns).

Different springs were used for prototype structures IV and V, where the connection relied only on friction (Figure 3b). Perfectly elasto-plastic relationship was considered, with the strength equal to the product of the axial force and the coefficient of friction (Magliulo et al. 2010).

Push-pull connections between panels and beams (spring C in Figure 2) were modelled by combining springs with different material characteristics (Figure 3c) in order to imitate cyclic behaviour, presented by Isaković et al. (2014). In addition, if the ultimate displacement was exceeded, stiffness and stress values were set to 0. The force-displacement relationship of the springs representing the panel-to-foundation connections depended on the type of connection. Rocking in prototype structures I and IV was simulated by rotational springs (spring D in Figure 2 and Figure 3d). Additionally to the rotational spring, the elasto-plastic model of the shear springs (spring E in Figure 2 and Figure 3e) was used to simulate the effect of friction. In system II, where connection to foundation was considered, linear springs with high stiffness parameters were provided for all degrees of freedom (Figures 3d and 3e). In this case, the failure of the vertical cladding panels was controlled by the connection at the top of cladding panels. In order to model shear interaction between the panels (spring F in Figure 2), linear force-displacement relationship was used (Figure 3f) based on the shear modulus (Malla et al., 2011) and geometry of silicone sealant (e.g., Lafarge, 2014 and RIBA, 2014). If the adhesive strength (Jiesheng et al., 2013) was exceeded, stiffness and stress values were set to 0.

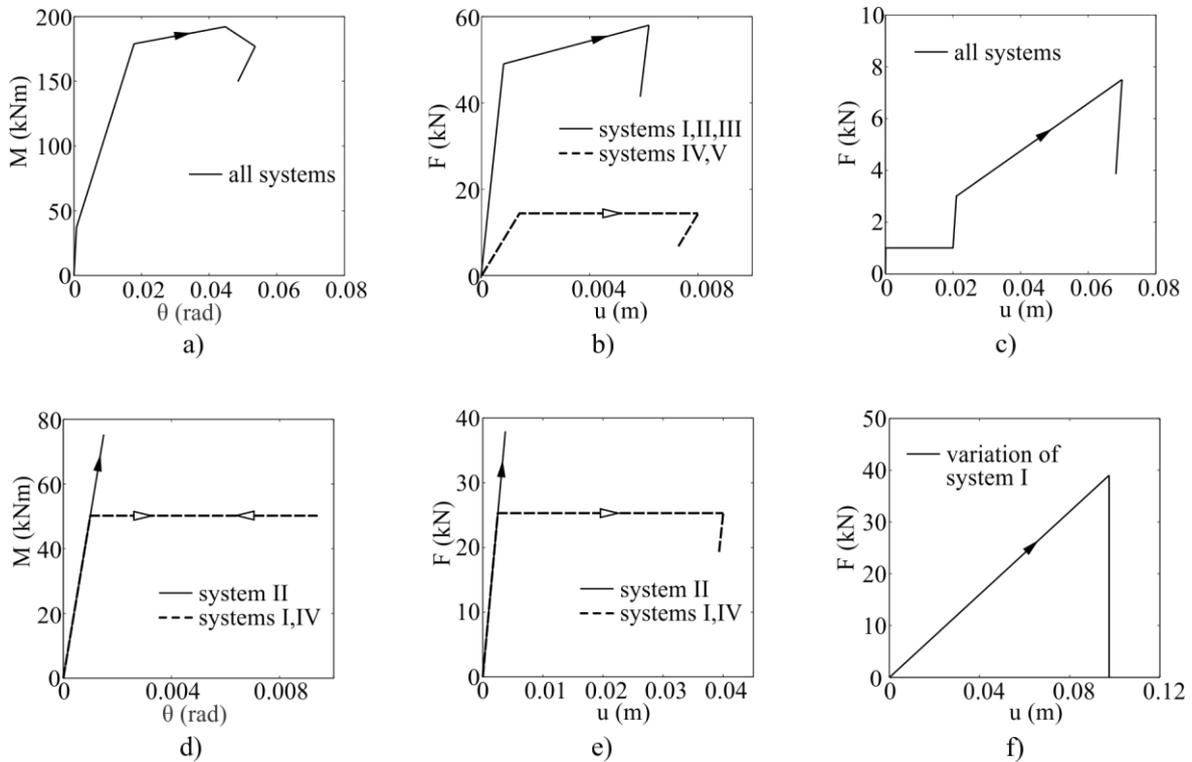


Figure 3. a) rotational spring at the base of the column, b) shear spring at the beam-to-column connection, c) shear spring at the panel-to-beam connection, d) rotational spring at the panel-to foundation connection, e) shear spring at the panel-to foundation connection and f) shear spring at the panel-to-panel connection

The limit state of significant damage (SD) and that of collapse (C) were defined. The SD limit state was associated with the partial collapse of the building, which was defined as either the dislocation of cladding panels (relevant in the case of systems I, II and IV) or the slippage of the roof (relevant in the case of systems IV and V). The dislocation of cladding panels was conditioned by the ultimate displacement of the panel-to-beam connection (Isaković et al. 2014), whereas the slip of the roof occurred if the friction force was exceeded in the beam-to-column connection.

The C limit state was defined as a consequence of either the collapse of the roof (failure of the beam-to-column connection) or the collapse of the columns. It was assumed, that the collapse of the columns occurred, when the rotation in the plastic hinges of the columns exceeded the ultimate rotation (CEN, 2005b), which corresponded to the 80% of strength in the post-capping range. The definition of the failure of the beam-to-column connection depended on the type of connection. The

ultimate displacement associated with the failure of the dowel corresponded to the shear strength of the dowel (Zoubek et al. 2014). In the case of the friction connections it was assumed that the collapse of the roof occurred due to the concrete crushing, which was caused by the high tensile stresses once the contact length in the beam column connection was relatively small. Note that a critical value of the slip (or the length of the beam-to-column connection) was predetermined by a FEM analysis. In the case of prototype structure IV, the critical contact length was estimated to be 17cm.

Limit-state peak ground accelerations were based on incremental dynamic analysis (Vamvatsikos and Cornell, 2002), which was performed for a set of ground motions that were selected from the European strong motion database (Ambraseys et al., 2004) and Italian accelerometric archive (Massa et al., 2009). All analyses were done by OpenSees (PEER, 2013). Mass proportional damping was assumed with the 5% ratio of critical damping. Each analysis based on a horizontal and a vertical component of ground motion. Horizontal components of ground motions were selected to match the elastic spectrum according to Eurocode 8 for the soil type B (CEN, 2004b). Note that a modified variant of a procedure proposed by Jayaram et al. (2010) was used for selecting the ground motions, which are presented in terms of acceleration spectra (Figure 4). A good agreement can be observed between the mean acceleration spectra of the selected ground motions and the target spectrum. In the range from 0.1s to 3s the values of the mean spectrum of the selected ground motions were not below the 90% of the values of target spectrum as it is required by Eurocode 8 (CEN, 2005b). Vertical components corresponded to the selected ground motions. However, ratios between the components of the peak ground accelerations were determined according to equation (1)

$$\frac{PGA_{vertical}}{PGA_{horizontal,selected}} = \min\left(\frac{PGA_{vertical}}{PGA_{horizontal,1}}, \frac{PGA_{vertical}}{PGA_{horizontal,2}}\right), \quad (1)$$

where $PGA_{horizontal,1}$, $PGA_{horizontal,2}$, $PGA_{horizontal,selected}$ and $PGA_{vertical}$ were peak ground accelerations of the two horizontal components, the actual selected horizontal component (either $PGA_{horizontal,1}$ or $PGA_{horizontal,2}$) and the vertical component of the same earthquake recording, respectively.

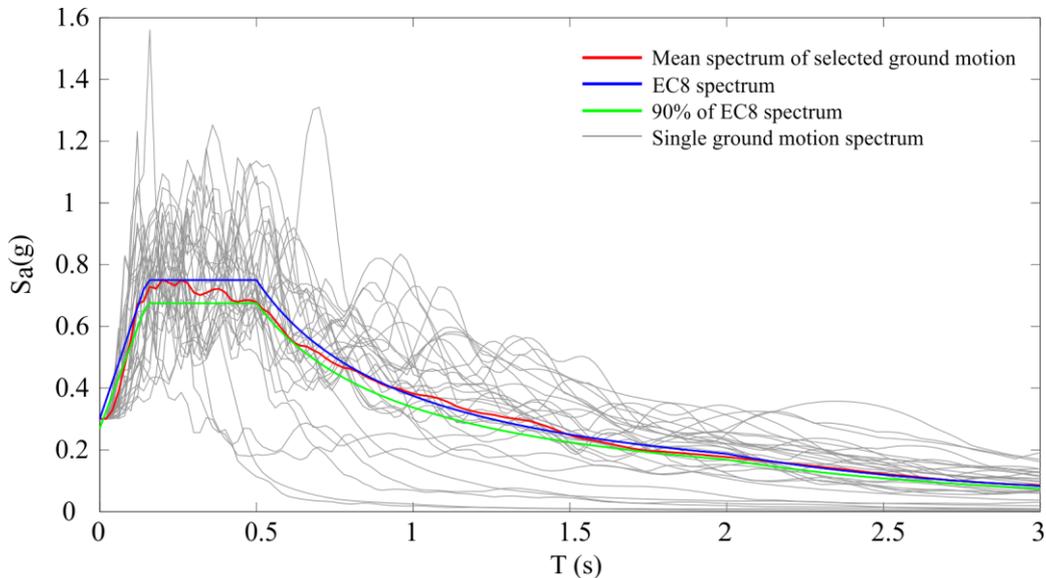


Figure 4. Spectra of the selected ground motions, the mean spectrum of the selected ground motions, the target spectrum (Eurocode 8) and the 90% of the target spectrum

RESULTS OF FRAGILITY ANALYSIS

Fragility functions were obtained on the basis of limit-state intensities from the incremental dynamic analysis. Some peak ground accelerations, causing collapse, were extremely high, i.e. up to 20g. These extreme values are not realistic for at least two reasons. Firstly, the relationship between the occurrence rate and the extreme value of intensity measure is very uncertain, which means that there is practically no added value if the analyst tries to precisely assess the fragility curves for the extreme values of intensity measure. Secondly, the large limit-state intensities are usually based on the extremely large scale factors, which makes the result of incremental dynamic analysis unrealistic since the frequency content, duration and other parameters of ground motions in general differs with respect to the value of the intensity measure. Therefore, the so-called truncated IDA (Baker, 2014a) was used, where the maximum PGA was considered on the basis of the maximum magnitude of earthquake from the hazard analysis. In the case of this study, the maximum magnitude of 7.0 was adopted (Lazar and Dolšek, 2013). The maximum PGA was then assessed to be 1.43g, which is equal to the value of two standard deviations above the median PGA, if using the Akkar & Bommer (2010) ground motion prediction equation for the case of the adopted magnitude and the zero source-to-site distance. The percentage of ground motions, which did not cause collapse before exceeding the maximum PGA, amounted to 43%, 36%, 43%, 17% and 17% for the structural models from I to V, respectively. Although analyses above the maximum PGA are not required in the case of truncated IDA, the collapse PGAs for these ground motions were estimated in order to investigate the scale factors at very high intensities. It was found that 60% of PGAs, which caused collapse of the prototype structures and were greater than the maximum PGA, were obtained by scaling the ground motions for more than 100 times. Note that both limit-states fragility functions for all the models were obtained by utilizing a procedure according to Baker (2014b).

Fragility functions for all prototype structures and both limit states are illustrated in Figure 5a. The system notation is consistent with that from Table 1, while the limit states LS 1 and LS 2 represent significant damage (SD) and collapse of the structure (C), respectively. Only small differences between the collapse (LS 2) fragility functions for systems I, II and III can be observed. The median collapse PGA, \overline{PGA}_{LS2} , for all these systems amounted to about 1.29g. Consequently it can be concluded that the presence of cladding panels, either connected to the foundation or not, did not significantly influence the collapse of the prototype structures. However, the opposite was observed in the case of the friction beam-to-column connection. The collapse fragility functions for prototype structures IV and V were significantly shifted to the left (median PGA was estimated to be approximately 0.95g) in comparison to those, which corresponded to prototype structures with the dowel beam-to-column connection. This illustrates the presence of the dowel significantly reduced the probability of collapse of the prototype structures. Similar observations were made when inspecting the LS 1 fragility functions. Significant damage in prototype structures IV and V (friction connection) was caused by the slippage of the roof, rather than the dislocation of the panels, which resulted in lower limit state intensities. While the median PGA for prototype structure I was estimated at 0.66g, the median PGAs for systems IV and V were around 0.3g. Furthermore, it was observed that the median value of the limit-state intensity was reduced (to around 0.57g) if the welded panel-to-foundation connection was used. This effect was caused by the increase of the panel stiffness, which prevented the panel to follow the response of the frame and consequently enlarged the demand on the panel-to-beam connection.

Results of the sensitivity study, which was performed at the level of structural system of the prototype structure I, are summarized in Figure 6. Parameters of the fragility functions (median value of limit-state PGA, \overline{PGA}_{LS} , and corresponding logarithmic standard deviation, β_{LS}) are presented for all parameters of the sensitivity study. Note that the parameters of the sensitivity study were divided into so-called component groups in order to illustrate the impact of different structural components on the fragility functions. It can be observed that the impact on the \overline{PGA}_{LS} was, in general, greater than that on the β_{LS} . The \overline{PGA}_{LSI} varied between 60% and 160% of that based on the base-case model of prototype structure I (0.66g), while the β_{LSI} was assessed between 68% and 112% of that based on the base-case model. The variation of fragility parameters (\overline{PGA}_{LS2} and β_{LS2}) for LS 2 was relatively smaller in

comparison to that for LS 1. The relative variation of \overline{PGA}_{LS2} and β_{LS2} was observed from 44% to 114% and from 83% to 119% (1.29g and 0.36), respectively.

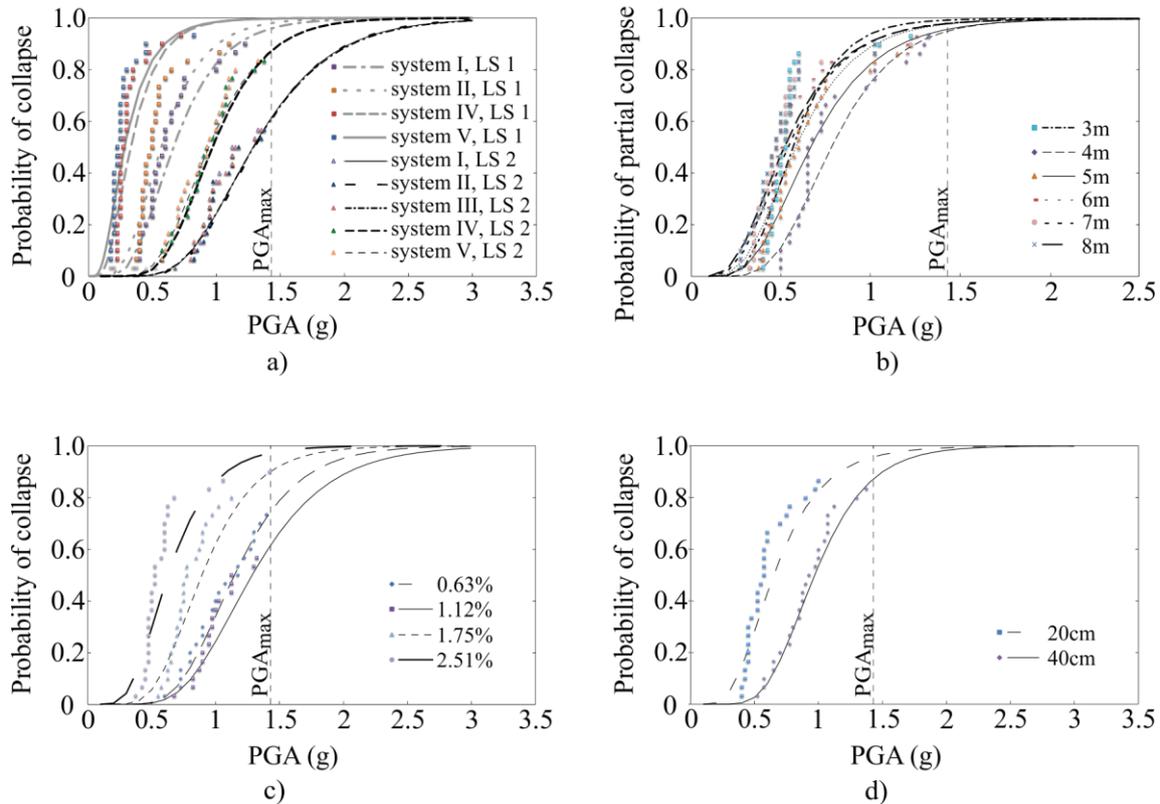


Figure 5. a) LS 1 and LS 2 fragility functions for five prototype structures, b) the effect of the height of the column on the LS 2 (significant damage) fragility functions of prototype structure I, c) the effect of the longitudinal reinforcement ratio of the columns on the collapse fragility functions of prototype structure I, d) the effect of the length of the beam-to-column contact on the collapse fragility function of prototype structure IV.

Note that from Figure 6 it can be clearly observed that in the case, if one component group had significant impact on the fragility function of one limit-state, this was not the case for the other limit-state.

LS 2 was affected mainly by parameters of the Column component group (i.e., by the height of the column and the amount of reinforcement) and the Beam component group. However, influence of the Connection component group was enlarged, if the beam-to-column connection represented the weak element of the structure. This was the case, if the diameter of the dowel was reduced to 12mm. The switch between the failure of the columns and the failure of the dowel was also observed when the height of the column was reduced from 4m to 3m and when the ratio of longitudinal reinforcement was increased from 1.12% to 1.75% (Figure 5c). Both of these changes had unfavourable effect on the fragility curves of the structure. The \overline{PGA}_{LS2} was decreased by 55%, if the column height was reduced from 4m to 3m. This is significantly more than the reduction of \overline{PGA}_{LS2} by 2%, which was observed when the height of the column was reduced from 5m to 4m. The increase of the longitudinal reinforcement ratio caused the decrease of \overline{PGA}_{LS2} by 31%. If the reinforcement ratio was further increased to 2.5%, \overline{PGA}_{LS2} was decreased substantially (for 51%) in comparison to the \overline{PGA}_{LS2} corresponded to the column reinforcing ratio of 1.12%.

The Panel component group (i.e. the number of panels, force-displacement relationship of panel-to-beam connection and the level of interaction between panels) and the Beam component group had impact on the significant damage (LS1). Additionally, the influence of the column height was observed to be important. The latter input parameter had a beneficial effect when it was gradually decreased from 8 m to 4 m. However, the fragility function was significantly shifted to the left in the case when the column height was set to 3m (Figure 5b) (\overline{PGA}_{LS2} was equal to 0.57g) since the switch between the failure modes occurred. Consequently, LS 1 and LS 2 were exceeded simultaneously.

The sensitivity parameters of the fragility curves to the variation of the contact length between the beam and column of the prototype structure IV was also investigated (Figure 5d). Half of the column width (20cm) was assumed as the length of the contact between the column and the beam in addition to that from the base-case model of prototype structure IV, where the contact length was assumed equal to the width of the column (40cm). The result is quite expected, since the reduction of the length of the friction connection to 50% of original value caused a reduction of the median PGA_{LS2} for about 40%, while the β_{LS2} did not change significantly. In the case where the beam-to-column connection is constructed without dowel, the length of the contact between the beam and the column is a very important parameter provided that the coefficient of friction is more or less constant for such types of connections.

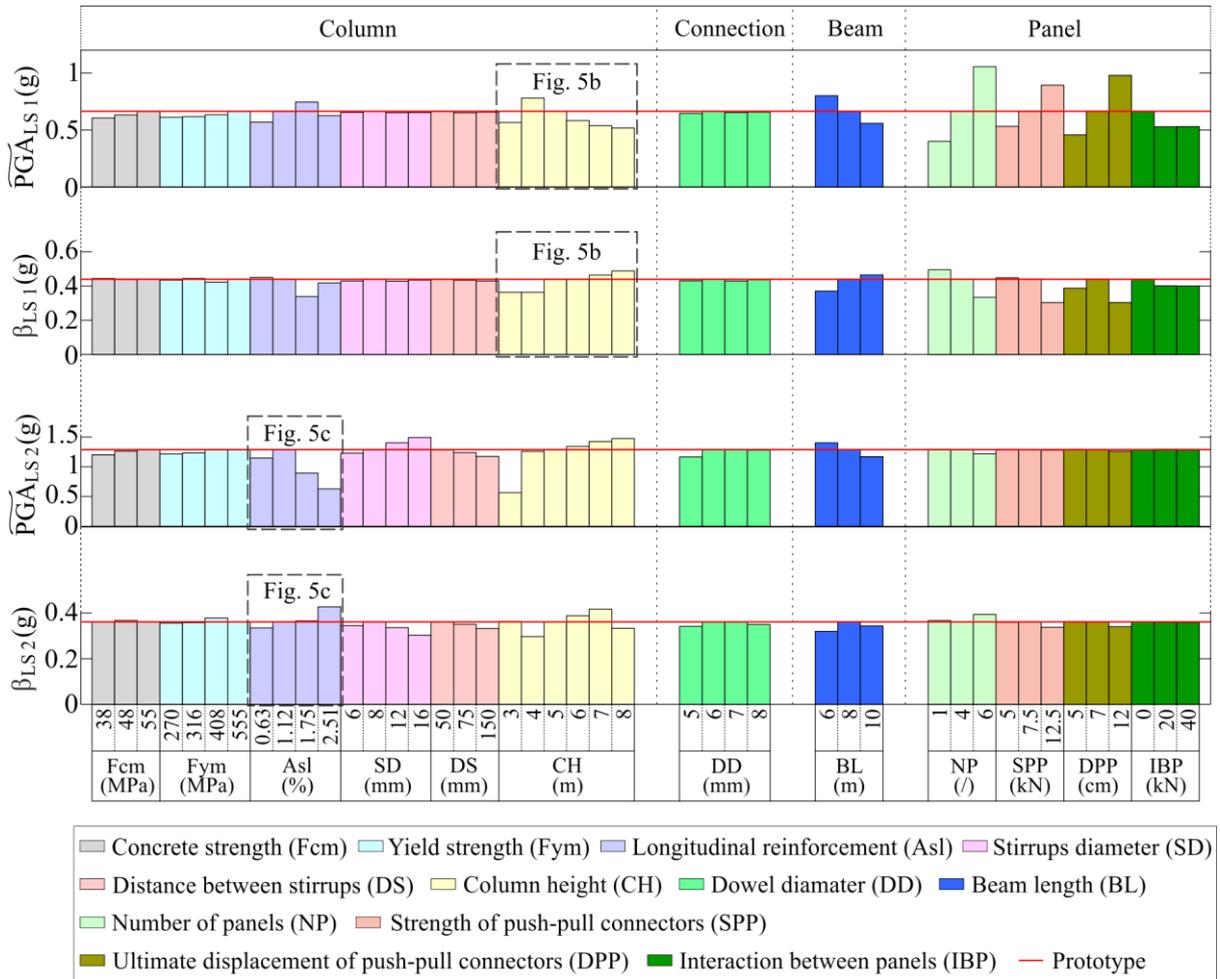


Figure 6. Median value of limit-state PGA in the case of LS 1 (\overline{PGA}_{LS1}) and LS 2 (\overline{PGA}_{LS2}) and corresponding standard deviation in log-domain (β_{LS}). The results based on prototype structure I.

CONCLUSIONS

Fragility functions associated with the collapse and the limit state of significant damage were derived for five structural systems with different beam-to-column connections and properties of cladding panels. It was observed that the presence of vertical cladding panels did not impact collapse of the structure. However, the performance of structures, which had the beam-to-column connections implemented by the dowel (median collapse PGA = 1.29g) was significantly improved in comparison to the structures with friction connections (median collapse PGA = 0.97g). Standard deviations of collapse intensities in log domain were quite similar for all five prototype structures. They were observed in the interval from 0.34 to 0.37. Significant damage occurred due to the slippage of the roof, rather than the dislocation of the panels, if the prototype structure had no dowel in the beam-to-column

connection (median PGA = 0.28g and standard deviation in log domain = 0.52). If a dowel was used, significant damage was caused by the failure of cladding panels. Median PGA causing significant damage was estimated to 0.57g in the case of the restrained cladding panels and 0.66g in the case of the unrestrained panels. The corresponding standard deviation was estimated at 0.44 for both cases.

Furthermore, a sensitivity study was performed at the level of structural system of the prototype structure I having the dowel connection and cladding panels, which were unrestrained to foundation. It was noticed that the collapse intensities were affected by parameters different than those, which had high impact on the limit-state intensities that caused significant damage. The parameters, which affected the collapse of the structure, were primarily the length of the beam, which directly affected the mass of the structure, and characteristics of the column, among which the amount of reinforcement and the column height caused the greatest impact on the fragility functions. However, by varying certain parameters (i.e. the height and the ratio of longitudinal reinforcement of the column), the switch between the failure modes was observed. In this case the beam-to-column connection became the weak link of the structure, which increased the importance of the diameter of the dowel. Significant damage was mainly affected by the parameters of cladding panels, i.e., by the number of panels attached to the structure, the force-displacement relationship of the panel-to-beam connection and the level of shear interaction between the panels.

REFERENCES

- Akkar S and Bommer JJ (2010) "Empirical Equations for the Prediction of PGA, PGV, and Spectral Accelerations in Europe, the Mediterranean Region, and the Middle East", *Seismological Research Letters*, 81(2):195-206
- Ambraseys NN, Smit P, Douglas J, Margaris B, Sigbjörnsson R, Ólafsson S, Suhadolc P, Costa G (2004) "Internet site for European strong-motion data", *Bollettino di Geofisica Teorica e Applicata*, 45(3):113-129
- Baker JW (2014a) "Efficient analytical fragility function fitting using dynamic structural analysis", *Earthquake Spectra* (in press)
- Baker JW (2014b) "Efficient analytical fragility function fitting using dynamic structural analysis", (code supplement)
<http://purl.stanford.edu/sw589ts9300>
- Bolognini D, Borzi B, Pinho R (2008) "Simplified Pushover-Based Vulnerability Analysis of Traditional Italian RC precast structures", *Proceedings of the 14th World Conference on Earthquake Engineering*, Beijing, China, 12-17 October 2008
- Borzi B, Pinho R, Crowley H (2008) "Simplified pushover-based vulnerability analysis for large-scale assessment of RC buildings", *Engineering Structures*, 30(3):804-820
- Casotto C (2013) *Seismic vulnerability of Italian RC precast industrial structures*, M.Sc. Thesis, The Institute for Advanced Study in Pavia, Italy
- Celarec D and Dolšek M (2013) "The impact of modelling uncertainties on the seismic performance assessment of reinforced concrete frame buildings", *Engineering Structures*, 52:340-354
- CEN (2004a) "Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings", European Committee for Standardization, Brussels, Belgium
- CEN (2004b) "Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings", European Committee for Standardization, Brussels, Belgium
- CEN (2005a) "Eurocode 8: Design of structures for earthquake resistance - Part 2: Bridges", European Committee for Standardization, Brussels, Belgium
- CEN (2005b) "Eurocode 8: Design of structures for earthquake resistance - Part 3: Assessment and retrofitting of buildings", European Committee for Standardization, Brussels, Belgium
- CSI (2012) "SAP2000 Ultimate 15.2.1, Structural Analysis Program", Berkeley, California, USA
- Dolšek M (2010) "Development of computing environment for the seismic performance assessment of reinforced concrete frames by using simplified nonlinear models", *Bulletin of Earthquake Engineering*, 8(6):1309-1329
- Fischinger M, Kramar M, Isaković T (2008) "Cyclic response of slender RC columns typical of precast industrial buildings", *Bulletin of Earthquake Engineering*, 6(3):519-534
- Isaković T, Zoubek B, Lopatič J, Fischinger M (2014) "Experimental research of typical cladding panel connections in industrial buildings", *Second European conference on earthquake engineering and seismology*, Istanbul, Turkey, 25-29 August 2014

- Jayaram N, Lin T, Baker JW (2010) "A computationally efficient ground-motion selection algorithm for matching a target response spectrum mean and variance", *Earthquake Spectra*, 27(3):797-815
- Jiesheng L, Shaopeng W, Dong E (2013) "Effect of Coupling Agent as Integral Blend Additive on Silicone Rubber Sealant", *Journal of Applied Polymer Science*, 128(4):2337-2343
- Lafarge (2014) "Architectural Precast Concrete Sealant & Joint guide"
http://www.lafarge-na.com/Sealant_And_Joint_Brochure.pdf
- Lazar N and Dolšek M (2013) "Incorporating intensity bounds for assessing the seismic safety of structures: Does it matter?", *Earthquake Engineering & Structural Dynamics*, 43(5):717-738
- Liberatore L, Sorrentino L, Liberatore D, Decanini LD (2013) "Failure of industrial structures induced by the Emilia (Italy) 2012 earthquakes", *Engineering Failure Analysis*, 34:629-647
<http://dx.doi.org/10.1016/j.engfailanal.2013.02.009> (9.1.2014)
- Magliulo G, Capozzi V, Fabbrocino G, Manfredi G (2010) "Neoprene-concrete friction relationships for seismic assessment of existing precast buildings", *Engineering Structures*, 33(2):532-538
- Magliulo G, Ercolino M, Petrone C, Coppola O, Manfredi G (2013) "Emilia Earthquake: the Seismic Performance of Precast RC Buildings" *Earthquake Spectra* (in press)
<http://dx.doi.org/10.1193/091012EQS285M> (18.12.2013)
- Malla RB, Shrestha MR, Shaw MT, Brijmohan SB (2011) "Temperature Aging, Compression Recovery, Creep, and Weathering of a Foam Silicone Sealant for Bridge Expansion Joints", *Journal of Materials in Civil Engineering*, 23(3):287-297
- Massa M, Pacor F, Luzi L, Bindi D, Milana G, Sabetta F, Gorini A, Marcucci S (2009) "The Italian ACcelometric Archive (ITACA): processing of strong-motion data", *Bulletin of Earthquake Engineering*, 8(5):1175-1187
- Negro P, Mola E, Ferrara L, Zhao B, Magonette G, Molina FJ (2007) "Precast structures EC8: Seismic Behaviour of Precast Concrete Structures with respect to EC8 (Contract Number G6RD-CT-2002-00857)", Final report of experimental activity of the Italo-Slovenian Group
- PEER (2013) "Open System for Earthquake Engineering Simulation (OpenSees)", Pacific Earthquake Engineering Research Center, University of California, Berkeley, USA <http://opensees.berkeley.edu> (18.12.2013)
- RIBA (The Royal Institute of British Architects) (2014)
<http://www.ribaproductselector.com/Product.aspx?ci=443&pr=ARDEX-ARDEXST>
- Senel SM and Kayhan AH (2009) "Fragility based damage assesment in existing precast industrial buildings: A case study for Turkey", *Structural Engineering and Mechanics*, 34(1):39-60
- Vamvatsikos D and Cornell CA (2002) "Incremental dynamic analysis", *Earthquake Engineering and Structural Dynamics*, 31(3):491-514
- Zoubek B, Fischinger M, Isaković T (2014) "Seismic response of dowel connections in precast industrial buildings", *Second European conference on earthquake engineering and seismology*, Istanbul, Turkey, 25-29 August 2014