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INFLUENCE OF BEAM-COLUMN DOWEL CONNECTION DETAILING ON THE SEISMIC FRAGILITY OF PRECAST INDUSTRIAL BUILDINGS

B. Zoubek [(1)], M. Fischinger [(2)], T. Isaković [(3)]

[(1)] *Post-Doc, Faculty of civil and geodetic engineering University of Ljubljana, blaz.zoubek@fgg.uni-lj.si*

[(2)] *Professor, Faculty of civil and geodetic engineering University of Ljubljana, matej.fischinger@fgg.uni-lj.si*

[(3)] *Professor, Faculty of civil and geodetic engineering University of Ljubljana, tatjana.isakovic@fgg.uni-lj.si*

Abstract

Dowel connections are the most common type of the beam to column connection, which are typically used in precast industrial buildings in Europe. Nevertheless, the knowledge about their seismic behavior was recently relatively poor as long as they were examined in the frame of the European FP7 project SAFECAST - Performance of Innovative Mechanical Connections in Precast Buildings Structures under Seismic Conditions. One of the results of this research were new formulas, which can be used to predict the force-displacement response of dowel connections. Due to their simplicity they can be used in the design practice.

These formulas take into the account several important parameters influencing the strength and deformation capacity of dowels, such as: an amount of the confining reinforcement in the adjoining precast elements, the position of the dowel, the level of relative rotations between the beam and the column, etc. Exploiting the newly obtained knowledge, robust macro numerical models for dowel connections were proposed. These elements were used in the extensive probabilistic dynamic parametric studies of precast industrial buildings.

A set of 15 single-storey industrial buildings was carefully selected in order to cover the wide range of typical buildings. All these structures were designed for peak ground acceleration of 0.25g and soil of type C according to Eurocode 8. Each building was designed considering three different types of dowel connection, which differed regarding the amount of the confining reinforcement in the adjoining precast elements and the position of the dowel. The conditional probability of failure was calculated for all buildings and for all dowel connection details taking into the account both, record-to-record variability and modelling uncertainties. The results of the seismic fragility analysis clearly illustrate the importance of the confining reinforcement constructed in the area around the dowel as well as the importance of the distance of the dowel from the edge of the connected precast elements.

Keywords: precast buildings, dowel connections, connection detailing, confinement reinforcement, seismic fragility

1. Introduction

Dowel connections (Fig. 1) are the most common type of the beam to column connection, which are typically used in precast industrial buildings in Europe. The importance of the detailing of such connections has been recognized in several studies, reported in [1-7]. In the frame of the European FP7 project SAFECAST [2] multiple experimental and analytical studies of beam-column dowel connections were performed. The main goal of these studies was to evaluate the effects of different parameters (e.g. number of dowels, position of dowels, density of confining reinforcement in the connection region, etc.) on the global force-displacement response. Based on these new findings, a macro model of beam-column dowel connections was formulated and presented in detail in [7] (a brief description of the model can be found in Section 2). With reliable models of connections being available, nonlinear dynamic analysis and studies of the seismic safety of precast industrial buildings seems to be the next reasonable step forward [8].

In this paper, an extensive parametric investigation of the seismic fragility of precast industrial buildings is presented. The study took into the account two most probable failure mechanisms, which can endanger the stability of the whole precast building: the failure of columns, and the failure of the beam-column dowel connection. Being aware of several possible shortcomings in such studies, the main goal was to prove the importance of adequate connection's detailing for the seismic safety of precast industrial buildings.

First, a set of 15 single-storey precast industrial buildings was carefully selected (Section 2) in order to represent the whole range of typical buildings. All selected structures were designed according to Eurocode 8 [9] (Section 2). Each of them was designed considering three types of dowel connections, which differed regarding the amount of the confining reinforcement around the dowel and regarding the distance of the dowel from the edges of the columns and beams. Seismic fragility for each building was calculated taking into the account record-to-record variability and modelling uncertainties (Section 3). Based on the results of the parametric study, the suitability of analyzed types of dowel connections was discussed (Sections 3 and 4).

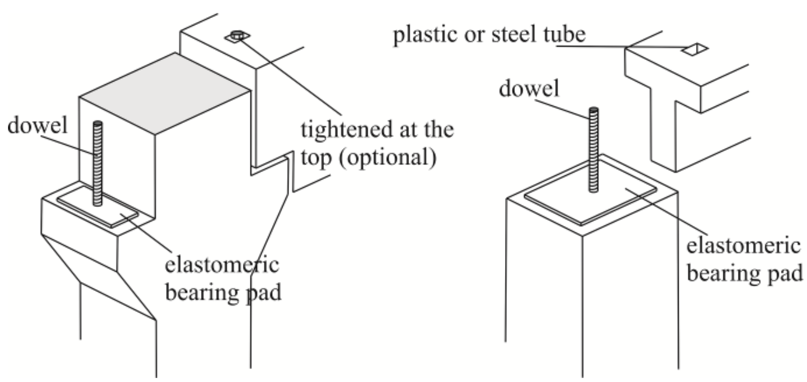

Fig. 1 – The scheme of the beam-column dowel connection assembly.

2. Representative set of buildings – design and modelling

As already mentioned, for the purpose of this study, a set of 15 single storey precast industrial buildings was selected. Structural system of the analyzed buildings is shown in Fig. 2. It consists of identical cantilever columns. The tops of the columns are tied together by internal pitched beams, perimeter beams and roof elements (the roof elements are not illustrated in Fig. 2). The pitched and perimeter beams are connected to columns by means of dowel connections (Fig. 1).

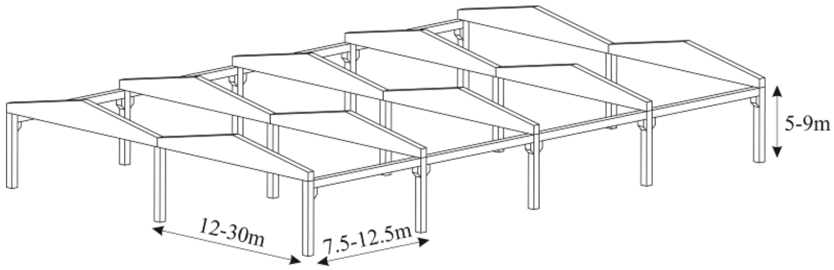

Fig. 2 – Structural system of the analyzed single-story precast industrial buildings.

In the parametric study 12-30 m long pitched beams as well as 7.5-12.5 m perimeter beams and roof elements were considered (Fig. 2). One and two-bay one-story structures were analyzed. The height of the buildings was varied between 5 m and 9 m, with the step of 1 m. Assuming a rigid diaphragm at the roof level, lateral load resisting system was modelled with single equivalent cantilever column. The corresponding mass was concentrated at the top of the column. The mass was defined taking into account the distributed load on the roof of w=5 kN/m^2. Considering the analyzed spans (see Fig. 2), the concentrated mass at the top of the equivalent column was in the range between m = 20 t and m = 100 t.

2.1 Design of columns and connections

A complete set of the 15 analyzed buildings, which cover the given geometric and mass ranges, are overviewed in Table 1. Each building is labeled. The label indicates the mass and the height of the building, respectively.

The buildings were designed according to the requirements of Eurocode 8 [9] for the ductility class medium (DCM). A behavior factor of q = 3 was applied, assuming that beam-column connections fulfill specific requirements of the code. A design acceleration of a_g = 0.25 g and soil type C were considered.

As it was already discovered in some previous studies of seismic response of precast industrial buildings [4], the dimensions of column cross-sections are, due to the column slenderness, most often determined by the limitations of the second order effects and interstorey drifts (Table 1). In case of buildings m20H5, m20H7, m20H9 and m40H9 minimal longitudinal reinforcement was sufficient while in all other cases the longitudinal reinforcement was selected based on the design loads.

In Fig. 3 the analyzed types of the beam-column dowel connections are presented. The connections differ in the position of the dowel and the amount of confining reinforcement around the dowel. The diameter of the dowel was chosen based on the design shear load obtained by the capacity design rule (diameters between 22 and 32 mm were considered). The shear strength of the connections was calculated as the pure shear strength of the dowel. This approach is often used in the Slovenian design practice.

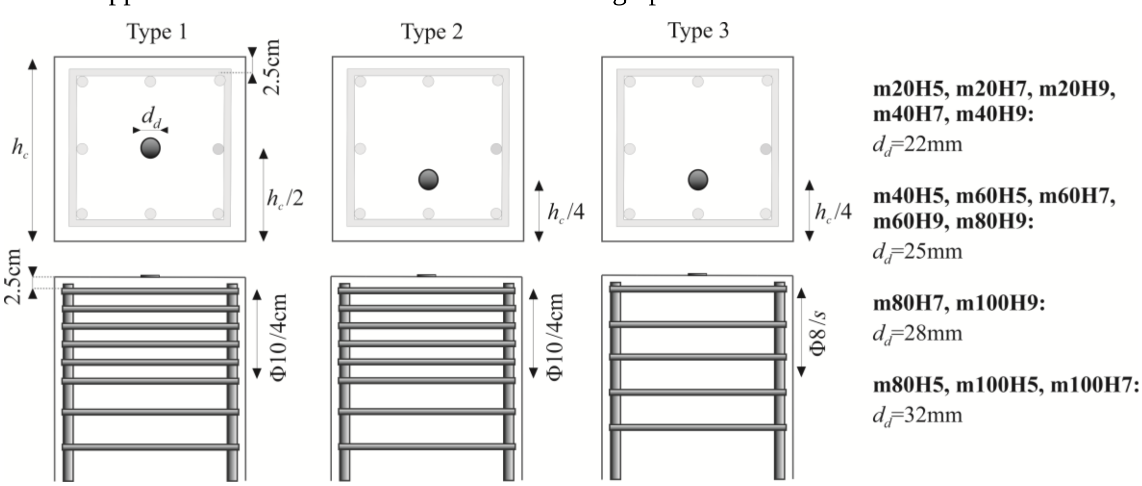

Fig. 3 – The three different types of the analyzed dowel connections.

Table 1 – The analyzed set of buildings

Label	Mass/column [t]	Height [m]	Column cross section [cmxcm]	Long. reinforce.	Transv. reinforce.
m20H5	20	5	40x40	8ϕ18	Φ8/14cm
m20H7	20	7	40x40	8ϕ18	Φ8/14cm
m20H9	20	9	50x50	12ϕ16	Φ8/12cm
m40H5	40	5	50x50	12ϕ20	Φ8/16cm
m40H7	40	7	50x50	12ϕ20	Φ8/16cm
m40H9	40	9	60x60	12ϕ20	Φ8/12cm
m60H5	60	5	50x50	12ϕ22	Φ8/16cm
m60H7	60	7	60x60	12ϕ22	Φ8/12cm
m60H9	60	9	60x60	12ϕ22	Φ8/12cm
m80H5	80	5	60x60	12ϕ25	Φ8/12cm
m80H7	80	7	60x60	12ϕ25	Φ8/12cm
m80H9	80	9	70x70	16ϕ20	Φ8/16cm
m100H5	100	5	60x60	12ϕ25	Φ8/12cm
m100H7	100	7	60x60	12ϕ28	Φ8/12cm
m100H9	100	9	70x70	16ϕ22	Φ8/16cm

2.2 Nonlinear modelling

The numerical model was defined as presented in Fig. 4. This model was similar to the one used in the design phase with a major improvement - nonlinear response of columns and dowel connections was included. Response of columns was simulated using lumped plasticity model (Fig. 4). Monotonic moment-rotation envelope (Fig. 5, left) and hysteretic rules assigned to the rotational spring were adopted from [10-12].

Nonlinear behavior of beam-column dowel connections was modelled by means of a shear spring [5,7]. Bilinear force-displacement response envelope (Fig. 5) was defined as described in [7]. Local and global failure mechanisms were considered (please see [5]). Strength reduction due to the relative rotations between the beam and the column was also taken into the account (Fig. 5). Since the relative rotations depend on the strength of the connections and vice-versa, an iterative Newton-Raphson procedure was used to obtain compatibility between these two quantities.

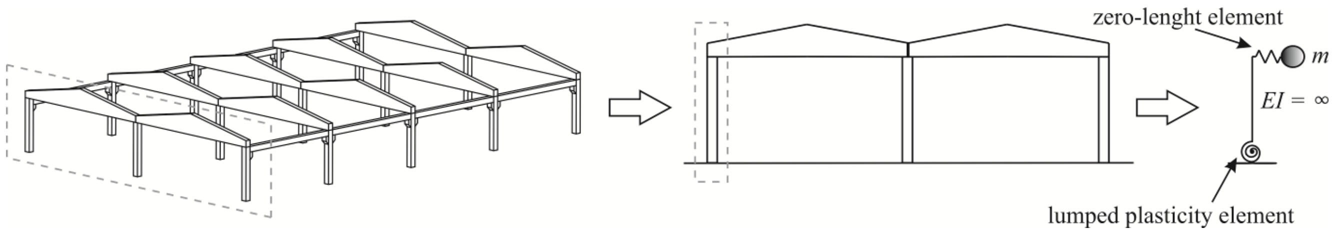

Fig. 4 – Nonlinear model of the single-storey industrial building taking into the account nonlinear response of columns and beam-column dowel connections.

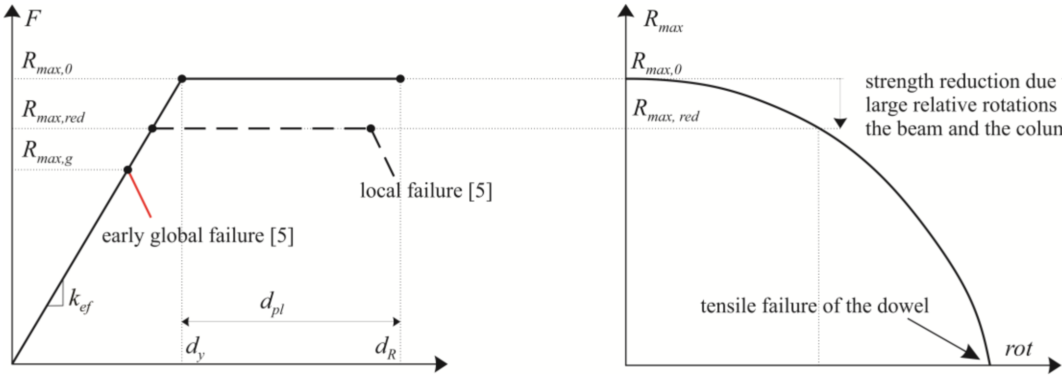

Fig. 5 –Force-displacement response envelope of dowel connections (left) and strength reduction due to the large relative rotations between the beam and the column (right)

3. Methodology

An extended incremental dynamic analysis [13] was performed for each of the analyzed buildings to estimate the median and dispersion values of their collapse capacities. For a single structure, a set of models was generated by utilizing latin hypercube sampling (LHS) method in order to take into the account modeling uncertainties [13]. Several input random variables were selected for this purpose. Their statistical characteristics are summarized in Table 2. Correlation between the input variables determining the response of columns was adopted according to the study performed by Ugurhan *et al* [14]. All other variables were assumed to be uncorrelated.

In addition to modeling uncertainties, record-to-record variability was considered by subjecting a structure to a set of 30 ground motion records (Fig. 6). The records were selected from the European [15] and Italian [16] ground-motion databases applying the iterative procedure proposed by Jayaram *et al* [17]. Eurocode 8 [9] spectrum for ground type C and the design ground acceleration of *PGA*=0.25 g was selected as the target spectrum. The goal was to achieve zero variance at the period T = 0 s and the smallest variance possible at all other periods. Additionally, when selecting the ground-motions, the following conditions were considered: magnitude should be between 4 and 8, source-to-site distance is between 4 and 60 km and maximum accelerogram scaling factor is not larger than 3.

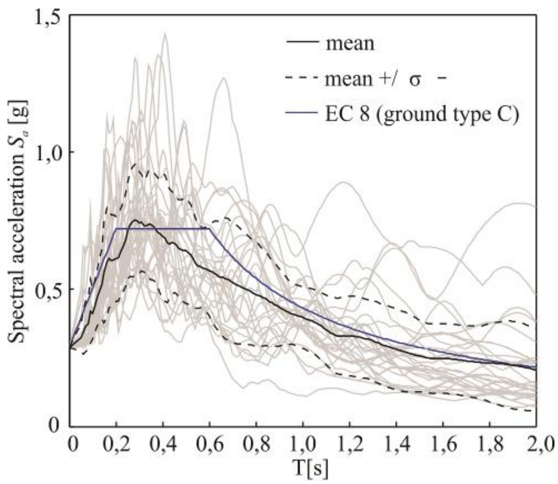

Fig. 6 – Spectra of the selected 30 ground-motion records and the Eurocode 8 spectrum for soil type C.

Table 2 – The selected input random variables and their statistical characteristics

Variable	Notation	Distribution function	COV	Reference
Yield drift	θ_y	lognormal	0.36	Haselton [11]
Hardening ratio	M_c/M_y	lognormal	0.10	Haselton [11]
Capping drift	θ_{cap}	lognormal	0.67	Haselton [11]
Post capping drift	θ_{pc}	lognormal	0.82	Haselton [11]
Normalized energy dissipation capacity	λ	lognormal	0.52	Haselton [11]
Mass per column	m	normal	0.10	Haselton [11]
Distance between the stirrups in the connection region*	s	normal	0.15	PCI [18]
Concrete cover in the connection region*	c	normal	0.20	JCSS [19]
Concrete compressive strength*	f_{cc}	normal	0.10	Melchers [20]
Yield strength of steel*	f_y	lognormal	0.05	Melchers [20]
Coefficient of viscous damping	ξ	normal	0.40	Porter *et al* [21]

*Dispersion of the variable is taken into the account for the calculation of the beam-column connection response parameters only.

4. Seismic fragility curves

The methodology described in the previous section was used to obtain seismic fragility curves (empirical log-normal cumulative probability density functions; Figs. 7, 8 and 9) for each of the analyzed structures (Table 1) and each analyzed type of the dowel connections (Fig. 3). The conditional probability of collapse $P(DS>DSi|PGA)$ at each intensity (*PGA*) was calculated as the ratio between the number of cases, in which the failure of the column or that of the connection occurred, and the size of the sample. Based on the empirical log-normal cumulative distribution functions (which is a discrete function) the underlying analytical (true) cumulative distribution functions were estimated. For this purpose, the maximum likelihood method was utilized. The agreement between the empirical and true cumulative distribution functions was tested by means of the chi-squared test.

4.1 Buildings with centric dowel connections

Fragility curves for the buildings with designed centric dowel connections, denoted as “week” connections are shown in Fig. 7. (red curves). For comparison, fragility curves for buildings with “strong” connections (it is assumed that the connections are strong enough that only column’s failure is possible) are also plotted in the same diagrams. In the case of eight buildings (m20H5, m20H7, m20H9, m40H7, m40H9, m60H9, m80H9 and m100H9) the strength of the centric dowel connections was sufficient, since the fragility curves of the structures with “weak” connections coincide with the fragility curves of the structures with “strong” connections.

It can be observed in Fig. 7 that the influence of the connections on the conditional probability of collapse increases together with the increase of the mass. At the mass of 60 t and height of 5 m (m60H5) the median capacities for structures with “weak” and “strong” connections are $\tilde{m}_c$=0.21 g and $\tilde{m}_c$=0.96 g, respectively (Fig. 7). The reason for such discrepancy is the design procedure, which is typically used in the Slovenian practice. The design shear strength of the connection was calculated as the design shear strength of the dowel itself. In this way the actual strength of the connection was considerably overestimated.

Furthermore, Fig. 7 clearly shows that the conditional probability of collapse decreases with the increasing height of the buildings. To explain this phenomenon let us first look at a specific case, i.e. buildings with a mass of 60 t per column. Median capacities of buildings m60H5, m60H7 and m60H9 with “weak” connections are $\tilde{m}_c = 0.21$ g, $\tilde{m}_c = 0.61$ g and $\tilde{m}_c = 1.33$ g, respectively. Capacity increases with the increasing height of the buildings. As already mentioned, when designing the structure, the actual strength of the connections was considerably overestimated. Using the 25 mm dowel the design requirements for all three buildings were fulfilled. However, since the actual strength was considerably smaller than the design strength, the failure occurred in the connection instead in the column. The shear forces in lower buildings are larger and therefore the conditional probability of failure is also larger (note that the strength of the connections is the same in all three buildings). It is interesting to notice that in the case of the building m60H9 the actual strength of the connections was still large enough so that the failures occurred in the column and not in the connection. In this way a larger dissipation of the seismic energy was possible and consequently the median collapse capacity $\tilde{m}_c$ was more than two times larger than in the case of the building m60H7. This result undoubtedly illustrates the importance of the design of the connections according to the capacity design principle, for which a sufficiently accurate estimation of the capacity of the connection is needed.

4.2 Buildings with eccentric dowel connections and with a relatively large amount of stirrups around the dowel

Fig. 8 shows that the fragility of buildings with eccentric dowel connections and with a relatively large amount of stirrups around it (Φ10/4cm; for more details, please see Fig, 3, type 2) is similar to the fragility of buildings with centric dowel connections (Fig. 7). Due to the large amount of confinement, the brittle global failure (see Fig. 5, left) is prevented. Instead a local failure of the connection occurred. Because dowels of the same diameters were used for all types of connections, the conditional probabilities of collapse for connection type 1 and 2 are more or less the similar (Figs. 7 and 8), except in three cases (e.g. m80H9, m100H7 and m100H9). The largest difference was observed in the building m80H9, where the median collapse capacities of the structures with centric and eccentric dowel connections were $\tilde{m}_c$=1.31 g and $\tilde{m}_c$=0.43 g, respectively.

Eccentric dowel connections are most often used at the internal columns, where two beams are connected to a single column. For this reason, an additional set of analyses was performed, taking into the account (dashed red lines in Fig. 8) such configuration. In this case, design shear load was two times smaller. Consequently, in almost all buildings (the exceptions are m80H5 and m100H5), a 25mm dowel fulfilled the design requirements. The results show that the fragility curves for buildings with eccentric dowel connections with large amount of confining reinforcement around the dowel and two connections per column, in the majority of cases coincide with the fragility curves for buildings with “strong” connections. The only exceptions are buildings m80H5 and m100H5 (Fig. 8).

4.3 Buildings with eccentric dowel connections and with a relatively small amount of stirrups around the dowel

The worst response was observed in buildings with eccentric connections with small amount of stirrups around the dowel (for more details please see Fig, 3, type 3). In all buildings and in each incremental dynamic analysis, failures of the connections was observed. The strength of the connections in the case of the connection type 3 is much lower than in case of connection types 1 and 2. Consequently, the conditional probability of failure is larger. In lower buildings with larger mass per column the median collapse capacity $\tilde{m}_c$ is even under 0.1 g. However, it should be noted that the actual capacity of the connections was at least slightly underestimated due to the following reasons: 1.) Global failure was assumed to be completely brittle (Fig. 5, left); 2.) Additional resistance due to the friction between the neoprene bearing pad and the adjoining concrete elements was neglected.; 3.) In reality, failure occurs when the beam loses support (slips off the column), for which a certain relative displacement between the beam and the column is needed.

It should be also noted that the prevailing practice is to provide a large amount of stirrups around the dowel (this can be confirmed at least for Slovenian contractors). Somewhat large fragility of buildings with eccentric dowel connections and relatively small amount of stirrups (Fig. 9) confirms the important role of the stirrups to the failure of the dowel connections.

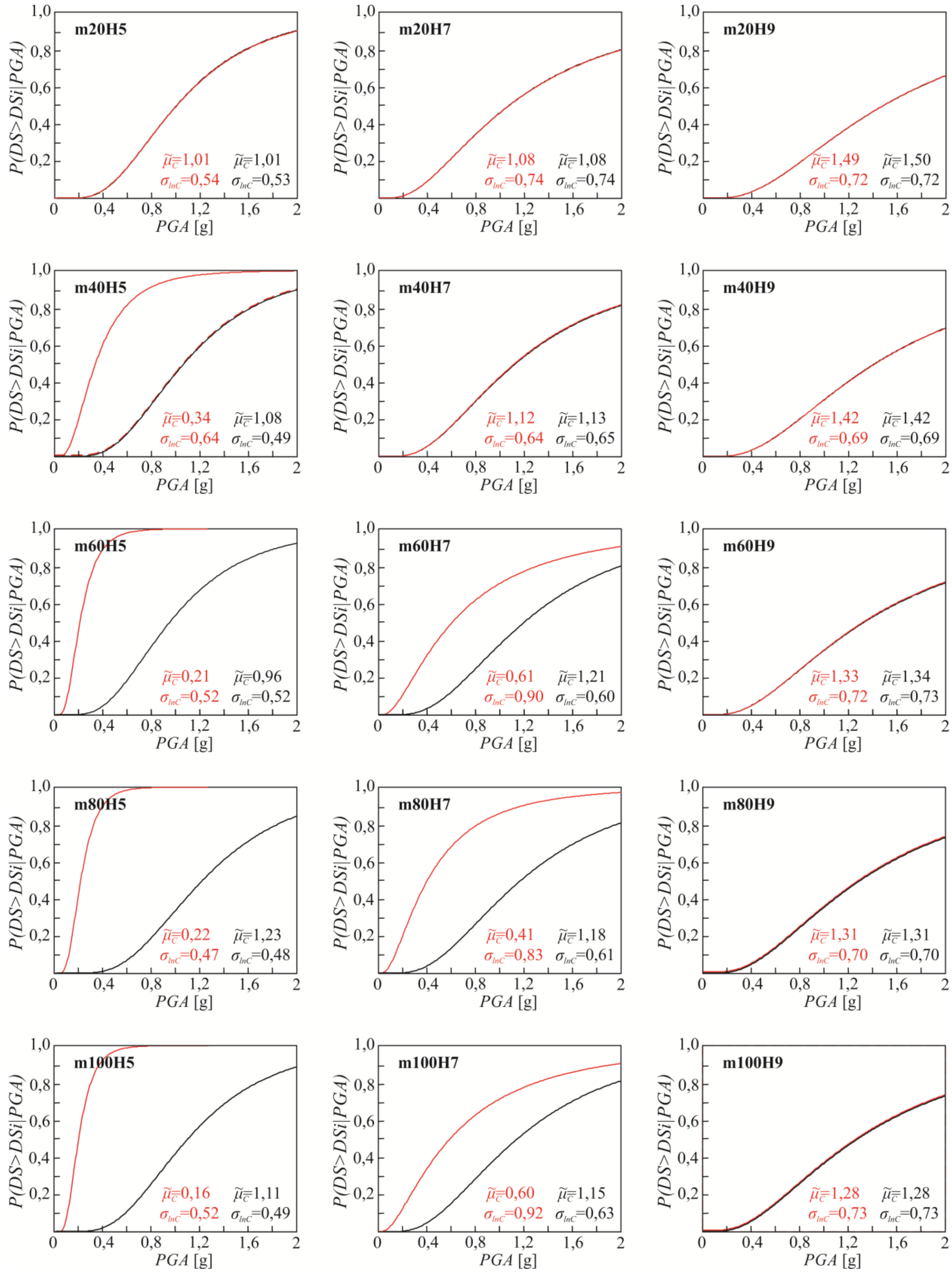

Fig. 7 – Fragility curves for buildings with "strong" (black curves) and "weak" (red curves) centric dowel connections – type 1 (see Fig. 3). It is assumed that only one beam is connected to a column.

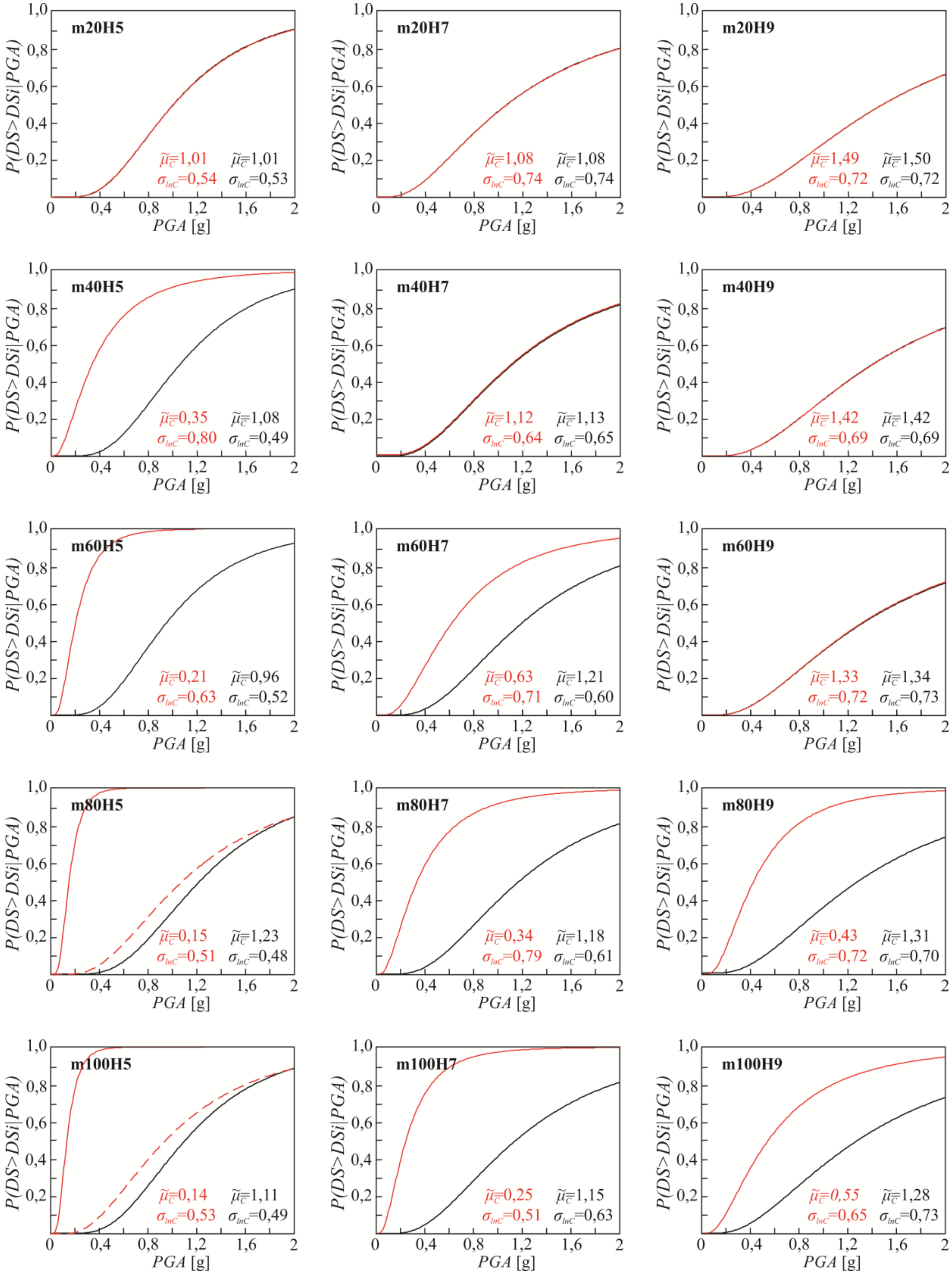

Fig. 8 – Fragility curves for buildings with "strong" (black curves) and "weak" (red curves) eccentric dowel connections with a relatively large amount of stirrups around the dowel – type 2 (see Fig. 3). It is assumed that one (continuous red curves) or two beams (dashed red curves) are connected to a column.

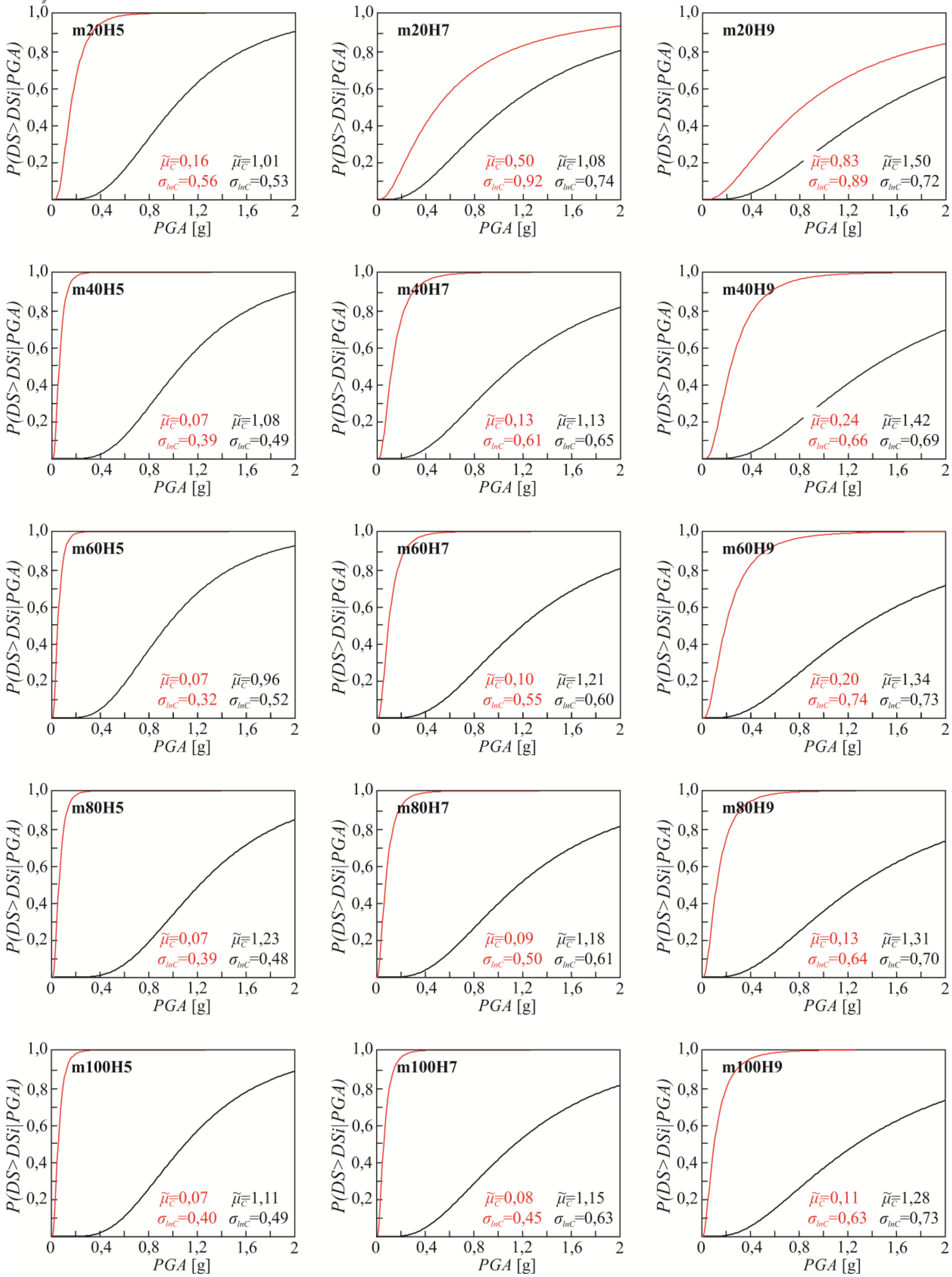

Fig. 9 – Fragility curves for buildings with “strong” (black curves) and “weak” (red curves) eccentric dowel connections with a relatively small amount of stirrups around the dowel – type 3 (see Fig. 3). It is assumed that only one beam is connected to a column.

5. Conclusions

In the paper, an extensive parametric study of the seismic fragility of precast industrial buildings typical for European practice was presented. The study took into the account two most probable failure mechanisms, which can endanger the stability of the whole precast building: the failure of columns, and the failure of the beam-column dowel connection. The main goal of the presented research was to confirm the outmost importance of adequate detailing of dowel connections to the seismic fragility of precast industrial buildings.

Seismic fragility curves were calculated for a representative set of 15 single-storey precast industrial buildings, designed according to the requirements of Eurocode 8 [9]. Record-to-record variability and modelling uncertainties were considered. Each building was analyzed taking into account three types of dowel connections, which differed regarding the amount of the confining reinforcement around the dowel and the position of the dowel relative to the edges of the column/beam. Dowels were designed according to Slovenian design practice, where the capacity of the dowel is typically defined based on the dowel shear strength.

The most important findings of the study are the following two:

1) In the beam-column dowel connections it is not possible to dissipate a considerable amount of seismic energy. Therefore, capacity design principle should be applied when designing such connections. The strength of the dowel connections cannot be defined based on the shear strength of dowels (which is the typical practice in Slovenia). In this way the strength is considerably overestimated and the probability of failure is considerably increased. The strength of the dowel connection can be defined as proposed in [5].

2) The results of the parametric study clearly demonstrated that a sufficient amount of stirrups around the dowel can prevent premature brittle failure of the eccentric dowel connections (dowels are constructed close to the edge of the column or beam). In this way the considerable impact of the stirrups around the dowel to the fragility of precast industrial buildings was confirmed.

Acknowledgements

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