SEISMIC COLLAPSE SAFETY OF RC COLUMNS IN PRECAST INDUSTRIAL BUILDINGS

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ABSTRACT:

The seismic collapse risk of one-storey precast industrial buildings is discussed. Due to the strong connections, the behaviour of the structures was dominated by columns having high shear span ratio. Full-scale pseudo-dynamic and cyclic tests provided important information about the behaviour of such columns when subjected to large deformations, as the structure approaches collapse. Based on the experimental results obtained, numerical model capable of modelling global collapse was improved for such columns with large shear spans. The model was applied to the seismic risk assessment of precast structures. Seismic risk was evaluated by means of probabilistic analysis, taking into account the randomness in seismic excitations and other sources of uncertainty. A verified probabilistic method was used to assess the seismic risk of the whole range of the analyzed precast structures, as built in practice. It was found that the minimum detailing requirements according to Eurocode 8 usually provide such structures with sufficient overstrength so that the seismic risk is acceptably low (the probability of collapse is 0.1–1.2 % in 50 years). However, if only design reinforcement is provided in the structures, the conservative estimate of seismic risk is high (the probability of collapse is 1.0–8.5 % in 50 years). The results have been used to obtain a quantitative evaluation of the force reduction factor used in Eurocode 8 standard.

KEYWORDS: precast reinforced concrete structures, slender cantilever columns, seismic collapse risk, force reduction factor

1. INTRODUCTION

Precast RC buildings represent an important share in the construction industry and house many businesses, people and equipment. However, there has been relatively little experimental and field evidence of the seismic behaviour of such systems. Several catastrophic collapses were reported in the past, e.g. in Armenia (Fardis, 1995). Some damage, specific to prefabricated industrial buildings in Europe was reported in Bulgaria (Tzenov, 1978), Montenegro (Fajfar, 1981) and Turkey (AIJ, 2001). Some collapses were catastrophic but in general the damage was rather limited. Obviously the design requirements in the codes have to be implemented, so that the likelihood of structural collapse will remain at an acceptably low level. For that reason it is necessary to expand the knowledge of the seismic behaviour of the precast concrete buildings.

Specific type of precast industrial building addressed in this paper consists of an assemblage of cantilever columns tied together with a roof system, which is essentially rigid in its own plane (capacity design of the connections is preventing the failure of joints). Seismic behaviour of such structures depends mainly on the cyclic behaviour of columns. It should be noted that these columns have two main characteristics – very large shear span ratio, and relatively low average axial compressive loading.

The main objective of this research was to assess the seismic collapse risk of the addressed structural system. The requirements of the Eurocode 8, and in particular the seismic force reduction factor for buildings of the treated type, were interpreted on the basis of the study of seismic risk. The main question was: is the solution from the latest version of the Eurocode 8 standard, which, under certain conditions (a limitation in the axial loading, properly designed connections), permits the use of the same reduction of seismic forces as in the case of monolithic reinforced concrete frames, satisfactory?

2. EXPERIMENTAL INVESTIGATION

Two full-scale models, representing single-storey precast industrial buildings, were tested. Both prototypes consisted of six slender cantilevered columns, which were connected to a roof structure. The two prototypes differed in the orientation of roof elements with respect to the applied earthquake loading: roof elements are parallel to the applied loading in case of Prototype 1 and perpendicular to the loading in case of Prototype 2. All the connections which held together the roof structure were designed according to the capacity design rule, so that the load-carrying capacity of the connections was relatively high. These connections produced a stiff diaphragm effect at the level of the roof structure, thus ensuring uniform distribution of seismic forces among the columns. Hence, the behaviour of both structures was similar, regardless of the roof configuration applied.

The plan of the Prototype 2 is illustrated in Figure 1 (the same dimensions apply for both prototypes). The ground plan dimension of the structure was 16 × 8 m. The columns were 5 m high and attached to the ground by means of traditional precast foundation sockets. The roof was made of prestressed concrete I-beams, which supported Π-panels on top (note: the opposite orientation of I-beams and Π-panels is characteristic for Prototype 1). The hinged connections between the columns and the beams were made of steel dowels and neoprene pads. Uniaxial material tests of the materials used for columns demonstrated mean cylindrical strength of concrete $f_c' = 55$ MPa and mean yield stress of steel $f_y = 555$ MPa. The total mass of the prototypes was 57.9 t (Prototype 2) and 62.3 t (Prototype 1) which result in average mass of 9.6 t (Prototype 2) and 10.4 t (Prototype 1) per individual column. Two characteristics of the columns are significant – very large shear span ratio ($L_s/h = 12.5$), and low average axial compressive loading (ν = 1.3%–1.5%).

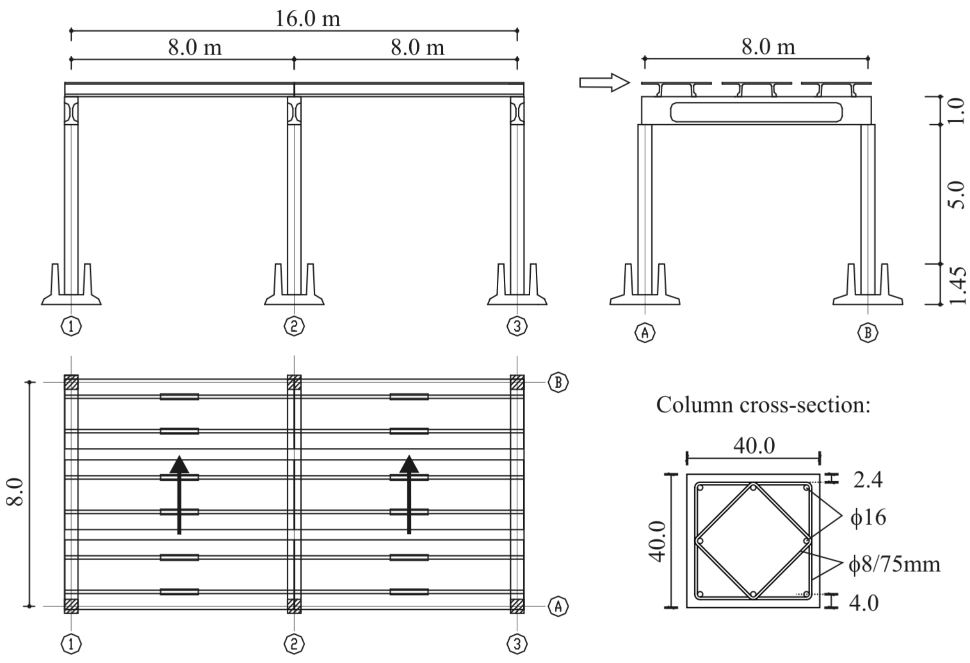

Figure 1 Plan of the Prototype 2

Due to the small mass of the specimen, design seismic forces of columns were rather small. Thus, minimum amount of flexural reinforcement according to EC8 ($\rho_{l\,min}$ = 1%) governed the design of columns (for both prototypes). It is important to realize that such reinforcement would be required by the code if the columns were designed for the design acceleration of 0.7 g, provided that the force reduction factor $q = 4.5$ for the ductility class high (DCH), which is the same as in the case of monolithic frame structures, was used.

The transverse reinforcement within the critical regions at the base of the columns was determined by the structural rules for the ductility class high (DCH) requirements. It consisted of two stirrups ϕ8 at a spacing of 50 mm (the corresponding ratio of transverse reinforcement was equal to $\rho_{sh} = 0.0057$).

The prototypes were first tested with a series of PsD tests. The seismic action was simulated by an artificial accelerogram generated to be compatible with the Eurocode 8 response spectrum for soil type B (dense sand or gravel). PsD testing was performed in four load steps: 0.05 g, 0.14 g, 0.35 g and 0.525 g. The capacity of the connections was high enough to enable the roof to act as a rigid diaphragm. The deformability and the deformation capacity of the structures (columns) were very high. Yielding in the columns was not observed until the last PsD test (0.525 g). The equivalent yield drift, estimated on the basis of the bilinear representation of the response, yielded about 2.6–2.8 %, which is much more than reported for columns with smaller shear spans. After the PsD testing, the final displacement-controlled cyclic test, with a constant step of 4 cm, was performed. The columns exhibited quite stable response up to a large drift close to 7 %. Buckling of the longitudinal reinforcement bars then led to subsequent tension failure of the bars in the first column, at about 7 % drift. At that drift the test of Prototype 1 was stopped. However, the test of Prototype 2 was continued, following considerable strength degradation and the flexural failure of several columns. At the 8 % drift the 20 % drop of strength was recorded, and the test of Prototype 2 was stopped as well.

3. NUMERICAL MODELLING

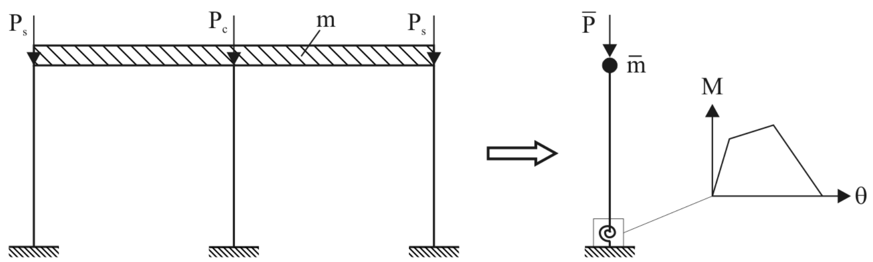

Figure 2 Simplified numerical model of the single-storey precast building

With regard to the experimental results (rigidity of the roof system), the whole building was modelled as an equivalent cantilever column with one sixth of the mass of the whole structure and the average axial compressive loading (Figure 2). A lumped plasticity model with a zero length inelastic spring at its base was applied to the column. The moment-rotation relation of the spring was defined by a hysteretic model developed by Ibarra et. al (2005) and calibrated for RC columns with predominant flexural behaviour by Haselton (2006). Based on the experimental results of the prototype structures, numerical model was improved for such columns with large shear span ratio (Fischinger et. al, 2008). The selected hysteretic model is capable of simulating the in-cycle as well as repeated-cycle strength degradation. This property of the model can be decisive importance when estimating seismic risk of the structure.

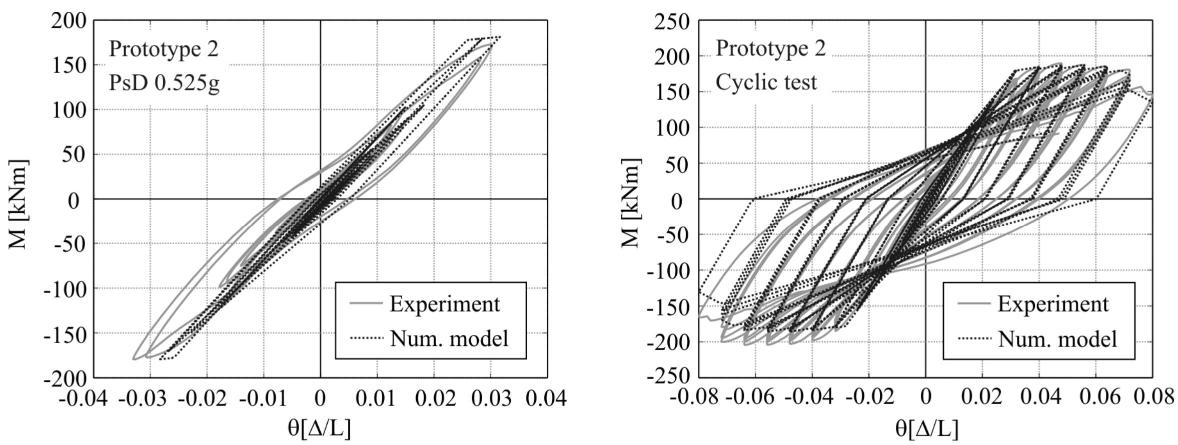

Figure 3 Numerical versus experimental results for Prototype 2

Using the selected numerical model, very good agreement with the experimental results was obtained for both tested structures (hysteretic response at the plastic hinge of Prototype 2 is shown in Figure 3; similar results were obtained for Prototype 1) so the model was chosen for future use in seismic risk studies.

4. PROBABILISTIC ANALYSIS

By means of the selected numerical model it is possible to obtain a fairly reliable estimate of the load-carrying capacity of a structure in the case of a predetermined load. It is, of course, well-known that seismic loadings are random and unpredictable and, apart from this, there are the other variables, which relate to the quality of the materials and the properties of the numerical model. When it is necessary to prepare a credible assessment of seismic risk for structures, then the methods of probability analysis have to be used. For this purpose a recently popular "PEER" methodology was used. The final result of the methodology is the probability of exceeding a structural limit state. With regard to the limit state definition different variations of the general "PEER" methodology can be applied. Most commonly, limit state is estimated by using pre-established limit value of the damage measure (*DM*) – e.g. displacements, deformations, etc. Thus, probability of exceeding a structural limit state is calculated according to the "*DM*-based approach". In this research, the limit state of the structure (column) was defined as the inability of a system to support gravity loads because of excessive lateral displacement. The collapse capacity of the structure (column) was predicted with the deteriorating numerical model considering P-delta effects. Therefore, the collapse was described by means of the Intensity Measure (*IM*) rather then Damage Measure (*DM*). The solution strategy called the "*IM*-based approach" was used.

4.1. General methodology

The *IM*-based approach is illustrated in Figure 4. The method is based on the Incremental Dynamic Analysis (IDA). IDA involves a series of dynamic analyses performed under several values of the intensity. The result is an IDA curve which is a plot of response values (i.e. damage measure - *DM*) versus the intensity levels (i.e. intensity measure - *IM*). The collapse of the structure occurs when the *DM*s increase in an unlimited manner for exceedingly small increments in the *IM* (collapse is indicated as the black dot on the IDA curve in Figure 4). Considering the record-to-record variability and the uncertainty in the numerical modeling, large number of IDA curves corresponds to the same structure, thus resulting in large number of limit state intensities (S_c). Separate analysis is involved in order to determine the seismic hazard function (H_s). The hazard function is defined as the probability that the intensity of the future earthquake will be greater than or equal to the specific value. Finally, limit state probability is calculated as the hazard function multiplied by the probability density function (PDF) of the limit state intensity and integrated over all values of the intensity. Presuming the lognormal distribution of the limit state intensity and exponential form of the seismic hazard function, limit state probability of the structure can be derived analytically (Jalayer, 2003).

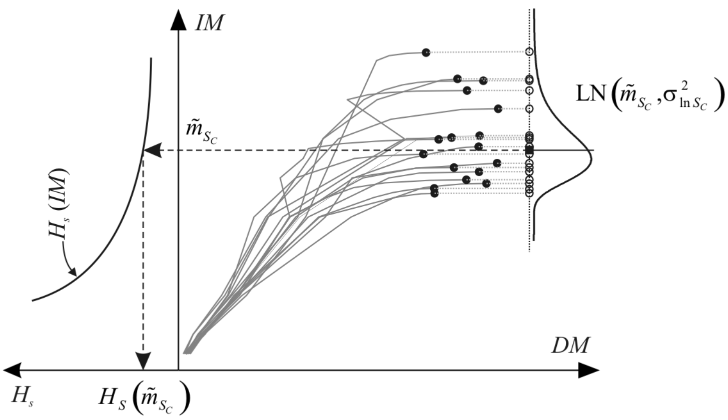

Figure 4: Schematic of the IM-based approach

4.2. Application to the RC precast structure

The *IM*-based approach was used to assess the seismic collapse risk of the RC precast buildings (i.e. an equivalent cantilever column). Common examples of the IM are the Peak Ground Acceleration (PGA) and the Spectral Acceleration at the structure's first-mode period ($S_a(T_1)$). In general, $S_a(T_1)$ produces lower dispersion over the full range of *DM* values (Vamvatsikos and Cornell, 2002). However, a comparative research has shown that the dispersion is similar for both measures of the intensity in case of the analyzed structure; therefore the PGA was chosen because it provides a better engineering feeling about the capacity of the structure.

Record-to-record variability was considered by means of 50 artificially generated accelerograms. The accelerograms were artificially generated to simulate the seismic action according to EC8 – i.e. the mean spectrum of the group of accelerograms approximately coincides with the EC8 elastic response spectrum for a soil type B (Figure 5a). The same design provisions were considered when determining the seismic hazard function. The hazard function was derived from the design acceleration values for return periods of 475 (0.25g), 1000 (0.3g) and 10000 years (0.55g) for the area of Ljubljana (Figure 5b).

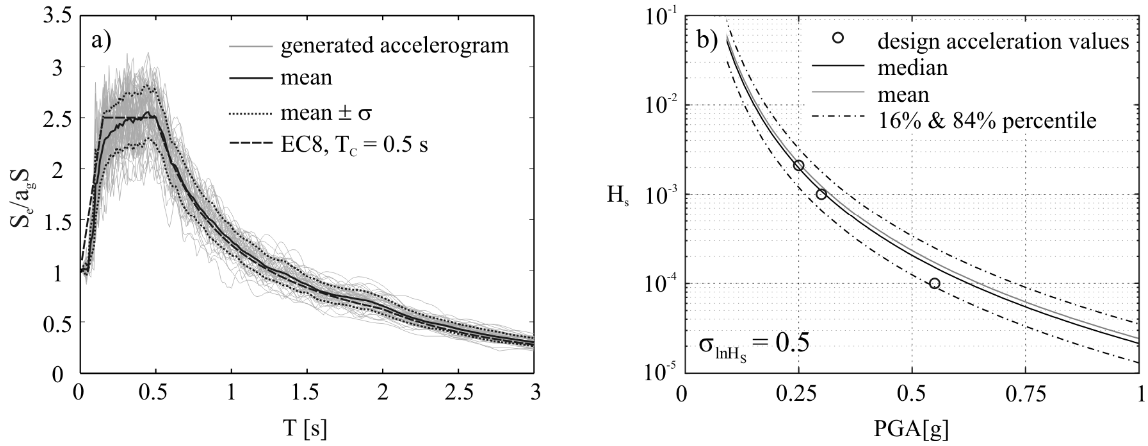

Figure 5 Normalized elastic response spectra for the generated accelerograms (a) and the seismic hazard function (b)

In addition to the record-to-record variability, the variance of collapse capacity due to the uncertainty in the numerical model was considered. The additional variance was computed using the first-order method; considering the model parameters like ductility capacity, post-capping stiffness, strength hardening and normalized energy dissipation capacity as random variables with lognormal distribution and standard deviation according to Haselton (2006). The advantage of the first-order method in comparison to the Monte Carlo method is the reduction of analyses needed to evaluate the variance. Relatively low computational cost enabled the evaluation of the variance for the whole range of columns with various characteristics (Kramar, 2008) discussed in the parametric study (chapter 5).

Using the described procedure, limit state probability of the tested structures yields 0.1-0.4% in 50 years, which is considered as low probability of failure. However, the mass of the prototype structures is relatively small in comparison to the "real" structures. In order to check the behaviour of the structures (columns) at more severe conditions, seismic assessment study was extended to a whole range of one-storey precast systems with different values of mass and corresponding amount of reinforcement.

5. PARAMETRIC STUDY

In the parametric study equivalent columns with large variation in mass were adopted. Concentrated mass was ranging from 10 t (small mass which can be attributed to vertical load 2.5 kN/m^2 acting on a tributary roof area of 40 m^2) to 150 t (large mass corresponding to load 6.5 kN/m^2 and tributary roof area of 230 m^2). All of the columns were 5 m high, while three different cross-sections were discussed: 40×40 cm, 50×50 cm and 60×60 cm. Columns were designed according to the latest Eurocode 8 standards, allowing for the force reduction factor q = 4.5 for the ductility class high (DCH), which is the same as in the case of monolithic frame structures. Design acceleration a_g = 0.25 g and soil type B were considered. It was found that the P-delta effects determine relatively large cross-sectional dimensions of the columns. Thus, it is not allowed to apply an equivalent mass larger than 30 t to the column of 40×40 cm. Likewise, the upper limits for the columns of 50×50 cm and 60×60 cm are 70 t and 150 t respectively. Due to the large depth of the cross-section, the design longitudinal reinforcement in columns is small and the minimum amount of longitudinal reinforcement for columns according to Eurocode 8 ($\rho_{l,min}$ = 1%) governs the design of all columns. Furthermore, the transverse reinforcement in the critical regions is determined by the structural rules for the DCH degree of ductility (note: minimum requirements and structural rules were decisive also in the case of prototype structures).

The main objective of the parametric study was to obtain a quantitative evaluation of the force reduction factor used in Eurocode 8. Two groups of columns were formulated: First group of columns was designed according to the latest Eurocode 8 standards, while taking into account all of the structural requirements for the DCH degree of ductility (Figure 7). In the second group of columns only “calculated reinforcement” was taken into account, without the implementation of the minimum requirements for longitudinal and transverse reinforcement (Figure 6). Such classification enabled the evaluation of the overstrength due to the minimum requirements and structural rules in the Eurocode 8 standard.

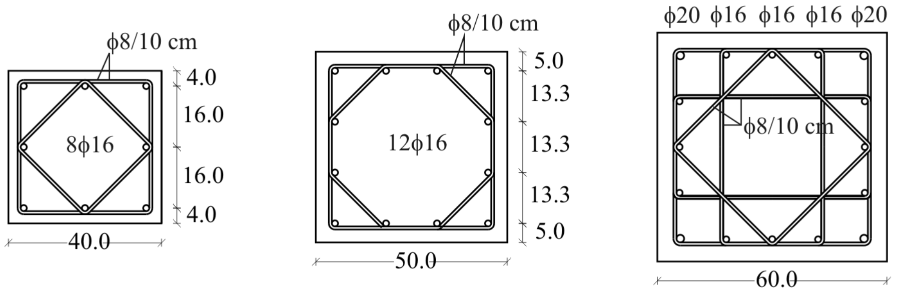

Figure 6 Cross-sections of columns with reinforcement according to EC8 considering all the minimum detailing requirements

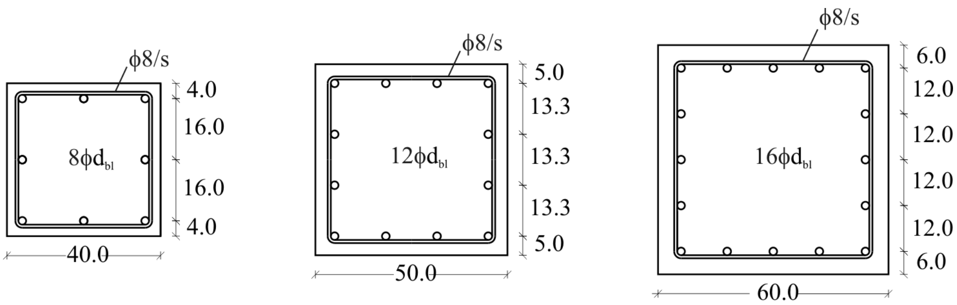

Figure 7 Cross-sections of columns with only the calculated reinforcement (minimum detailing requirements not considered)

Probabilistic method discussed in previous chapter was used to assess the seismic risk of precast structures (columns). Seismic risk was estimated based on the following criteria:

1. **Capacity of the structure expressed in terms of PGA (PGA_C)**
   Reference value − 5 % percentile of PGA_C, was compared to the design acceleration of 0.25 g.
2. **Probability of collapse in 50 years for the area of Ljubljana ($H_{LS,50}$)**
   $H_{LS,50}$ was compared to the target reliability suggested by the Joint Committee on Structural Safety. Target failure rate for structures with moderate consequences of failure and large relative cost of safety (seismic loading) is equal to 2.5 % in 50 year reference period (JCSS, 2001).

6. RESULTS AND CONCLUSIONS

Results for both groups of columns with cross-sectional dimensions 60×60 cm are presented in Figure 8 and Figure 9. It can be observed that the seismic risk exceeds the limit values for most of the columns with only calculated reinforcement ($PGA_{C,5\%}$ = 0.2−0.4 g, $H_{LS,50}$ = 1.0−8.5 %). However, the seismic risk is considerably reduced when all the minimum requirements and structural rules are considered ($PGA_{C,5\%}$ = 0.4−0.6 g, $H_{LS,50}$ = 0.1−1.2 %). Very similar results were obtained for the columns with smaller cross-sections.

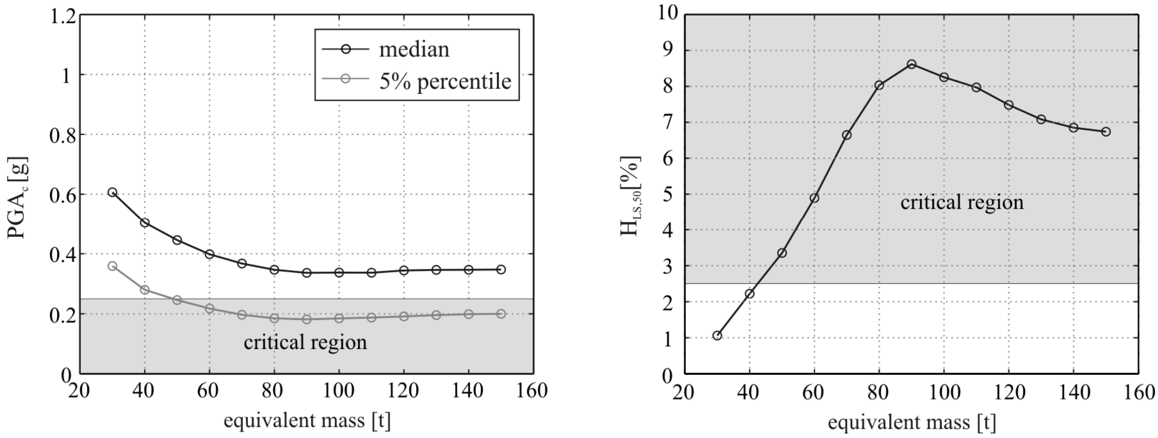

Figure 8 Seismic risk of columns with "design reinforcement" (h = 60 cm)

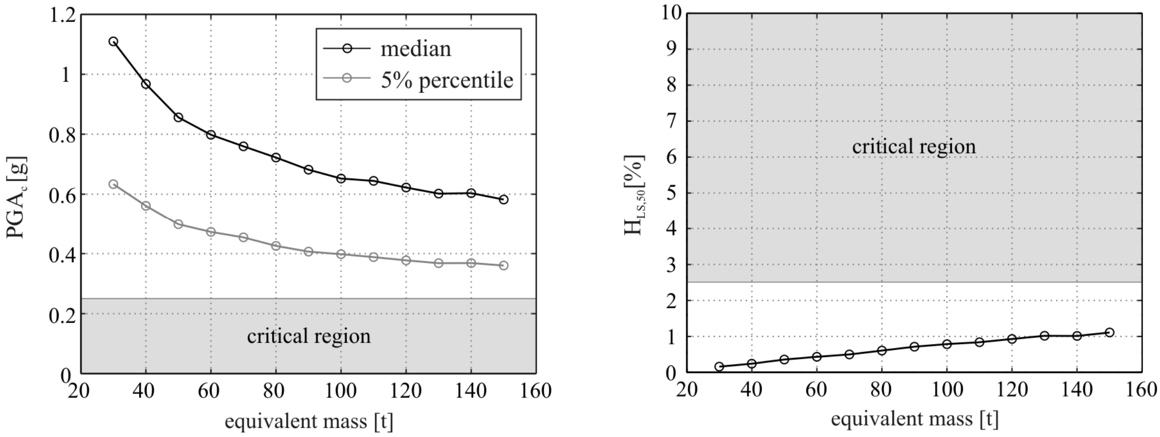

Figure 9 Seismic risk of columns designed according to EC8 including all the detailing requirements (h = 60 cm)

Based on these results, the force reduction factor of $q = 4.5$ could be justified for the columns designed according to Eurocode 8, considering all the minimum requirements. However, such factor should not be used for the columns which do not meet the minimum requirements and structural rules according to the latest Eurocode 8 standard.

In general, the basic value of the force reduction factor in Eurocode 8 is multiplied by the factor α_u/α_1 which accounts for the redistribution of loading among the elements for the multi-degree of freedom systems. The multiplier α_u/α_1 = 1.0 should be used for the one-storey precast structures, since these structures are basically single-degree of freedom systems without the capability of the redistribution of forces. Thus, the final value of the force reduction factor for the one-storey precast structures is equal to the basic value q = 4.5.

The results of this research have confirmed the solution from the latest version of Eurocode 8 standard, which, under certain conditions (a limitation in the axial loading, connections designed according to the capacity design rule, minimum requirements considered) permits the use of the same basic value of the force reduction factor as in the case of monolithic frames (q = 4.5). Nevertheless, we believe that monolithic structures, which are designed with the same basic force reduction factor as precast structures, are safer. One of the reasons is large overstrength of the monolithic systems due to the redistribution of loading among the elements (multiplier $\alpha_u/\alpha_1 = 1.1$ is permitted for the one-storey monolithic frame structures; in reality, larger values are expected). Besides, the monolithic frame structures have a larger number of plastic hinges; moreover the hinges are located at the edges of beams which are typically more ductile than the columns.

ACKNOWLEDGMENTS

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