Seismic Risk Assessment of Multi-Storey Precast Structures

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SUMMARY:
Single-story precast structures with the pinned connections between the columns and the beams have been frequently used in Europe. However, a similar structural solution has been extended to the multi-storey buildings. The aim of this research was to assess the seismic safety of multi-storey precast structures with pinned connections. On the basis of the experiments performed in the framework of the SAFECAST project, a numerical model for both the columns and the connections was calibrated. Using the calibrated models seismic risk was evaluated for the whole set of the regular multi-storey structures which are usually built in practice. The observed differences in the probability of exceeding the limit state as well as the parameters contributing to seismic safety were discussed. In buildings with masses corresponding to the normalized axial force in columns above approximately 0.15, seismic risk increased substantially due to the possibility of the failure of standard pinned connections.

Keywords: Multi-storey precast structures, seismic risk assessment, beam-column connections

1. INTRODUCTION

In this paper, multi-storey precast system with hinged beam-to-column connections is discussed. Assuming that cladding panels do not contribute to the lateral resistance of the structure, the resistance relies only on the multi-storey cantilever columns, which are typically very slender. In addition, recent research (Fischinger et. al., 2012) indicates that the horizontal forces acting on the beam-to-column connections are much larger than considered in traditional design. This could cause a shear failure of the connections. Therefore, the main objective of the research was to assess the seismic risk of multi-storey precast structures considering realistic properties of the beam-to-column connections.

First, the numerical model of multi-storey precast structure was calibrated with the experimental results of the SAFECAST (Colombo, 2012) project. The columns were calibrated based on the pseudo-dynamic and static-cyclic tests of the 3-storey full-scale precast structure performed at ELSA laboratory in Ispra (Italy). In addition, the connections between the columns and the beams were calibrated based on the test results of the intermediate-storey beam-column connection typical for the Slovenian practice. The calibrated model was used to assess the seismic risk for the whole set of structures of regular 2- and 3-storey precast buildings, which can occur in the design practice.

The 'PEER methodology' was used to assess the seismic collapse risk. This methodology involves the use of conditional probabilities, which are used to propagate the uncertainties from one level of analysis to the next, resulting in a probabilistic prediction of performance. With regard to the limit state definition, different solution strategies can be applied. In this research the collapse limit state of the structure was discussed. The collapse was defined either by the formation of plastic hinges at the base of a column, or by the failure of the connections. In accordance with the collapse limit state definition, the solution strategy for evaluating collapse, i.e. the intensity measure approach, was applied. This solution employs a deteriorating numerical model, which can predict collapse in terms of an intensity measure instead of using a predefined limit value of the response.

2. CALIBRATION OF THE NUMERICAL MODEL

The numerical model of the investigated system is presented in Fig. 2.1. The structures were represented with a single "frame" oriented in the direction of loading. The height of the first and the subsequent floors was 7 m and 5 m, respectively. In order to model the response within the connections, the connections between the columns and the beams were modelled as springs in the horizontal direction. Each spring was actually a zero-length element with the prescribed non-linear force-displacement relationship. In the direction of the rotation, connections were considered perfectly hinged. The mass of each floor was divided in two 2 parts, located in the middle of the beams.
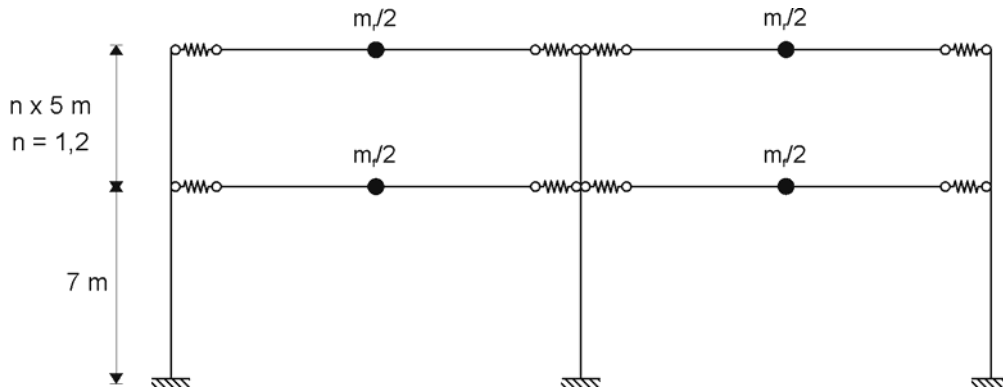

Figure 2.1. Numerical model of multi-storey precast structures

The numerical model was calibrated based on the experiments performed in the framework of the SAFECAST project (Colombo, 2012). The key tests of the project were pseudo-dynamic and static-cyclic tests of the 3-storey full-scale precast structure (Fig. 2.2) performed at the ELSA laboratory in Ispra, Italy. Four structures with different lateral force resisting systems were investigated: one with the wall (prototype 1) and three with bare frames (prototype 2-4). In Prototype 1 and 2 all the connections between the columns and the beams (and columns and the walls) were hinged. In prototype 3 the hinged connections in the upper floor are replaced with moment-resisting connections. The same was done in the rest of the floors in Prototype 4. In this paper, the main interest was taken on the multi-storey cantilever system with hinged beam-to-column connections (Prototype 2) which resembles the system analyzed in this paper. This prototype was used to calibrate the columns of the numerical model.

In prototype 2, the connections between the columns and the beams were overdesigned. Therefore, the response of the connections in the horizontal direction was mainly elastic. For this purpose, other tests of the isolated beam-to-column connections were used to calibrate the behaviour of the connections.
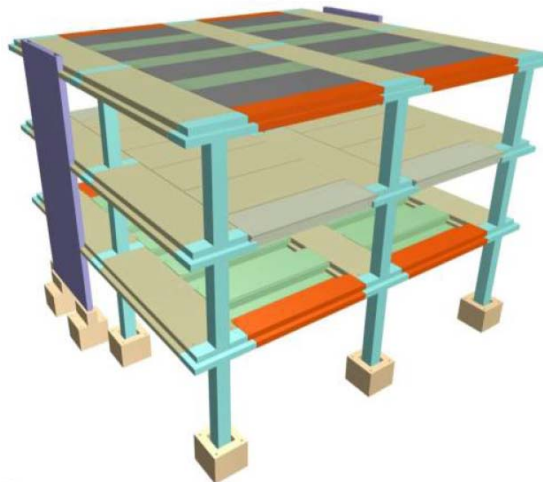

Figure 2.2. ELSA structure

2.1. Calibration of the model for the columns

Due to the hinged beam-to-column connections, the whole lateral resistance of the investigated structure relies on the multi-storey cantilever columns. Therefore, a special attention was paid to the modeling of such columns. In multi-storey cantilever columns the bending moments distribution is changing over the time. Because of this, the columns cannot be accurately modelled with the concentrated plasticity models. Therefore, the distributed plasticity element with the force-based formulation (i.e. forceBeamColumn element in OpenSees) was used. The required number of integration points was calibrated with the experimental results of Prototype 2.

The behaviour within the integration points was modelled with the pre-defined moment-curvature envelopes which were associated with the Takeda hysteretic rules. The moment-curvature envelopes were calculated with the section analysis, considering median material characteristics and confinement of the concrete core (Mander, 1988). Different approximations of the envelopes were considered and compared to the experimental results. The best agreement with the experiments was obtained with the 4-linear backbone curve determined by the crack point, yielding and the maximum strength. The maximum strength corresponded to the concrete spalling while the yielding was determined with the equal energy rule. After achieving the maximum strength, severe strength degradation was considered. With the moment-curvature model described above and 5 integration points along the elements, very good correlation was obtained for different levels of the intensity (Fig. 2.3). The same model, with appropriate modifications relating to the geometry and material characteristics was used to model the columns in the parametric study (Section 4).
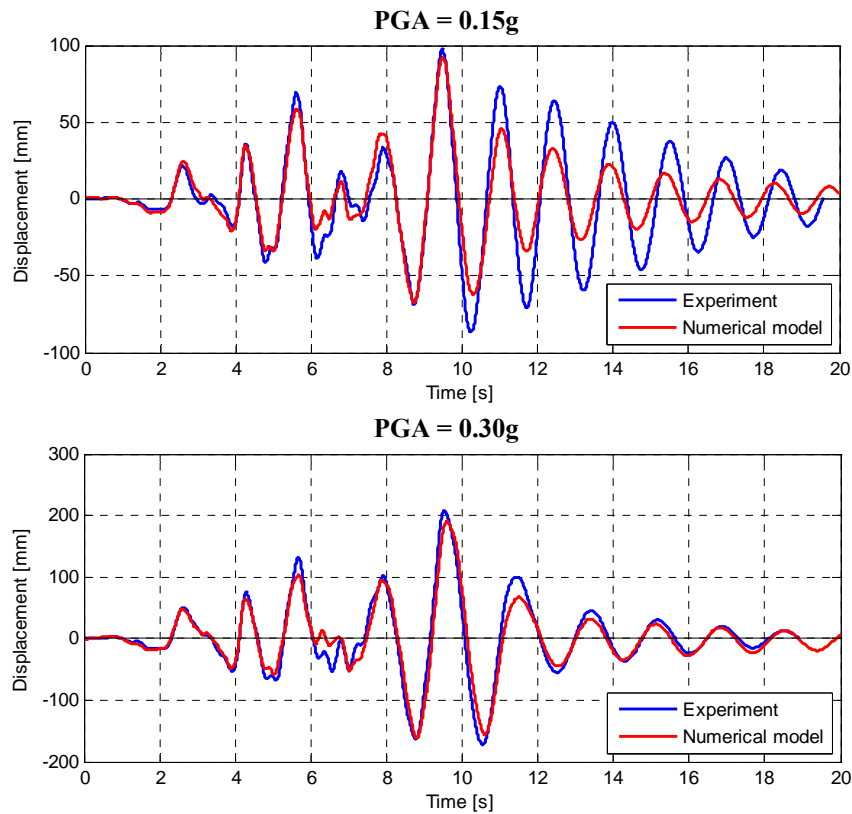

Figure 2.3. Top displacement time-history of Prototype 2 (ELSA structure)

2.2. Calibration of the model for the connections

Horizontal connections between the columns and the beams were calibrated using the results of the cyclic tests performed by the authors on the intermediate-storey beam-column connections typical for the Slovenian practice (Fig. 2.4). The same numerical model was applied to all connections in the parametric study.
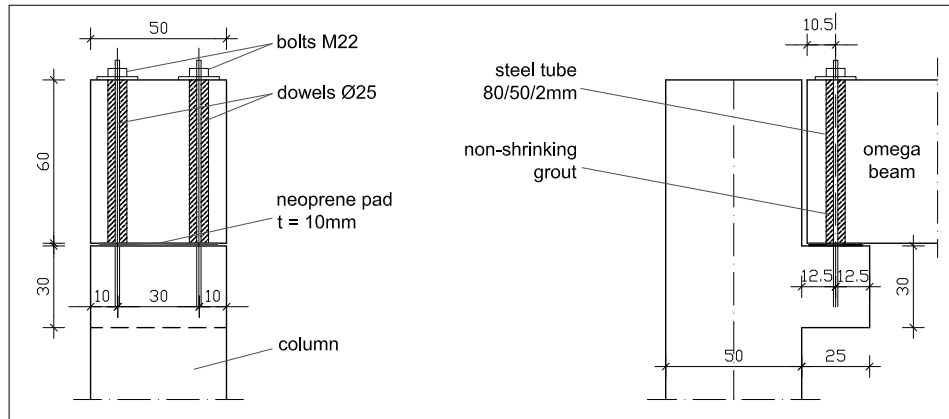
Figure 2.4. Intermediate storey connection tested by UL

Fig. 2.5 shows the response of the connection in terms of horizontal force (Fh) versus relative displacement (L1). It can be observed that the response is asymmetrical. The strength degradation only occurs when the beam is pulled away from the column (positive L1). In the opposite direction, the behaviour is defined by the empty space between the beam and the column (20 mm). Until there is no contact with the column, the behaviour is similar to the positive direction. However, when the beam touches the column (L1 = -20 mm), large increase in stiffness is observed.

The observed behaviour was modelled (Fig. 2.5) with a *Hysteretic material* object integrated in OpenSees (2010). Hysteretic material accounts for pinching of force and deformation, damage due to ductility and energy, and degraded stiffness based on ductility. In this particular case, only the damage due to ductility was considered (damage1 = 0.0012) while the damage due to energy was neglected. The pinching effect was found to be pretty large (pinchX = 0.9, pinchY = 0.5). The degraded unloading stiffness was assumed equal to 0.5. The monotonic response depended on the direction of loading. In the positive direction (positive L1) strength degradation was considered with a negative slope of a backbone curve. As follows from the experimental results, no strength degradation was considered in the negative direction (negative L1). Instead, displacements were limited with the minimum displacement (-20 mm) which represents the empty gap between the beam and the column. This limitation was realized with a parallel material object made up of previously-defined Hysteretic material and additional Gap material.
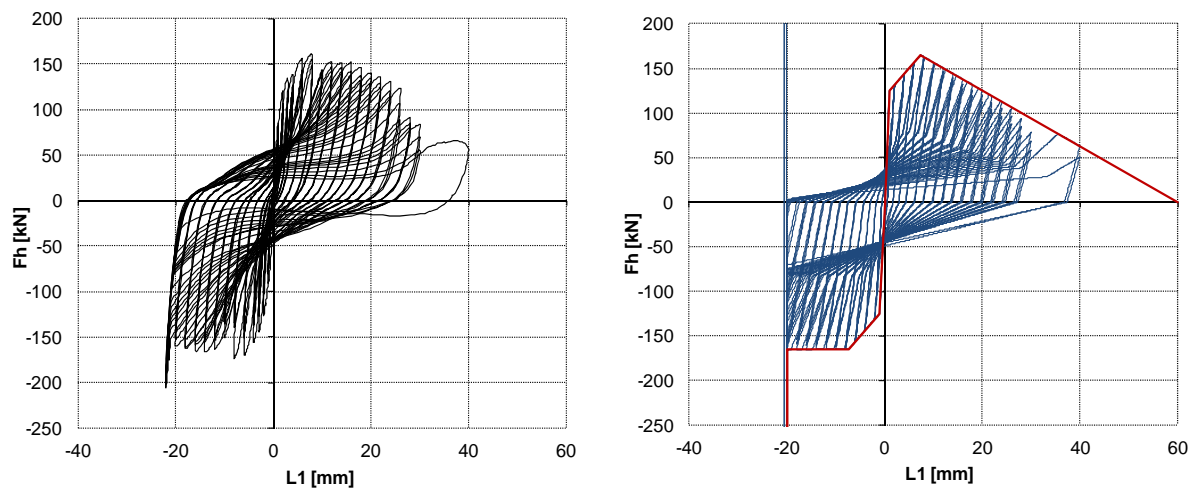

Figure 2.5. Connection: experimental vs. numerical results

In order to assess the seismic risk, the failure of the connection should be perceived by the numerical model. This happens at the moment when the resistance of the connection is reduced to zero (i.e. when the displacement reaches 60 mm or even earlier, depending on the damage due to the cyclic degradation). The failure never occurs in the negative direction.

3. PROBABILITY-BASED SEISMIC PERFORMANCE ASSESSMENT

Seismic performance assessment was made according to the "PEER" methodology (Cornell, 2002). This methodology involves the use of conditional probabilities, which are used to propagate the uncertainties from one level of analysis to the next, resulting in a probabilistic prediction of performance. With regard to the limit state definition, different solution strategies can be applied. In this research the collapse limit state of the structure was discussed. The collapse refers either to the failure of the columns (at the base) or the failure of the connections. In accordance with the collapse limit state definition, the solution strategy for evaluating collapse, i.e. the Intensity Measure approach, was applied (Jalayer, 2003). This solution employs a deteriorating numerical model, which can predict collapse in terms of an intensity measure instead of using a predefined limit value of the response.

3.1. Overview of the global collapse methodology (the IM-based approach)

The IM-based approach is illustrated in Fig. 3.1. 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 DMs increase in an unlimited manner for exceedingly small increments in the IM (collapse is indicated as the black dot on the IDA curve in Fig. 3.1). Considering the record-to-record variability and the uncertainty in the numerical modelling, 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, the mean limit state probability can be derived analytically (Jalayer, 2003):

$$\bar{H}_{LS} = \tilde{H}_S\left(\tilde{m}_{S_C}\right) \cdot \exp\left(\frac{1}{2}k^2\beta_{RTR}^2\right) \cdot \exp\left(\frac{1}{2}k^2\beta_{MDL}^2\right) \cdot \exp\left(\frac{1}{2}\beta_{Hs}^2\right) \tag{1}$$

where:

- $\tilde{H}_S$ is the median seismic hazard function with a lognormal standard deviation β_{Hs}; approximated by a power-law relationship: $\tilde{H}_S(x) = k_0 \cdot x^{-k}$;
- $\tilde{m}_{S_c}$ is the median capacity of the structure expressed by the intensity measure;
- β_{RTR} is the record-to-record uncertainty (lognormal standard deviation);
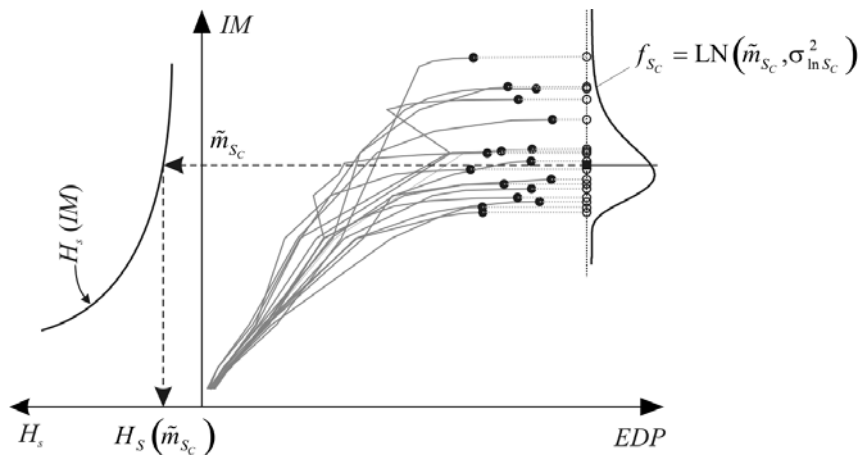- β_{MDL} is the is the modelling related uncertainty (lognormal standard deviation);

Figure 3.1. Schematic presentation of the IM-based methodology

3.2. Application to the multi-storey precast structures

In this research, PGA was chosen for the IM mainly due to the lack of seismic hazard data in Slovenia related to the spectral accelerations. The RTR variability (β_{TOT}) was evaluated based on the response to the 50 accelerograms which were artificially generated to match the EC8 elastic response spectrum for ground type B (Fig. 3.2a). In addition to the RTR variability, the modelling related uncertainty (β_{MDL}) was considered in accordance with the ATC recommendations (ATC, 2008). The numerical models used in this research can be classified as "Good" according to ATC report. Hence, the dispersion value $\beta_{MDL} = 0.3$ was adopted for all cases. The seismic hazard function was derived from the design acceleration values for different return periods for the area of Ljubljana (Fig. 3.2b). Therefore, relatively large value of the seismic hazard dispersion was assumed ($\beta_{Hs} = 0.5$).
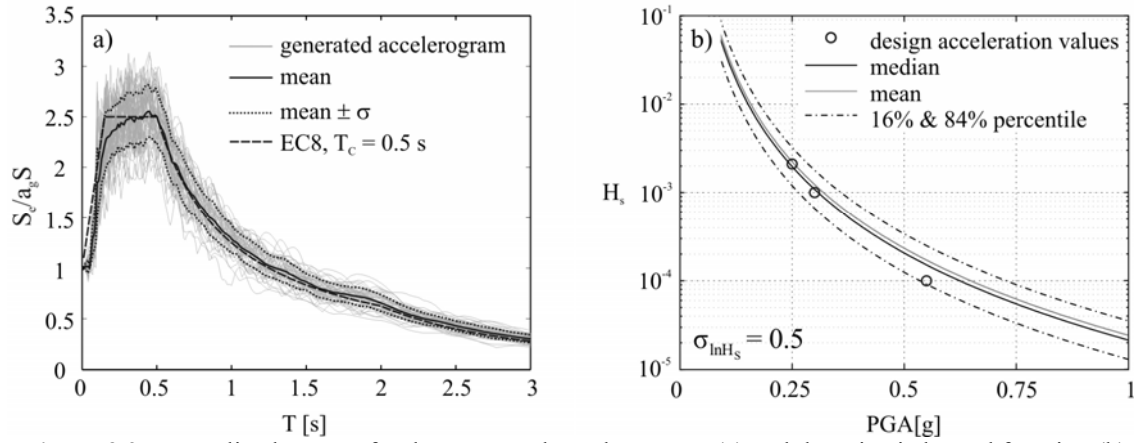

Figure 3.2. Normalized spectra for the generated accelerograms (a) and the seismic hazard function (b)

4. PARAMETRIC STUDY

The procedure described above was performed for realistic variations of regular multi-storey precast buildings, which are commonly used in the Slovenian/European practice. The investigated structural system (Fig. 4.1) consisted of 3 multi-storey cantilevers connected with the hinged beams. In accordance with the common practice, the structure had either 2 or 3 floors. The height of the first storey was assumed equal to 7 m, while the height of the subsequent stories was taken equal to 5 m. The amount of mass (i.e. vertical loading) and thus the size of the column cross-sections were varied within the range determined by the Eurocode standards. In addition, the ratio of the floor loading to roof loading (r) was varied from 1 to 6 in order to capture different mode shapes.
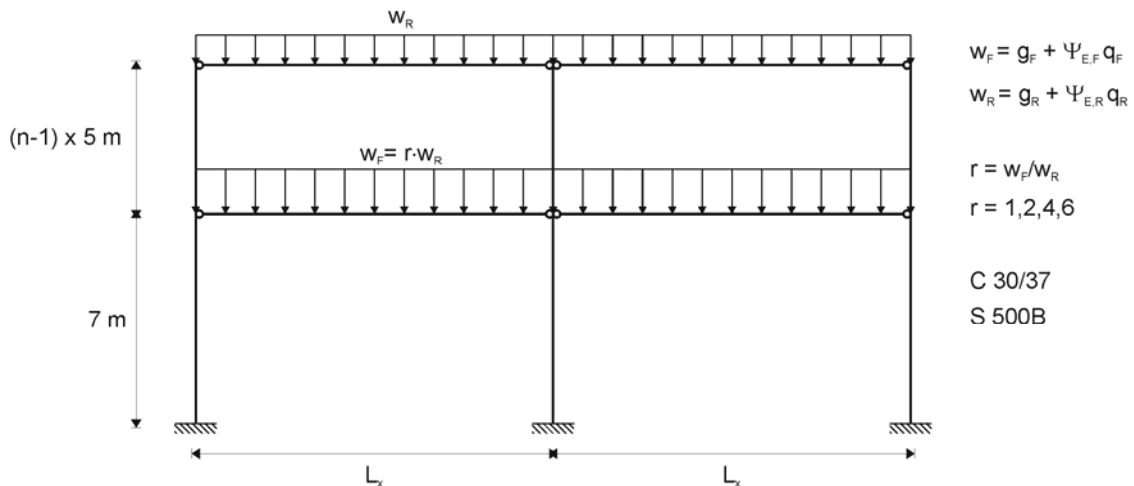

Figure 4.1. Numerical model considered in the parametric study

All cases considered in the parametric study are schematically presented in Tab. 4.1-4.2. Sixteen 2-storey structures and sixteen 3-storey structures were analyzed (32 structures all together). The structures vary depending on the column cross-section (bxh), maximum normalized axial force measured at the base of a middle column (v_d), and the ratio of floor to roof weight (note: r1 means r = 1, r2 means r =2, etc). The cases were selected to carry as much weight as possible but still meet the requirements regarding the drift limitations ($d_r \leq 0.025h$) and second order effects ($\theta \leq 0.3$) which govern the design of these structures (the common design assumption that the cladding panels do not affect the structural stiffness was considered, although this assumption is questionable). This means that the structures with larger column cross-sections were loaded with larger axial force and vice versa. For the same reason, the peak normalized axial force of the 3-storey structures was smaller compared to the 2-storey structures.

Table 4.1. 2-storey structures considered in the parametric study

b [cm]	$v_d = 0.1$	$v_d = 0.15$	$v_d = 0.2$	$v_d = 0.25$
60	r1, r2, r4, r6			
70		r1, r2, r4, r6		
80			r1, r2, r4, r6	
90				r1, r2, r4, r6

Table 4.2. 3-storey structures considered in the parametric study

b [cm]	$v_d = 0.05$	$v_d = 0.075$	$v_d = 0.1$	$v_d = 0.125$	$v_d = 0.15$	$v_d = 0.175$
60	r1, r2	r4, r6				
70		r1, r2	r4, r6			
80			r1, r2	r4	r6	
90				r1	r2	r4, r6

The structures were designed according to Eurocode 8 (2004) assuming design ground acceleration 0.25g and type B ground. Ductility class medium (DCM) was considered, with the behaviour factor equal to 3.0 (although the actual force reduction was usually smaller as explained below).

The analyzed structures are very flexible. Therefore, the cross-sectional dimensions of the columns were determined by the damage limitations rather than by the required strength. Consequently, the minimum longitudinal reinforcement ($\rho_1 = 1\%$), which should be provided to all primary seismic columns according to Eurocode 8, was sufficient in all cases. Similarly, the confinement reinforcement at the base of the columns was determined by the minimum value of the mechanical volumetric ratio of confining hoops applicable to DCM columns ($\omega_{wd,min} = 0.08$). The same section was assumed throughout the column height, regardless of the column location (external-central).

The structures were modelled in accordance with the procedure defined in Section 2. The non-linear dynamic analysis of the structures was performed in OpenSees (2010). In addition to the basic characteristics (bxh, r, v_d), the structures varied depending on the type of connections:

1) **Structures with strong connections**. In this case, beam-to-column connections were considered infinitely strong. This situation refers to the case when the connections are designed according to the capacity design rule and are therefore much stronger than the seismic demand. Hence, the collapse (and PGA capacity) was determined by the collapse at the base of a column, followed by the severe strength degradation of the structure.

2) **Structures with weak connections**. In this case, non-linear behaviour of the connections was included in the numerical model in accordance with Section 2.2. In practice, seismic demand in connections is typically not calculated and the designers are using the standard solutions/proportions of the connections. Hence, the same typical connection was used for all types of structures, regardless of the seismic demand in the connections. The collapse of the structures was defined either by exceeding the flexural capacity at the base of the columns, or by the failure of the connection (whichever occurs first).

4.1. Results

In Fig. 4.2-4.3 seismic risk of structures with strong and weak connections is compared for different values of normalized axial force (v_d) and mass ratio r = 2, 4 (results for other ratios can be found in (Fischinger et.al., 2012). The results are presented in terms of median PGA capacity (Fig. 4.2) and frequency of exceeding the limit state in 50 years (Fig. 4.3).
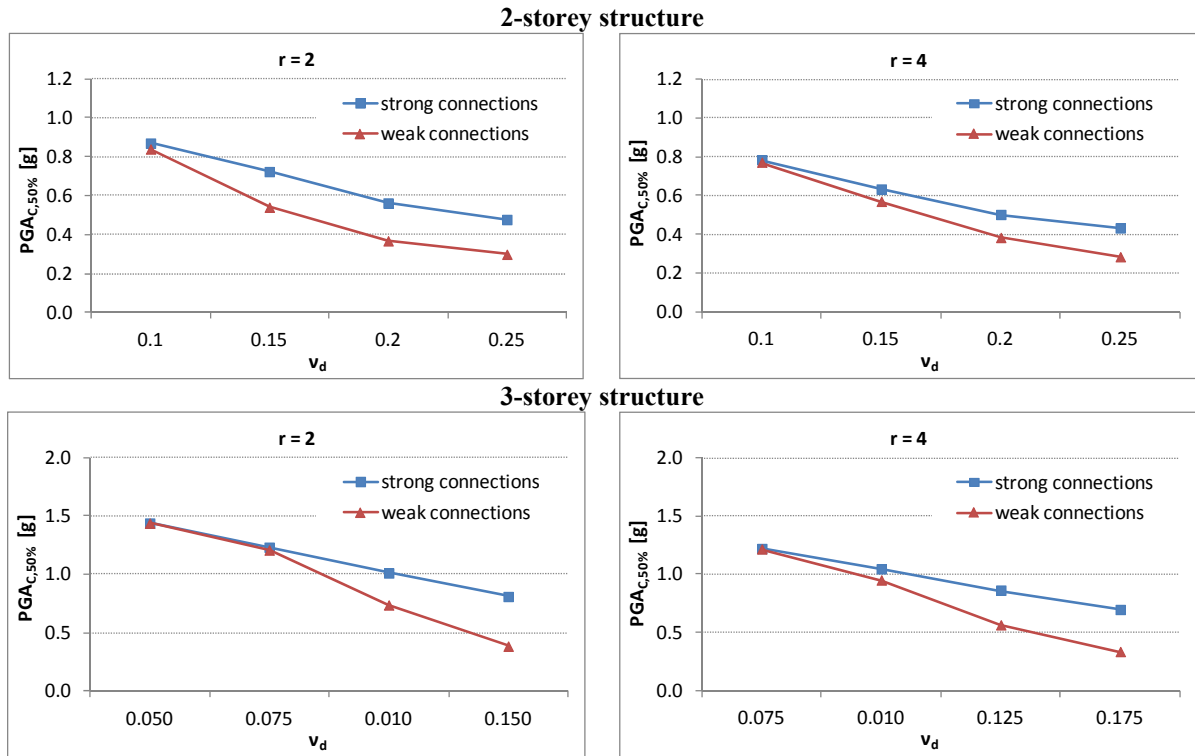
2-storey structure

3-storey structure

Figure 4.2. Median PGA capacity of the structures

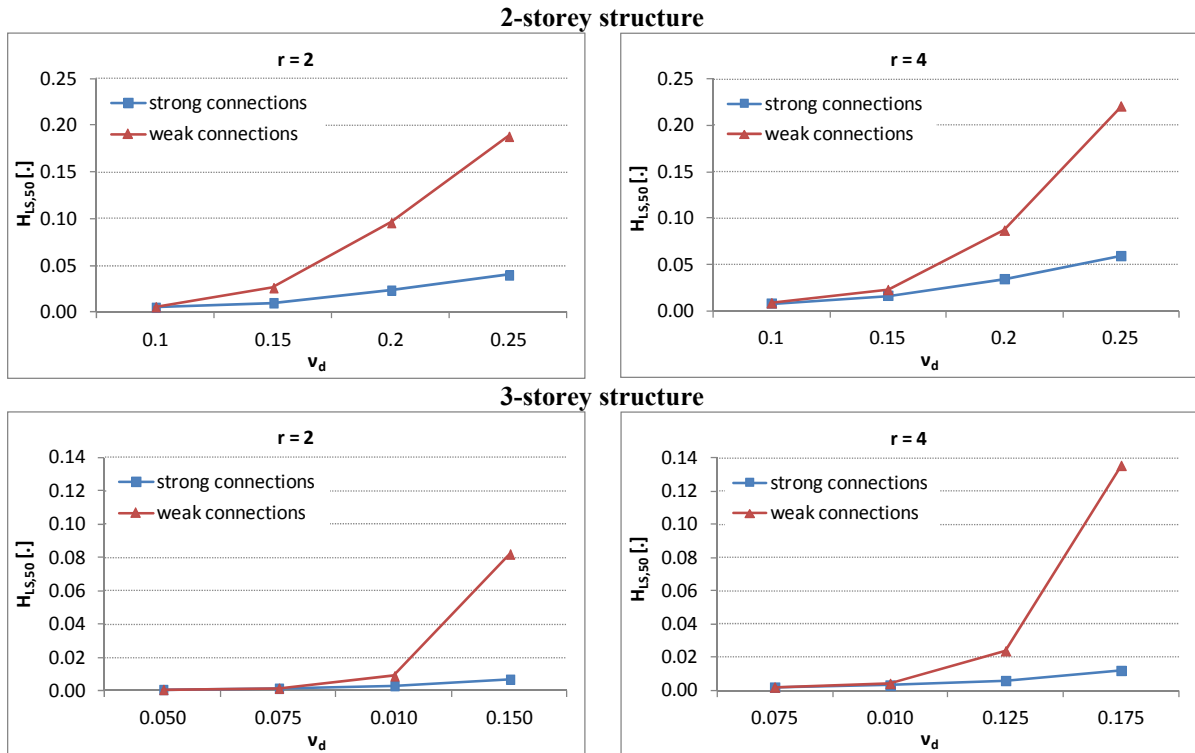**2-storey structure**

3-storey structure

Figure 4.3. Frequency of exceeding the limit state in 50 years

As expected the seismic risk is the smallest in the case of structures with small normalized axial force corresponding to small masses (small risk is indicated by large value of median PGA capacity and at the same time small frequency of exceeding the limit state). As already explained in Section 4, columns with small normalized axial force have very large overstrength which results from the minimum reinforcement requirements. The more the normalized axial force (mass of the structure) is increased, the more the overstrength is reduced and consequently the capacity is smaller. The results are also slightly different in terms of the ratio r ("r" is the ratio between the floor load/mass and the roof load/mass). It seems that the most unfavourable are large ratios; however, the differences between structures with different ratios are small. When comparing the 2-storey and 3-storey structures, structures with the same normalized axial force should be considered. If such comparison is made, it can be concluded that results are similar for both structures.

In order to quantify the risk, limit values have to be suggested for $H_{LS,50}$. The appropriate limit value for the probability of collapse is very difficult to define. In this study, the recommendations suggested by the Joint Committee on Structural Safety (2001) were considered. For the analyzed industrial buildings (with moderate consequences of failure) and seismic action (a large uncertainty of loading) the Committee has suggested a target reliability of 2.5% in 50 years. In case of 3-storey structure with strong connections the requirements are met in all cases. However, in these structures the normalized axial force is relatively small. In 2-storey structures with strong connections the normalized axial force is in general larger and the probabilities of exceeding the limit state are increased. Consequently, some structures with $\nu_d \geq 0.15$ (depending on ratio r) do not meet the above requirements. However, it should be noted that structures with such large normalized axial force carry large vertical load which is not always realistic. Moreover, the probabilities $H_{LS,50}$ were calculated based on conservative estimate of seismic hazard function at large PGA values.

Comparison of the structures with strong and weak connections shows that the results are almost identical for the structures with low normalized axial force ($\nu_d \leq 0.15$). However, when the normalized axial force is increased ($\nu_d > 0.15$), the difference between the structures with strong and weak connections is amplified. This is because the demand on the connections is increased and exceeds the connection capacity. Therefore, the connections fail prior to the columns and the capacity of the structure is reduced. This is demonstrated by the reduction of the PGA capacity of structures with weak connections by up to 50% compared to the structures with strong connections. Accordingly, the frequency of exceeding the limit state is increased: $H_{LS,50}$ comes close to 14% in case of the 3-storey structures and close to 25% in case of 2-storey structures (note: this is because 2-storey structures have larger normalized axial force).
According to these results, beam-column connection with median capacity of 165 kN (this have been experimentally determined value for the connections typically used in the design practice) should only be used for the structures where ν_d does not exceed approximately 0.15. In other cases stronger connections should be used. In general, this results show that beam-column connections cannot dissipate a large amount of the energy introduced by the seismic loading. Soon after the yielding occurs, the failure of the connections follows, resulting in high seismic risk. Therefore, the yielding of such connections should be avoided by using the capacity design rules. If this was the case, the response of all structures would improve and safety would be equal to the structures with the strong connections.

5. CONCLUSIONS

Seismic risk assessment has been performed for all possible variations of regular multi-storey precast buildings, which can occur in the design practice. The peak ground capacity and the probability of failure were assessed. Structures with strong and weak (realistic) connections were analysed. The appropriate limit value for the probability of collapse has been proposed based on the recommendations suggested by the Joint Committee on Structural Safety (2001). It is important to note that only regular buildings were analysed.

The design of multi-storey cantilever columns in precast structures is governed by drift and slenderness limitations. If these limitations in EC8 are followed, the resulting cross-sections are large – in most realistic cases between 60x60 and 80x80. Then, taking into account the minimum longitudinal reinforcement requirement (1%), this typically results in a considerable overstrength. So the peak ground acceleration capacity for structures with strong connections was frequently (for ν_d between 0.1 to 0.15) several times higher than the design ground acceleration. Accordingly, the probability of collapse in 50 years was sufficiently low in comparison with the recommended values. In the analyzed structures value $\nu_d = 0.15$ corresponds approximately to the vertical load of 10 kN/m^2 acting on a tributary area of 100 m^2. Larger loads than this could be considered as rather exceptional. However, for structures carrying such large masses, overstrength is not so pronounced. In these cases, the stiffening of the system by concrete walls/cores or the use of dissipative elements is needed.

Another goal of the research was to study the influence of the realistic (weak) connections on the seismic risk. For structures with lower masses (ν_d from 0.1 to 0.15) the risk did not increase compared to the risk assessed in the case of structures with strong connections (indicated that the strength of standard connections was sufficient). However, in structures with larger masses the connections were damaged and the risk drastically increased. This confirms the conclusion that the capacity design of connections is needed.

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