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Seismic Response of Reinforced-Concrete One-Storey Precast Industrial Buildings with Horizontal Cladding Panels

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Abstract: An extensive parametric study of the seismic response of one-storey precast buildings with horizontal cladding panels frequently used in Central Europe was conducted to analyse the panels' influence on the overall response of buildings and to find out if the panels can be considered non-structural elements when they are attached to the main building with the connections typically used in practice in Central Europe. The studied structural system consisted of reinforced concrete columns and beams connected by dowels. Horizontal cladding panels were attached to columns using one of the most frequently used isostatic fastening systems. The top connections provided out-of-plane stability, and the bottom connections supported the panel in the vertical direction. The parametric study was preceded by extensive experimental research, including cyclic tests on connections and full-scale shaking table tests of whole buildings. The results of experiments were used to reveal the basic response mechanisms of panels and connections and to develop, validate and calibrate numerical models employed in the parametric study presented herein. Fifteen generalised structures with different masses and heights were subjected to 30 accelerograms with two peak ground acceleration (PGA) intensities of 0.3 g and 0.5 g, corresponding to significant damage and near-collapse limit states. The effects of the construction imperfections in connections, the silicon sealant panel-to-panel connections and different types of connections of the bottom panel to the foundation were analysed. The crucial parameter influencing the response was the displacement capacity of the connections, which was considerably affected by the construction imperfections and, consequently, difficult to estimate. It has been observed that in some buildings, particularly in shorter structures with smaller mass, cladding panels can have a somewhat more notable influence on the overall response. However, in general, when the considered types of connections are used, the panels can be considered as non-structural elements, which do not importantly influence the response of the main building. Owing to structural imperfections and relatively short available gaps, the failure of the considered top connections and falling of the panels is very likely in the high seismicity regions. In the most adverse cases, it can occur even in the moderate seismicity regions.

Keywords: RC precast structures; horizontal RC cladding; sensitivity analysis; cladding connections

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1. Introduction

Precast industrial buildings house a large share of European industrial activity. Because of their rapid construction, open space and low cost, they are becoming a more popular structural system throughout Europe. In fact, approximately 50 million square metres of precast buildings are built yearly in Europe [1], demonstrating this structural type's importance.

Reinforced concrete (RC) precast structures have been used for industrial purposes and large shopping centres with tens of thousands of visitors daily. For reference, one of the largest shopping centres in Slovenia has 21 million visitors per year [2]. Damage or collapse of RC precast buildings can cause human casualties as well as considerable direct and indirect economic losses due to production disruption, as observed during earthquakes in northern Italy [3–5].

The estimated economic losses are enormous. Magliulo et al. [4] reported that the direct financial loss after the two Emilia earthquakes amounted to about one billion EUR, while the induced or indirect financial loss due to production interruption amounted to about five billion EUR. Some sources report even higher numbers—according to the CATDAT report [6], the Italian government's final loss estimate for direct economic losses for the earthquakes in the Emilia-Romagna region was above 12 billion EUR.

Because of potential risks to human life and economic prosperity, precast structures should be designed using rigorously validated and reliable procedures. One primary concern during the design of RC precast industrial buildings is whether the concrete cladding panels influence the overall stiffness of these buildings. The answer to this question greatly depends on the type of panel-to-structure connections and panel-to-panel connections. Different cladding connections may provide different levels of interaction between the panels and the main structure.

The general classifications of cladding connections can be found in the literature [7–9]. The integrated solution provides full integration of the cladding panels into the main structural system [10]. The main structure and panels are restrained, and the displacements are coupled between the connecting parts. In such a system, the panels' stiffness has an important influence on the overall response of a precast structure. On the other hand, the isostatic solution isolates panels from the main precast structure, and the effect of the panels' stiffness on the seismic response of the main structural system is small. This type of fastening allows relative displacements between the panels and the main structural system by keeping the panels as the non-structural elements [11,12]. In between the two approaches is the dissipative solution. In those cases, the fastening system of cladding panels or the connections placed between adjacent panels are used as an important source of energy dissipation [11,13,14].

The connections typically used in Central Europe and Italy are considered in this study. They are commonly classified as isostatic connections, which isolate panels from the main structure. Consequently, panel stiffness is assumed to have negligible influence on the seismic response of the main structural system, and the panels are considered non-structural elements. Many of these connections were developed for non-seismic regions without any consideration of their seismic response [1,3,4,15–18], and even in the case of isostatic solution with separated panels and main structure, the panels are connected among themselves by various sealants or other means, creating a much stiffer panel assembly.

After numerous failures of cladding panels in the 2012 Emilia-Romagna earthquakes, extensive studies of the connections commonly used to attach the vertical and horizontal claddings to the main structural system of precast buildings were performed. The University of Ljubljana (UL) conducted experimental and analytical studies of single connections [19,20] for the FP7 Safecladding project [7,8,21]. Further research on connections was performed by Menichini et al. [22].

Since the tests on single components could not fully reveal dynamic effects on the seismic response, the UL team extended the research in the frame of the national research project funded by the Slovenian National Research Agency. The cladding panel influence on precast buildings' overall seismic response was evaluated within an extensive shaking table experimental campaign [20,23–25], which revealed several key properties of the response. Firstly, under seismic loads, the panels moved translationally in their plane and mostly followed the movements of the structure, although sliding occurred in the connections. At high intensities, the sliding was so large that the gaps in the fastening connections completely closed and panels collided with columns. Impacts caused load spikes in the elements, but tests and numerical simulations showed that they had little effect on the dynamic response of the main precast system. The tests also made it possible to develop a robust numerical model of the structure and calibrate all parameters. The validated and calibrated numerical model is presented in detail by Gams et al. [24], and was used in this parametric study. This study also considers vital results obtained within other research campaigns reported in the literature [7,13,15,26] and, in particular, the findings about the panel-to-panel connections [27,28] using silicon sealant.

The presented research is aimed at investigating the influence of cladding panels on the seismic response of one-storey precast industrial buildings and how they should be accounted for in the design. The particular point of interest was whether the horizontal panels attached to the structure using typical Central European fasteners, which were not developed primarily for seismic regions, could be considered non-structural elements. The goal was to identify which circumstances and parameters primarily caused them to fall. Different parameters were analysed, such as the effects of the construction imperfections in connections, the silicon sealant panel-to-panel connections and different types of connections of the bottom panel to the foundation.

Recently, Tornaghi et al. [9] studied the effect of horizontal panels on the response of the structure by pseudo-dynamic tests on a full-scale one-storey precast structure. Their research dealt with similar questions as the research presented herein (e.g., Are cladding panels non-structural elements? How do the connections of the bottom panels to foundations influence the response?). It was concluded that horizontal panels should be considered in the design and could increase the structure's stiffness. However, the connections considered in this study were significantly different from those taken into account herein.

The parametric study, including the description of the studied precast buildings, cladding panels and their connections, the range of the crucial parameters considered and the summary of analysed buildings with their most critical features, are presented in Section 2. Section 3 reviews the numerical models of the main structural system, cladding panels, panels-to-main structure connections and panel-to-panel connections. A detailed evaluation of the used models can be found elsewhere [24,25]. Then, parameters such as the displacement capacity of the connections, construction imperfections in connections, the silicon sealant panel-to-panel connections and different types of connections of the bottom panel to the foundation, which significantly affect the response and their effects, are analysed in Section 4. Finally, Section 5 presents conclusions from the study.

2. Parametric Study

2.1. Typology of the Analysed Buildings

The study analysed the most common RC precast structural system in Central Europe (see Figure 1a), a single-storey building with equally spaced cantilever columns tied with roof girders and beams. The distance between columns ranges from 6 to 12.5 m in the longitudinal direction, reaching up to 30 m in the transverse direction. These buildings are typically single-, two- or multi-bay in the transverse direction and multi-span in the longitudinal direction. The column heights range from 5 to 10 m.

Dowels connect columns and beams. A neoprene pad is present between beams and columns. The roof usually consists of precast RC elements acting as a rigid diaphragm. Considering the type of column-to-beam connections, the main structural system of the analysed buildings consists of an assembly of cantilever columns.

The façade of analysed buildings consists of horizontally placed prefabricated RC cladding panels attached to columns. The length of cladding panels L_{p} spans the entire distance between two adjacent columns. Their width h_p varies between 1.2 and 2.5 m. The thickness of panels with no thermal insulation is usually 15–20 cm. They are thicker when thermal insulation is provided.

The horizontal cladding panels are connected to columns. Figure 1c illustrates the connections typically used in Central Europe. They comprise a pair of bolted top connections that provide cladding panel horizontal out-of-plane stability and a pair of bottom cantilever connections that support the panels' weight.

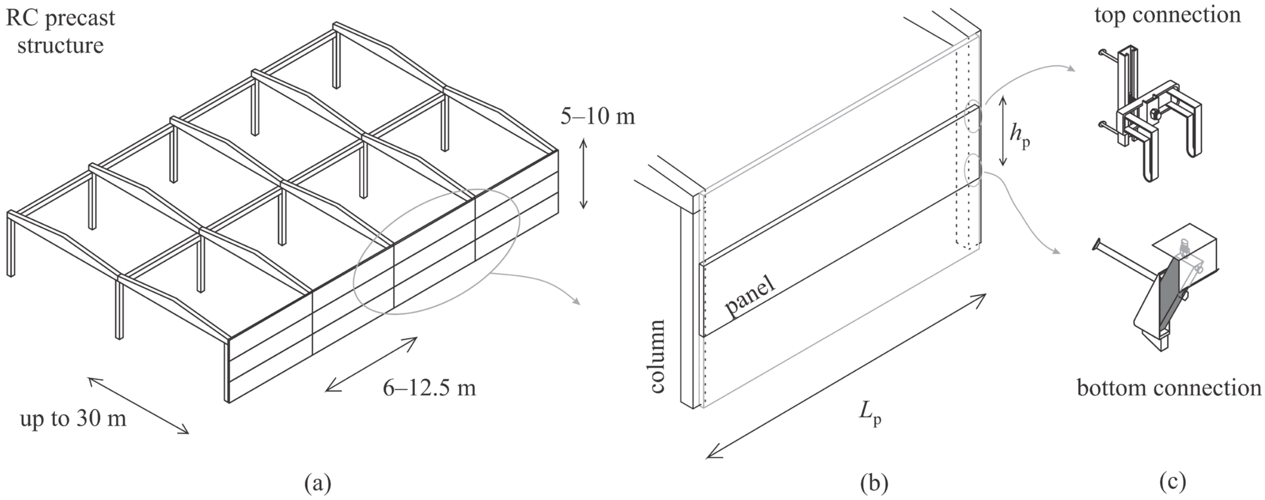

Figure 1. (**a**) Typical RC precast building with (**b**) horizontal panels and (**c**) types of connections considered in the parametric study.

In-plane horizontal sliding of the panel's top is prevented as long as the friction due to the bolt tightening is not exceeded. Once the friction is exhausted, the cladding panel can slide in its in-plane horizontal direction. The gaps of the top connections limit this sliding. Additionally, in-plane horizontal panel sliding can occur at the bottom connection level once the friction due to the panel weight is exceeded. The gaps in the bottom connections limit these movements.

Adjacent panels are typically connected by slots and ribs. The joints between the panels are filled with silicone strips (Figure 2b). The primary role of the sealant is to provide waterproofing and cover irregularities of the gap to improve the building's appearance. The sealant with a width-to-depth ratio of 2:1 is usually placed at both (external and internal) sides of the panels. The minimal silicone width depends on the joint length, ranging from 20 mm to 35 mm for 6 m and 10 m long panels, respectively. Dal Lago et al., and others [7,27,29] performed several experiments on concrete blocks, sub-assemblies and full-scale structures with cladding panels sealed with silicone sealant. Their recommendations were considered when modelling panel-to-panel connections, as described in Section 3.

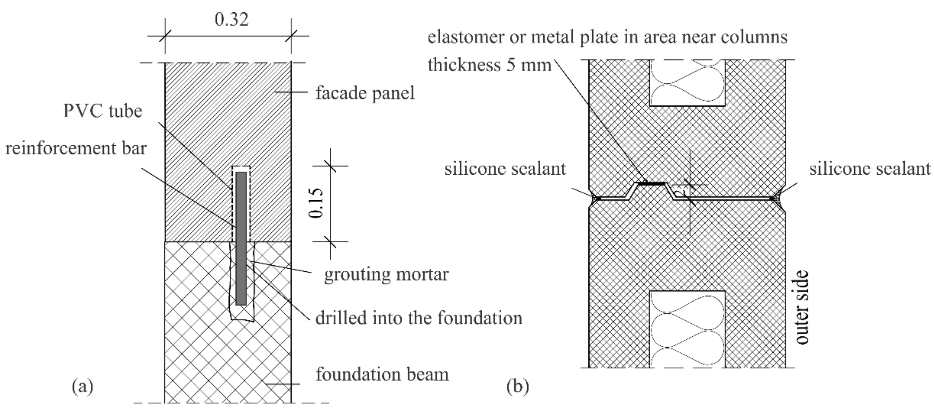

Figure 2. Cladding connections: (**a**) a connection between the cladding panel and the foundation beam and (**b**) a connection between adjacent cladding panels. Dimensions in m.

A large variety of the lowest panel connections with the foundations can be observed in European practice. This parametric study considered two types of these connections. In the first case, the lowest panel was attached to the foundations with steel anchors, hammered into the façade panel and mounted into pre-drilled holes in the foundation beam (Figure 2a). This type of connection, denoted as a fixed connection (F), limits relative movements between the panel, which can increase shear forces at the bottom of the column. In the second case, the bottom panel was slotted and simply laid on the foundation rib (Figure 2b). Consequently, the bottom panel could move over the foundation in its in-plane horizontal direction. This type of connection is denoted as a sliding connection (C).

2.2. Basic Set of the Analysed Buildings

First, a basic set of 15 one-storey RC precast structures was defined, as summarised in Table 1, considering a range of typical one-storey precast buildings found in practice [30,31]. Columns were assumed to be connected by a rigid diaphragm, and the buildings comprised an assembly of equal cantilever columns. Thus, the design was performed considering the typical column and corresponding tributary mass, which ranged from 20 to 100 t in increments of 20 t [31]. Three column heights of 5, 7 and 9 m were evaluated. Each analysed building is denoted based on the tributary mass and typical column height. For example, structure m60H7 consists of 7 m tall columns with a tributary mass of 60 t on each column.

Table 1. Main properties of the basic set of analysed buildings.

Structure	m [t/Column]	H [m]	b [m]	d_{bL} [mm]	s [m]	n_p	h_p [m]	m_p [t]	T [s]
m20H5	20	5	0.4	18	0.14	3	1.67	5.0	0.94
m20H7	20	7	0.4	18	0.14	4	1.75	5.3	1.56
m20H9	20	9	0.5	16	0.12	5	1.80	5.4	1.46
m40H5	40	5	0.5	20	0.16	3	1.67	5.0	0.85
m40H7	40	7	0.5	20	0.16	4	1.75	5.3	1.41
m40H9	40	9	0.6	20	0.12	5	1.80	5.4	1.43
m60H5	60	5	0.5	22	0.16	3	1.67	6.7	1.05
m60H7	60	7	0.6	22	0.12	4	1.75	7.0	1.20
m60H9	60	9	0.6	22	0.12	5	1.80	7.2	1.75
m80H5	80	5	0.6	25	0.12	3	1.67	6.7	0.84
m80H7	80	7	0.6	25	0.12	4	1.75	7.0	1.39
m80H9	80	9	0.7	20	0.16	5	1.80	7.2	1.49
m100H5	100	5	0.6	25	0.12	3	1.67	8.3	0.94
m100H7	100	7	0.6	28	0.12	4	1.75	8.8	1.55
m100H9	100	9	0.7	22	0.16	5	1.80	9.0	1.66

m—tributary mass, H—the height of the columns, b—the width of column cross-section, d_{bL}—longitudinal bar diameter, s—hoop spacing, n_p—number of panels along the column height, h_p—height of one panel, m_p—mass of a panel, T—fundamental vibration period of a structure.

All structures were designed according to standard Eurocode 8 [32] considering the design peak ground acceleration (PGA) of a_g = 0.25 g and ground type C (Zoubek [31]). Since isostatic connections of cladding panels were assumed, the design was performed assuming that cladding panels do not influence the stiffness of the main structural system consisting of an assembly of cantilever columns.

Columns were designed as ductility class medium (DCM) with a behaviour factor q = 3.0. The parameters for concrete class C 40/50 and steel reinforcement B500C were applied. The design resulted in four column types, as presented in Figure 3. Table 1 summarises the dimensions of the column cross-sections and the corresponding reinforcement.

The presented transverse reinforcement corresponds to potential plastic hinge regions. Outside this region, the concrete without shear reinforcement provides adequate shear resistance. Consequently, these parts of columns were reinforced using minimum shear reinforcement. Stirrups Ø8 mm with 24, 26 and 30 cm spacing were used in columns reinforced by longitudinal bars with diameters 20, 22, 25–28 mm, respectively. Calculations of the shear resistance employed the requirements of the EC2 standard [33] and the corresponding Slovenian National Annex [34].

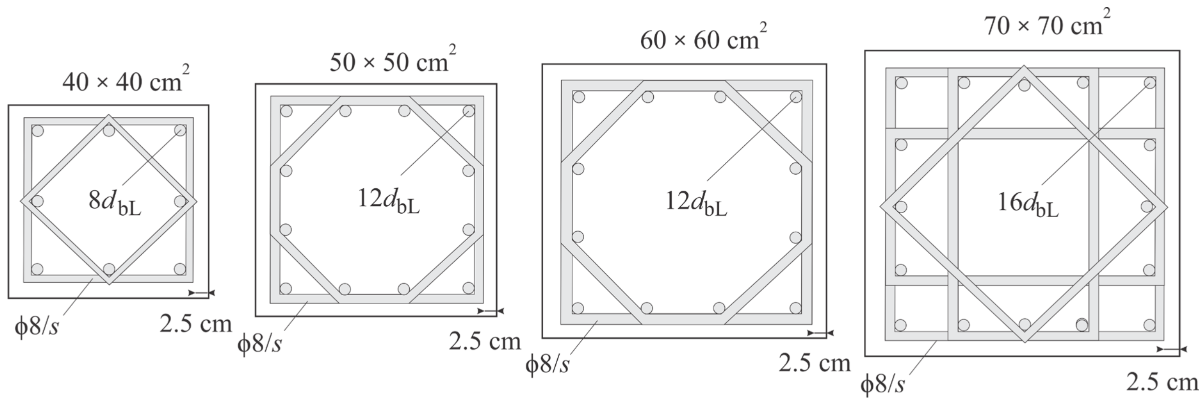

Figure 3. Types of the column cross-sections [31].

Three to five horizontal panels were attached (i.e., n_p = 3–5) to the perimeter columns, depending on their height. In each single building, all panels had the same width h_p. The panel thickness was 16 cm in all buildings.

Since all columns were connected with a rigid diaphragm, the analysis within the parametric study was performed on the model of an equivalent column, similar to the one considered in the design. However, the properties of these two columns differed since the corresponding tributary floor areas were different.

In the design, the tributary floor area of the typical column was defined by equally distributing the total mass/weight of the building to all columns. However, in the parametric study, different tributary floor areas were considered. To address the interaction with cladding panels, the floor area corresponding to one panel/span was taken into account (see Figure 4). Since the length of the panels equalled the span length, the tributary area was defined as the product of the span length in the considered direction and half the total length of the structure in the perpendicular direction. In most cases, this tributary area was larger than the tributary area corresponding to the single column considered in the design. Consequently, the properties of the corresponding equivalent column differed from those of the single column considered in the design.

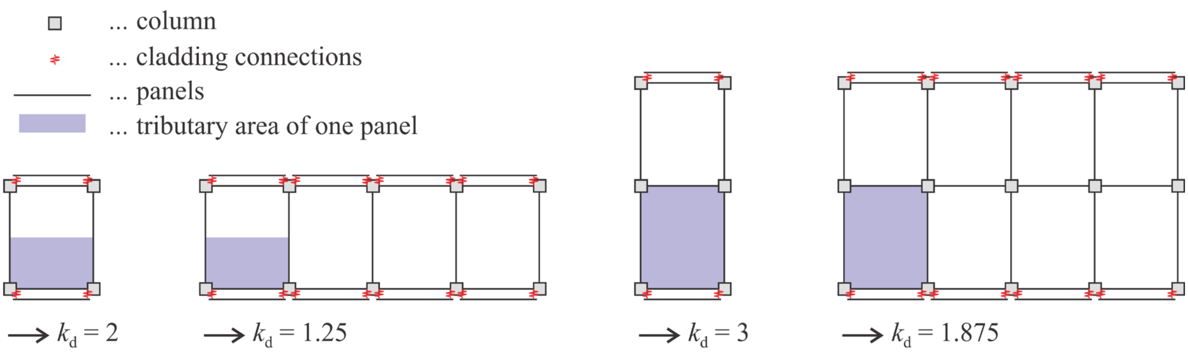

Figure 4. Effective number of columns k_d in the longitudinal direction of different buildings.

To provide the same dynamic properties of the equivalent column and the whole building (e.g., the same fundamental period of vibration), the properties of the equivalent column (e.g., the stiffness, strength and tributary mass) were defined by multiplying the properties of the single column by the effective number of columns, supporting the tributary area corresponding to one panel:

$$k_d = \frac{A_\mathrm{pan}}{A_\mathrm{scol}} = \frac{A_\mathrm{pan}}{\frac{A_\mathrm{tot}}{n_\mathrm{col,tot}}} = \frac{L_\mathrm{d} L_\mathrm{p} \cdot \frac{n_\mathrm{p}}{2}}{n_\mathrm{d} L_\mathrm{d} n_\mathrm{p} L_\mathrm{p}} n_\mathrm{col,tot} = \frac{n_\mathrm{col,tot}}{2 n_\mathrm{d}} \quad (1)$$

where k_d is the effective number of columns in direction d (e.g., x and y are the longitudinal and transverse direction of the building, respectively); A_{pan} and A_{scol} are tributary floor areas corresponding to one panel and one column, respectively; $n_{col,tot}$ is the total number of columns supporting the building; and n_d, n_p and L_d, L_p are the number of spans and span lengths in two orthogonal directions, respectively.

The coefficient k_d is illustrated in Figure 4. For example, in the two-bay, four-span building, the tributary area corresponding to one panel equals one-eighth of the total area, which is supported by 15 columns. Therefore, the effective number of columns is $k_d = 15/8 = 15/(2 \times 4) = 1.875$. Larger k_d values correspond to shorter buildings with more columns. In single- and two-bay buildings, large k_d values are more likely in the transverse direction, while smaller k_d values are more likely in the longitudinal direction, where the number of spans is larger.

To account for a wide range of typical single- and two-bay precast buildings, coefficient k_d ranged between 1 and 10 for all 15 buildings, as presented in Table 1, resulting in 150 structures defined. In all of these cases, the initial position of the panel-to-column connections was assumed centred (i.e., equal gaps in both in-plane panel directions), the bottom panel was considered fixed to the foundations, and the panel-to-panel interaction was assumed to be linked by silicone sealant (for more details, see Section 3).

2.3. Additional Sets of Analysed Buildings

Along with the basic building set, additional structures were included in the parametric study to analyse the influence of the following building properties on the overall seismic response:

(a) Initial position of the panel-to-column connections. The initial position of the panel-to-column connections affects the initial size of the connection gaps in the in-plane panel direction, further influencing the interaction between the columns and panels. Three cases were considered (see Figure 5): (a) the centred (as in the basic set of buildings) top and bottom connections (denoted as MM), (b) top and bottom connections shifted to the most left position (denoted as LL) and (c) top and bottom connections shifted to the most left and right position, respectively (denoted as LR).

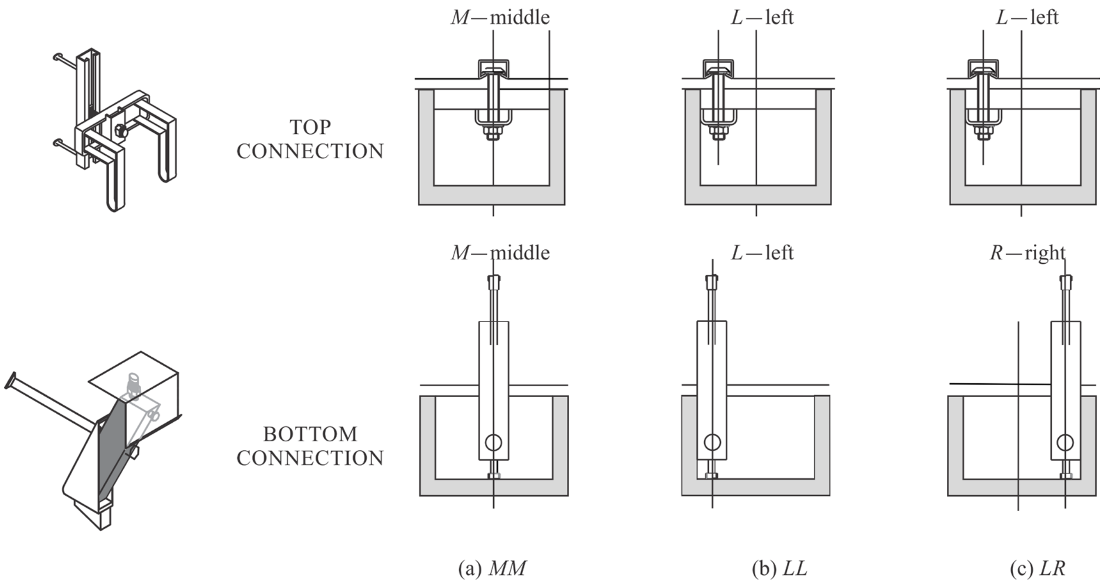

Figure 5. Different positions of the top and bottom connections considered in the study (top view).

(b) Panel-to-panel interaction. As in the basic set of buildings, the panel-to-panel interaction was taken into account considering the silicon sealant (cases denoted as P); in the other case, the panel-to-panel interaction was neglected (case denoted as N)

(c) Type of bottom-to-foundation connections. Two cases were considered regarding the bottom-to-foundation connections: (a) fixed (F) and (b) sliding (C). For an explanation, see Section 2.1.

In all additional sets of buildings, the column-to-panel ratio coefficient k_d was set to 2. All analysed buildings, including the basic set, are summarised in Table 2.

Table 2. Summary of all analysed buildings.

k_d / **Building**	**1–10**	**2**	**2**	**2**	**2**
m20H5	MM-P-F	LL-P-F	LR-P-F	MM-N-F	MM-P-C
m20H7	MM-P-F	LL-P-F	LR-P-F	MM-N-F	MM-P-C
m20H9	MM-P-F	LL-P-F	LR-P-F	MM-N-F	MM-P-C
m40H5	MM-P-F	LL-P-F	LR-P-F	MM-N-F	MM-P-C
m40H7	MM-P-F	LL-P-F	LR-P-F	MM-N-F	MM-P-C
m40H9	MM-P-F	LL-P-F	LR-P-F	MM-N-F	MM-P-C
m60H5	MM-P-F	LL-P-F	LR-P-F	MM-N-F	MM-P-C
m60H7	MM-P-F	LL-P-F	LR-P-F	MM-N-F	MM-P-C
m60H9	MM-P-F	LL-P-F	LR-P-F	MM-N-F	MM-P-C
m80H5	MM-P-F	LL-P-F	LR-P-F	MM-N-F	MM-P-C
m80H7	MM-P-F	LL-P-F	LR-P-F	MM-N-F	MM-P-C
m80H9	MM-P-F	LL-P-F	LR-P-F	MM-N-F	MM-P-C
m100H5	MM-P-F	LL-P-F	LR-P-F	MM-N-F	MM-P-C
m100H7	MM-P-F	LL-P-F	LR-P-F	MM-N-F	MM-P-C
m100H9	MM-P-F	LL-P-F	LR-P-F	MM-N-F	MM-P-C
Number of structures	150	15	15	15	15

2.4. Ground Motion Records

Each building was subjected to 30 accelerograms. Seismic records were selected from the RESORCE database [35] considering a design PGA of 0.3 g (soil type C, return period 475 years) and the EC8 elastic spectrum as a target spectrum (see Figure 6). The ground motions were selected using a slightly modified procedure of Jayaram et al. [36], where the target dispersion was set to zero for all periods. Since the analysed precast structures have different fundamental periods, the T = 0 s was used as a conditional period (i.e., the spectra of ground motions were scaled to PGA in the process of selecting ground motions). Therefore, the dispersion of the spectra of selected ground motions was equal to zero only at period T = 0 s. Additionally, the source to site distance was limited to 5 ÷ 55 km, the magnitude to 4–8.

Nonlinear response history analyses were performed for two intensity levels: PGA of 0.3 g and intensity of 0.5 g, corresponding to the 2475 years return period and near-collapse (NC) damage state defined in Eurocode 1998-3 [37].

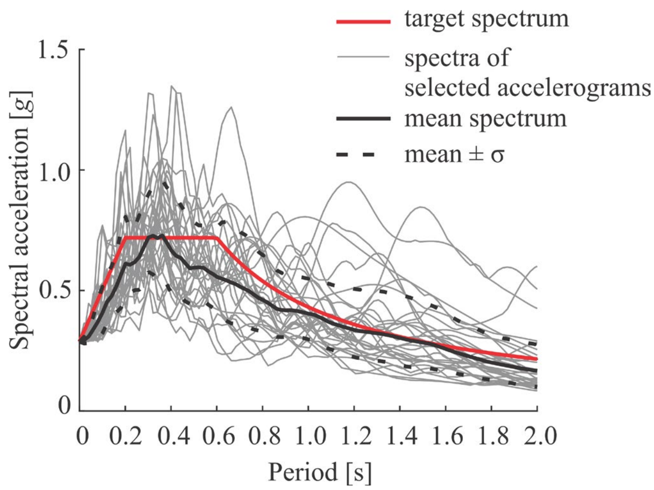

Figure 6. Acceleration spectra of selected accelerograms and the target elastic acceleration Eurocode 8 spectrum for the ground type C (PGA = 0.3 *g*).

3. Numerical Models

The numerical models of all elements considered in the parametric study (main structure, panels, panel-to-column connections and panel-to-panel connections) are overviewed. The simplified model, presented in the following subsections, was verified via a comparison with the full 3-D model of two typical buildings. This verification and additional data about the numerical models of different components can be found in Section 6.2.4 of Starešinič's open-access Ph.D. dissertation [25].

3.1. Main Structure-Equivalent Column

Since the analysed structures consist of equal cantilever columns connected with a rigid diaphragm, the main structural system of buildings was modelled using a single equivalent column (see Figure 7). The properties of this column were defined as described in Section 2.2, multiplying the stiffness, strength and tributary mass of single column by the effective number of columns k_d (see Equation (1) and Section 2.2). The column was modelled using nonlinear force-based beam column elements with five integration points, as defined in OpenSees [38]. The number of these elements was equal to the number of panels attached to them (see Figure 7).

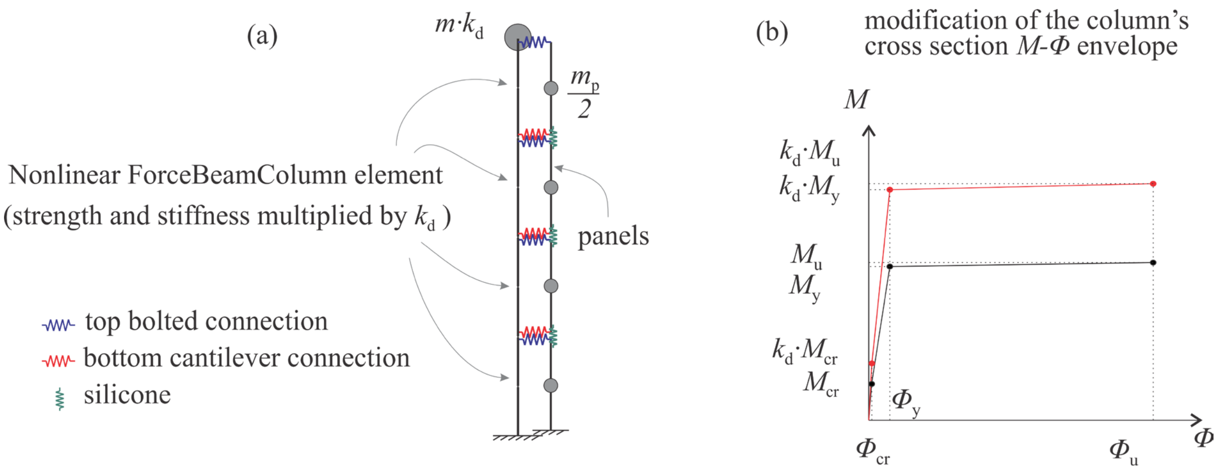

Figure 7. Numerical model of the equivalent column with attached horizontal cladding panels: (**a**) equivalent column, cladding panels and their connections and (**b**) the moment–curvature relationship of the single column modified by the effective number of columns k_d (M_{cr}—cracking moment, M_y—yield moment, M_u—ultimate moment).

Their nonlinear response was defined by the appropriate moment–curvature envelope corresponding to specific cross-sections of the single column (see Figure 3) modified by the effective number of columns k_d (see Figure 7b). Table 3 presents the characteristic features of moment–curvature envelopes for all considered cross-sections. They were defined considering the mean compressive strength of concrete f_{cm} = 48 MPa and mean yield strength of steel f_{ym} = 575 MPa.

Table 3. Basic features of moment–curvature envelopes of the cross-sections shown in Figure 3.

Column	Φ_{cr} [10^{-3}/m]	Φ_y [10^{-3}/m]	Φ_u [10^{-3}/m]	M_{cr} [kNm]	M_y [kNm]	M_u [kNm]	V_{Rd} [kN]	M_u/H [kN]
m20H5	1.7	13.1	257	64	223	244	170	49
m20H7	1.7	13.1	257	64	223	244	170	35
m20H9	1.3	9.7	190	116	338	378	224	42
m40H5	1.5	10.1	172	133	539	553	253	111
m40H7	1.5	10.1	172	133	539	553	253	79
m40H9	1.1	8.0	157	212	662	742	301	82
m60H5	1.7	10.5	152	149	658	664	282	133
m60H7	1.2	8.4	165	232	836	885	331	126
m60H9	1.2	8.4	165	232	836	885	331	98
m80H5	1.3	8.3	167	251	1036	1167	362	233
m80H7	1.3	8.6	146	251	1071	1085	362	155
m80H9	1.1	8.0	137	366	1165	1177	419	131
m100H5	1.4	8.3	168	271	1077	1210	389	242
m100H7	1.4	8.7	132	271	1305	1317	404	188
m100H9	1.1	8.0	117	389	1378	1399	451	155

Φ_{cr}—cracking curvature, Φ_y—yield curvature, Φ_u—ultimate curvature, M_{cr}—cracking moment, M_y—yield moment, M_u—ultimate moment, V_R—shear resistance.

The hysteretic response was defined following the Takeda [39] hysteretic rules. Figure 8 illustrates the typical hysteretic response. For each column, the design shear resistance V_{Rd} outside the potential plastic hinges was also calculated considering the contributions of concrete and minimum shear reinforcement (see Table 3). In all cases, V_{Rd} was significantly larger than the maximum shear force, calculated by dividing the ultimate bending moment by column height ($V = M_u/H$) in single columns in buildings without cladding panels.

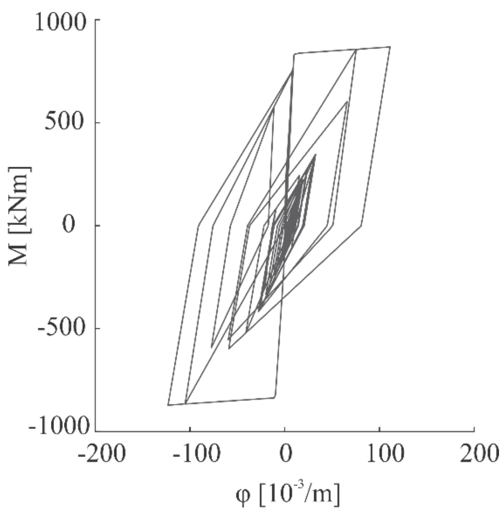

Figure 8. Hysteretic moment–curvature response of a cross-section in structure m60H7.

3.2. Cladding Panels

Cladding panels having length equal to the entire span were attached to the effective column (one panel at each elevation—see Figure 7a). They were modelled using the elastic elements. The mass of each panel was applied to the middle of the panel.

3.3. Panel-to-Column Connections

Because one column supports two halves of the panels in the adjacent spans, two bolted top connections and two bottom cantilever connections per panel were considered (see Figure 7a). Figures 9 and 10 present the numerical models of the top and bottom connections, respectively. In both cases, an initial strength owing to the friction is provided (R_{fr}). When it is exceeded, sliding between the panel and column occurs. Then, after the sliding displacements exceed the gap available at the connection, a collision between the panel and column occurs, considerably increasing the demand and the stiffness of the connection.

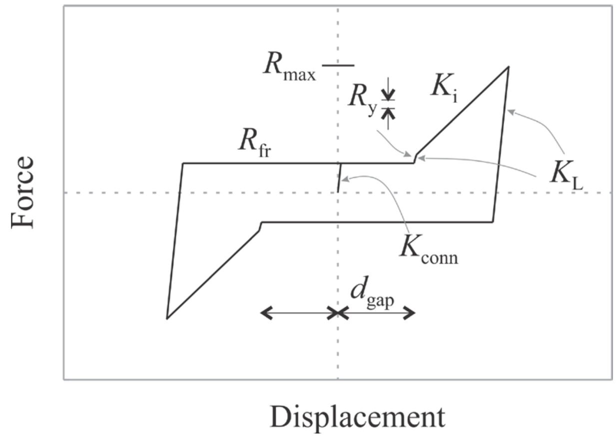

Figure 9. Numerical model of top connections.

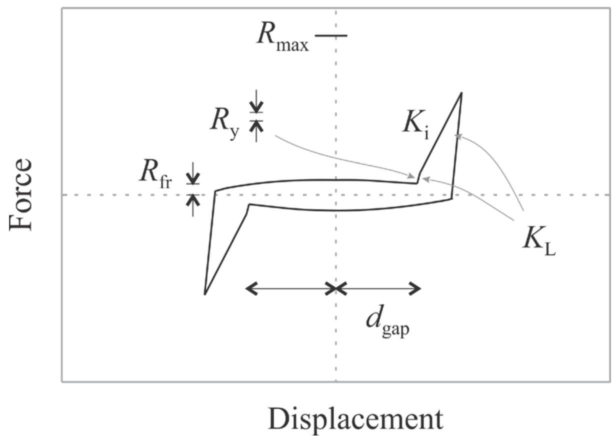

Figure 10. Numerical model of bottom connections.

Table 4 lists the basic parameters considered in the numerical models of the top and bottom connections. These values were carefully evaluated using the results of extensive cyclic and dynamic tests of single components and shaking table tests of whole buildings [20,24]. A detailed analysis and evaluation of the connection numerical models can be found in the literature [25].

Table 4. Basic parameters of the numerical models for the top and bottom connections.

Connection	R_{fr} [kN]	R_{max} [kN]	R_y [kN]	K_{conn} [kN/m]	K_i [kN/m]	K_L [kN/m]	d_{gap} [cm] *
Top	15.0	58.0	0.01	2×10^4	1.5×10^3	1.0×10^4	±4.0
Bottom	2.00	176	0.01	2×10^3	1.5×10^4	1.0×10^4	±4.5

* Corresponds to centred connections (MM). R_{fr}—static friction, R_{max}—capacity of the connection, R_y—auxiliary, K_{conn}—initial stiffness of the connection, K_i—stiffness after contact with column, K_L—unloading stiffness.

3.4. Panel-to-Panel Connections

In most of the analysed buildings, panels were connected to each other with silicon sealant. Figure 11 shows the model of this connection, defined based on the experiments performed by Dal Lago et al. [27]. Using the results of these experiments, the numerical model was proposed by Menichini and Isaković [12,28,40]. Pinching4 material, as defined in OpenSees [38], was used to describe the nonlinear response of silicone sealant. All properties of the material used for the modelling are summarised in Figure 11.

Table 5. Pinching model 4 parameters for modelling silicone sealant.

Parameter Value	*ePf1*, *eNf1* $0.25 \cdot l_s \cdot b_s$	*ePf2*, *eNf2* $0.21 \cdot l_s \cdot b_s$	*ePf3*, *eNf3* $0.224 \cdot l_s \cdot b_s$	*ePf4*, *eNf4* 0.0	
Parameter Value	*ePd1*, *eNd1* $0.146 \cdot t_s \cdot 10^{-3}$	*ePd2*, *eNd2* $1.6 \cdot t_s \cdot 10^{-3}$	*ePd3*, *eNd3* $2.35 \cdot t_s \cdot 10^{-3}$	*ePd4*, *eNd4* $3.0 \cdot t_s \cdot 10^{-3}$	
Parameter Value	*gK1* 0.0	*gK2* 0.0	*gK3* 0.3	*gK4* 0.2	*gKLim* 100
Parameter Value	*gD1* 0.15	*gD2* 0.0	*gD3* 1.0	*gD4* 1.0	*gDLim* 100
Parameter Value	*gF1* 0.0	*gF2* 4.0	*gF3* 1.0	*gF4* 1.0	*gFLim* 100
Parameter Value	*rDispP*, *rDispN* 0.3	*rForce P*, *rForceN* 0.04	*uForceP*, *uForceN* 0.15	*gE* 43	*dmgType* "energy"
Parameter Value	t_s 15 mm	b_s 15 mm	l_s 4.66 m		

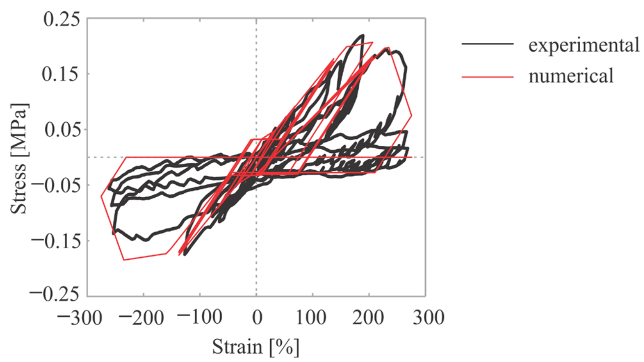

Figure 11. Numerical model of the silicone sealant and comparison with the experiment (parameters in Table 5).

4. Response of the Analysed Buildings

4.1. General Observations about the Response, Capacity and Demand of the Connection

In this subsection, the basic features of the response observed within the parametric study are summarised. In general, the analysis was in good agreement with the full-scale shaking table surveillances of the whole buildings [20,23].

At low seismic excitations, panels were pinned at the top and slid at the bottom (the friction force of bottom connections was exceeded at very low excitation levels). They moved translationally in their in-plane directions (see Figure 12). When the demand was increased, sliding also occurred at the top connections. Of note, the top and bottom relative displacements between columns and panels ($d_{\text{panel,top}}$ and $d_{\text{panel,bot}}$) were in opposite directions. Such displacements were observed in the shaking table experiments and later confirmed by the analysis. When gaps in the connections were depleted, the stiffness of connections considerably increased, owing to the impact between the panel and the column, which also increased the connection force demand.

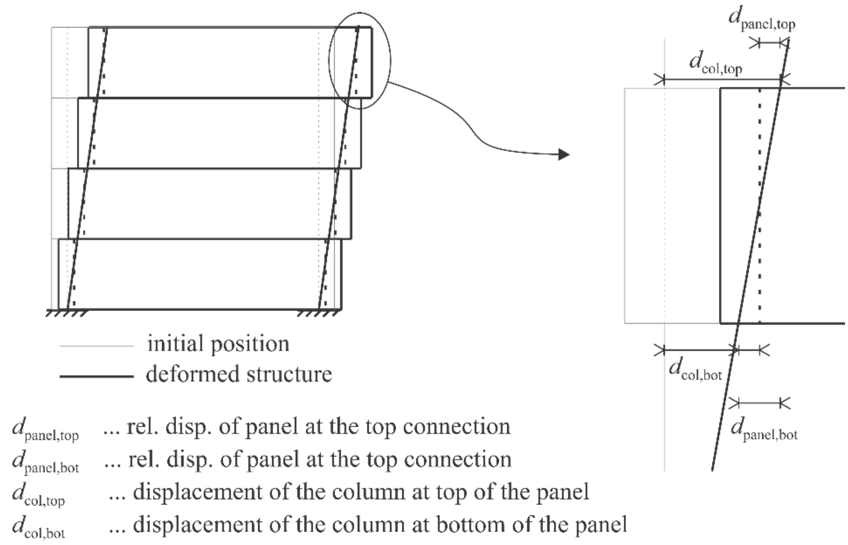

Figure 12. Displacement response of panels and columns, in which relative displacements between panels and columns at the top and the bottom of the panels ($d_{\text{panel,top}}$ and $d_{\text{panel,bot}}$) are in the opposite direction.

In general, the panels tended to move such that the impact with the column occurred simultaneously at the top and the bottom connections. When the impact was strong enough, the failure of the weaker top connections occurred, causing the panel to fall.

The top connections failed when their displacement capacity was exceeded. This capacity is defined as the size of the gap plus the maximum possible displacement of the connection after the impact with the column, which is about 3 cm ($d_{\max}$ = $(F_{\max} - F_{\text{tr}})/K_{\text{i}}$ = (58 − 15) kN/15·kN/cm = 2.9 cm; see Figure 9 and Table 4). Since the size of the gap is 4 cm, the top displacements can resist ±7 cm of displacement relative to the columns.

Such displacement capacity of the top connections is available only when the connections are centred in their initial position (MM), meaning that the fastening bolt is placed directly in the middle of the steel cantilever. In practice, the initial position of the bolt may significantly be off-centre owing to various construction imperfections, which can significantly affect the displacement capacity of the top connections and cause the panels to fall sooner.

The effects of the initial position of the connections are analysed in Section 4.3, which is preceded by the analysis of the response of various building configurations and the influence of the panels on the overall response (Section 4.2). The effects of the silicone sealant between panels and the effects of different types of connection of the bottom panels to the foundations are presented in Section 4.4 and Section 4.5, respectively.

4.2. Response of the Basic Set of Buildings and the Effects of the Panels on the Overall Response

In this section, the response of the basic set of buildings, described in Section 2.2, is analysed for two ground acceleration intensities of 0.3 *g* and 0.5 *g*, corresponding to design level excitations and NC limit state.

In Figures 13 and 14, the median values and the standard deviations of the maximum displacements defined in the top and bottom connections corresponding to the excitation intensity of 0.3 *g* and 0.5 *g* are presented, respectively.

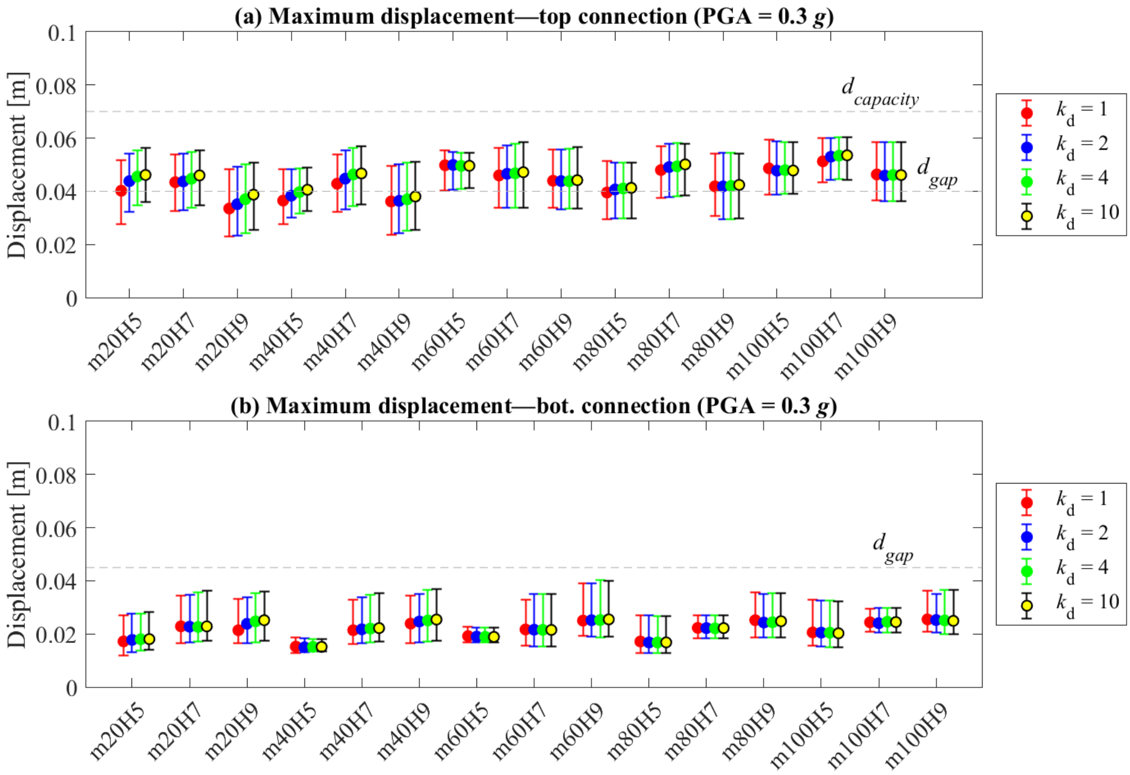

Figure 13. Maximum displacements of (**a**) top and (**b**) bottom connections at PGA of 0.3 *g*. Legend: d_{gap}—the size of the gap, d_{capacity}—displacement capacity of the connection.

When subjected to the design level earthquakes (PGA = 0.3 *g*), the impacts between panels and columns are observed only at the top connections. However, these impacts are limited in number and intensity (compare the displacements with the size of the gap d_{gap}). The displacement and force demand are well below the maximum displacement capacity and strength. No failure of the top connections occurs in any building, nor is there any impact between the panels and columns at the bottom connections. On average, their relative displacements are well within the available gap in the bottom connections.

At the NC level excitation (PGA = 0.5 *g*), impacts occur more often, and the forces in the top connections are higher than those in the design level. However, they are significantly less than the capacity (see Figure 15a), and no top connections fail. Impacts between the panels and columns also occur in the bottom connections. The intensity of the force demand is, on average, less than in top connections, but the upward scatter is larger (see Figure 15b).

The panels do not have a notable influence on the global response of the whole building, particularly at the design level earthquakes. Figure 16 shows this result, where the median values of the roof (top) displacements in different buildings (considering different k_{d}) are compared with the roof displacements of the buildings without cladding panels.

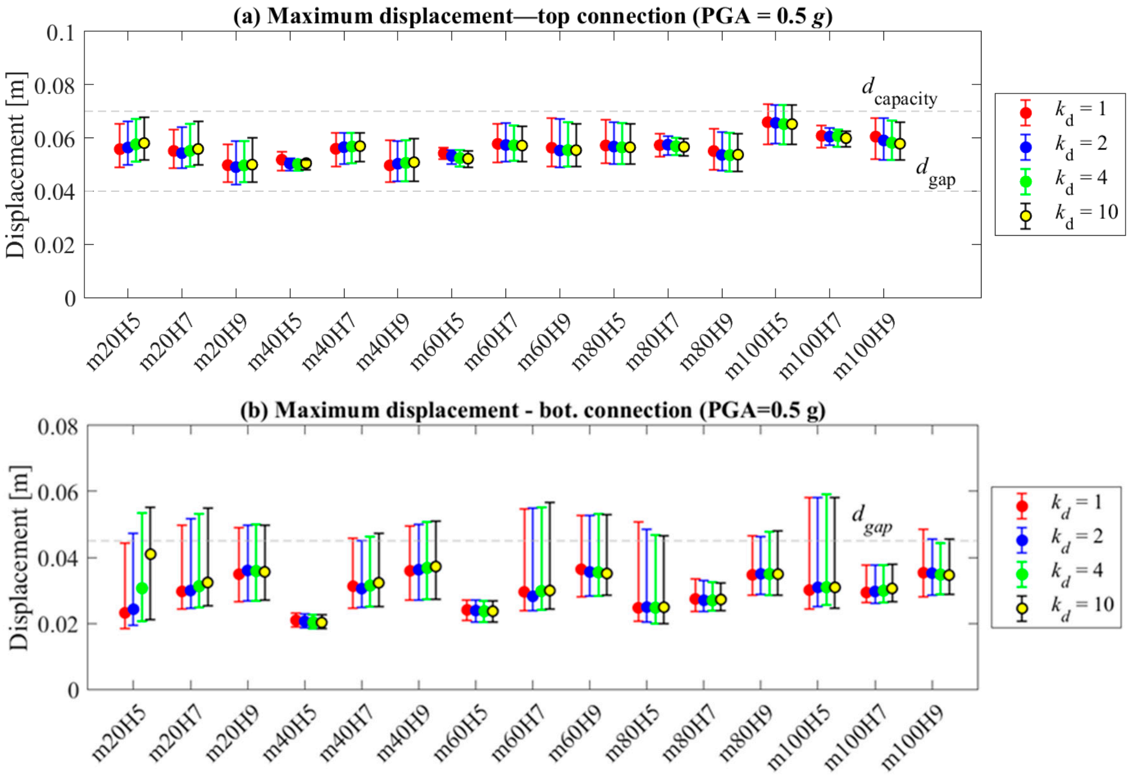

Figure 14. Maximum displacements of (**a**) top and (**b**) bottom connections at PGA = 0.5 *g*. Legend: d_{gap}—the size of the gap, $d_{capacity}$—displacement capacity of the connection.

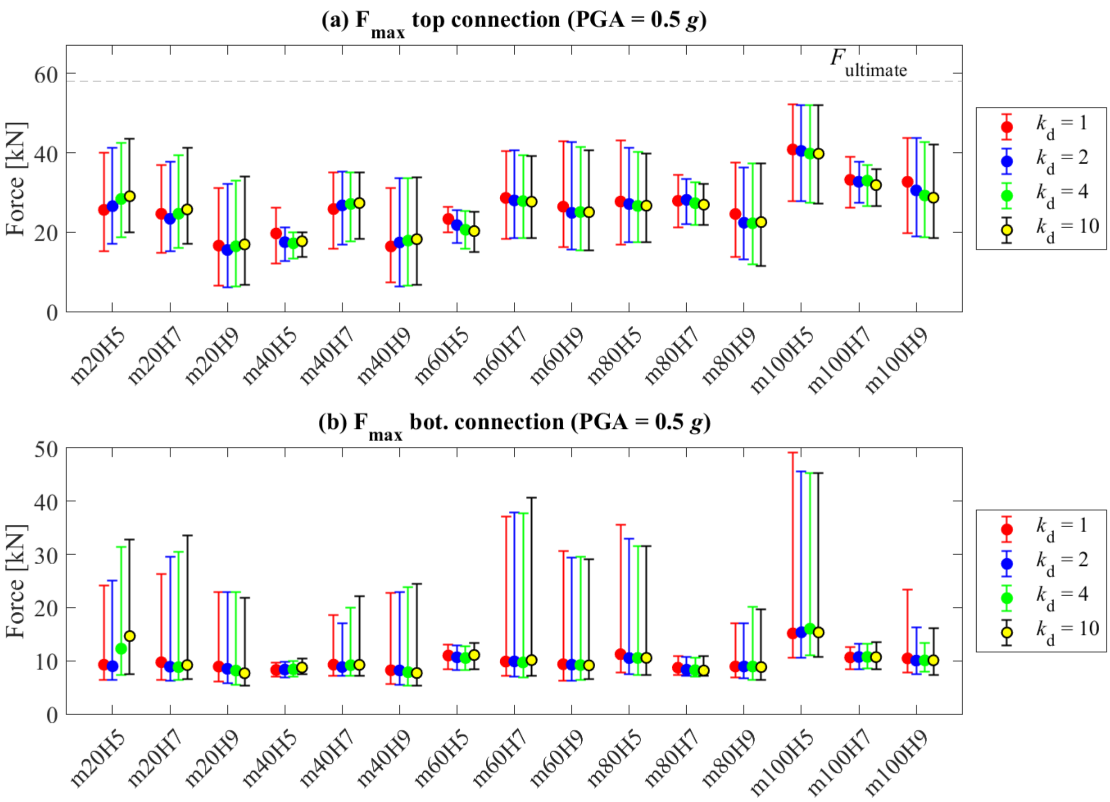

Figure 15. Maximum forces in (**a**) top and (**b**) bottom connections at PGA = 0.5 *g*. Legend: $F_{ultimate}$—strength of the connection.

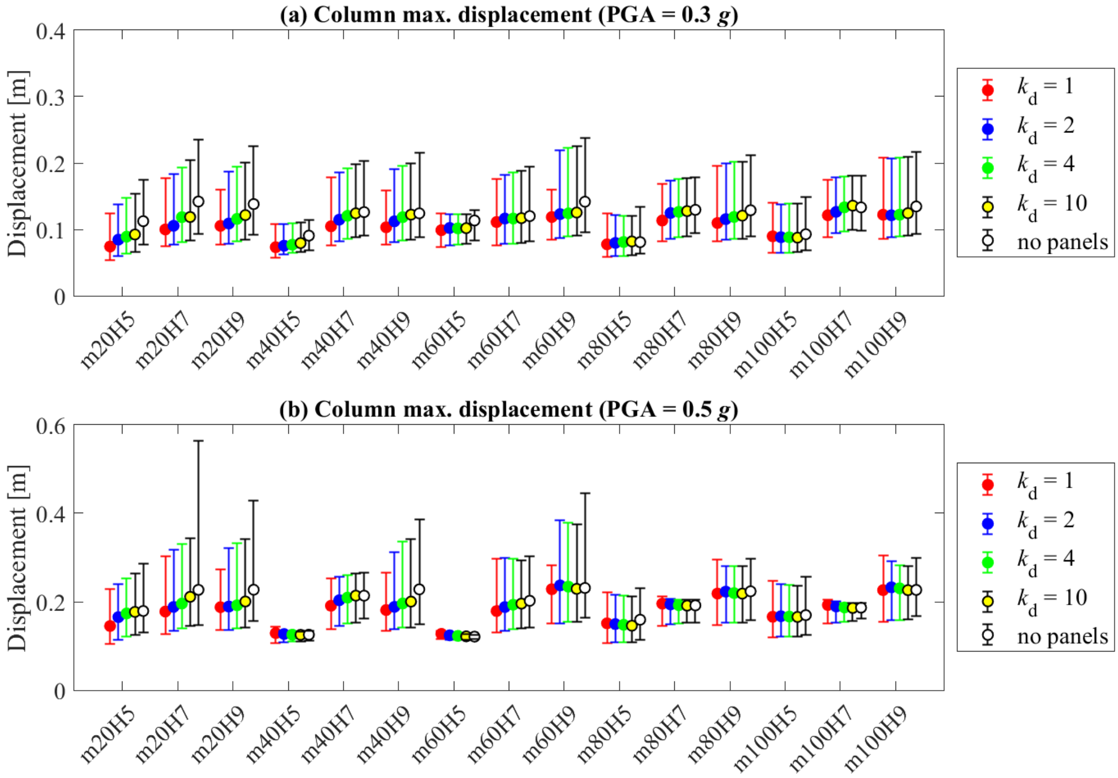

Figure 16. Top displacements of buildings for (**a**) PGA = 0.3 g and (**b**) PGA = 0.5 g.

Because the impacts between the panels and columns are limited for the design level earthquakes in most buildings of the same height and mass, the top displacements differ negligibly. A slightly larger difference (up to 33%) can be observed only in buildings with very low mass (e.g., with short spans, such as m20H5, m20H7 and m20H9).

At PGA = 0.5 g, the influence of the panels on the overall response increases because of the more intense impacts with the columns. However, this influence can still be neglected. As noted above, only in buildings with small mass can the panels influence the stiffness of the whole building. This is more notable in structures with a relatively large number of panels compared to the number of columns (smaller k_d).

Figure 17 presents the median values of maximum shear forces in columns compared with the shear forces corresponding to the columns' flexural capacity M_u/H, which are the maximum shear forces that can occur in columns in buildings without cladding panels.

At PGA = 0.5 g, shear forces in some columns are somewhat larger than the maximum expected forces in buildings without panels. This increase is the consequence of fixed foundations of the bottom panels and silicone sealant between panels. Their effects are discussed in Sections 4.4 and 4.5. Despite the increase, the shear forces in the columns are still well within the columns' shear capacity.

Based on the presented results, the cladding panels in most of the cases do not affect the response of the main structural system of buildings considerably. A more notable influence can be observed only in buildings with smaller masses and smaller k_d (shorter buildings with a larger number of panels compared to the number of columns). As a result, studies of construction imperfections, silicon sealant and types of foundations of bottom panels consider only k_d value of 2.

4.3. Initial Position of Connections and Construction Imperfections

In the previously presented analysis, all connections are centred (MM) in the initial position, and the gaps on both sides are equal in size. In practice, this is an unlikely scenario, particularly considering that the gaps are designed primarily to compensate for different construction imperfections. Because the gap size influences the interaction between panels and columns, additional analyses are performed to evaluate the effects of different initial positions of the connections to the response.

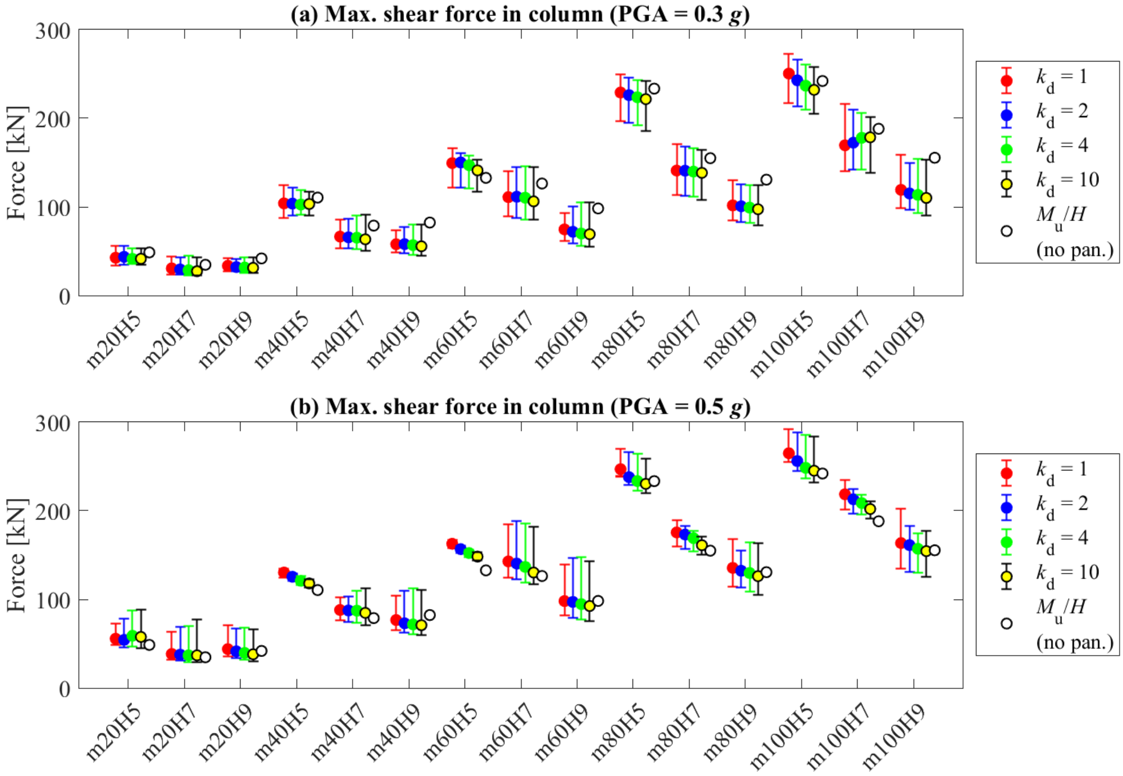

Figure 17. Maximum shear forces in columns for (**a**) PGA = 0.3 g and (**b**) PGA = 0.5 g.

In practice, various combinations of initial positions of top and bottom connections are possible. Herein, three extreme cases are analysed, as presented in Figure 5 in Section 2.3. In addition to the centred connections (denoted as MM), two eccentric cases are considered: (a) LL, where top and bottom connections are shifted in the same direction at the end of the available gap, and (b) LR, where they are shifted to the end of the available gaps but in the opposite direction.

The initial position of the connections influences the position and number of impacts between panels and columns. When they are placed eccentrically in the most extreme positions, the impact in one direction can occur immediately (depending on the accelerogram). In contrast, in the opposite direction, the size of the gap is doubled, and the impact is less probable. Figure 18 illustrates the influence of the analysed connection positions using the example of structure m60H9. Although the impacts, to a great extent, depend on the properties of certain accelerograms, the effects of imperfections are the most severe when the connections shift in opposite directions. In such cases, impacts will occur mostly along the column height but only in one direction. The impacts are less likely in the opposite direction since the available gap is doubled.

Figure 19 shows that the LR position is the most adverse case, where the median values of maximum forces in the connections are presented. No connection failures are observed at the design level excitations, regardless of the connection type. However, the forces corresponding to the LR position are considerably larger than in the other two cases. In some connections, the forces more than double.

When subjected to NC-intensity earthquakes, failures of the top connections are common for the LR case (see Figure 20), while in the other two cases, the maximum force demand in the top connections is still below the maximum allowed $F_{\max}$.

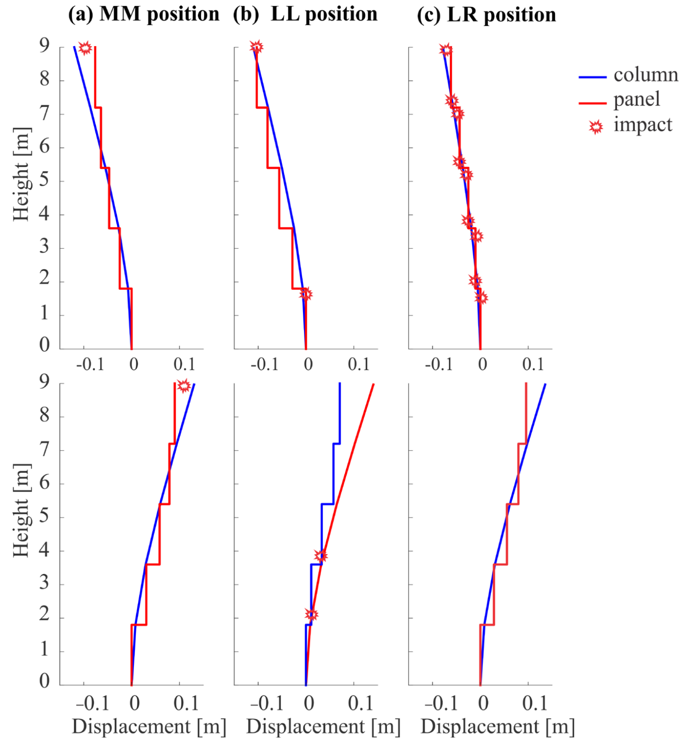

Figure 18. Impacts between panels and columns in the two opposite directions, considering different initial positions of connections: (**a**) MM—centred, (**b**) LL—top and bottom connections shifted in an extreme position in the same direction and (**c**) LR—top and bottom connections shifted in an extreme position in the opposite directions.

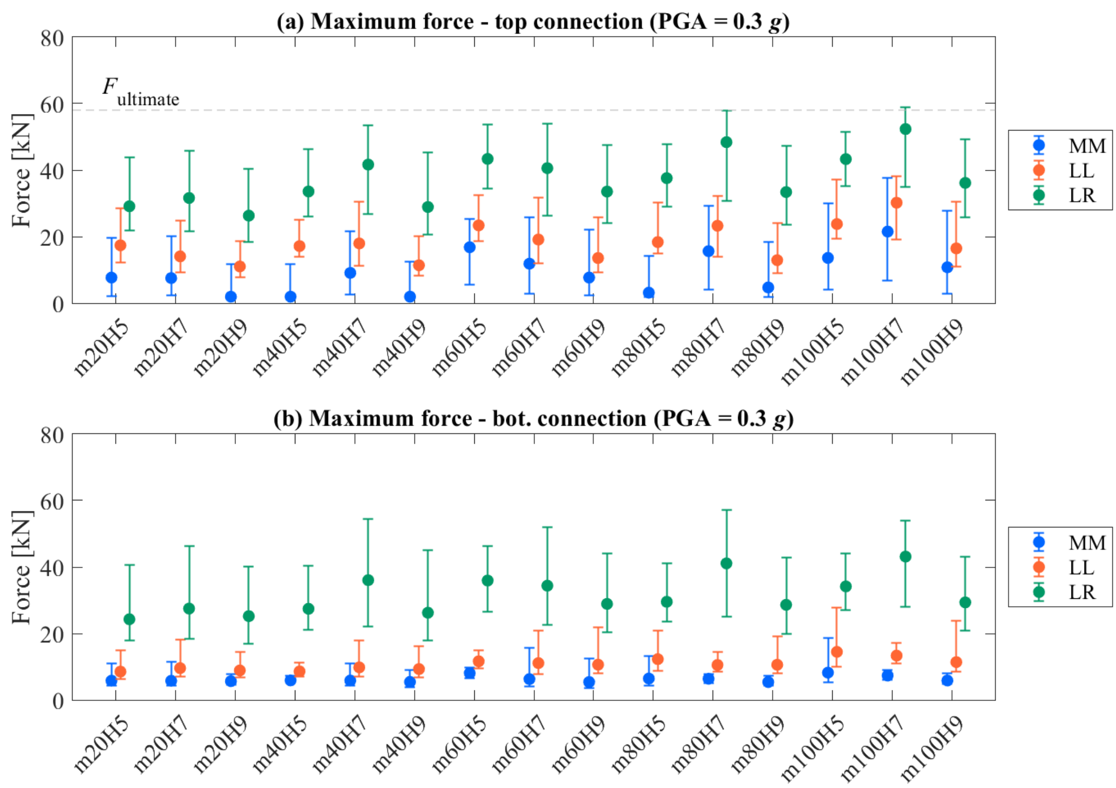

Figure 19. Maximum forces in (**a**) top and (**b**) bottom connections at different initial positions of connections at PGA = 0.3 g. Legend: F_{ultimate}—strength of the connection.

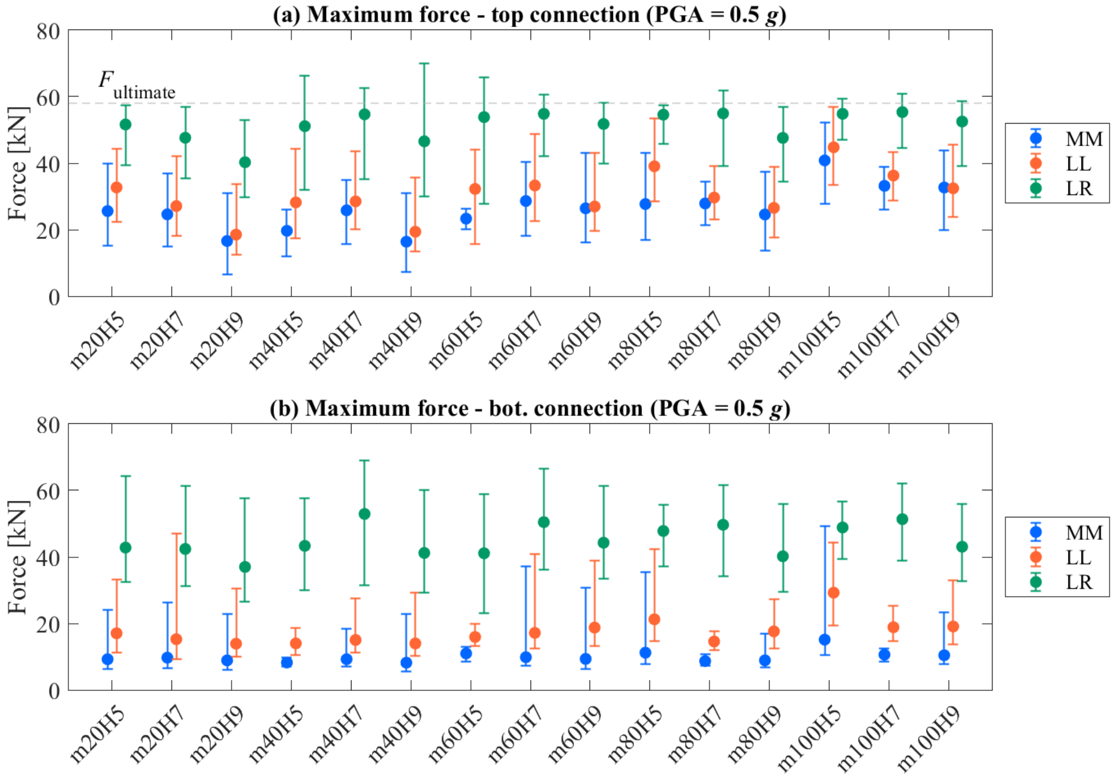

Figure 20. Maximum forces in (**a**) top and (**b**) bottom connections at different initial positions of the connections at PGA = 0.5 *g*. Legend: F_{ultimate}—strength of the connection.

The initial positions of connections primarily affect the stability of panels, which fall when the top connections fail. The influence on the global response of the building is less pronounced (see Figures 21–23). Almost no influence on the response is evident at the design level earthquakes. Again, only slightly more noticeable effects can be observed in buildings with small masses. At the NC seismic excitation level, the top displacements of some buildings more notably increase in the LR case, but the differences compared to the MM and LL are smaller than the scattering of maximum displacements at certain acceleration excitations.

4.4. Influence of Interaction between Panels

The effect of the interaction between panels linked by silicone sealant is illustrated in Figure 24 for m60H9. The relatively strong connection created by the assembly of cladding panels increases their total stiffness and the difference from the stiffness of the columns. Consequently, the relative displacements between columns and panels increase, particularly at the top of the column (see Figure 24).

As concluded in the previous sections, the interaction between the panels does not considerably influence the response of columns. The columns' shear forces can somewhat increase, particularly when the mass and k_d are small. Despite the limited effects, the panel interaction can affect the distribution of shear forces along the column, as illustrated in Figures 25 and 26 for m60H5, m60H7 and m60H9. Since the relative displacements between the panels and columns particularly increased at the top of the columns, the shear forces increased, and maximum shear forces occurred at this part of the column.

4.5. Influence of the Connection of Bottom Panels to the Foundation

The difference in the bottom panel's response in the fixed (F) case compared to just lying on the foundations (C) is presented in Figure 27. When the bottom panel lies on the foundation, the response of the bottom panel is similar to the other panels. Relative displacements with respect to the column can be accommodated at the top and bottom of the panel. These displacements are in opposite directions, just like in the other panels (Figure 27a).

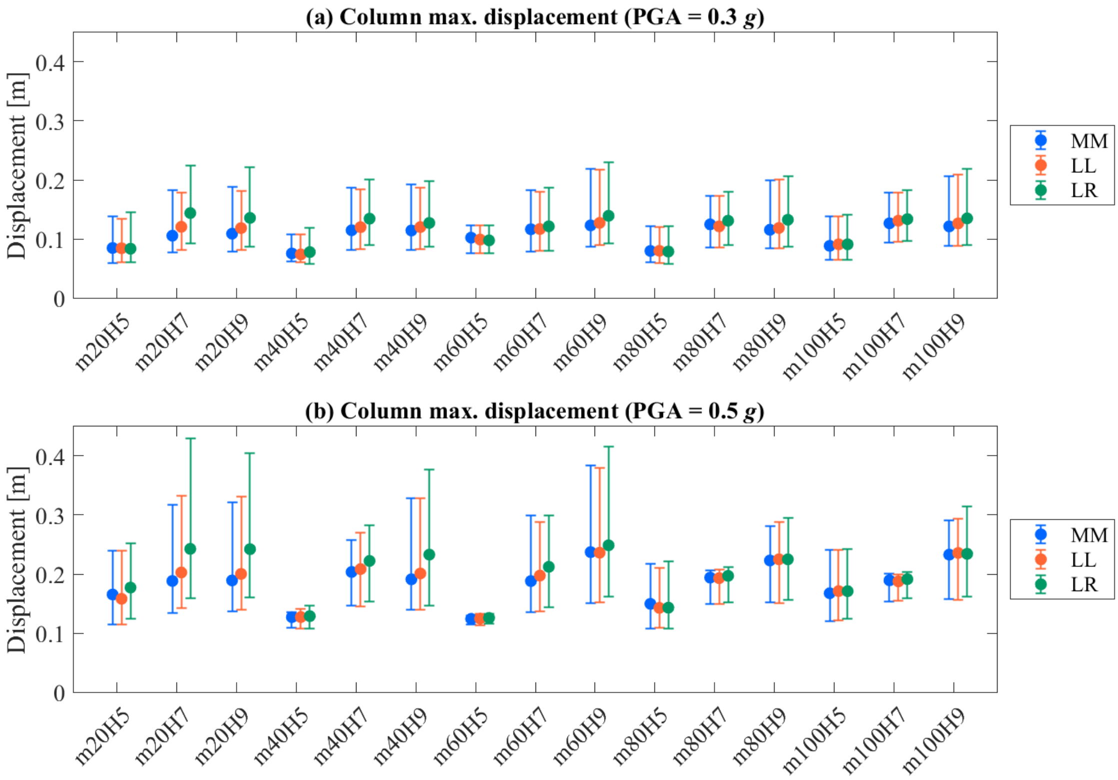

Figure 21. Maximum top displacements of buildings at (**a**) PGA = 0.3 *g* and (**b**) PGA = 0.5 *g*.

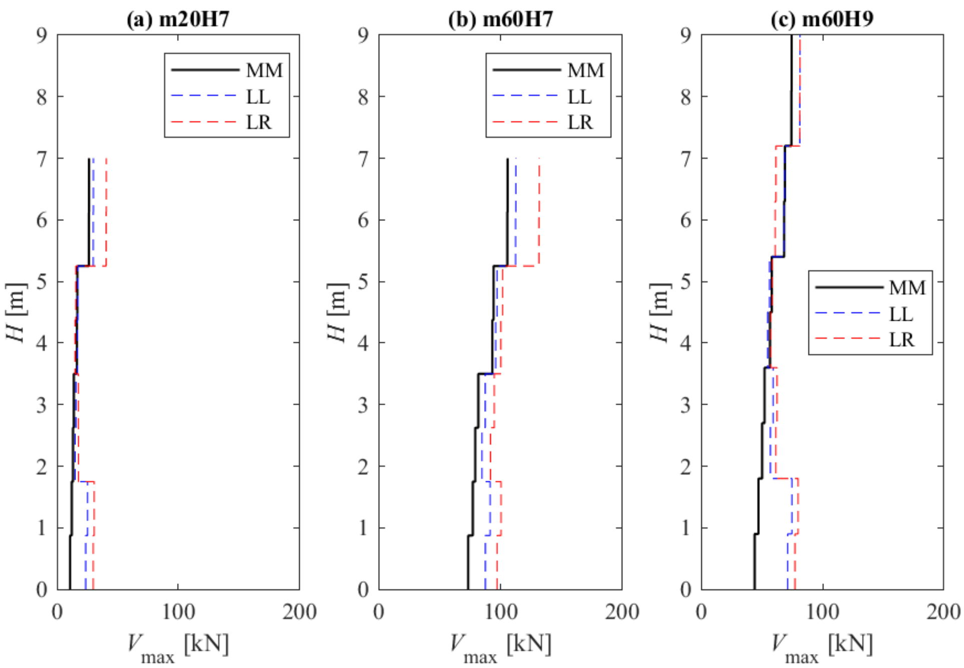

Figure 22. Max. shear forces in columns at PGA = 0.3 *g*.

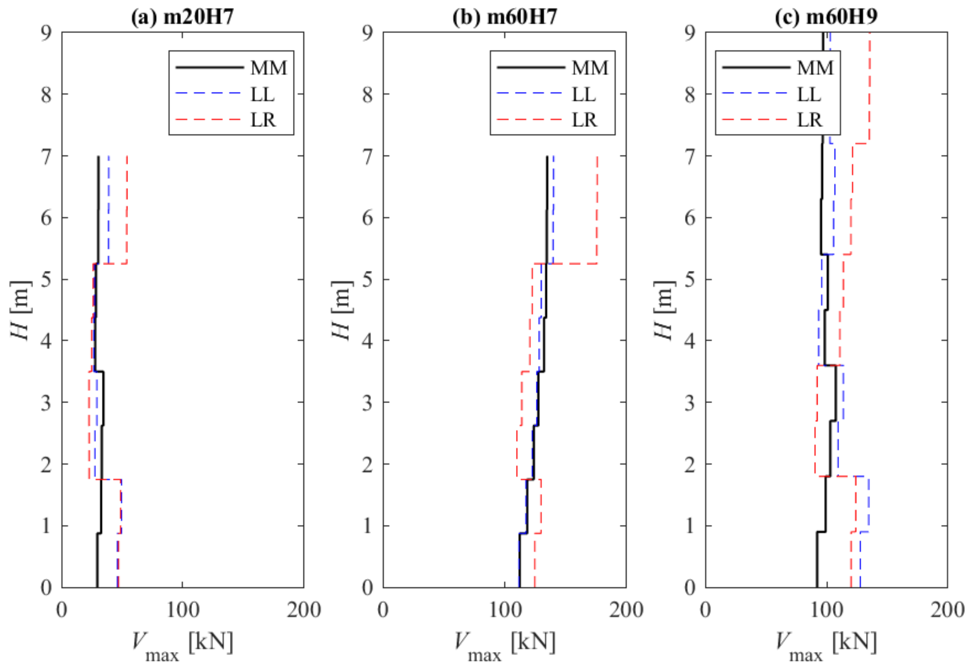

Figure 23. Max. shear forces in columns at PGA = 0.5 *g*.

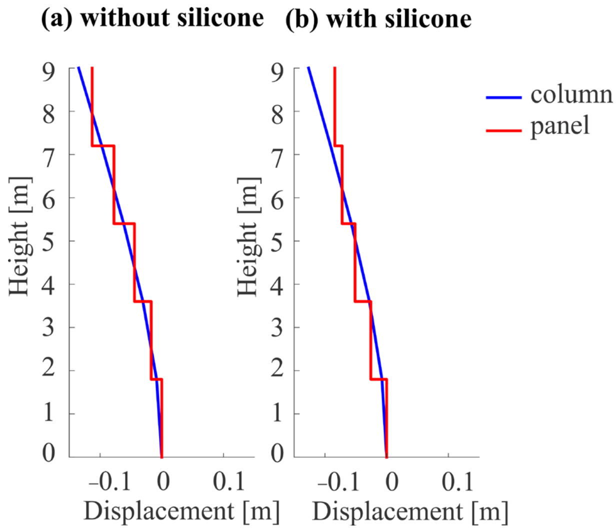

Figure 24. Response of precast structure m60H9—(**a**) without silicone sealed joints and (**b**) with silicone sealant.

When the bottom panel is fixed to the foundation, relative displacements occur only at the top of the bottom panel (Figure 27b). Consequently, the demand in the top connections is considerably increased, leading to an earlier failure of the fastening system. Although such failure does not necessarily cause a collapse of the panel (owing to the panel being fixed at the bottom), repair can be difficult. All upper panels must be removed to repair the bottom panel.

Because the top connections of the bottom panel fail earlier when the panel is fixed to the foundation, the shear forces in columns are not considerably affected (see Figure 28). The maximum values are only slightly larger than those of the C-type foundations of the bottom panels.

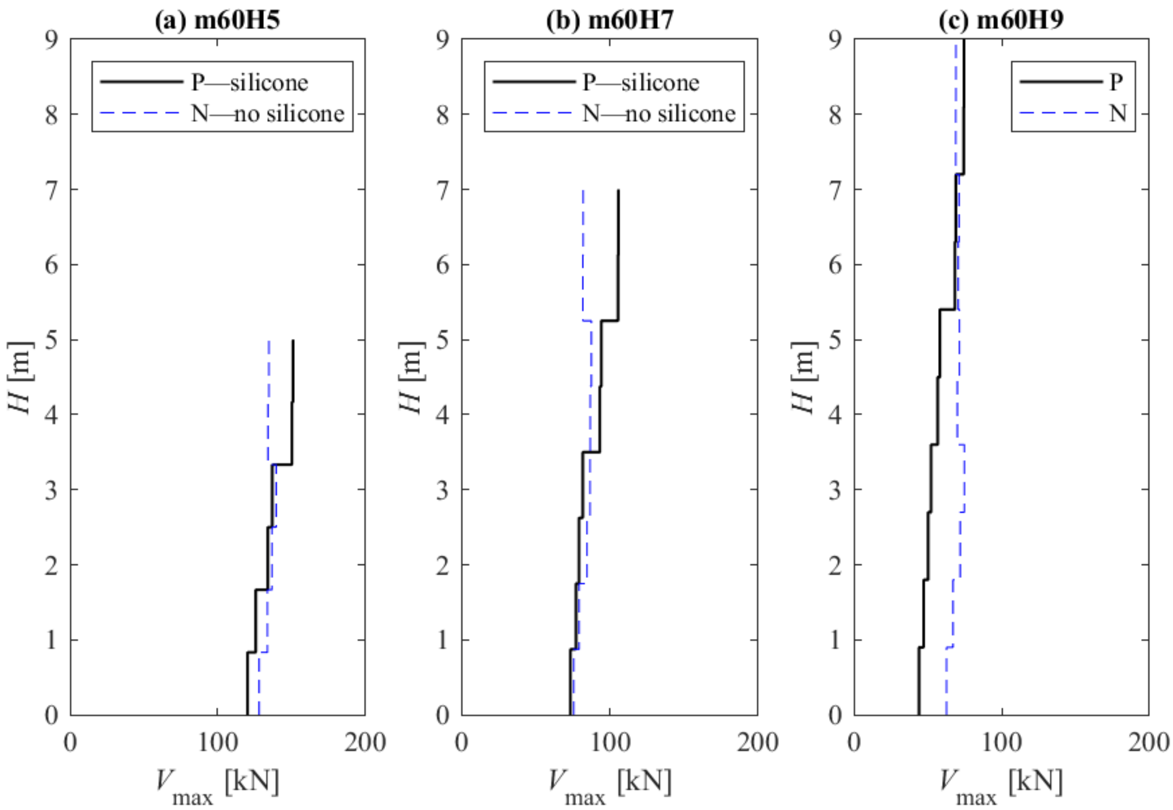

Figure 25. Max. shear forces in columns at PGA = 0.3 *g*.

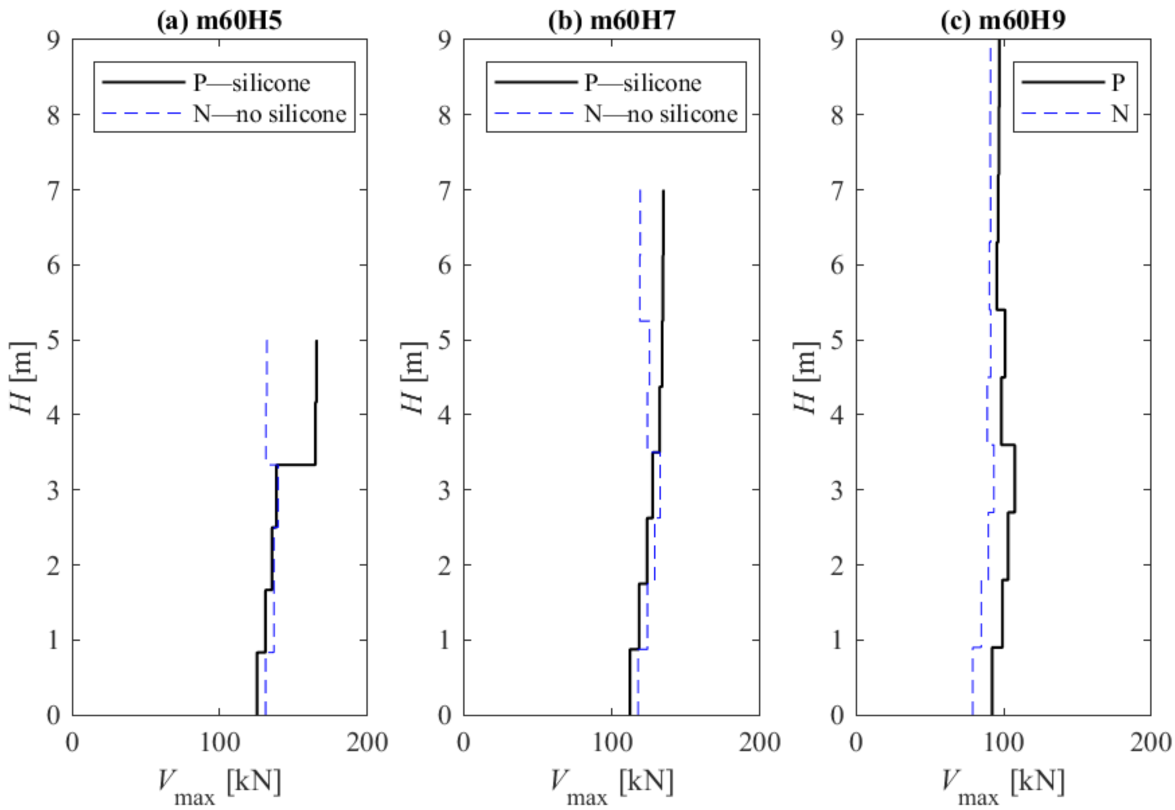

Figure 26. Max. shear forces in columns at PGA = 0.5 *g*.

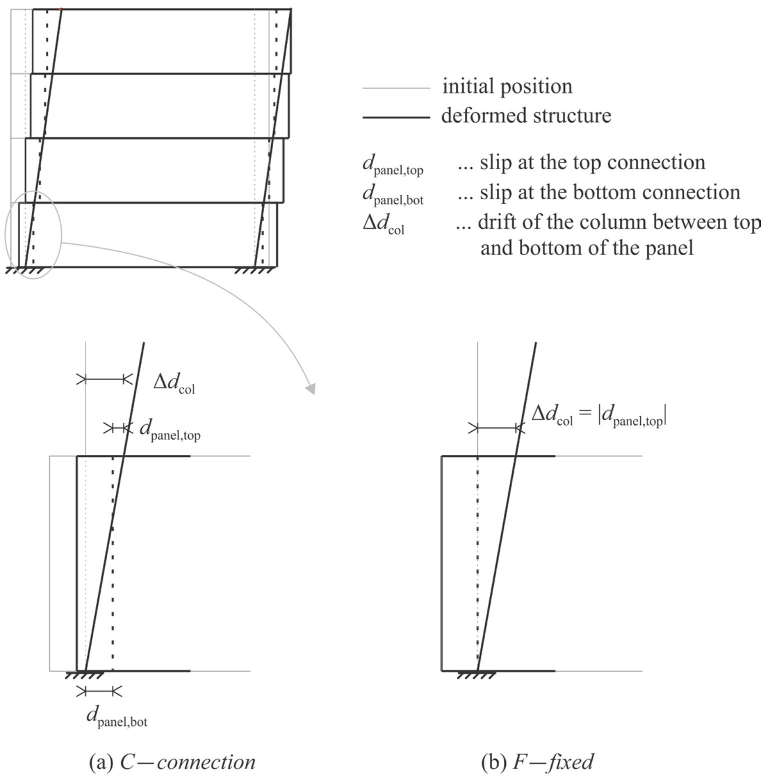

Figure 27. Response of the structure with horizontal cladding panels: (**a**) bottom panel connected to the column with cantilever connection and sealed with silicone to the foundation and (**b**) bottom panel fixed to the foundation.

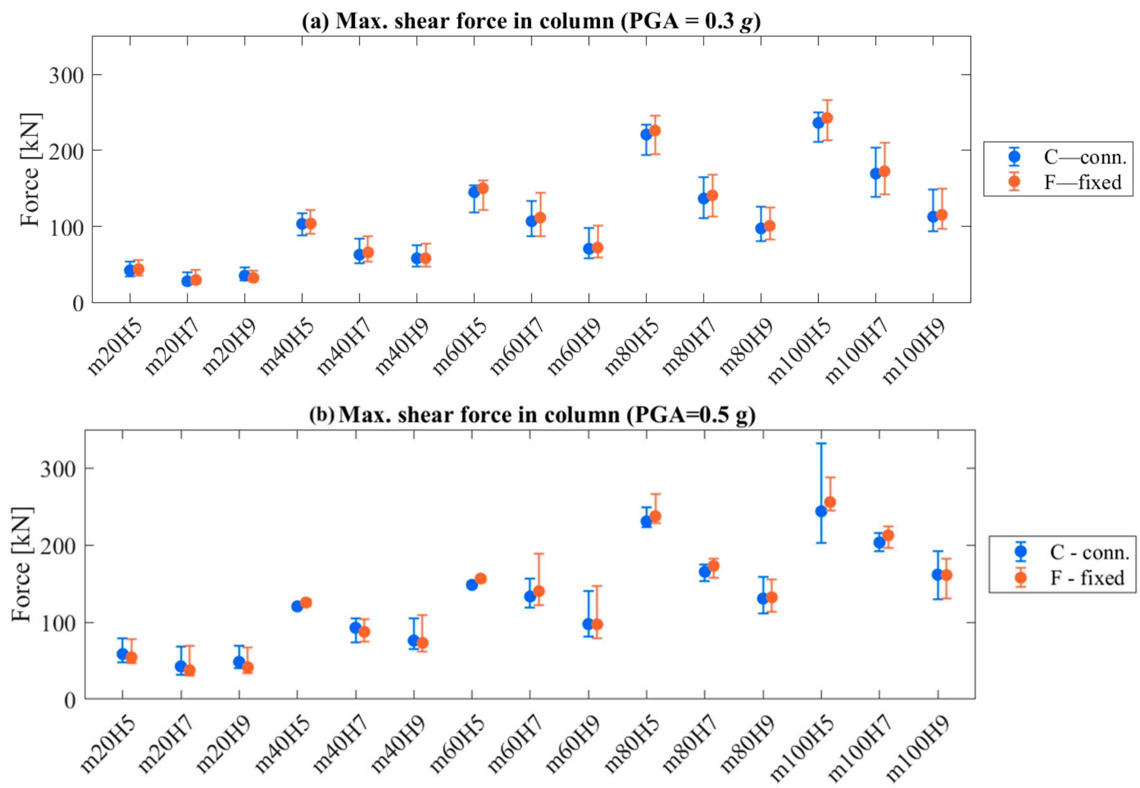

Figure 28. Max. shear forces in columns at (**a**) PGA = 0.3 g and (**b**) PGA = 0.5 g in the case of C- and F- type foundations of the bottom panel.

5. Conclusions

A parametric study of typical one-storey precast RC buildings with horizontal panels has been performed, focusing on the response of panels and their influence on the overall seismic response of structures. The main structural system resisting the seismic actions consisted of an assembly of cantilever columns. They were designed according to standard Eurocode 8, considering the design PGA of 0.3 g. The panels were attached to the main building with connections typically used in Central Europe and were not explicitly designed considering seismic action. They allow relative displacements between the main building and panels so long as their gaps are not closed.

The study has shown the impacts between columns and panels can be expected since the available gaps in the connection were too small. It has been observed that the critical parts of the fastening system were the top connections. When they failed, the panels fell. The possibility of their failure primarily depended on the initial positions of the top and bottom connections. When both top and bottom connections were ideally centred, impacts between the panels and columns occurred only at stronger seismic excitations (PGA = 0.5 g). The possibility of failure of the connections was limited.

However, an ideal connection casting is difficult to achieve, particularly considering that the gaps in connections are designed primarily to compensate for construction imperfections. When the initial position of connections was off-centre, the possibility of failure was much higher since the available gap at one side was smaller, and premature impact with the column was more likely to occur. The most adverse case was when the initial positions of the top and bottom connections were in opposite extreme positions. In such situations, the failure of the top connections can occur even when subjected to design seismic action (PGA = 0.3 g), and failure is almost inevitable during NC earthquakes (PGA = 0.5 g).

It has been found that horizontal panels and their connections have limited influence on the overall seismic response of the building. The effects were slightly more notable only in shorter-span structures (smaller mass) with more panels, where a more noticeable increase in the maximum shear demand in columns can occur. However, the shear demand was still well within the available shear capacity of columns.

The effects of the silicon sealant between panels and the type of the bottom foundations on the seismic response of the connections, panels and the overall response of the primary structural system were studied. The silicone sealant increased the relative displacements between panels and the top parts of columns. Consequently, the shear forces were redistributed along the column, increasing at the top parts of the columns. However, this was not critical in any case since the shear capacity was far above the demand.

The bottom panels fixed to the foundations did not significantly increase the maximum shear forces in the columns. However, the demand for top connections was considerably increased, leading to an earlier failure of the fastening system. Although such failure does not necessarily cause a collapse of the panels (owing to the panels being fixed at the bottom), repair can be complicated.

The analyses have confirmed that precast structures with horizontal claddings can be designed considering panels as non-structural elements in the case of the analysed type of connections. It has been found that the risk of failure of these connections is considerable, particularly in the high seismicity regions. Their failure might also occur in the moderate seismicity regions in the most adverse cases. The presented conclusions are related to the specific considered type of connections. However, it is believed that they can also be extended to similar isostatic connections, which can accommodate certain relative displacement between panels and the main structural system and have relatively small gaps.

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