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Research Paper

The Influence of the Connection Properties of Prefabricated Large-Panel Walls on the Static and Dynamic Behavior of a Building

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The aim of this article is to present a method of assessing the load-bearing capacity of prefabricated large-panel buildings built after the 1950s. Since the main problem in the existing large-panel system buildings is the actual condition of the joints between the panels, it was decided to investigate their impact on the behavior of the entire structure. Therefore, analyses were carried out in which the influence of the adopted connection models on the static and dynamic behavior of the building was examined. Several calculation models of the building were analyzed using three different types of connections between wall and ceiling panels: rigid, elastic and hinged, each representing a distinct state of the actual connection. The criterion used for the comparisons were the values of extreme internal forces and static displacements as well as the dynamic responses of the structure. The parameters that have the greatest influence on the static and dynamic analyses' results are described and commented on in the conclusion. The analyses' results are supported by data obtained from source materials: inspection reports, case studies and technical documents. The research helps to understand better the static and dynamic behavior of a building erected using a system of prefabricated large-size panels and to determine the main parameters of the structure influencing this behavior. However, as a result of the research, it was found that changes in the state of panel connection in large-panel system buildings do not have such a significant impact on the overall static and dynamic responses of the structure as initially expected.

Keywords: large panel system (LPS); prefabricated buildings; precast connections; seismic analysis.

1. Introduction

The load-bearing system of residential buildings can consist of several different structures, such as masonry walls, frame systems (columns and beams), shear walls, floor slabs, etc. A special construction system that uses prefabricated reinforced concrete panels to create the main load-bearing structure of the building is the large panel system (LPS). According to [1], the LPS was first developed in France by Raymond Camus and Eduard Coignet in the 1950s. From France, the system was exported to other European countries. This type of construction was widely implemented in the post-war period in Europe for the purpose of rebuilding the continent after the devastating World War II. Now, such structures can be found not only in Europe but also on other continents. According to [2], the first examples of this type of construction appeared in the early 1950s, but their large-scale use began in the 1960s. Nowadays, there are many such buildings around densely populated European cities, especially in Eastern Europe. As stated in [3], some of these buildings have reached the end of their service life and local authorities are running special renovation programs for these structures. If their renovation is infeasible, they are destined for demolition. Although seismic activity is low in most European countries, these buildings also exist in other parts of the world where ground excitation is an important factor in the design of building structures. In [4], the author describes the typical structural problems encountered in LPS buildings located in Poland and explains the methods used to retrofit and repair joints to improve structural safety.

Buildings erected in LPS technology are structures consisting of prefabricated panels with specific dimensions, thickness and strength characteristics. On the other hand, individual geometric features, such as openings (e.g., windows, doors, etc.) and reinforcement distribution, are determined in the prefabrication plant. After the prefabricated elements have been transported to the construction site, the panels are assembled vertically and horizontally. Their structural arrangement, and thus the static scheme of the panels, can be designed in various ways, e.g., as one-way or two-way systems.

The joint of large prefabricated panels in a building is defined as the area where at least two wall members and/or floor slabs are in contact with each other. Those connections can be divided into horizontal and vertical joints. Vertical connections occur within each story, where the vertical edges of successive wall elements meet. Horizontal connections are constructed between the wall panels on successive stories along their horizontal edges, where they usually intersect with the floor slabs.

The connections can be constructed in several different ways, e.g., using in-situ reinforced concrete, infill grouting, by means of mechanical connectors, welding of reinforcement ties or plates, or using reinforcement loops.

With the advancement of technology in recent decades, many different solutions [5, 6] have been invented that allow large panels to be joined so that the connection has the same stiffness as assumed in the design of the computational model. In contrast, when mass production of LPS buildings began to increase in the early 1950s, the quality of the connection between the panels was a rather poorly recognized parameter, the importance of which was underestimated. There are several factors that may influence the behavior of these vertical and horizontal connections, such as the quality of the filling material, the accuracy of the assembling of the rebars/fasteners, the accuracy of the formwork before grouting, the straightness of bars and/or walls themselves, etc. The structural designer can ignore all these inaccuracies or incorporate them into the design. One of the goals of this study is to evaluate these two design approaches and determine whether consideration of joint accuracy has a significant impact on the overall structural behavior of a building. Taking into account the variable behavior of the connections caused by the above-mentioned factors, this comparative study proposes four various connection models. It is worth mentioning that within the same building, the condition and quality of connections may vary, which may require further detailed research and analysis. In the first case, it is assumed that all performance criteria are met and there are no inaccuracies, due to which the stiffness of the connection is high, close to the stiffness of monolithic walls made at the construction site. For this case, the first calculation model (RG model) is proposed. A different approach is for the case where not all performance criteria are met, so that inaccuracies will be included in the definition of panel connections. Three models are proposed that differ in the definition of rotational stiffness along the edges of the panels: vertical released rotational edges (vertical hinges – VH model), horizontal released rotational edges (horizontal hinges – HH model) and limited, defined elastic stiffness (EE model) along all edges. The comparison of the RG model with other models is aimed at demonstrating the effect of actual weaknesses or cavities in joints (vertical and horizontal) on the static and dynamic response of the structure, as well as the effect of reduced joint stiffness on this response.

2. Research motivation

Many previous studies have extensively discussed the technical condition of LPS buildings, the accuracy of the assembly of the prefabricated elements and the results of their on-site testing. On the other hand, to the authors' knowledge, the extent to which element connections affect the global static and dynamic response of LPS buildings has yet to be thoroughly investigated.

An element of novelty in this work is the inclusion in the numerical modeling of the LPS building structure of certain limitations or lack of certain properties (e.g., rotational stiffness along a horizontal or vertical edge, the flexibility of joints) resulting from the method or quality of assembly of the prefabricated walls.

One of the main objectives of this study is to statically and dynamically analyze the construction of LPS buildings when different connection models were used to represent defects in the assembly of prefabricated walls. According to many studies available in the literature, the quality of the connections between prefabricated elements was an important safety issue for LPS structures [6–8, 10, 11]. Advanced FEM analysis, including a time-history analysis, was carried out to assess the response of the LPS structure. The results of the analyzes can be used to explain why LPS structures built in the past with poor construction skills are probably in better condition than expected. In addition, building authorities can better assess the actual quality of LPS buildings and more effectively plan to upgrade or repair their structures.

3. Literature review

The literature review in this article is aimed at providing extensive information on technical issues related to LPS buildings, with a focus on connections, different structural systems and valuable inspector reports/examples of existing buildings that can help better evaluate LPS building construction.

In [8], the authors presented the results of visual, non-destructive and destructive testing of a more than 40-year-old LPS building scheduled for demolition. It was pointed out that the vertical and horizontal joints connecting the individual prefabricated elements are potentially weak points, important for the durability of large-panel building structures. Moisture penetrating the outer layers of the wall spreads through the joints to the inner parts of the wall. In contrast, water from moisture-generating rooms (kitchens and bathrooms) seeps into the joints immediately adjacent to them. In addition, it was stated that the condition of the precast elements is variable due to factors such as the quality control of the precast elements at the production facilities, the method of transport and storage on site, the accuracy of their assembly, the quality of the concrete filling of the joints and, in the next stage, the proper maintenance of the building [8]. It was found that administrative and politico-social changes in Central Europe over the decades have negatively influenced the planning for the modernization of LPS structures.

In [9], the difference between the energy dissipation capacity of vertical joints compared to horizontal joints under seismic loading was explained. The authors also explained that although the provision of flexural ductility is limited in LPS buildings, their seismic resistance is not solely related to the presence or absence of ductility but to the overall ability of the structure to dissipate energy. This ability mainly comes from the connection between the panel walls, where a large amount of energy is dissipated. So the real challenge in seismic resistance of an LPS building is to improve the ability of the connections to dissipate energy. The authors pointed out that the desirable location for energy dissipation in such structures is the vertical connection since slippage at the horizontal connection will be permanent due to the lack of corrective elastic forces acting to straighten the building, while the vertical connections, after energy dissipation, return to their original alignment with little or no permanent damage due to the elastic action of the cantilevered shear walls. In addition, the slippage that occurs at vertical joints reduces the lateral stiffness of the building, thus extending the basic period of vibration, which is desirable for seismic loading.

It was mentioned in [10] that even today, the regulations do not address all the needed regulations for LPS construction. In the early days of LPS building design, the lack of necessary regulations caused a series of events that led to a move away from LPS technology in some countries and partial improvements to regulations in others. The authors classified the basic problems in existing LPS buildings that arose during the construction phase. During the design and construction phases, some of them are poor knowledge of the guidelines, inconsistencies between theoretical knowledge and practical application, lack of knowledge of material parameters and their behavior, poor concrete quality and deviations in panel dimensions, and lack of qualified workers to assemble wall panels in terms of making structural connections.

The problems of LPS buildings built in Poland in the early days of this technology were mentioned in [11]. All structures are susceptible to gradual and natural degradation due to external factors. The degree of deterioration also depends on the type of technology used and the negligence made during design [11]. In addition to the typical problems that any structure experiences over time, LPS buildings can have additional damage due to their specific characteristics. It is important that the permissible dimensional and installation tolerances of the panels are established before construction to avoid problems associated with the wrong depth of support of the floor slabs, the construction of joints, the installation of prefabricated elements, and consequently, problems associated with the interconnection of the panels. Errors can occur at any stage of the construction process, and without established and acceptable tolerances, it may be impossible to take further action. When LPS technology began, there was a lack of construction experience with this type of structure.

The welded connections proposed by the designers were theoretically correct, while it was not always possible to make these connections during assembly according to theoretical assumptions. Lack of knowledge of the material's strength parameters caused post-construction problems. Later, attempts were made to overcome them, but using incorrect solutions. In [11], the causes of inaccuracies and deviations in prefabricated products arising during production are listed:

- poor quality of concrete molds/forms, as well as their locks,
- insufficient stiffness and high deterioration of metal parts,
- defects caused by flexible form parts.

According to [12], due to a lack of proper maintenance, LPS buildings that have been in existence for more than 30–40 years in many countries are in poor condition. In addition, the authors pointed out improper modifications, such as reconstructions for additional living space, openings in load-bearing walls, and enlarging existing buildings horizontally and vertically, which risk structural safety. One example of enlargement modification is given in Fig. 1, where additional floors have been added on top of the existing LPS building.

FIG. 1. The superstructure on an existing LPS building (photo: P. Knyziak, CC BY) [13].

The authors of [13] capture the problem of LPS buildings in an interdisciplinary way, taking into account urban, architectural, social and technical issues. Their research summarizes analyses of 110 LPS buildings located in Poland in terms of their technical condition, structural condition and internal installations. The authors also summarized the factors influencing the technical wear and tear of LPS buildings and described the problems occurring in them with proposals for their solutions.

A study of the technical condition of the steel hangers connecting the insulation layer to the load-bearing part of the wall panel was presented in [14]. They described their effect on the entire building rather than on individual panels, especially when leaks cause precast panels to become damp. Leakage of the layer can be the result of errors in placement, in the number and diameters of hangers, and the use of ordinary steel instead of stainless steel.

Although many researchers have mentioned country-specific deficiencies and their effects on LPS technology in terms of structural vulnerability, LPS buildings have been known to perform well when collapse prevention was the criterion. In [15], the author described the case of an LPS building located in Poland that leaned out from the vertical by 60 cm as a result of ground subsidence caused by mining activities in the area, and was restored to the vertical as a result of repair works of lifting and concreting the base. In addition, the author presented the results of a study of 300 structural joints from more than 100 LPS structures built between 1961 and 1994 in Poland in terms of their technical condition and concluded that these joints show a low degree of degradation and do not pose a safety hazard.

According to [16], a large proportion of LPS buildings built in seismically active regions of the former Soviet Union were known to withstand and protect their occupants during earthquakes.

The case study conducted by the authors in [17] and [18] provides a better understanding of what factors may influence the behavior of panel walls and their connections. It presents the results of the visual inspection of defects and tests of material properties of the existing LPS buildings located in Poznań (Poland), built in 1986. The condition of the outer walls of a residential building constructed in the LPS technology was examined through non-destructive and destructive tests to determine its residual material properties. Based on the observations, the number of panels with irregularities, taking into account damage or defects, was determined, and the test results are presented in Table 1.

Table 1. List of panels with irregularities or defects [17].

Damage or defect	Number of examined panels	Number of panels with irregularities	Percentage of irregularities
Visible scratching, cracks	224	41	18.30
Visibly washed, falling grit	180	25	13.90
Excessive thickness of the external layer and lagging (>16 cm)	224	59	26.30
Visible hangers, pins (detached covering)	224	12	5.40

Table 2 shows the results of strength tests, i.e., the average compressive strength of samples taken from three boreholes for each wall panel. The test was carried out for five different walls in the structure, and three borehole samples with a diameter of 43–44 cm were taken from each wall. The boreholes were drilled using a non-impact drill.

Table 2. Concrete strength test results [17].

Wall panel	Mean compressive strength of three samples σ [MPa]	Stress after correction σ [MPa] (Annex A of the standard [19])
1	28.9	31.8
2	20.9	22.9
3	26.6	29.2
4	39.7	40.4
5	29.6	32.5

An N-type Schmidt hammer was used to test the hardness of the outer concrete layer at a temperature of 20°C to 25°C. The tested concrete of the external layer was in an air-dry state, and the surface of the component was prepared for measurement each time by splitting the surface layer of grit, grinding and smoothing with a wire tip. A hammer was applied perpendicularly to the surface to be tested each time. Between 9 and 12 measurements were taken on each component at 6 different locations. The results, the characteristic mean concrete compressive strengths (f_{ck}) determined for the 10 selected panels, are given in Table 3.

Table 3. Concrete strength values obtained in the Schmidt hammer test [17].

Panel	1	2	3	4	5	6	7	8	9	10
f_{ck} [MPa]	13.1	18.5	24.1	23	11.1	12.1	20.9	19.7	8.6	11.5

The observation of the reinforcement system was made with the use of ferromagnetic devices for a wall panel with dimensions of 180 × 180 cm. Reinforcement was found in the form of a double mesh with a mesh spacing of 20 to 22 cm and a diameter of reinforcing bars from 6 to 8 mm. The range of the carbonization zone in the collected samples was determined by the phenolphthalein method. The maximum carbonization depth was measured as 18 mm with an average range of 15 to 16 mm for drilled holes with a diameter of 44 mm.

In [20], the authors attempted to assess the impact of mining activities on the LPS buildings. Their study is important due to the proposed calculation model for wall connections in LPS buildings. The numerical model based on the FEM analysis for a typical wall-wall and wall-slab connection was developed for a nonlinear material model (plasticity of damaged concrete). In order to determine the stiffness of the joint in the transverse, vertical and rotational directions, the displacements of the point selected in the center of the connection were analyzed. Figure 2 shows a two-dimensional representation of the developed calculation model.

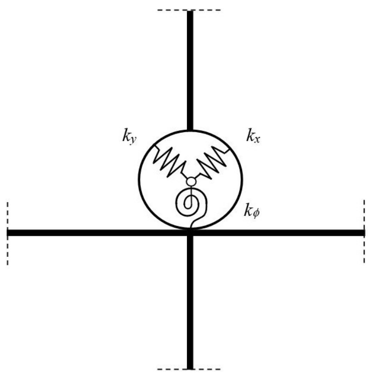

FIG. 2. Simplified, equivalent numerical model of the analyzed connection [20].

As a result of the conducted analyses, the translational stiffness of the panel joint was determined the same in both directions: $k_z = k_x = 2\,230$ MN/m^2 and the rotational stiffness $k_\varphi = 223$ MN/rad [20].

In [6], experimental and numerical analysis was conducted for a proposed new type of precast wall connector. Along with the determination of the dispersive capacity, residual displacement values and conclusions about the failure mode in experimental studies, in addition, the numerical analysis found that an effective way to simulate the behavior of prefabricated shear walls is achieved by modeling them as shell elements.

There are several ways to design and model LPS building structures. An important parameter with a large impact on the global behavior of the structure is the type and method of supporting the horizontal panels erected above the vertical walls. The work published by the authors of [1] explains that the type of LPS building structure is determined by the location of load-bearing walls and whether the ceilings are supported as one-way or two-way slabs. According to [1], three solutions can be distinguished with regard to the type of panels and the method of their support:

1) Transverse support system (Fig. 3a). The system consists of one-way slabs. Only the outer walls are load-bearing walls. The longitudinal walls are generally non-load-bearing walls. In order to be able to transfer the horizontal load to the wall elements, the floor slabs must be joined together to form a diaphragm.
2) Longitudinal support system (Fig. 3b). Only the outer longitudinal walls are load-bearing that supports the one-way floor slabs. Pre-stressed hollow core slabs are used in this system as they allow large spans between the walls to be covered.
3) Cross load bearing system (Fig. 3c). The system consists of two-way floor slabs. The floor slabs are usually the same size as the room above which they are and are usually 150 mm thick. The floor slabs are connected to the vertical walls to form a box structure. With this system, it is easier to achieve structural stability in both directions.

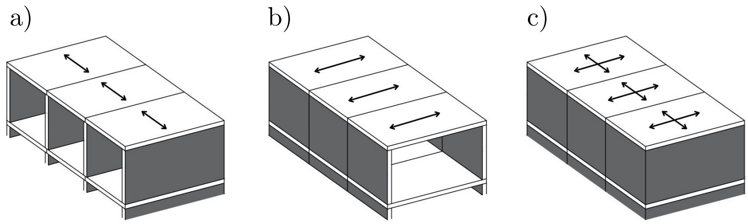

FIG. 3. Load bearing systems in LPS buildings [21]:
a) cross wall, b) longitudinal wall, c) all-wall.

Figure 3 shows three types of LPS building load bearing systems, where arrows show the direction of the floor slabs' support.

As a result of many years of experience under different conditions and on different projects, it was concluded in [21] that the best structural solution is offered by a transverse wall system when the load-bearing walls are across the longitudinal dimensions of the building. In this system, the longitudinal exterior walls have no imposed requirements, and the floor plates are as economical as those supported on all four edges. The least economical system is the one with floor panels resting on the longitudinal walls (the face of the building). The span of the floor elements becomes large and the longitudinal exterior walls must be designed as load bearing, which greatly restricts the designer. Therefore, the longitudinal wall system (Fig. 3b) was mainly implemented in non-residential buildings.

The authors in [22] compare the dynamic response of three different LPS structures, which differ in the assumptions concerning joints between the panels: with a monolithic joint, weak vertical joints, and weak horizontal joints. Illustrative deformations of a multi-story structure for various models of panel connections caused by an earthquake are shown in Fig. 4.

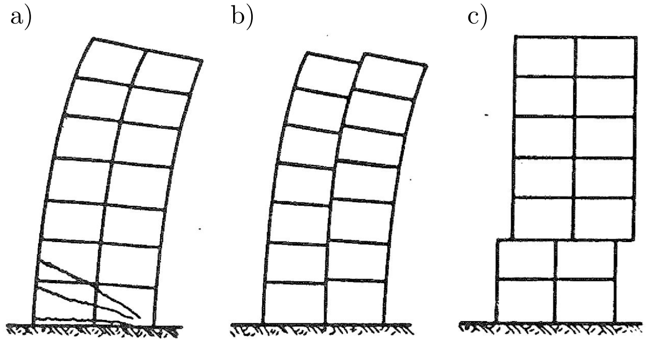

FIG. 4. Dynamic response of the structure for different panel-to-panel connection assumptions [22]: a) monolithic behavior as a single cantilever, b) weak vertical joints with vertical slip, c) weak horizontal joints with horizontal slip.

The results of the analyses confirm that the weak horizontal joints (Fig. 4c) behave similarly to the "soft story", which is a disadvantageous solution leading to a dangerous destruction mechanism. Better, safer results are given by the model of weak vertical joints (Fig. 4b).

Another study [23] was carried out for multi-family buildings built before 2000 across 27 cities in 20 countries of Europe and Central Asia. The study examined the susceptibility of 7 different categories of multi-family residential buildings to damage caused by earthquakes. The general results of the research for the adopted categories in terms of structure sensitivity are shown in Fig. 5.

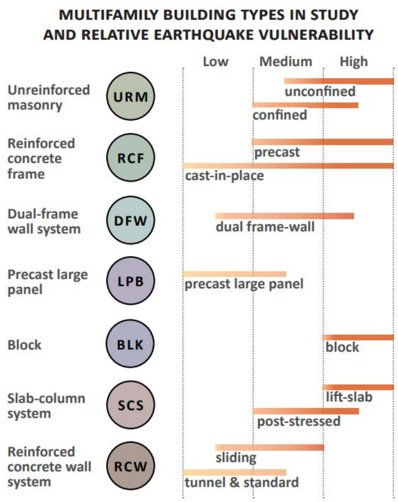

FIG. 5. Sensitivity to earthquakes for different types of multi-family buildings [23].

Although LPS buildings appear to be less susceptible than other multi-family units, the same authors conducted another study and gave examples of LPS buildings and their vulnerability in [23]. Based on its findings, several factors that make these buildings vulnerable to seismic activity, for example deterioration of prefabricated elements or inadequate renovation of internal partition walls, are listed.

Another study [24] mentions one of the most famous failures of LPS buildings in Great Britain, Ronan Point Tower (1968), which was the reason for introducing changes to the national regulations for this type of structure. The gas explosion broke the outer panel of the structure, which caused the progressive collapse of the edge of the structure along the entire height of the building (Fig. 6).

FIG. 6. Ronan Point Tower crash on May 16, 1968, in East London
(photo: Derek Voller, Ronan Point collapse, Canning Town, CC BY-SA).

The inspector's report presented in [25] on the Armenian earthquake in Spitak in 1988 contains a lot of valuable information about the LPS buildings and their seismic resistance. According to this report, 87% of the structures in the city of Spitak collapsed or sustained severe damage; however, most of the damaged structures were either masonry or had a prefabricated frame structure. At that time, there was one 5-story LPS building in the city of Spitak, while in the neighboring city of Gyumri, there were sixteen LPS buildings of various heights up to 9 floors. The condition of these structures after the earthquake was rated as very good with slight visible damage. Although they were built using the poor technologies available at that time, they showed better resistance compared to other structural systems. The photo in Fig. 7, given in [26] and taken after the devastating Spitak earthquake in 1988, shows the buildings after the disaster.

Fig. 7. Damage caused by the 1988 earthquake of Spitak (Armenia) (authors of photos: Vsatinet, CC BY-SA 4.0 (on the left); C.J. Langer. U.S. Geological Survey, public domain (on the right)).

4. Modeling of panel connections

In this study, a ten-story building structure constructed with LPS technology was analyzed. Data on the architecture of the building and data on material properties were taken from the published report [26]. The analysis was carried out for the material properties adopted in the project, as there was no information on the current state of the structure. While the main purpose of the study was to test various assumptions about panel connections, this approach was justified. On the other hand, it should be noted that analyses performed for other purposes may require the use of current parameters. While the main construction details and information about the building were included in the report, some necessary simplifications had to be made for modeling, e.g., the subsoil-structure interaction was omitted.

Along with newly proposed and researched prefabricated wall assembly methods [5, 6, 27], a variety of systems [4, 8, 11, 16, 28, 29] are still being used in LPS buildings. In the analyzed LPS building, vertical joints of wall panels were created by welding a horizontal reinforcing bar left from the adjacent panels to the central bar placed between them. The number of welds between the panels ranged from 2 to 5 [26]. The horizontal connection between the walls was created by means of dowel joints and welding of the hook rebar with the adjacent wall reinforcement. This type of connection is shown in Fig. 9. After the bars are welded, all the joints are grouted with concrete of the same strength as the concrete used for the production of the panels. In general, the aim was to connect all the panels in such a way that the joints would be able to transfer all forces in three perpendicular directions. Horizontal joints were placed between successive floors. Vertical dowels were placed between the lower edge of the above panel and the upper edge of the lower panel and connected to each other by welding hook-type reinforcing bars in a similar manner to the vertical wall joints (Fig. 9). Also, the horizontal connections between the floor slabs and the vertical walls were created using the same methodology. An example of this type of connection is shown in Fig. 8. In each room, the floor slab is supported by four wall panels forming a box-type spatial structure (Fig. 3c).

FIG. 8. The connection between the floor slab and vertical panels [26].

FIG. 9. Vertical connection of wall panels [26].

The type of material filling the joint and the type of reinforcement that connects the adjacent panels affect the joint operation. On the one hand, it can be assumed that the concrete is well bonded to the adjacent slabs and rebars, so it behaves like a rigid connection, and another idea might be that the material filling the gaps between the slabs is so weak that the slabs are only joined by rebars. Taking into account the possible differences in the quality of panel joints, four different calculation models were created with identical parameters except for one case where the joint parameters were defined. All joints were modeled along the edges of the panels as continuous connections rather than as point connections.

Although the behavior of connections is highly dependent on the technology used to assemble panel walls, due to the characteristic assembly method of the analyzed building (Figs. 8 and 9), it was computationally possible to release rotational stiffnesses and/or define elastic stiffnesses that would simulate the probability of inaccuracy. In addition, the authors [9, 22] discuss differences in the dynamic properties of LPS buildings depending on the orientation of weak connections, analyzing whether a vertical or horizontal orientation would be more favorable. Thus, the VH and HH models were proposed to test the effect of the orientation of negligible (zero) rotational stiffness on the solution. In the EE model, the connections were modeled in a way that expresses joint degeneration through a limited value of rotational elastic stiffness. Such a connection behaves neither as rigid nor as hinged, but it is closer to the actual state and can be compared with the rigid connection. For this purpose, the RG model was proposed, which expresses the original goal of joint design to transfer all loads uniformly in three perpendicular directions.

The first model behaves like a monolithic cantilever structure constructed with shear walls that are cast-in-place structures. This model is assumed to be the ideal system for this case study and is the basis for comparison with other models that have different assumptions for connections. To simplify further descriptions, the designation RG was adopted for the model with perfectly rigid connections.

In the second and third models, the hinge connection is defined by a linear rotary release along the vertical or horizontal edges, respectively. In the case of the second model, the designation VH was adopted, where V stands for a vertical and H for a hinge. The third model was marked with the abbreviation HH (horizontal hinge).

The fourth model has neither rigid connections between the panels nor hinged ones. and The connections are to be elastic, for which the stiffness coefficients are taken from [20]. For the fourth model, the abbreviation EE was adopted, referring to the word elastic.

Figure 10 shows the assumptions for the various connection models for both the horizontal and vertical edges of the panels.

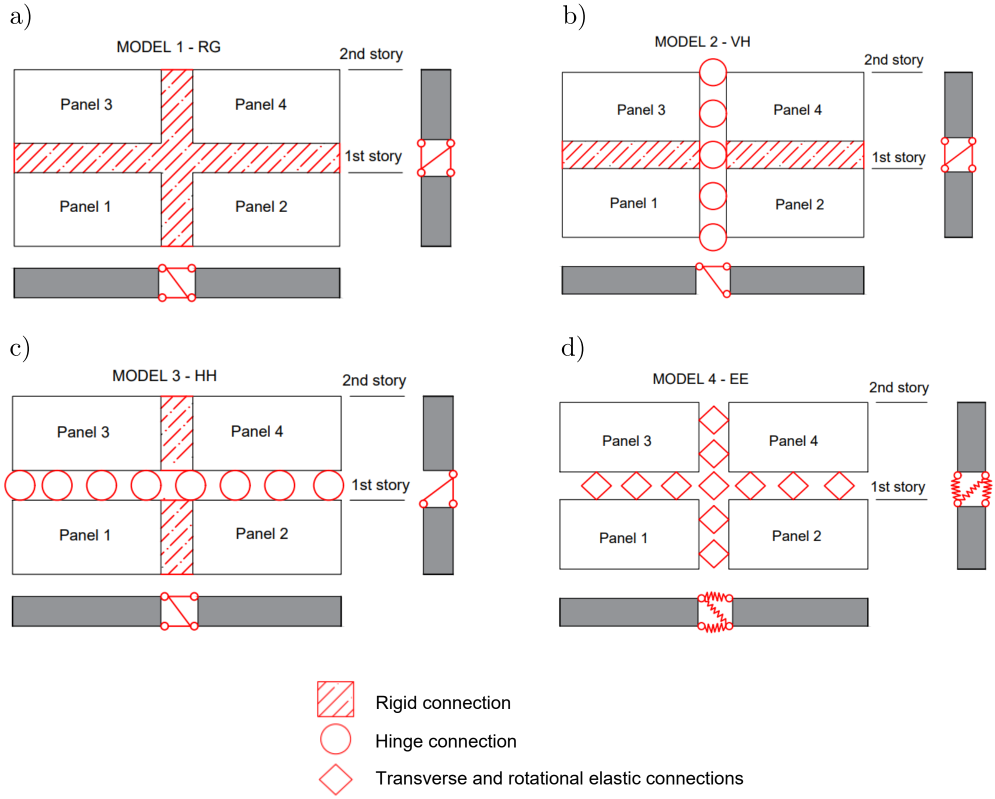

FIG. 10. Types of vertical and horizontal connections in the analyzed models: a) rigid – RG, b) vertical hinge – VH, c) horizontal hinge – HH, and d) flexible – EE.

The same material parameters were adopted for all the analyses. For all floor slabs and wall elements, concrete with a compressive strength of 37 MPa (cube test) and reinforcing steel with a yield stress of 390 MPa were adopted.

Autodesk Robot Structural Analysis (RSA) software was used for the static and dynamic analysis of the considered LPS building, which uses the finite element method for spatial analysis. Table 4 summarizes the material parameters for concrete adopted in the RSA software.

Table 4. Concrete material properties defined in RSA.

Material	Young's modulus [MPa]	Shear modulus [MPa]	Poisson's ratio	Unit weight [kN/m^3]	Design resistance [MPa]
C30/37	33 000	13 333	0.20	24.53	30.00

The loads imposed on the structures were assumed to be identical for all models and amounted to a dead load of 2.0 kN/m^2, and live load of 1.5 kN/m^2. Wind loads were taken into account using an automatic wind load generation option, assuming a design wind velocity of 30 m/s. Load combinations in static calculations were created automatically in RSA according to Eurocode standards.

The spatial model of the building prepared in the RSA program is shown in Fig. 11. The spacing of the transverse walls, along the global x-direction, is 3.6 m and 3.4 m, while the spacing of the longitudinal walls, measured along the global y-direction, is 5.4 m and 2.1 m.

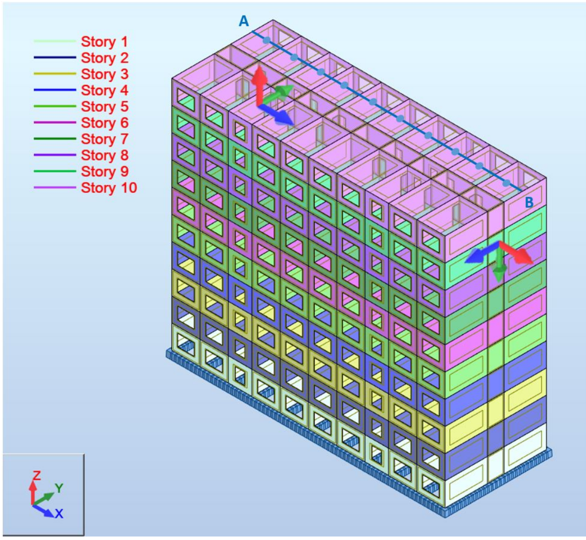

FIG. 11. A spatial model of the building prepared in the RSA program.

The main parameters of the model, including the number of structural elements and the corresponding number of nodes and finite elements, are given in Table 5.

Table 5. Numerical data of the computational model.

Description	Number
External wall panels (30 cm thick)	26
Interior wall panels (16 cm thick)	47
Floor panels (16 cm thick)	28
Openings	49
Finite element nodes	3710
Finite elements	4230

A shell element with the Coons meshing method created automatically by RSA software was adapted as a computational model for the panels. Using this method, all points created on the selected edge of a rectangular panel are connected by parallel lines to points on the opposite edge of the panel. The point of intersection of mutually perpendicular lines determines the node inside the region. In this way, regular finite element meshes consisting of surface 4-node square elements of 0.5 meters in each direction were created for panel walls and floor plates without smoothing, as shown in Fig. 12. The effects of soil-structure interaction were ignored, and the connection to the foundation was defined as fixed.

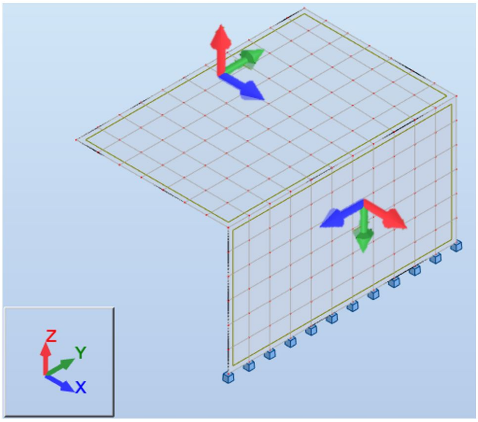

Fig. 12. Example finite element meshes of the wall and floor slab.

5. The results of the analyses

The static and dynamic analyses were carried out for the construction models according to design assumptions presented in the previous sections. A linear, elastic material model was used to solve the static problem of the considered system. As part of the dynamic analysis, a modal analysis was first performed, and then a seismic analysis according to the general approach given in Eurocode EN 1998-1 [30]. Finally, as part of dynamic analysis, time history analysis was performed using an exemplary accelerogram recorded during an earthquake.

The results of the static analysis are obtained for the combinations of loads adopted in accordance with the EN 1990: 2002 standard for individual calculation models (RG, VH, HH, and EE).

The overall stress distribution σ_{YY} [MPa] for the gable wall of the structure, for different computational models, is given below in Fig. 13. The concrete wall layer carries the greatest amount of stress depending on the direction of the applied load. When panel walls are subjected to bending, shear or bend-

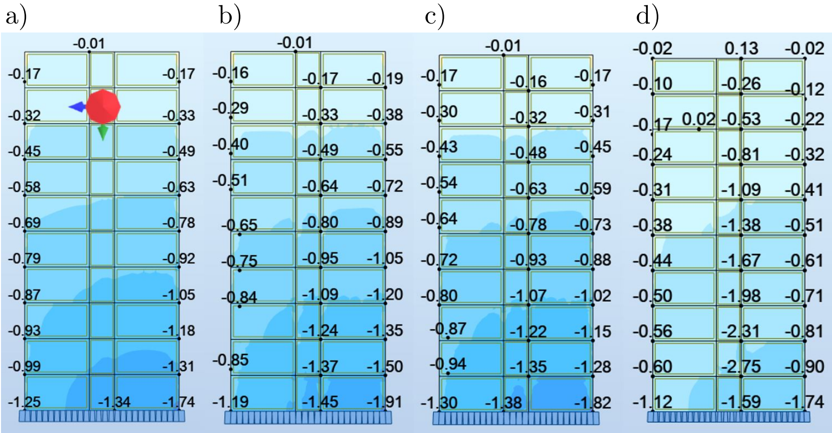

FIG. 13. Stress distribution σ_{YY} [MPa] for the gable wall, obtained from the static analysis for individual models: a) rigid (RG), b) vertical hinges (VH), c) horizontal hinges (HH), and d) flexible (EE).

ing in tension/compression, different stress states can occur in different layers throughout the wall thickness. In the following case, the maximum values are shown for a selected combination of loads for the middle surface (mid-thickness) of concrete wall panels.

To illustrate the overall static response of two-way floor slabs for different wall connection configurations, Fig. 14 shows the bending moment values of the center of the first-floor slab for the selected load combination. In order to be able to correctly interpret the internal forces in the joints of LPS elements, it is necessary to carry out a mesh refinement along the edges of the elements. Since the panel walls and floor slabs were uniformly meshed as a simplification in this

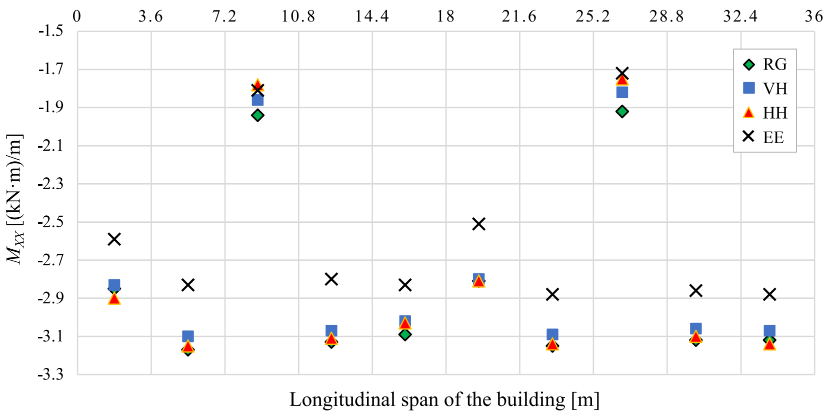

FIG. 14. Bending moment values M_{XX} [(kN · m)/m] for the first-story floor slabs along the line A-B for: rigid connections (RG), vertical hinges (VH), horizontal hinges (HH), and flexible connections (EE).

study, only the bending moments determined for the center of the floor slab were listed to compare the overall structural response. Figure 14 shows the values of spanwise bending moments M_{XX} [(kN · m)/m] per unit length of the cross-section for each slab along the A-B line shown in Fig. 11.

The maximum horizontal displacements of individual floors for the load combinations adopted according to EN 1990: 2002 for the four models are shown in Fig. 15. The values of the maximum displacements are given as the maximum horizontal displacements of the finite element mesh nodes located at the corner of each floor. The dimension of the building along the y-axis is much smaller than along the x-axis, so the weak axis of stiffness of the structure is the x-axis (the x-y plan in Fig. 11), so transverse displacements U_{YY} [cm] along the y-direction were chosen as presented.

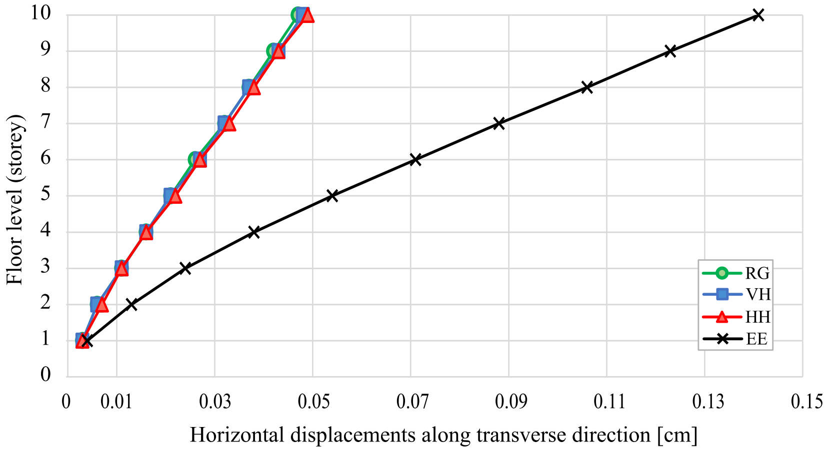

FIG. 15. Graph of maximum horizontal displacements at the corners of each floor.

The required reinforcement areas in the reinforced concrete cross-section were calculated for the respective ultimate (ULS) and serviceability (SLS) limit states for floor slabs and panel walls on a repeatable story. Although the maximum amount of reinforcement did not change, and the general reinforcement distribution was similar in all four models, the reinforcement density in individual zones varied slightly depending on the assumptions made for the connections between the panels.

In order to design an earthquake-resistant structure, the forces acting on the structure during ground movement should be appropriately and accurately assessed by engineers. While there are different ways to define seismic forces acting on structures, such as probabilistic estimates or deterministic approaches based on actual earthquake records, the goal is the same, which is to define the maximum forces that can arise from ground motion.

Among the three types of dynamic analysis procedures given in EN 1998-1, i.e., the equivalent lateral force method, spectral analysis and time history analysis, in this study, spectral analysis and time history analysis were implemented for the adopted models of structure.

Before starting the dynamic structure analysis according to the procedure given in EN 1998-1 in Chapter 3.2.4, some simplifications should be introduced, e.g., service loads should be converted into mass, which should also be taken into account during seismic excitation.

First, a modal analysis was performed, and the main modes of vibration were identified. It is necessary to carry out a modal analysis because the dynamic characteristics of structure derived on the basis of eigenvalues and eigenvectors are the input data for further seismic analysis: spectral or time history analysis.

In the calculation program, the mass matrix type was selected as "lumped without rotations", which required less computational effort, and at the same time accurately reflects the behavior of panel structures. Another type of mass matrix, i.e., a consistent mass matrix, is usually used for bar structures.

According to the assumptions specified in the standard EN 1998-1 in Chapter 4.3.3.2, the modal responses in two modes of vibration can be considered independent of each other if their periods of vibration differ from each other by more than 90%. If the vibration modes are independent, the square roots of sum of squares (SRSS) methodology can be implemented for summing modal responses. If the condition regarding the independence of the vibration modes is not satisfied, the complete quadratic combination (CQC) methodology is suggested. In the study, modal responses were summed up using the CQC method, which is more precise.

The EN 1998-1 standard states that for the purposes of dynamic analysis, the ratio defining the percentage of mass in motion in horizontal directions must be greater than 90%. This condition is relatively easy to achieve in frame structures. In the case of LPS buildings, the method of modeling the mass distribution along the building height depends on the number of stories. In the case of small buildings (up to 4–6 stories), the mass is not concentrated at the level of the floor slabs, which makes it impossible for the structure to obtain high values of the mass participation coefficients for horizontal displacements.

The diagram in Fig. 16 shows the values of the natural frequencies (Hz) for the first ten modes of vibration for the considered calculation models of the building. Figures 17 and 18 show the form of building deformation for the first two vibration modes for a rigid structure model (RG).

The seismic action was excited using the elastic design spectrum of type 1 given in EN 1998-1 for the type A subsoil.

In order to simplify the analysis, the seismic excitation acting only along the y-direction of the tested structure (Fig. 11) was applied. With this method of

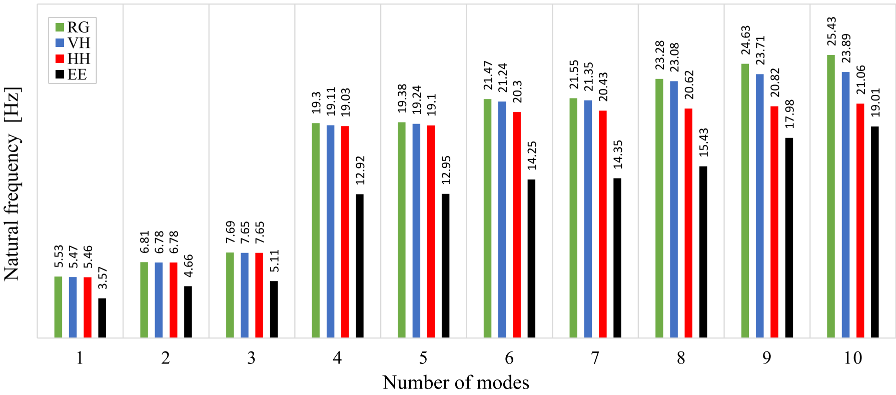

FIG. 16. The values of natural frequency [Hz] for four different models (modes of vibrations).

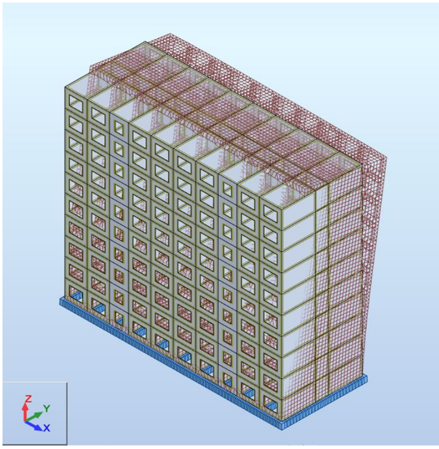

FIG. 17. The first mode of vibration ($f = 5.53$ Hz) – the RG model.

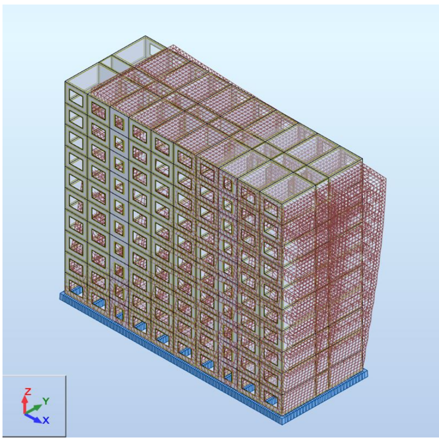

FIG. 18. The second mode of vibration ($f = 6.81$ Hz) – the RG model.

loading, the building was bent around the x-axis, which was the weak axis in the x-y cross-section. The stresses acting along the y-direction on the surface of the so-called blank walls or gable walls (walls without openings) were determined. The extreme values of the determined stresses were definitely below the material strength limit, and the differences between the analyzed models were minimal. A map showing the stress zones resulting from seismic action for models along the y-direction of the panels is shown in Fig. 19.

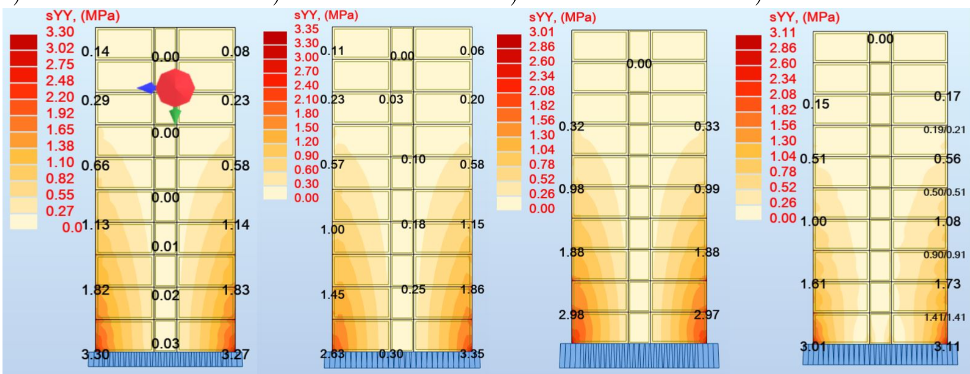

FIG. 19. Stress σ_{YY} [MPa] distribution in the gable wall: a) RG, b) VH, c) HH, d) EE.

Diagrams of the drift of successive stories in individual models along the y-direction are presented in Fig. 20. The dynamic responses for the first three models are similar, while different and much higher values were obtained for the fourth model-the model with elastic connections (EE).

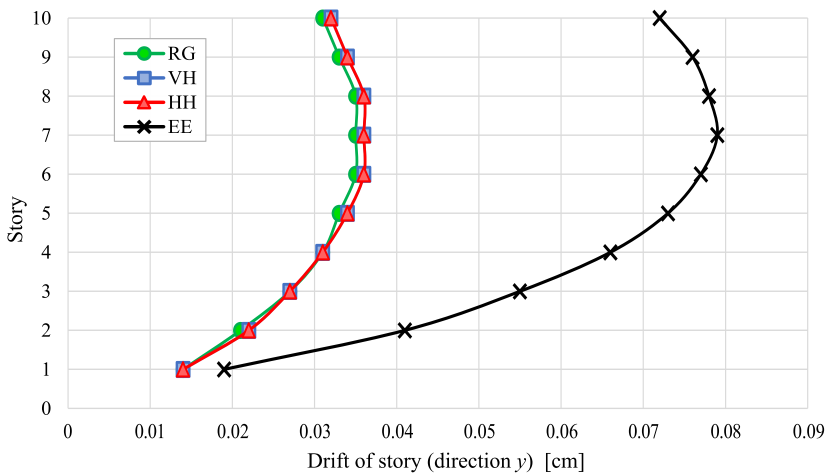

FIG. 20. The drift of the stories along the y-direction for the four models.

Based on the results of static and dynamic spectral analysis, it was found that the behavior of the rigid and hinge models does not differ from each other in terms of static deflections, internal forces and dynamic response. Therefore, the time history analysis was performed only for the model with rigid connections (RG) and for the model with elastic connections (EE).

The record of the 1999 earthquake in Izmit, Turkey (magnitude 7.6) was selected for the time history analysis because it was one of the most devastating earthquakes in Turkish history. Some characteristics of the selected earthquake record are given in Table 6.

Table 6. Data for the 1999 earthquake record in Izmit, Turkey.

Definition	Value
Sampling interval [s]	0.005
Duration [s]	252.89
Peak ground acceleration (PGA) [cm/s^2]	158.52
Peak ground velocity (PGV) [cm/s]	14.69
Peak ground displacement (PGD) [cm]	12.58
Moment magnitude [–]	7.6
Fault type	Strike-slip

Due to the adopted small dimension of the finite element in a relatively large spatial model, and due to the necessary, short integration step, the dimension of the problem was large. In order to shorten the calculation time, a 5-second interval was separated from the accelerogram describing the entire earthquake. Only two selected models (RG and EE) were used for dynamic calculations, as it was previously observed in dynamic and static results that RG, VH and HH models behave similarly. The time interval was selected in which the acceleration amplitudes were large and had an increasing tendency (Fig. 21), i.e., from the time $t_1 = 58.54$ s to the time $t_2 = 63.54$ s.

The results of the time history analysis obtained in the form of a dynamic response at a selected point, i.e., in the corner of the building on the top floor, for the two tested models are presented in the graphs, respectively: displacement (Fig. 22), velocity (Fig. 23), and acceleration (Fig. 24).

The maximum amplitudes of horizontal displacement of successive stories obtained during the considered range of seismic excitation are given in Fig. 25.

In the case of time history analysis, the obtained extreme stresses in the panels were 211 MPa for the model with RG and 179 MPa for the model with EE. For both models (RG and EE), during the time history analysis, RG of the lowest-story panels with the foundation were assumed. It is worth noting that the way in which the subsoil under the building or the boundary conditions at

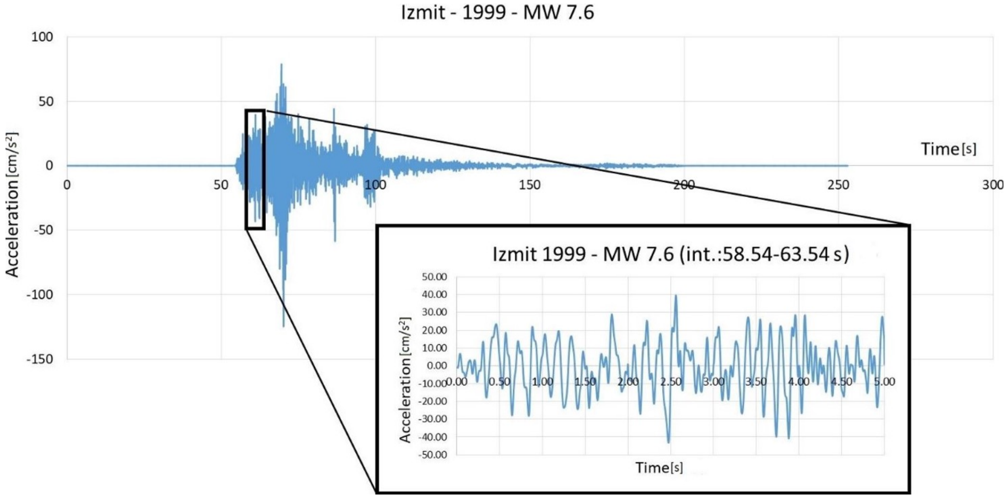

FIG. 21. Selected fragment of the record of the 1999 Izmit earthquake – MW 7.6 (time interval 58.54–63.54 s).

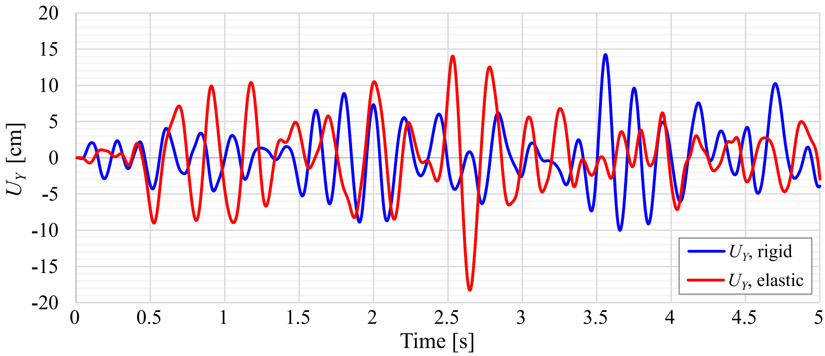

FIG. 22. Displacement plot for a selected point – the result of the time history analysis.

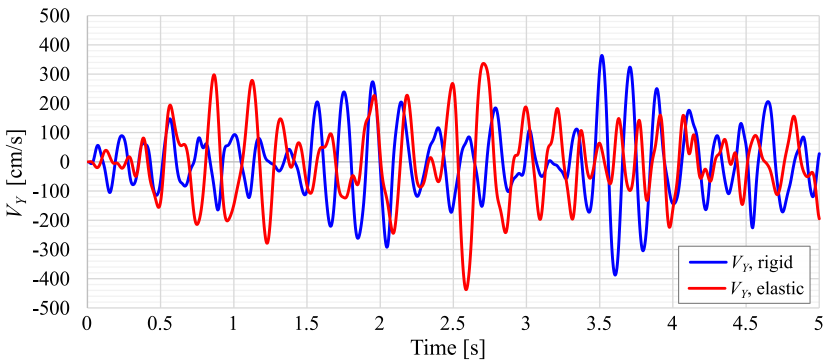

FIG. 23. Velocity plot for a selected point – the result of the time history analysis.

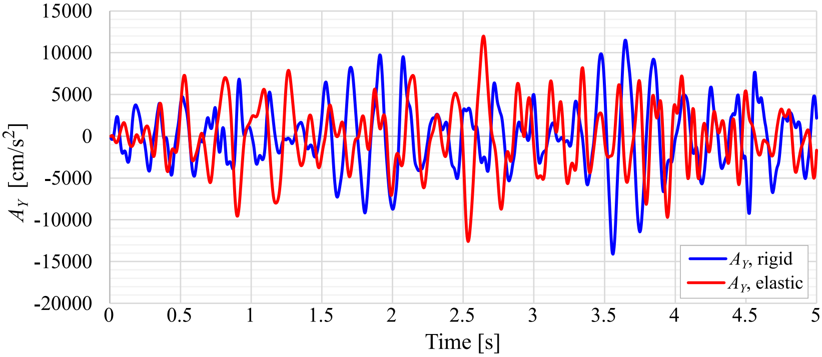

FIG. 24. Acceleration plot for a selected point – the result of the time history analysis.

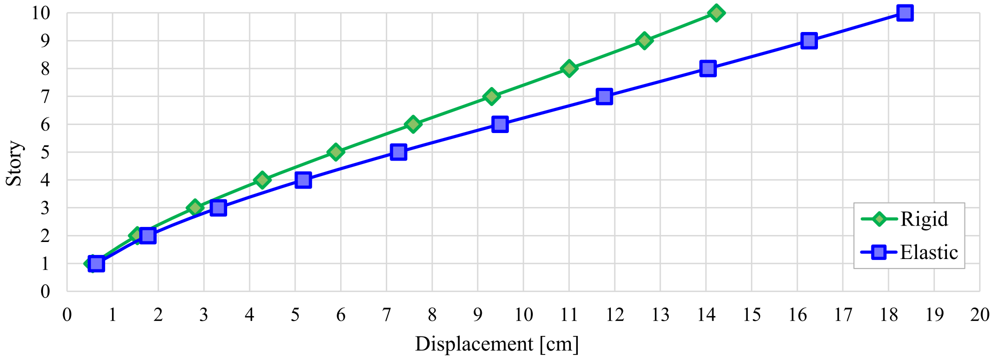

FIG. 25. Maximum horizontal displacements of successive stories for the analyzed models.

the foundation level are defined has a significant impact on the dynamic response of the structure, so this problem must be taken into account in future studies.

6. Concluding remarks

LPS buildings are specific in their structural behavior, compared to traditional high-rise buildings where the local failure of an element does not necessarily lead to a catastrophe or partial collapse of the structure. The real challenge for computer analysis of this type of objects is the modeling of connections between prefabricated elements. For this reason, an attempt was made to evaluate the influence of various models of panel joints on the values of internal forces in prefabricated elements as well as on the values of displacement and global behavior of the structure. According to KONCZ [1], the system of large-panel buildings is very suitable for high-rise residential buildings. The author explains that the load-bearing capacity of the walls can be used effectively and the thickness of the walls can be relatively small due to the inherent structural integrity of the system. The author emphasizes that the best structural solution to achieve this integrity is a load-bearing system including all walls with two-way floor slabs, where stability is ensured in both directions without additional measures.

1) Based on the results obtained from the static analysis, it can be concluded that for the adopted models, the differences are insignificant in terms of internal forces. It was observed that the general static behavior of all models was very similar to each other, while the discrepancies in the results in the form of local extremes occurred at the edges or corners of the panels. Similar observations were made with regard to the obtained stress distribution and the area of the required reinforcement.
2) The most stiffened model is the rigid model (RG), which represents an insitu reinforced concrete structure with shear walls that behave like monolithic cantilevers in the vertical plane. The spatial integrity of the building is so great that the loads are transferred evenly through all elements without a significant increase in stress at the edges or near the joints. As the panel walls in the modeled LPS building were well-serrated, the use of hinges on vertical or horizontal joints did not cause significant increases in stress. Higher stress values could only be observed locally at the edges of the panels.
3) The results of the static analysis showed that the prefabricated elements used in the construction of LPS buildings are so stiff in the plane that placing them perpendicular to each other ensures a very high spatial stiffness. Also, other factors such as simplicity of construction, symmetry in plane and elevation, regularity and structural integrity of the elements explain why changing the assumptions in the calculation model for panel joints did not have a large impact on the global behavior of the structure in terms of internal forces and displacements. However, it should be noted that differences in the results are possible, especially in terms of local extremes, for other types of LPS buildings constructed with different methods or with different components, e.g., when there are one-way slabs, non-load-bearing external or internal walls, etc.
4) Overall, the obtained similarity of the results without a large difference between the considered models shows that LPS buildings built even with poor construction skills and without proper control are likely to perform better than expected.
5) One of the key interests in this study was to see if LPS technology could be successfully applied in zones of high seismic activity. The results obtained from seismic analysis and time history analysis are promising even for

earthquakes of moderate magnitude. The stress levels in the prefabricated elements remained well below the material strength in the case of the seismic analysis performed for the type 1 spectra given in EN 1998-1.

6) High spatial stiffness, uniform distribution of shear walls, homogeneity and simplicity of construction can be given as the reasons why LPS buildings have high seismic resistance with their potential to eliminate structural deficiencies. It should be added that seismic analyses were carried out for type A soil according to European standards, which was favorable for the structure since spectral accelerations reach higher values on weaker soils. It may be necessary to conduct further analyses for different soil parameters.

Among the most important achievements in the present work are:

- The flexibility of prefabricated wall connections has an insignificant effect on the dynamic response of the structure and the distribution of static internal forces.
- Previously erected LPS buildings without experience in construction methodology, with a lack of proper regulations and built by unskilled workers are likely to perform better than expected.
- Instead of technical degradation, a more important criterion for the structural integrity of LPS buildings is the load-bearing wall layout adopted in the design.
- Properly designed and constructed large-panel buildings can be used in regions prone to seismic loads.
- Potentially, the results of this study can be used to better understand the numerical modeling of LPS structures and simulate their performance in static and dynamic terms. The results of this study, along with a review of the literature, may prompt construction authorities to reassess the lifespan of LPS structures and plan their retrofits more effectively.

Finally, it can be summarized that in the past, LPS technology buildings performed their functions quite satisfactorily. However, in high seismic risk zones there are few buildings of this type, and their number is much smaller than the number of buildings with a different type of structure. Therefore, we do not have a sufficient data set to judge their resistance to seismic loads that have already occurred in the past. Currently, there are many modern solutions enabling the design of seismically safe prefabricated structures, even in the most seismically active regions in the world, e.g., post-tensioning of structural elements in orthogonal directions [31, 32]. However, a general concern about existing LPS buildings is related to the uncertainty of connections between panels, as poor connections in some load-bearing system configurations can lead to progressive devastation and, eventually, building collapse.

Acknowledgments

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