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Principles of Energy Efficient Construction and their Influence on the Seismic Resistance of Light-weight Buildings

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Abstract: Recently, an increasing trend of passive and low-energy buildings transferring from non earthquake-prone to earthquake-prone regions has thrown out the question about the seismic safety of such buildings. The paper describes the most commonly used details of energy efficient construction, which could be critical from the point of view of earthquake resistance. The paper focuses on the prevention of ground floor slab thermal bridge and presents a case study on the seismic response of multi-storey wooden buildings founded on the RC foundation slab lying on a thermal insulation (TI) layer made of extruded polystyrene (XPS). The structural response is investigated with reference to the following performance parameters: the building's lateral top displacement, the ductility demand of the superstructure, the friction coefficient demand, the maximum compressive stress in the TI layer and the percentage of the uplifted foundation. A comparison between the response of models founded on a fixed base and models founded on a layer of TI with the same wooden crosslam structure differing in the number of storeys, strength capacity and subjected to earthquakes with different levels of seismic intensity is done. Regarding the building's top displacements, the maximum compressive deformation in the TI layer, and the percentage of the uplifted foundation, the results have shown that the potentially negative influences of inserting the TI under the foundation slab could be expected only for high-rise buildings subjected to severe earthquakes. Oppositely, for the superstructure's ductility demand and for the friction coefficient demand it was demonstrated that the largest demands could be expected in the case of low-rise buildings. The control of friction coefficient demand, which was recognized as critical parameter for analyzed wooden buildings, has shown that the capacity value could be exceeded yet in the case of moderate earthquake occurrence.

Keywords: Earthquake response, Extruded polystyrene (XPS), Foundations on thermal insulation layer, Low-energy buildings, Seismic analysis, Thermal bridge, Wooden multi-storey building.

1. INTRODUCTION

In the past 40 years many new insights on the climate change and its impact on the environment were developed. The results of several researches indicate that people with their activities largely contribute to the changing climate. The latter dictates that we must adapt and accept new strategies in order to prevent or at least limit the significant climate change. It is necessary to take into account that buildings alone comprise 40% of the total energy consumption in the European Union (EU) [1]. Reducing energy consumption and producing energy from renewable sources therefore represent an important measure in the building sector. As a consequence an increasing trend of low-energy buildings is notable in the last 15 years.

The implementation of the directive [1] has set new requirements for the buildings' energy efficiency. After the year 2020 it will not be possible to get a building permit unless the building will be near zero-energy. One of the Directive 2010/31/EU principles is, that energy efficiency measures should not affect other requirements in buildings such as accessibility, safety and intended use of the building. On the subject of ensuring the safety of low-energy buildings there is no sufficient literature to investigate to what extent and in which cases can the construction of such buildings be dangerous in earthquake-prone regions. In those parts of Europe in which low-energy buildings have already become an established practice, earthquakes are for the most part unknown and therefore the verification of new construction details is not necessary. In recent years, however, the low-energy buildings standard has slowly been gaining ground in areas where earthquakes (including strong earthquakes) are frequent, such as Spain, Portugal, Italy, Greece, Croatia and Slovenia. The suitability of such details in earthquake areas needs to be verified, and appropriate solutions found. The paper describes the critical details of low-energy buildings and focuses on the prevention of ground floor slab thermal bridge. Under the given assumptions a case study of multi-storey wooden buildings founded on a layer of thermal insulation (TI) is presented. A comparison between the response of models founded on a fixed base and models founded on a layer of TI with the same wooden structure is presented.

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2. CRITICAL DETAILS OF LOW-ENERGY BUILDINGS FROM THE POINT OF VIEW OF EARTHQUAKE RESISTANCE

In practice there are no uniform requirements for thermal efficiency and construction detailing methods, which will define the unique low-energy buildings [2]. On the other hand, in the case of passive houses, these requirements are more accurately defined and determined by regulations of the Passivhaus Institute [3]. In order to achieve low-energy consumption passive houses must expose well isolated and air-tight envelope without thermal bridges. In this way, extremely low transmission heat losses are achieved. Furthermore passive houses must have a controlled ventilation system preheated with heat from the exhaust air, which also reduces ventilation losses. By achieving all of the Passivhaus Institute requirements the energy consumption of the building is reduced to the level, that no active heating system is required.

The demand of constructing buildings without thermal bridges is a trend, which applies to all new built buildings, regardless of the different definitions of low-energy buildings and the use of different passive and active systems to reduce energy consumption. Already a small thermal bridge can endanger the environmental concept of such buildings [4]. The problem exposed in this article relates to the fact that the construction of low-energy buildings is also present in earthquake-prone areas. However, the specific details to prevent thermal bridges have not been adequately verified on dynamic seismic loads [5]. Structural control for seismic load is necessary, because the majority of problematic junctions is resolved by inserting thermal insulative parts between the load-bearing structural elements and can cause weakening of the structure in the most crucial parts of the building. On the account of improving thermal comfort of the building structural integrity/stability can be threatened.

First low-energy buildings were low-rise buildings, which are not so vulnerable to the changes on the building envelope from the point of view of structural resistance [6]. The latter is the main reason, that structural seismic safety of low-energy buildings has not been thoroughly researched until now. Solutions for new critical details in passive and low-energy buildings are mainly developed and experimentally tested by manufacturers of construction products and architecture designers according to the requirements of an individual building project. In the Fig. (**1**) an example of a building with the applied energy efficient building principles has been presented. In this schematic representation of a building section, the problematic details of the interaction between the buildings' structural elements and the thermal insulation have been exposed. The building in the Fig. (**1**) is only an overview of problematic details, regardless of the structure material or structural system and is not further used in the analysis. The special details of passive and low-energy buildings, which could be critical in the case of dynamic seismic loads, are shown in Fig. (**1**) and can be divided as following:

A. Installation of a TI layer with suitable compressive strength beneath the ground floor slab, foundation slab or strip foundations. For this purpose most frequently used materials are extruded polystyrene (XPS) boards and foam glass boards/granulate. More details of the insertion of TI layers beneath the foundation slab and their influence on seismic response are presented in chapter 3.

B. Interruption of the thermal bridge at the junction of the outside wall with the strip foundation or foundation slab by means of a so-called insulation base made of a material with suitable compressive strength and thermal conductivity. Insulation base is usually a thermal insulating block produced from one of the following materials: aerated concrete, light concrete, foam glass and extruded polystyrene (XPS). Such thermal insulation blocks can be mainly used in masonry structures and are not reinforced as it is in the case of load bearing TI elements (Detail C). For this reason thermal insulation blocks can be used only to withstand compressive stresses. The manufacturers of these elements usually limit their use regarding the number of building storeys, which is the main parameter for defining the axial compressive stress on each block. Insulation base is mostly suitable for low-rise buildings with less than three storeys, because the compressive strength of thermal insulation blocks is limited on the account of their good thermal conductivity properties. The axial compressive stresses on the insulation base can be even further increased, if the building is exposed to seismic shaking. From the point of view of

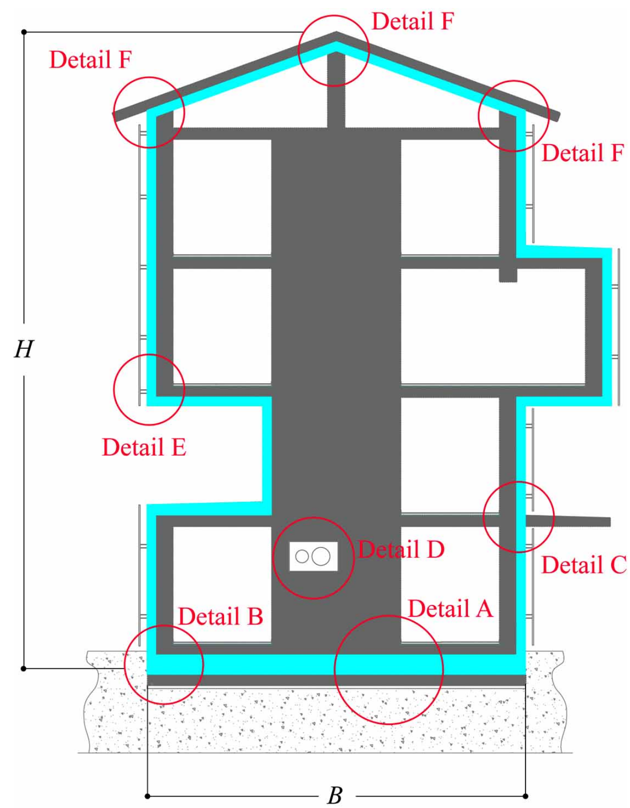

Fig. (**1**). Schematic representation of low-energy buildings' details critical from the point of view of earthquake resistance.

earthquake resistance the position of the blocks at the buildings' base is undesirable, because the seismic forces are as a rule largest at the base. Furthermore seismic shaking can also induce shear and tensile stresses in the thermal insulation blocks, which are not designed to overcome the combination of compressive and shear stresses or tensile and shear stresses.

C. In the case of preventing thermal bridges between the balcony cantilever and the internal slab, special innovative solutions of different load bearing TI elements were proposed by manufacturers of construction products. These products are experimentally tested on vertical loads and their results are published by manufacturers in the form of different material for building designers. Some research [7], which compares the experimental and analytical results, was obtained with an intention to present a variety of potentially dangerous failure mechanisms of such load bearing thermal insulation elements exposed to vertical loads. When exposed to seismic shaking other critical issues could emerge. In the case of long cantilever the induced vertical oscillating could result in the local failure of the load bearing thermal insulation element. Moreover, such a detail can also be of critical concern for the global structural seismic response in the case when it interrupts the load bearing shear wall or column. In this case the structural vertical integrity is affected and could result in the deteriorated overall structural response. The problem is similar as in the case of thermal insulating blocks (detail B).

D. Interruptions in the structural system, because of the new requirements of the controlled mechanical ventilation system. The installation pipes used for the mechanical ventilation system are in most cases placed at the parts of the structure, which are crucial for their stability (structural walls, columns, slabs and beams). From the point of view of earthquake resistance, the interruptions in the structural elements cause a new weakening point, which can resolve in a different plastic mechanism under severe structure ductile behaviour (e.g. soft storey mechanism [8]).

E. In the process of energy efficient construction, the most effective measure is to increase the thermal insulation thickness. However, the increased thermal insulation thickness of the outer wall can endanger the mounting of external façade elements, which is more difficult to assure. Furthermore, the thermal bridges through the façade elements are avoided by using special fixation elements with minimal thermal conductivity, which could prove to be critical in the case of strong seismic shaking.

F. Roof construction fittings without thermal bridges. This detail could be more critical, when exposed to strong wind than in the case of strong earthquakes. However, the roof construction is usually considered as a stiff diaphragm, which binds the vertical construction elements to work as a whole in the case of a horizontal load. If a stiff diaphragm on top of the building is not assured, the seismic response of the whole building could be endangered.

3. SEISMIC ASPECT OF THE INSERTION OF THE TI LAYER BENEATH THE GROUND FLOOR SLAB

From the point of view of earthquake resistance, it should be pointed out that, by inserting the flexible layers of TI between the reinforced concrete (RC) foundation slab and the layer of blinding concrete on the ground, the fundamental period of the structure will be prolonged, since, due to the horizontal shear deformability of the insulation layer, the building will oscillate more slowly than on a firm ground. The fundamental periods are additionally increased by rocking effects, which are a consequence of the vertical deformability of the insulation layer. Most passive houses are low-rise buildings with short fundamental periods which could be elongated by the insertion of a TI layer, and thus moved into the resonance part of the design response spectrum (into the period of constant accelerations). In such cases the expected top accelerations of the structure could increase by a factor of two or three in comparison with a structure on a fixed base (Fig. (**2**)). Such an increase could lead to damage to the superstructure or its content, which should not be ignored [5, 9-11]. However, if the fundamental period of the superstructure is already on the plateau of constant accelerations, the insertion of TI under the foundation slab might prolong the structural period into the descending branch, so that the seismic forces acting on the structure might be reduced. Only in this case will the TI layer act as a traditional seismic base isolation system [12-14], so that the earthquake induced forces would be reduced.

The results of the preliminary studies [5] and extensive parametric studies [11] have shown that the designers of multi-storey buildings founded on the TI layer under the ground floor slab should pay additional attention to the seismic behaviour of such structures. In the case of stronger seismic excitation the following limit states (or their combination) could be expected: (i) formation of a plastic mechanism in the superstructure (i.e. the selected strength was too small), (ii) overturning of the superstructure (i.e. its selected slenderness was too high), (iii) exceedance of the allowable compressive strength of the TI (i.e. the weight of the building was too large), and (iv) the allowable shear strength of the TI, or the allowable friction capacity between the TI layers, is exceeded. Both studies [5, 11] also proved

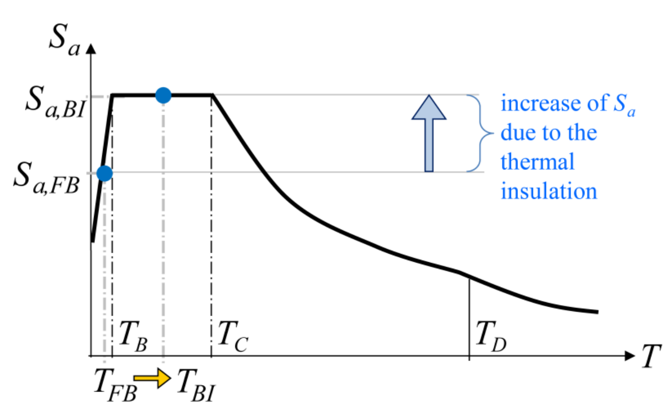

Fig. (**2**). Roof accelerations of the stiff superstructure in relation to its fundamental period – comparison of the fixed base (FB) and base isolated (BI) building.

that the control of maximum shear stresses and maximum horizontal displacements at the TI layer is much less critical than the control of behaviour of TI in compression which becomes of critical concern when severe uplifting takes place. It was shown that during moderate seismic excitation the edge compressive stresses in the TI layer beneath the foundation slab could exceed the TI nominal compressive strengths already in the cases of heavy concrete buildings with more than two or three storeys [5]. The allowable number of storeys depends on numerous parameters such as seismic intensity, the floor plan aspect ratio (slenderness) of the superstructure, its stiffness, strength and cyclic behaviour, its seismic weight, quality and thickness of the TI [11]. In authors' related study [10] also the soil type and the stiffness of the ground floor slab have been investigated and recognized to be essential parameters in governing the seismic response of the buildings founded on the TI layer. In the present research, the seismic response of multi-storey wooden buildings founded on a TI layer is presented.

4. CASE STUDY OF WOODEN MULTI-STORY BUILDINGS FOUNDED ON THE TI LAYER

4.1. Numerical Modelling

The construction of low-energy or passive houses is a complex and multidisciplinary field. The decisions of choosing a construction type are governed by many criteria: economic aspect, construction safety, environment, well-being and many others. In [15] the comparison of different passive house construction types was performed by considering 15 criteria and using analytic hierarchy process (AHP). Four different types of construction for passive houses (solid wood, wood frame, aerated concrete, and brick) were compared on the basis of the selected criteria for a chosen case study. The AHP analysis revealed that the highest ranking criteria are notably well-being, the psychological aspect, and functionality. It was pointed out that wood as a renewable raw material was one of the best choices for energy efficient construction because it is also a good thermal insulator, has good mechanical properties, and ensures a comfortable indoor climate. As a result, the wood construction was found to be the best material for low-energy or passive houses among the construction materials. The advantages of wood as a construction material are: lower embodied global warming potential, embodied carbon, positively associated with well-being, aesthetic and eco-friendliness, and realistic end-of-life disposal options. In the light of the growing importance of energy-efficient building methods, it could be said that wood construction would play an increasingly important role in the future.

Furthermore the construction of wooden multi-storey buildings has in the recent years proved to be reliable also in earthquake-prone areas. Recent experimental studies [16-19] regarding seismic safety of wooden buildings have showed that these building can be built up to 7-storeys in earthquake-prone areas and that they perform very well under strong seismic shaking with maximum averaged inter-storey drifts in the order of 2% and suffer only minor non-structural damage. As can be seen in these studies, the failure modes of wooden structures are ductile with fastener bending and embedment. Brittle failures of the wooden structural parts are prevented by the plasticization of metal connections, which is achieved by designing the corresponding members for the overstrength of the connections [20]. In general two different construction types of wooden multi-storey buildings are used: (i) light frame wood structures in combination with steel frame [16, 18, 19, 21, 22] and (ii) solid wood structures with massive cross laminated timber panels [17, 20]. This paper deals with the seismic analysis of multi-storey crosslam buildings. Such systems are made from massive timber panels connected to each other and to the foundations using metal brackets and nails (or screws), and self-tapping screws or punched metal plate connections. During an earthquake, energy is dissipated in all the connections as well as in friction between timber panels, although it is suggested that the beneficial contribution of the latter is conservatively neglected until further investigations are performed [20].

In the present study comparison of wooden multi-storey structures with different number of storeys has been carried out with an intention to compare the response of conventionally founded multi-storey crosslam structures on a firm soil (FB models) with the response of the same structures founded on a layer of TI (BI models). In order to make a parametric presentation of the inelastic response of the large number of structures included in the study, the superstructure was conveniently modelled as a simplified, lumped-plasticity SDOF model with one plastic hinge at its base. The investigated models are within the domain of a parametric study performed in a companion paper [11], where the nonlinear time history analyses (NTHA) of superstructures with different stiffnesses, strengths, weights and ductility capacities, as well as height/width and floor plan depth/width ratios were performed. Thermal insulation of different types and thicknesses was also included. Various types of realistic regular passive houses made of different materials, using different structural systems, with different basic geometrical data and different weights can be recognized from this parametric study. Only a part of these results pointing out the performance of light-weight energy efficient buildings were presented in this paper. Similar simplified models for the superstructure have already been used for extensive parametric studies of fixed based structures [23-26], seismic isolation systems [27-29], soil structure interaction (SSI) phenomena [30-34] or to determine the probabilistic characteristics of seismic ductility demand [35, 36]. The behaviour of buildings founded on TI, when subjected to earthquake ground motions, is to a certain extent similar to the problems of nonlinear dynamic SSI and therefore a similar level of accuracy is presumed for the results obtained in this paper. The findings of previous researches [23-36] performed on simplified building models indicate that the results between the MDOF and equivalent SDOF models are in good agreement regarding buildings' top displacement response, whereas the simplified methods could lead to conservative estimates of the ductility demands for high rise structures. These facts are taken into consideration in this paper, where the results are mainly focused on low to mid-rise structures and on the distinction between the response of the FB and BI models.

For modelling purposes, an inverted triangular distribution of the horizontal seismic forces was assumed. As a result the mass of the superstructure (m) was lumped at 2/3 H [37], and the mass of the foundation slab together with the ground floor (m_b) was concentrated at the bottom of the building (see Fig. (**3**)). Geometric nonlinearities, including P-Delta effects, were considered in the analysis [38]. The used moment-rotation relationship of the superstructure is expressed by means of the effective yield moment (M_y), the yield rotation (θ_y), and the post-yield stiffness, as defined by a hardening ratio ($\eta = 0.03$) and a ductility capacity ($\mu_c = 3$) (Fig. (**3**)). The assumed yield point of the superstructure was related to the superstructure strength by the strength ratio parameter ($\alpha = \frac{F_y}{W}; W = m \cdot g$). Hysteretic behaviour was modelled by means of degrading pinched hysteretic behaviour [22]. The TI material (XPS boards) under the foundation was modelled by a set of vertical and horizontal springs with non-linear hysteretic behaviour. The material characteristics of the XPS boards and the corresponding hysteretic rules were obtained previously by static monotonic and cyclic tests for the investigation of the shear and compression properties of the XPS material [39]. It was assumed in the analyses that the foundation slab is rigid, and modelled by a set of rigid beams. The width of the modelled foundation is represented by the short side (B) of the foundation slab, and is divided into a fixed number of FE. Each node, where the beam is divided, is supported by a vertical spring with the characteristics of TI in the compression. The stiffness of each spring corresponds to the FE length and the long side of the foundation slab (A). The hysteretic behaviour of the vertical springs is modelled by an elastic perfectly plastic gap material [40] with a compression-only gap element. The secondary slope for the nonlinear backbone curve (i.e. the post-yield stiffness ratio) is defined by the hardening ratio ($\eta = 0.08$), and depends on the type of material used in the study. The shear horizontal stiffness was modelled by one lateral spring only, as shown in Fig. (**3**).

The input parameters of the simplified SDOF superstructure system were evaluated from finite element analysis of the previously studied wooden cross laminated structures [16, 17], along with basics of structural dynamics and are presented in Table **1**. The main goal of the study was to analyse the global behaviour of a BI model structure and compare it with the corresponding FB model structure. For this purpose a set of regular, symmetric superstructure models with different heights and designed according to the modern earthquake building codes were selected for the analyses. The examined structures were analyzed with the use of simplified SDOF superstructure models, which have already been used for wooden shear wall structures [41-43]. The SDOF structure model is adequate for estimating the global behaviour of wood shear wall structures, as their typical behaviour is driven by the first mode of vibration [42]. The simplified SDOF building model used in [42] was confirmed by comparison with correlated shake table tests of two-story single family house [22]. The comparison confirmed the accuracy of the simplified methodology in representing the behaviour of wood shear walls and the importance of incorporating degradation effects for global evaluation of the behaviour of wood structures.

The analyses performed in this paper were nonlinear time history analyses (NTHA). The selection of 7 ground motions

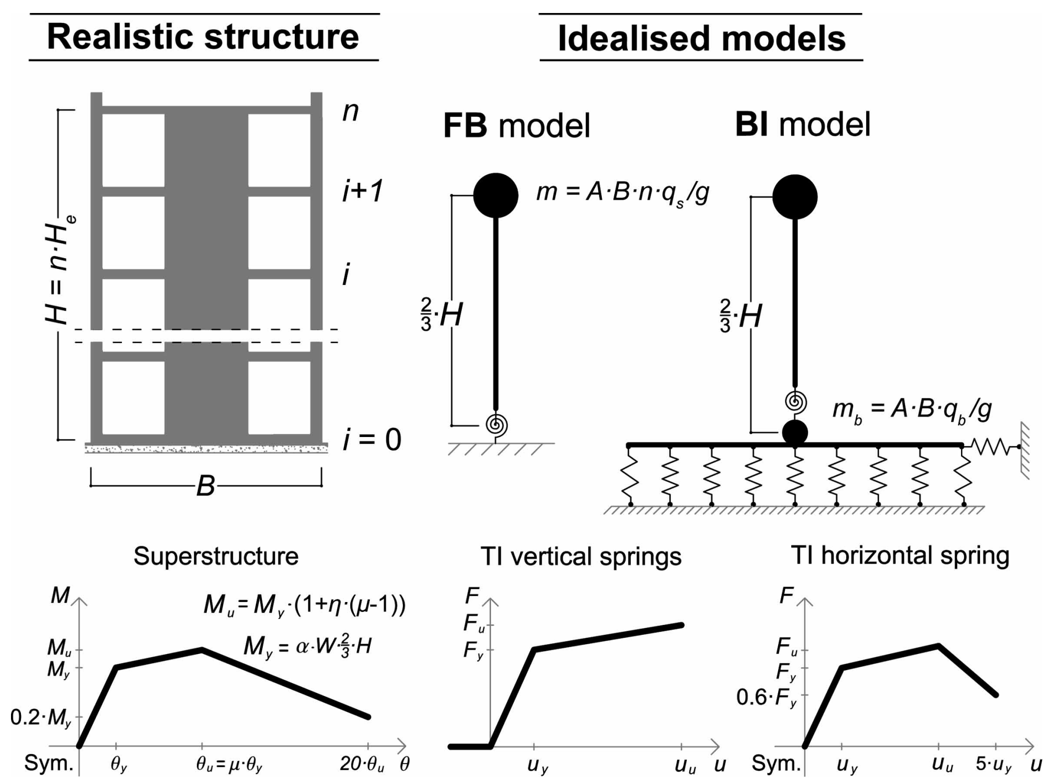

Fig. (**3**). Comparison of the fixed base (FB) and base isolated (BI) building model with nonlinear backbone curves.

Table 1. The fixed input parameters for the elastic and inelastic models.

Superstructure	Foundation and TI layer
Hysteresis type: *degrading pinching*	Type of TI: *XPS400*
Ductility capacity: $\mu_c = 3$	Thickness of TI: $d_{XPS} = 30$ cm
Seismic vert. load: $q_s = 6\,\mathrm{kN/m^2}$	Ground floor loading: $q_b = 10.5\,\mathrm{kN/m^2}$
Storey height: $H_e = 3.0\ m$	Foundation slab (rigid): $A/B = 15/7.5\,m$

from the ESD database [44] was made by REXEL software [45] and can be found in [11]. The results presented in this study are always calculated as an average value of 7 chosen ground motion accelerations response. The selection of the superstructure's stiffness for the analyzed case models was done indirectly through the calculation of the fundamental vibrational period with a simplified formula used in EC8 [46]:

$$T_{FB} = C_t \cdot H^{3/4} \tag{1}$$

The lateral stiffness obtained by this simplified formula is in agreement with several studies of wooden cross laminated structures and can be found in [17, 47]. The parameter $C_T = 0.50$ was selected and height of the superstructure (H) was calculated depending on the number of storeys, where $H_e = 3\ m$ is the storey height and was assumed to be constant along the height of the building. In order to generate the models for the superstructure the equivalent lateral stiffness of the superstructure was calculated based on the desired fundamental period T_{FB} and knowing the building's mass. The superstructure yield point was estimated with two strength factors ($\alpha = 0.25$ and $\alpha = 1.0$). The use of a strength factor $\alpha = 0.25$ exhibits a highly nonlinear behaviour with a degrading pinched hysteretic behaviour under cyclic load reversals, which is governed by the nonlinear behaviour of the wooden crosslam structure fasteners. On the other hand a strength factor $\alpha = 1.0$ was selected to present the contrary superstructure models with no inelastic behaviour. This high strength factor can be linked with structures, which were highly overdesigned in the design process or are subjected to earthquakes with intensities lower than the design seismic intensity. Other parameters of the case study are presented in the Table **1**.

It was presumed in the study that the ductile behaviour of the connectors is achieved. The presentation of the hold down anchors and angle brackets can be obtained from the foundation detail presented in Fig (**4**). The ductile failure mechanism in wooden crosslam structures is characterized by plasticization of connectors (hold-downs, angle brackets and screws) between adjacent wall panels and between panels and foundations [20]. The crosslam panels and the connections between adjacent floor panels must be designed for the overstrength of the connectors to ensure that they

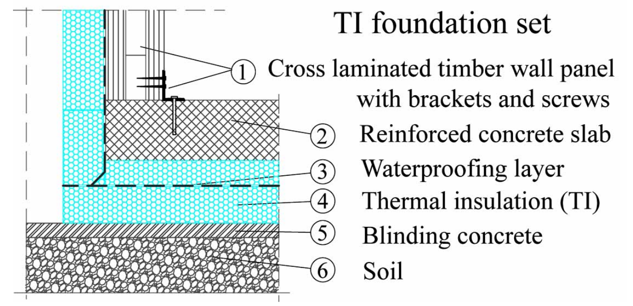

Fig. (**4**). The detail of a TI layer with XPS boards and inserted waterproofing (HI) foil.

remain elastic during the earthquake and the ductile failure mechanism is attained.

For presenting results in Fig. (**5**) and Fig. (**6**) bar graph representation was used. The response of the structures was presented by 5 main engineering design parameters (EDPs), where each of the EDP is presented by a single bar column in the bar graph representation. The coloured bar columns belong to the response of BI models, where the black bar columns show the response of the corresponding EDP for FB models. Detailed descriptions of the EDPs are given in the Table **2**. In order to show all of the EDPs on the same axis of bar graph presentation, a scaling of results with an appropriate reference value was obtained. Ultimate values for each EDP are presented with a dashed line surrounding the bar graph. For EDPs that exceed the ultimate values, the bar graph column crosses the dashed line, meaning that one of the failure mechanisms was reached.

4.2. Results and Discussion

The analyzed results are presented in Fig. (**5**) and Fig. (**6**). The initial lateral strength of the superstructures in the study is defined by a strength factor α ($\alpha = F_y/W$; W-weight of the entire superstructure). The results in Fig. (**5**) represent models with an expected inelastic behaviour of the superstructure under severe earthquakes ($\alpha = 0.25$). On the other hand the structures showed in Fig. (**6**) expose very high overstrength values and remain in elastic state even under severe earthquakes ($\alpha = 1.0$). Each row of the figures represents a structure with different number of storeys (n) and each column the response of the same structure exposed to a different peak ground acceleration (a_g). All together 24 structures differing in the foundation conditions (FB and BI models) and number of storeys $(n = 2 - 7)$ have been analyzed for 4 different a_g values ($a_g = 0.1 - 0.4$ g). In Table **3** the calculated fundamental periods for all FB (T_{FB}) and BI models (T_{BI}) are presented in accordance with the number of storeys. It can be seen that the fundamental periods of the BI buildings are always longer than those of the FB buildings (the ratio T_{BI}/T_{FB} is always larger than 1.0), and that the differences are bigger in the case of buildings with a larger number of storeys. The reasons for the increase in the fundamental periods are the vertical (rocking) and horizontal (shear) deformability of the TI layer.

Table 2. The output EDPs, together with their definitions and domains.

	Label	Description	Ultimate Values	Remarks
FB and BI models	D_{top}	Absolute top displacement [cm]	$\frac{H}{300}$	The chosen limit value $H/300$ for the serviceability limit state is presented in EC0 [48] for the FB models, and is therefore only a reference value for the BI models.
	μ_d	Ductility demand ($\mu_d = \theta/\theta_y$)	$\mu_c = 3$	The obtained value of μ_d can exceed the selected ultimate value, which in practice corresponds to structural damage.
	f	Shear/friction demand ($f = \frac{Q}{W}$); where Q is a base shear force	0.27 (XPS-HI)	The friction between the XPS with the insertion of a waterproofing layer (HI) was the lowest measured experimental value [39]. The detail is shown in Fig. (**4**).
only BI models	ϵ	Deformation in the TI spring at the edge of the foundation slab	$\epsilon_y = 2.1\%$	ϵ_y represents the experimentally determined yield deformation at 2.1% (XPS400) [39]. Because a stiff ground floor slab was assumed, the largest ϵ appears at the edge.
	T_p	Percent of foundation not in contact with the TI layer [% $A \cdot B$]	50%	For structures reaching a T_p of 50% or more, EC7 [49] regulates special precautions in the design of the foundation, as it is exposed to loads with large eccentricities.

Table 3. The fundamental vibrational periods for FB and BI models.

n	**2**	**3**	**4**	**5**	**6**	**7**
T_{FB}	0.192	0.260	0.322	0.381	0.437	0.490
T_{BI}	0.209	0.283	0.365	0.445	0.521	0.610
T_{BI}/T_{FB}	1.086	1.088	1.134	1.167	1.193	1.245

It can be seen from both figures that the differences between the FB and BI model response cannot be neglected. The BI models' values are almost in all cases larger than the corresponding values for the same superstructure founded on a fixed base. The changes between the response of both corresponding models are dependent on the number of storeys (n) and on the peak ground acceleration values (a_g). In the case of inelastic models ($\alpha = 0.25$), the amplifications in the BI models response of compared to the FB models amount up to 30% for D_{top} values, 25% for μ_d and even up to 260% for f. On the other hand in the case of elastic models ($\alpha = 1.0$) the amplifications are up to 40% for D_{top} values and much lower for f (up to 40%), whereas for ductility demand even some deamplifications were noticed, but it should be noted that the structures behave elastically in all investigated cases (μ_d remains under 1.0).

For structures presented in Fig. (**5**) it is assumed that they are designed according to modern seismic design principles, and are therefore capable of absorbing energy by plasticization of metal connections, used to connect the crosslam timber panels. The majority of new wooden structures should be built in this way, unless they are not overdesigned for any of the other reasons which might apply in the low-energy building design process. In these cases we might expect much more significant inelastic behaviour of the superstructure, which would be reflected in significantly differing impacts on the TI under the foundation slab. The comparison of D_{top} shown in Fig. (**5**) and Fig. (**6**) does not indicate significant differences and can be thus concluded that the equal displacement rule of elastic and inelastic structures was confirmed. The top displacements of the BI models are always larger than of the FB models. The main reasons for the increase of the top displacements are: (a) the deformation of the superstructure, (b) the rotation of the foundation slab, (c) the horizontal shear displacement of the TI layer, and (d) the possibility that the BI model captures the influence of the uplifting of the foundation slab.

The main difference between the elastic and inelastic response is in the superstructure's ductility demand (μ_d). For a realistic structure the available ductility capacity is usually much greater than the design ductility capacity (μ_c). However, for larger values of μ_c, i.e. values above 10, the structural collapse of these structures is very likely. In the presented case study the selected design ductility capacity was set to 3. It can be seen that the initial lateral stiffness is a parameter, which significantly influences μ_d. For stiffer inelastic structures shown in Fig. (**5**) the μ_C can be exceeded for earthquakes with higher peak ground accelerations (i.e. $a_g > 0.30$ g). In the case of presented wooden crosslam buildings the lowest initial lateral stiffness is reached for low-rise structures (e.g. $n < 4$), where the most probable failure mechanism is the inelastic collapse of the superstructure. On the other hand from the Fig. (**6**) it can be seen that μ_d is lower than the ductility capacity (μ_c) in all cases. This means that the load carrying capacity of a

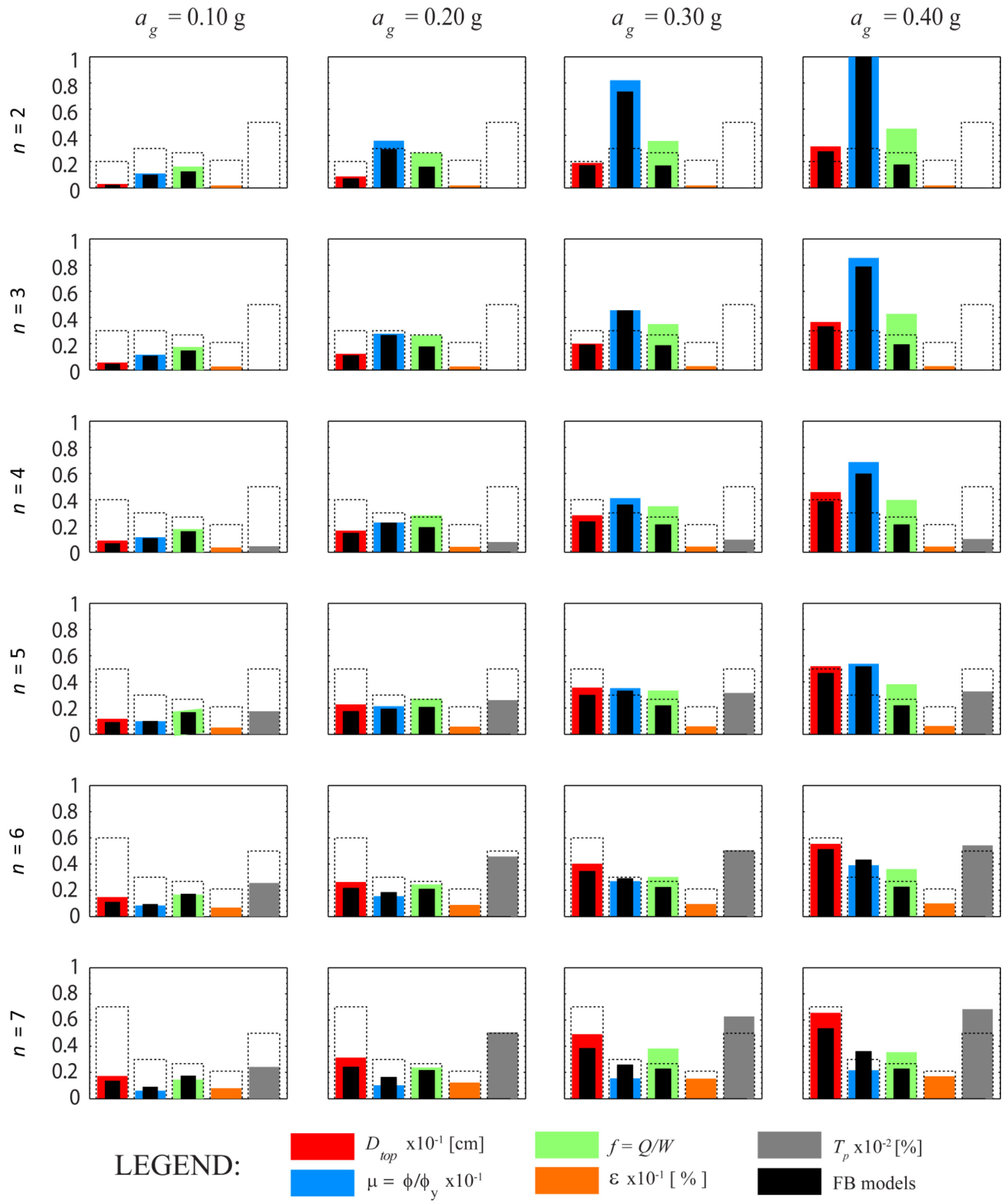

Fig. (5). The comparison of the fixed base (FB) and base isolated (BI) inelastic ($\alpha = 0.25$) building model response.

superstructure with $\alpha = 1.0$ is much greater than that needed by the considered earthquake ground motions.

The green bar column presents the ratio between the calculated seismic base shear and the superstructure weight together with the ground floor. In the case of the BI models f presents the seismic demand for the friction force in the TI foundation set composed of different constituent layers (RC foundation slab, XPS layer, HI layer, layer of blinding concrete, etc.). The friction coefficient demand parameter indicates the possibility of uncontrolled horizontal sliding at the critical contact plane between different layers of the TI foundation set. It can be seen that the friction force acting on the foundations is much smaller for the inelastic models ($\alpha = 0.25$), at the expense of the nonlinear behaviour of the wooden crosslam superstructure. In our particular case the calculated values for the inelastic structures remain below 0.45, which is much lower than for elastic structures ($\alpha = 1.0$), where the values can be more than twice as high. Furthermore, the value f seems to be dependent on the number of storeys (n), where buildings with lower number of storeys reveal as critical ones (i.e. the highest value of f is reached). In the case of a TI foundation set with a

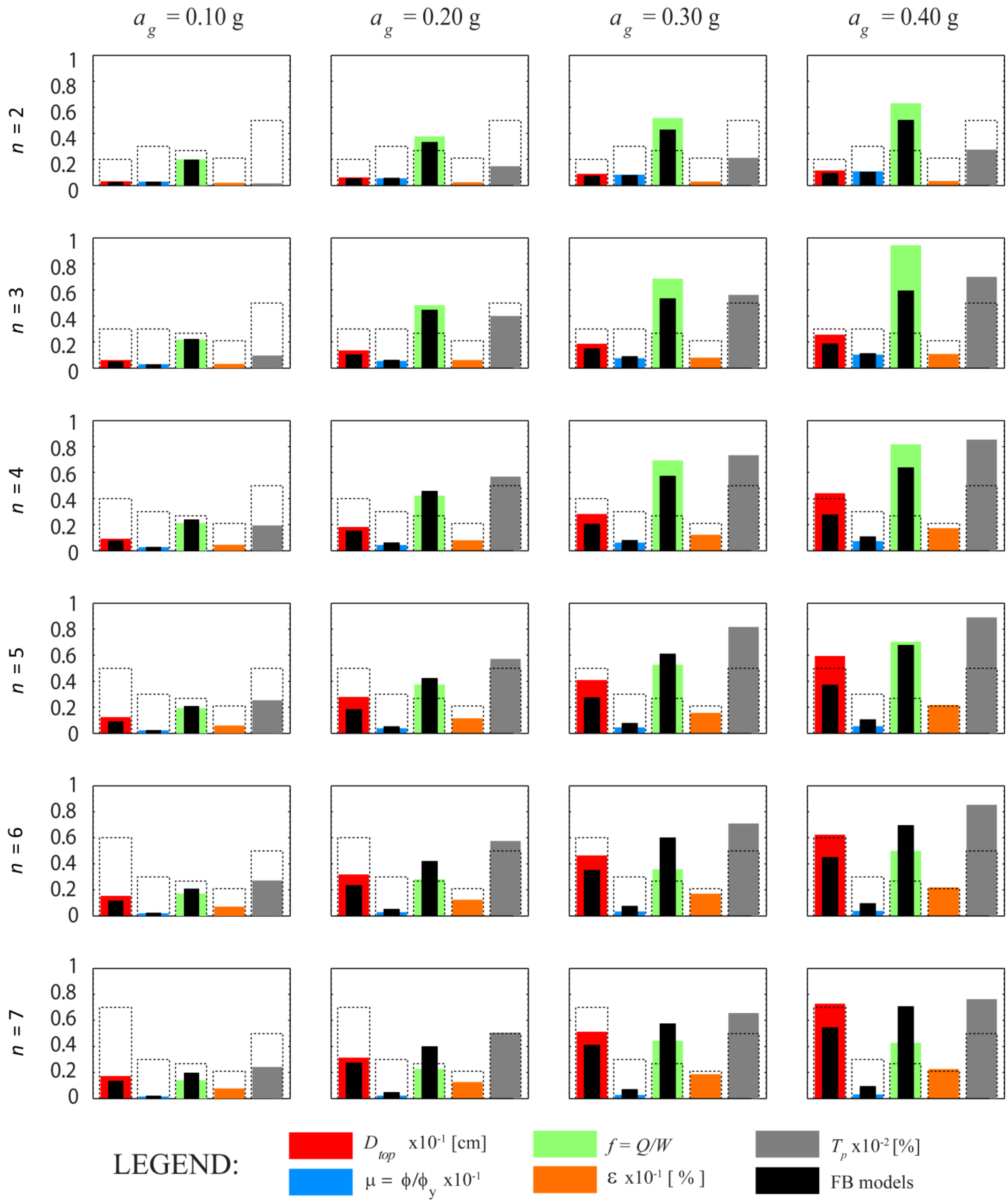

Fig. (**6**). The comparison of the fixed base (FB) and base isolated (BI) elastic ($\alpha = 1.0$) building model response.

waterproofing layer between the XPS boards (Fig. (**4**)), the experimentally determined limit value for the friction coefficient is very low ($f < 0.27$) [39]. This means that most of the light-mass wooden crosslam buildings founded on a TI layer with intermediate waterproofing layer would slip from its initial position in the case of a strong earthquake. This phenomenon could be avoided with a use of an improved TI foundation set of higher friction value and/or by increasing lateral support of the foundation plate. However, this phenomenon could also be used as a seismic fuse, which would prevent significant structural damage or any other failure mechanism. If the horizontal displacement of the buildings' base would be controlled, then the mechanism of exceeded friction could have a similar positive influence as the seismic base isolation devices.

In the case of stronger seismic excitations, the maximum compressive stress in the thermal insulation layer could exceed the material's nominal compressive strength, indicating irreversible damage to the TI, as well as the possibility of permanent leaning of the building to one side. The TI vertical compressive deformation (ϵ) of the elastic and inelastic models almost in all cases remains in elastic state for the selected material XPS400. The material yield deformation ($\epsilon_y = 2.1\%$) is exceeded only in the case of elastic high-rise buildings ($n > 5$) for severe earthquakes ($a_g = 0.40$ g). For inelastic structures ($\alpha = 0.25$), ϵ_y is never exceeded. In general it can be concluded that light-mass wooden buildings are not exposed to permanent (inelastic) vertical deformation of the TI layer. For wooden buildings with higher mass (e.g. buildings with higher number of storeys) the exceedance of ϵ_y and the occurrence of permanent vertical displacement in the TI layer could be prevented by the use of a material with higher compressive strength (e.g. XPS and cellular glass boards with higher nominal compressive strength than XPS400 used in the study). However, a deep nonlinear behaviour of the TI layer has been noticed in case of heavier and slender buildings – see the extensive parametric study [11].

The grey bar columns represent the maximum part of the foundation slab, that is not in contact with the TI layer (T_p) during the dynamic analysis. The rocking phenomenon [37] causes the structure's foundation slab to uplift, and at this particular moment only one part of the foundation slab is in contact with the TI layer. Furthermore the foundation slab uplift could also be the consequence of irreversible TI layer compressive deformation, which might cause permanent gap between the edge of the foundation slab and the TI layer. Slender structures (higher $H{:}B$ ratio) with a larger number of storeys (n) are more exposed to rocking, and so reach very high values of T_p. The limit value used from EC7 (i.e. 50%) is presented only as an orientation value, although for structures that exhibit T_p above the limit value, overturning in the case of seismic loads is quite possible. The structures which seem to perform in a very stable fashion in the sense of overturning are those with lower number of storeys ($n < 4$) and low strength factor (i.e. $\alpha = 0.25$). For these structures only 10% or less of the foundation slab is exposed to uplift even under severe earthquakes. In general, for inelastic buildings ($\alpha = 0.25$) problems with overturning (limit T_p is exceeded) would appear only for buildings with ratio $H{:}B \geq 2.4$ ($n \geq 6$) exposed to higher earthquake ground motions ($a_g > 0.30$ g). On the other hand, in the case of elastic superstructures ($\alpha = 1.0$), overturning is a much more significant problem. From the Fig. (**6**) it can be seen that the limit value of these structures can be exceeded already for the ratio $H{:}B > 1.2$, which could mean that overturning of low-rise buildings is also plausible.

CONCLUSION

Article presents the critical issues of energy efficient construction on seismic safety of low-energy and passive houses. Different critical details of low-energy construction were recognized, where the foundation on the TI layer has been pointed out as one of the most problematic. Furthermore, the case study of regular and symmetric wooden crosslam buildings, founded on a TI layer, has been presented. Nonlinear time history analyses have been carried out on 24 different models, differing in the number of storeys, structural strength capacity and boundary conditions of the foundations (FB and BI models). Considering the limitations of the case study (superstructure with floor plan $A/B = 15/7.5$ m modelled by a simplified SDOF model, stiff ground floor slab lying on XPS layer and firm soil) the main findings of the study are presented as follows:

- The calculated EDPs for BI models are almost in all cases larger than those from FB models. The amplifications are in extreme cases up to 40% for top (roof) displacement, up to 25% for ductility demand and even up to 260% for friction demand coefficient. In general, the differences are larger in the case of high-rise than in the case of low-rise buildings. The large amplifications, due to the vertical and shear deformability of the XPS layer, can be significant in the seismic design of wooden crosslam buildings and cannot be neglected.
- In the case of wooden buildings presented in the study, the yield compressive deformation of the XPS layer (XPS400) is rarely exceeded, because the main parameter influencing the deformability is the building's weight. In our study the exceedance could only be expected in the case of wooden structures with a larger number of storeys ($n > 5$) and subjected to severe earthquakes ($a_g \approx 0.40$ g). It can be concluded that light-weight wooden buildings are not exposed to permanent (inelastic) vertical deformation of the TI layer.
- The study has shown that friction coefficient demand is the most critical EDP for wooden crosslam buildings. The friction problems are more critical in the case of low-rise structures, which are lighter than the higher structures. It was also shown that the friction demand is much smaller (up to two times) in the case of inelastic models, at the expense of the nonlinear behaviour of the superstructure. The use of a XPS layer with the inserted waterproofing foil (friction capacity lower than 0.27 [39]) has proved to be inadequate in the case of earthquake intensities larger than 0.20 g.
- Regarding the building's maximum compressive deformation in the XPS layer, the percentage of the uplifted foundation and the friction coefficient demand, the results have shown that the potentially negative influences are more critical for elastic than for inelastic superstructure behaviour. Namely, the inelastic behaviour of the superstructure appears to be favourable for the TI layer since it reduces the forces and transfers smaller moments onto the foundations.
- With proper detailing of the TI layer (the use of TI materials with high compressive strength and TI foundation sets with friction capacity above 0.50), the problems of uncontrolled sliding and rocking are not critical for wooden buildings on areas with seismic intensity lower than 0.20 g.

CONFLICT OF INTEREST

The authors confirm that this article content has no conflict of interest.

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REFERENCES

[1] Directive 2010/31/EU of the European parliament and of the council of 19 May 2010 on the energy performance of buildings, 2010.

[2] X. Dequaire, "Passivhaus as a low-energy building standard: contribution to a typology," *Energy Efficiency*, vol. 5, pp. 377-391, August 2012.

[3] W. Feist, "Life-cycle energy analysis: low-energy house, passive house, self-sufficient house," In: *International symposium of CIB* W67, Vienna, Austria, 1997, pp. 183-190.

[4] W. Feist, Wärmebrücken und Tragwerksplanung - die Grenzen des Wärmebrückenfreien Konstruirens. Darmstadt: Passivhaus Institute, 2007.

[5] V. Kilar, D. Koren, and M. Zbašnik-Senegačnik, "Seismic behaviour of buildings founded on thermal insulation layer," *Građevinar*, vol. 65, pp. 423-433, 2013.

[6] M. Zbašnik-Senegačnik, "Large buildings built as passive houses," *Građevinar*, vol. 63, pp. 903-906, 2011.

[7] T. Heidolf and R. Eligehausen, "Design concept for load bearing thermal insulation elements with compression shear bearings," *Beton- Und Stahlbetonbau*, vol. 108, pp. 179-187, 2013.

[8] A. Tena-Colunga, "Review of the soft first story irregularity condition of buildings for seismic design," *Open Civil Engineering Journal*, vol. 4, pp. 1-15, 2010.

[9] D. Koren, V. Kilar, and M. Zbašnik Senegačnik, "Seismic safety of passive houses founded on thermal insulation," In 17th *International Passive House Conference* 2013, Frankfurt am Main, 2013.

[10] D. Koren and V. Kilar, "Seismic vulnerability of reinforced concrete building structures founded on an XPS layer," Submitted for publication in Earthquakes and Structures, *An Int'l Journal*, 2014.

[11] B. Azinović, D. Koren, and V. Kilar, "The seismic response of low-energy buildings founded on a thermal insulation layer - a parametric study," Submitted for publication in *Engineering Structures*, 2014.

[12] V. Kilar and D. Koren, "Seismic behaviour of asymmetric base isolated structures with various distributions of isolators," *Engineering Structures*, vol. 31, pp. 910-921, 2009.

[13] A. Tena-Colunga, "Seismic design of base-isolated buildings in mexico. part 1: guidelines of a model code," *Open Civil Engineering Journal*, vol. 7, pp. 17-31, 2013.

[14] C. Christopoulos, A. Filiatrault, and V. V. Bertero, Principles of passive supplemental damping and seismic isolation: *IUSS Press*, 2006.

[15] M. K. Kuzman, P. Grošelj, N. Ayrilmis, and M. Zbašnik-Senegačnik, "Comparison of passive house construction types using analytic hierarchy process," *Energy and Buildings*, vol. 64, pp. 258-263, 2013.

[16] J. W. van de Lindt, S. E. Pryor, and S. Pei, "Shake table testing of a full-scale seven-story steel-wood apartment building," *Engineering Structures*, vol. 33, pp. 757-766, 2011.

[17] A. Ceccotti, C. Sandhaas, M. Okabe, M. Yasumura, C. Minowa, and N. Kawai, "SOFIE project - 3D shaking table test on a even-storey full-scale cross-laminated timber building," *Earthquake Engineering & Structural Dynamics*, vol. 42, pp. 2003-2021, 2013.

[18] A. Filiatrault, I. P. Christovasilis, A. Wanitkorkul, and J. W. van de Lindt, "Experimental seismic response of a full-scale light-frame wood building," *Journal of Structural Engineering*, vol. 136, pp. 246-254, 2009.

[19] J. W. van de Lindt, S. Pei, S. E. Pryor, H. Shimizu, and H. Isoda, "Experimental seismic response of a full-scale six-story light-frame wood building," *Journal of Structural Engineering*, vol. 136, pp. 1262-1272, 2010.

[20] M. Fragiacomo, B. Dujic, and I. Sustersic, "Elastic and ductile design of multi-storey crosslam massive wooden buildings under seismic actions," *Engineering structures*, vol. 33, pp. 3043-3053, 2011.

[21] A. Filiatrault and B. Folz, "Performance-based seismic design of wood framed buildings," *Journal of Structural Engineering*, vol. 128, pp. 39-47, 2002.

[22] A. Filiatrault, H. Isoda, and B. Folz, "Hysteretic damping of wood framed buildings," *Engineering Structures*, vol. 25, pp. 461-471, 2003.

[23] Y. F. Zhang and X. B. Hu, "Self-centering seismic retrofit scheme for reinforced concrete frame structures: SDOF system study," *Earthquake Engineering and Engineering Vibration*, vol. 9, pp. 271-283, 2010.

[24] A. Ayoub and M. Chenouda, "Response spectra of degrading structural systems," *Engineering Structures*, vol. 31, pp. 1393-1402, 2009.

[25] G. D. Hatzigeorgiou and D. E. Beskos, "Inelastic displacement ratios for SDOF structures subjected to repeated earthquakes," *Engineering Structures*, vol. 31, pp. 2744-2755, 2009.

[26] G. D. Hatzigeorgiou, G. A. Papagiannopoulos, and D. E. Beskos, "Evaluation of maximum seismic displacements of SDOF systems from their residual deformation," *Engineering Structures*, vol. 33, pp. 3422-3431, 2011.

[27] D. Cardone, A. Flora, and G. Gesualdi, "Inelastic response of RC frame buildings with seismic isolation," *Earthquake Engineering & Structural Dynamics*, vol. 42, pp. 871-889, 2012.

[28] M. Dicleli and S. Buddaram, "Comprehensive evaluation of equivalent linear analysis method for seismic-isolated structures represented by sdof systems," *Engineering Structures*, vol. 29, pp. 1653-1663, 2007.

[29] S. Mahmoud, P.-E. Austrell, and R. Jankowski, "Simulation of the response of base-isolated buildings under earthquake excitations considering soil flexibility," *Earthquake Engineering and Engineering Vibration*, vol. 11, pp. 359-374, 2012.

[30] M. Nakhaei and M. Ali Ghannad, "The effect of soil-structure interaction on damage index of buildings," *Engineering Structures*, vol. 30, pp. 1491-1499, 2008.

[31] M. Moghaddasi, M. Cubrinovski, J. G. Chase, S. Pampanin, and A. Carr, "Probabilistic evaluation of soil-foundation-structure interaction effects on seismic structural response," *Earthquake Engineering & Structural Dynamics*, vol. 40, pp. 135-154, 2011.

[32] M. A. Ghannad and A. H. Jafarieh, "Inelastic displacement ratios for soil-structure systems allowed to uplift," *Earthquake Engineering & Structural Dynamics*, vol. 43, no. 9, pp. 1401-1421, 2014.

[33] I. Anastasopoulos and T. Kontoroupi, "Simplified approximate method for analysis of rocking systems accounting for soil inelasticity and foundation uplifting," *Soil Dynamics and Earthquake Engineering*, vol. 56, pp. 28-43, 2014.

[34] R. Kourkoulis, I. Anastasopoulos, F. Gelagoti, and P. Kokkali, "Dimensional analysis of sdof systems rocking on inelastic soil," *Journal of Earthquake Engineering*, vol. 16, pp. 995-1022, 2012.

[35] K. Goda, H. P. Hong, and C. S. Lee, "Probabilistic Characteristics of Seismic Ductility Demand of SDOF Systems with Bouc-Wen Hysteretic Behavior," *Journal of Earthquake Engineering*, vol. 13, pp. 600-622, 2009.

[36] H. Hong and P. Hong, "Assessment of ductility demand and reliability of bilinear single-degree-of-freedom systems under earthquake loading," *Canadian Journal of Civil Engineering*, vol. 34, pp. 1606-1615, 2007.

[37] F. Gelagoti, R. Kourkoulis, I. Anastasopoulos, and G. Gazetas, "Rocking-isolated frame structures: Margins of safety against toppling collapse and simplified design approach," *Soil Dynamics and Earthquake Engineering*, vol. 32, pp. 87-102, 2012.

[38] K. Faramarz, S. Mehdi, and P. Farzane, "P-delta effects on earthquake response of structures with foundation uplift," *Soil Dynamics and Earthquake Engineering*, vol. 34, pp. 25-36, 2012.

[39] V. Kilar, D. Koren, and V. Bokan-Bosiljkov, "Evaluation of the performance of extruded polystyrene boards – implications for their application in earthquake engineering," *Submitted for publication in (Elsevier) Polymer Testing* 2014.

[40] F. McKenna, G. L. Fenves, M. H. Scott, and B. Jeremic. (2000, 3.3.). OpenSees-Open system for earthquake engineering simulation. Available at: http://opensees.berkeley.edu/
[41] B. Folz and A. Filiatrault, "Cyclic analysis of wood shear walls," *Journal of Structural Engineering*, vol. 127, pp. 433-441, 2001.
[42] A. Ayoub, "Seismic analysis of wood building structures," *Engineering Structures*, vol. 29, pp. 213-223, 2007.
[43] G. C. Foliente, "Hysteresis modeling of wood joints and structural systems," *Journal of Structural Engineering*, vol. 121, pp. 1013-1022, 1995.
[44] N. Ambraseys, P. Smit, R. Sigjbjornsson, P. Suhadolc, and B. Margaris. (2002, 17.9.). Internet-Site for European Strong-Motion Data. Available: http://www.isesd.hi.is/ESD_Local/frameset.htm
[45] I. Iervolino, C. Galasso, and E. Cosenza, "REXEL: computer aided record selection for code-based seismic structural analysis," *Bulletin of Earthquake Engineering*, vol. 8, pp. 339-362, 2010.
[46] Eurocode 8: Design of strucures for earthquake resistance - Part 1-1: General rules, seismic actions and rules for buildings, *CEN*, 2005.
[47] A. Ceccotti, M. Follesa, N. Kawai, M. Lauriola, C. Minowa, and C. Sandhaas, "Which seismic behaviour factor for multi-storey buildings made of cross-laminated wooden panels," In: *Proceedings of 39th CIB W18 Meeting*, paper, 2006, p. 4.
[48] Eurocode 0: Basis of structural design, CEN, 2004.
[49] Eurocode 7: Geotechnical design - Part 1: General rules, *CEN*, 2005.

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