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Buildings Founded on Thermal Insulation Layer Subjected to Earthquake Load

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Abstract**—**The modern energy-efficient houses are often founded on a thermal insulation (TI) layer placed under the building's RC foundation slab. The purpose of the paper is to identify the potential problems of the buildings founded on TI layer from the seismic point of view. The two main goals of the study were to assess the seismic behavior of such buildings, and to search for the critical structural parameters affecting the response of the superstructure as well as of the extruded polystyrene (XPS) layer. As a test building a multi-storeyed RC frame structure with and without the XPS layer under the foundation slab has been investigated utilizing nonlinear dynamic (time-history) and static (pushover) analyses. The structural response has been investigated with reference to the following performance parameters: i) Building's lateral roof displacements, ii) Edge compressive and shear strains of the XPS, iii) Horizontal accelerations of the superstructure, iv) Plastic hinge patterns of the superstructure, v) Part of the foundation in compression, and vi) Deformations of the underlying soil and vertical displacements of the foundation slab (i.e. identifying the potential uplift). The results have shown that in the case of higher and stiff structures lying on firm soil the use of XPS under the foundation slab might induce amplified structural peak responses compared to the building models without XPS under the foundation slab. The analysis has revealed that the superstructure as well as the XPS response is substantially affected by the stiffness of the foundation slab.

Keywords**—**Extruded polystyrene (XPS), foundation on thermal insulation, energy-efficient buildings, nonlinear seismic analysis, seismic response, soil–structure interaction.

I. Introduction

THE new directives and standards about rational and efficient energy use strictly regulate that the thermal bridges have to be avoided and the thermal insulation (TI) layer should run without interruptions all around the building – even under the building or its foundations. For the TI layer various insulating materials such as cellular glass gravel, cellular glass boards, extruded polystyrene (XPS) boards and other materials with high compressive strength and water resistance, minimized creep, and durability can be applied. From the earthquake resistance engineering design viewpoint such buildings, already developed for non-earthquake prone areas, might not perform well in earthquake prone areas, and their suitability in such areas needs to be verified, and appropriate solutions found. At present, however, there is a lack of research dealing with the field of seismic response of buildings on TI layer. If we limit on the thermal insulation made of extruded polystyrene foam (XPS), the available studies concentrate on the material behavior, mostly from the energy efficiency point of view [1]–[3]. The XPS material has been investigated also in some special applications as a part of a vibration isolating screen installed in the soil near a test public transport track [4]. The study on the relation between the XPS foam microstructure and its response under compression stresses has been done in [5]. The XPS long-term mechanical properties (compressive creep strains and modulus) that are of key importance for thermal insulation beneath foundations have been analyzed in [6]. It should be noted that the XPS time-dependent behavior could substantially affect the structural seismic response. Due to the fact that the long-term mechanical properties of the XPS are significantly reduced compared to the short-term ones the response of the upper building as well of the XPS layer bellow the foundation slab could be more critical in the case of earthquake occurrence many years after the building was built. Additionally, in the same reference also the modeling of the foundation slab resting on thermal insulation layer has been schematically indicated.

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Some findings about the seismic response of the building structures founded on TI layer can be found in the recent studies of the authors of the current paper [7]–[11]. A preliminary study on the seismic response of such buildings modeled as a rigid box has been presented in [7], [8]. The authors conclude that in general the seismic safety of passive houses with the height up to 2 or 3 stories is not of critical concern. For higher (or slenderer) buildings however, the negative effects of insulation layer beneath foundation slab are more important. It should be noted that the insertion of the TI layers under the building's foundation changes its dynamic characteristics. As a consequence of the shear and vertical deformability of the TI layer between the RC foundation slab and blinding concrete on the ground, the fundamental period of the structure is increased. For most of energy-efficient or passive houses that are built as low-rise buildings with short fundamental periods this elongation might result in increased top accelerations and potential damage of the structure (i.e. the structural period of vibration is moved into resonance part of the Eurocode 8 response spectrum) – Fig. 1. In [10] the cyclic behavior of extruded polystyrene foam (XPS) in compression as well in shear has been experimentally researched. In the same reference, also the shear behavior of different composed thermal insulation (TI) foundation sets has been investigated and their friction capacity estimated. The obtained experimental data have been used also in the case study presented in the current paper. An extensive parametric study of seismic behavior of buildings founded on deformable thermal insulation layers has been presented in [9] where the upper structure was idealized by the single degree of freedom (SDOF) nonlinear system with stiff foundation slab lying on the XPS layer without taking into account the soil-structure interaction.

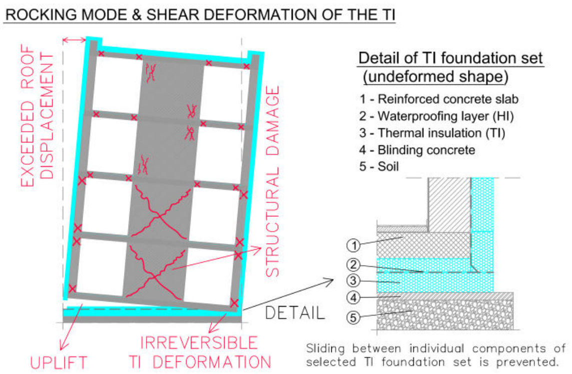

Fig. 1 Foundation of buildings on thermal insulation layer(s)and their potential seismic response

In the presenting paper the effect of implementing the XPS layer bellow the building's foundation slab on the seismic behavior of the XPS layer as well as the upper building structure has been analyzed. For this reason nonlinear dynamic and static analyses were performed on various 2D multi-storeyed RC frame structures with and without the XPS layer beneath the foundation slab of different stiffness. The complete nonlinear building models were assumed to be lying on real soil with the soil-structure interaction taken into consideration and subjected to a group of seven European ground motions normalized to two earthquake intensities a_g = 0.25g and 0.35g.

II. Investigated Structural Models and Seismic Input

A. Superstructure

A four-storeyed RC frame building with rectangular plan layout was used as a test example (Fig. 2). The selected structure represents a typical simplified passive or low-energy office-building and is assumed not to have any structural irregularities in plan as well as in height. In the presented study only the building variants without underground basement have been analyzed. In preliminary analyses various superstructures differing in stiffness was investigated utilizing linear dynamic response spectrum analyses. The aim of preliminary analyses was the assessment of the stiffness that gives the critical response of the base-isolated (BI) model (i.e. models founded on TI layer) compared to the response of the corresponding fixed-base (FB) one. Different stiffnesses of the frame were achieved by changing the columns' cross section dimensions.

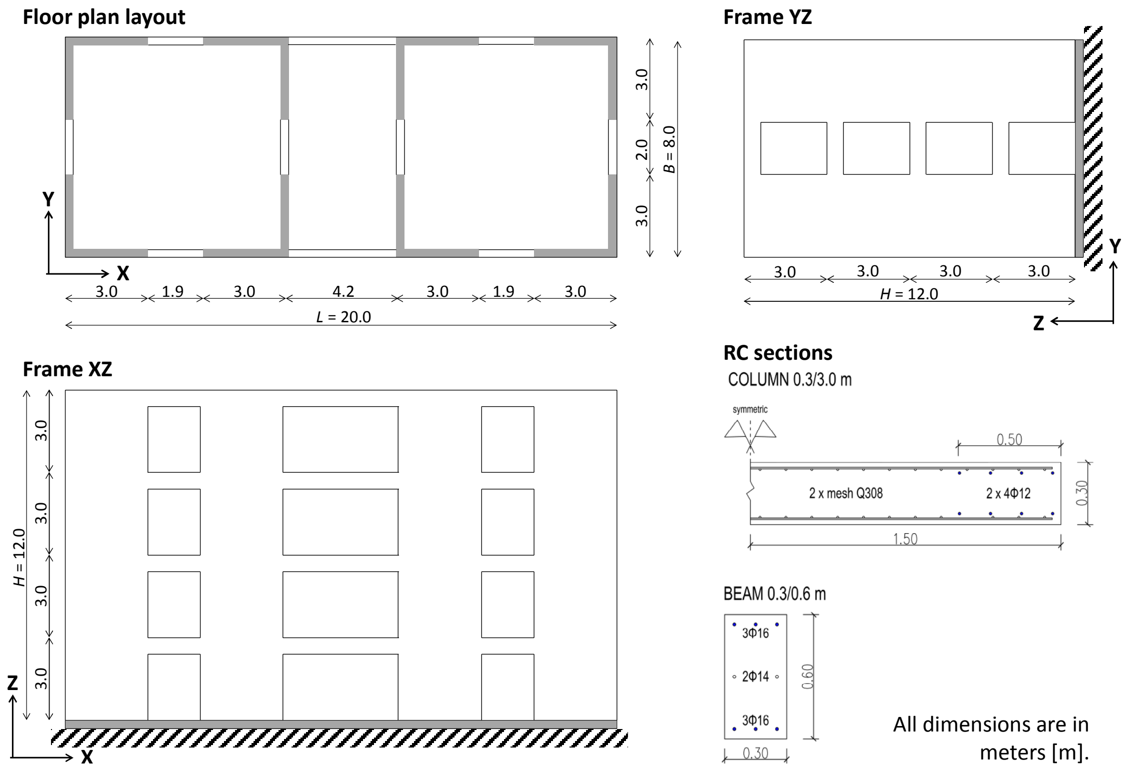

Fig. 2 Building's geometry with 2 characteristic frames and the cross sections with the longitudinal steel reinforcement considered in the study

Comparing the FB and BI structural response it was observed that the response amplifications of stiffer superstructures are more pronounced than the response amplifications of the flexible ones. The analyses have also shown that the fundamental periods of the BI models could be amplified more than 2 times in comparison to FB models. For these reasons the cross section dimensions for the further detailed analysis was chosen equal to 30/300 cm and 30/60 cm for the columns and beams, respectively. In order to study the effect of the building height, buildings with different number of storeys have been investigated. Fig. 2 presents the floor plan and two characteristic frames of the original (4-storeyed) structural variant, which is taken under observation in this paper. The effect of the foundation slab stiffness has been also researched by means of analyzing models with foundation slab of actual thickness (30 cm) and a variant with a foundation slab modeled as fully rigid. The fundamental periods of the FB and BI frames (on XPS400) are presented in Table I.

TABLE I
EFFECTIVE FUNDAMENTAL PERIODS [SEC] IN RELATION TO THE FOUNDATION SLAB STIFFNESS FOR FRAMES ON SOIL TYPE A

	Frame XZ		Frame YZ	
Foundation slab stiffness	FB	BI	FB	BI
Actual (d = 30 cm)	0.168	0.249	0.160	0.280
Fully rigid	0.165	0.208	0.158	0.270

The structural response was analyzed by means of the computer program SAP2000 [12]. All elements of the 2D load bearing structures (frame YZ and frame XZ) were modeled with frame line elements in the central lines of the actual cross sections. The connections between the beams and the central line of the walls were modeled by using rigid links within the walls' cross sections. Similarly, the RC foundation slab was also modeled with frame line element of rectangular shape with elastic behavior assumed. In order to obtain the sufficient density of the XPS and soil springs discretely modeled below the foundation slab the frame element was divided to sub-elements of equidistant length equal to 50 cm (central elements) and 25 cm (edge elements). The nonlinear behavior of the superstructure was modeled using a lumped plasticity model with plastic hinges at both ends of each beam and column. Two types of plastic hinges were investigated: (i) bending hinges (beams) and ii) combined axial-bending hinges (columns). For both the three-linear moment-curvature envelopes with a load drop were assumed (Fig. 3) with the initial (elastic part) stiffness taken equal to 0.5*EI* (cracked cross sections). In the figure the selected limit curvatures of the characteristics performance levels (IO, LS and CP)are also presented. The plastic hinge lengths were determined according to [13]. According to Eurocode 2 denotation the concrete C30/37 and the steel reinforcement S500 were used in the study. For the material cyclic behavior Takeda (concrete) and Kinematic (steel) hysteretic model were assumed. For the steel reinforcement of the selected concrete cross sections of the beams and columns the minimum longitudinal reinforcement for ductility class medium (DCM) according to EC8-1[14] was adopted (Fig. 2). The assumed reinforcement proved to be adequate for the design seismic intensity (design EC8 spectrum: a_g = 0.25g, soil A, q = 3).

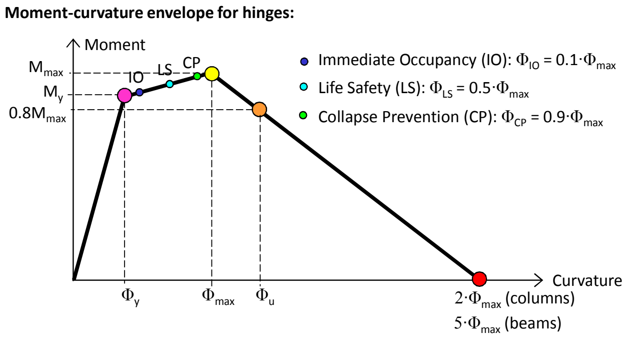

Fig. 3 The envelope of plastic hinges in columns and beams

B. Soil and XPS

The seismic soil-structure interaction (SSI) has been recognized to have possible detrimental effects on the behavior of the superstructure [15]–[18]. Thus, in the parametric study the SSI was taken into account and three different soil types (A, C and E) according to EC8 have been analyzed. For the soil density the constant value equal to ρ = 2000 kg/m^3 for all investigated soil types was assumed. The shear-wave (v_s) velocity were selected according to the EC8: v_s = 2500 m/s, 300 m/s and 100 m/s for soil type A, C and E, respectively). For the longitudinal- (primary-) wave (v_p) velocity the relationship $v_p = 2.5v_s$ was taken into account. The soil shear modulus (G) and Poisson ratio (v) were calculated according to the following expressions [17], [19]:

$$G = \rho \cdot v_s^2 v = \frac{0.5\cdot\left(\frac{v_p}{v_s}\right)^2 - 1}{\left(\frac{v_p}{v_s}\right)^2 - 1} \tag{1}$$

The behavior of the soil in shear was modeled by linear elastic springs. The soil springs were modeled as one joint link/support elements of MultiLinear Plastic – Kinematic type available in SAP2000 software with the stiffness [kN/m] corresponding to the effective area with the length of the finite elements equal to 0.50m and the considered effective width (0.5B for longitudinal (XZ) frame and 0.2L for transversal (YZ) frame) – see Fig. 2. Such discretization resulted in 40 and 16 springs for XZ and YZ frame, respectively. For the analyzed plan the dynamic stiffness coefficients [20] of values 1.0 and 1.25 were considered in the study for the vertical and longitudinal (k_Z, k_X) and for the lateral (k_Y) direction, respectively. The damping of the soil springs in both active directions (axial and shear) was neglected. Based on the fact that in practice the soil-structure contact tension resistance is not provided and that the foundation could partially uplift from the soil surface when the compressive stresses become tensile[21]–[23], the behavior of the soil in compression was modeled by nonlinear contact springs. Just for the comparisons also the structural models without modelling the soil were taken into account. In this case the joints of the foundation slab (FB models without soil) or the bottom joints of the springs presenting the TI layer (BI models without soil) were fully restrained and the potential uplifts were prevented.

For the XPS material behavior (in compression as well as in shear) the experimentally determined stress-strain curves [10], [11] were idealized and applied in the analyses. The XPS boards of two different nominal compressive strengths (400 and 700 kPa) were used in the analysis. In practice the TI foundation set is multilayered and consists of two or more XPS boards with the intermediate sheet (waterproofing insulation sheet or a polyethylene sheet) inserted. In the analysis the assumption that the sliding at contact between different layers is prevented was considered.

The base-isolated (BI) models were derived from the fixed-base (FB) ones by adding the XPS two-joint springs modeled in series with the one-joint soil springs bellow. The length of the XPS springs was equal to the XPS layer total thickness (24 cm). The axial cyclic behavior was modeled by kinematic hysteretic model while for the cyclic behavior in shear the Pivot hysteresis loop was assumed considering the following parameters: $\alpha_1 = \alpha_2 = 1.0$, $\beta_1 = \beta_2 = 0.25$ and $\eta = 0$. The effective stiffnesses were taken equal to the initial stiffness of the material. The rotational degree of freedom of the XPS spring was fixed.

III. Vertical and Seismic Loads

The structural analyses of the investigated frame systems were carried out by utilizing nonlinear static and dynamic (time-history) analysis response analysis. As the initial loads in the seismic analyses the vertical loads according to the seismic limit state defined in EC8 were assumed. In the 2D frame models the vertical loads were applied on all beams as uniform line loads equal to q_{sd} = 45.0, 45.4 and 56.9 kN/m for the base floor, interstorey floors and the roof, respectively. In seismic analyses the one-directional seismic excitation was applied. The vertical component of the seismic load was not applied in the presented study.

The nonlinear seismic response of the considered building models has been analyzed by the use of nonlinear static (pushover) analysis first. The lateral load distribution shape in pushover analyses corresponded to the mass proportional to the displacement of the 1st mode of the structure's eigen oscillation with initial stiffness considered. Besides the pushover analyses also the nonlinear dynamic analyses were applied. An ensemble of seven different ground motions with two different PGA levels (a_g = 0.25g and a_g = 0.35g) were considered. The selected ground motions were Eurocode 8 [14] spectrum-compatible, artificial accelerograms generated by the SYNTH program [24]. As a target spectrum, the EC8 elastic spectrum for selected soil type (A), scaled to a peak ground acceleration of a_g = 0.25g for a 5% damping ratio, was used. The original ground motion records that were used for the generation of artificial accelerograms are summarized in Table II. In the study only their N-S components were considered and applied in the direction of the analysis. The same group of ground motion records was used also in the authors' previous studies [25], [26].

In the nonlinear dynamic analyses the damping matrix was proportional to the mass and the initial stiffness matrices. 5% damping was assumed in the first and second mode of vibration. Damping of the soil as well as of the XPS layer was assumed to be negligible.

TABLE II
Characteristic Features of the Original Earthquake Records

State	Date	Station	a_g [g]	a_g/v_g [Hz]	Duration [s]
Italy	11.5.1976	Tolmezzo	0.349	16.9	15
Italy	15.9.1976	Forgaria	0.305	12.8	15
Montenegro	15.4.1979	Bar	0.364	8.7	25
Montenegro	15.4.1979	Petrovac	0.438	10.4	19.6
Montenegro	15.4.1979	Ulcinj	0.285	7.1	25
Bosnia and Herzegovina	13.8.1981	Banja Luka	0.516	21.1	10
Romania	30/31.8.1986	Focsani	0.279	7.5	21.7

IV. Seismic Response of the Analyzed Models

A. The Nonlinear Static Structural Response

By means of the nonlinear static (pushover) seismic analysis the global response of the selected building models has been evaluated. The target roof displacement was set equal to values from 10 to 25 cm depending on the stiffness of the system. Figs. 4 and 5present the selected results obtained by pushover analyses. As can be seen from the Fig. 4 the response of the building strongly depends on the direction of the analysis and the stiffness of the foundation slab. The latter was recognized to substantially affect the overall response of the structure, especially when the 3-bay frame (frame XZ) was taken under observation. If the foundation slab is fully rigid, the distribution of contact compressive stresses in the soil or XPS layer below is uniformly linear (triangular or rectangular) while in case of flexible foundation slab the contact stresses become discretely distributed. In Fig. 5 the distribution of the vertical base reaction forces are presented for the BI frame XZ with fully rigid and real (30 cm) foundation slab recorded at equal lateral roof displacements in the case of utilizing the pushover analysis. It should be noticed that at some parts of the foundation slab the vertical reaction forces are zero, i.e. the uplift of the foundation slab occurred and only the part of foundation was in compression. The corresponding effective periods of vibration are shown in Table I. Comparing the response of the frame XZ and YZ it is obvious that the frame XZ is stronger and in extreme cases reaches the complete plastic mechanism, what does not occur in the case of analyzing the YZ frame. The only exception was the case of FB variant of the YZ frame without modeling the soil stiffness. In all other investigated models a deep nonlinear behavior of the soil or XPS layer occurred and the plastic mechanism in the upper frames did not form. In such case the upper structure behaves like (semi-)rigid box on soft layer and the overall response is governed by the stiffness of the bottom layer. In extreme situations also the global stability of the structure (potential overturning) was threatened. The effect of the soil type has also exposed to have an important role to the structural response. Comparing the response on firm (A) and soft (E) soil the initial stiffness of the pushover curve is substantially decreased while soft soil are taken into account. The stiffness of the model on soil type A is comparable to the stiffness of the model without the soil stiffness included (in the Fig. 4 the pushover curve for BI model on soil A coincides with the pushover curve for BI model without modeling soil). It should be noted that the models without modeling soil were assumed to be fully restrained and the potential uplift of the foundation slab was not taken into account. Observing the FB and BI models on different soils it can be pointed out that the BI models are more flexible and the differences are more distinct in case of foundations on firm soils. The differences between the FB and BI model on soil E are negligible. For this reason the results presented further on show the response of the structures on soil A.

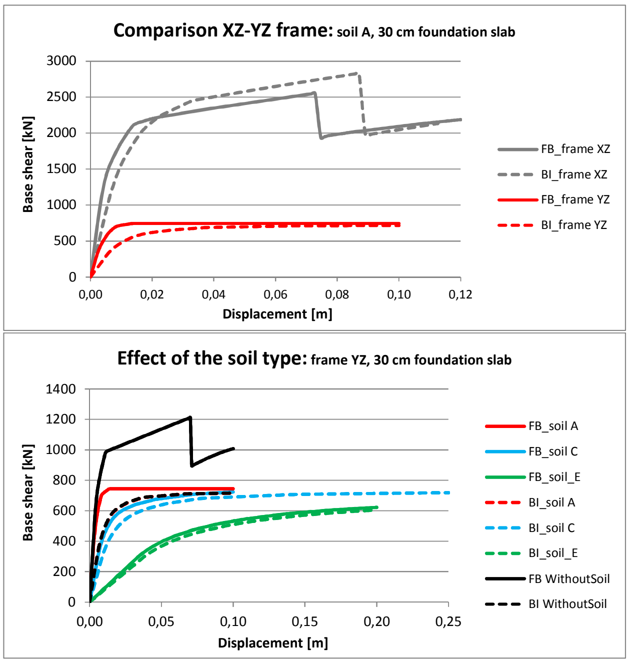

Fig. 4 The pushover response of 4-storey FB and BI (XPS400 $d = 24$ cm) frames XZ and YZ under different conditions

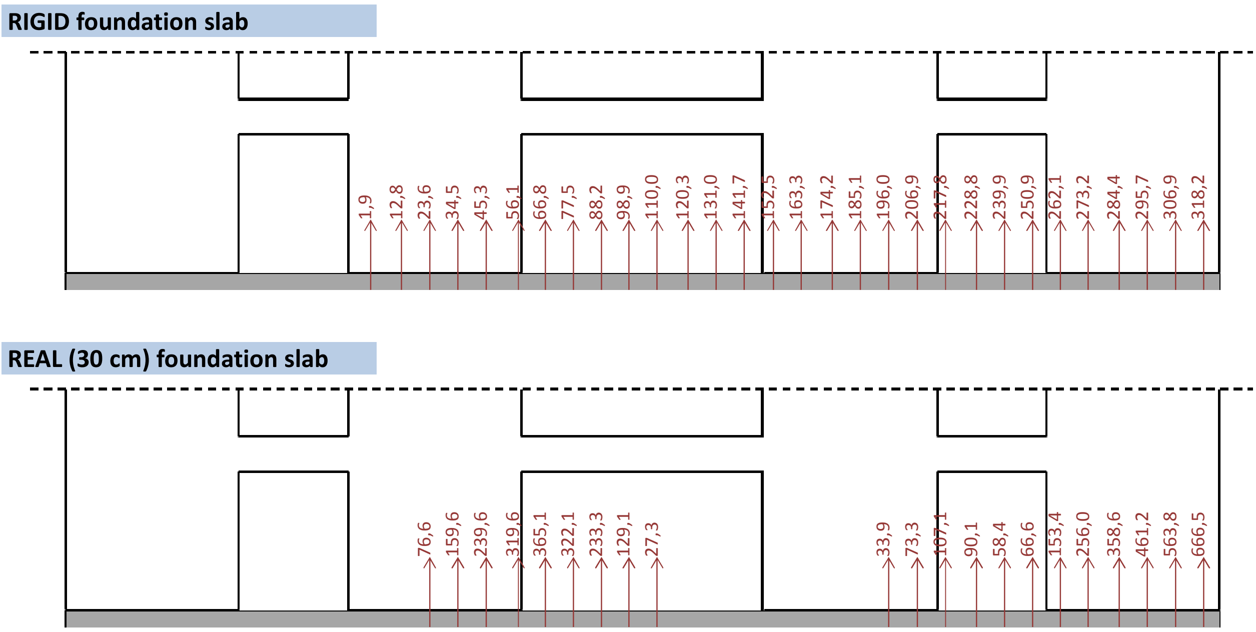

Fig. 5 The obtained vertical reaction forces [kN] patterns related to the stiffness of the foundation slab (BI frame XZ, soil A, $D_{x,\,roof} = 2.7$ cm)

B. The Nonlinear Dynamic Structural Response

The seismic response of the original structure (4-storey YZ and XZ frames, $a_g = 0.25$g, foundation slab 30 cm, XPS400 of thickness $d = 24$ cm) is presented in Figs. 6–8. The lateral roof displacements and maximum edge compressive deformations of the XPS layer in the case of BI frame YZ are presented in Fig. 6. The results are shown as the envelope values for each applied seismic ground motion record. As can be noticed from the figure the differences between the particular accelerogram are more distinct observing the building's lateral displacements, while in the case of surveying the obtained maximum compressive deformations in XPS layer a very good agreement of results was achieved. As can be seen from the figure, the obtained displacements in most cases do not exceed the limit values chosen equal to 0.33 % of the total building's height (H) and 2.1 % of the thickness of the TI (XPS) layer (d) for lateral roof displacement and XPS compressive deformation, respectively.

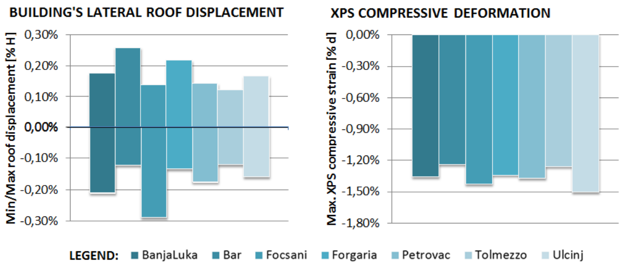

Fig. 6 The response of the 4-storey (H = 12.0 m) BI frame YZ in the relation to the applied seismic ground motion records: 0.25g, XPS400 d = 24 cm, foundation slab 30cm, soil A

In Fig. 7 the obtained typical damage patterns of the FB and BI frame YZ are presented together with the absolute maximum accelerations measured at different building's storey levels. The damage patterns show the performance levels (measured with the reference to the obtained curvatures in the generated plastic hinges – see Fig. 3) of the superstructure. It can be seen that the obtained damages for the considered seven ground motions are larger in the case of BI buildings. Moreover, in the analyzed FB frame structure no damage was recorded in columns, while there were some plastic deformations in the column(s) of the first storey of the BI frame. Comparing the response of BI and FB models in terms of the absolute maximum accelerations it was observed that the amplifications BI/FB are of significant value (~1.8)in the case of accelerations measured on the ground floor. Observing the BI/FB amplifications on the roof the values were actually the same. This means that the accelerations on the top of the ground floor slab are essentially larger in the case of buildings lying on the XPS layer. Comparing the soil (input) and the accelerations transferred to the top of the ground floor slab, it is obvious, that there are significant amplifications (~1.8) in the case of BI models, while in the case of FB models they are negligible (i.e. $a_{ground\ floor} \approx a_{soil}$ in the case of FB model). It should be mentioned that such a response is quite different than the response of structures base-isolated by means of seismic isolation devices (e.g. rubber bearings, friction-pendulum isolators, etc.) where the accelerations at the top of the ground floor are significantly reduced due to the considerable elongation of the fundamental period of vibration [25]–[27]. Additionally, for the structures base-isolated with seismic isolation devices the amplifications of the accelerations along the building's height are in general minimal what was to some extent observed also in our analysis case with structure isolated with XPS layer beneath the foundation slab. As can be seen from Fig. 7 the amplifications of the accelerations along the BI building's height ($a_{roof}/a_{g,floor}$) were around 1.5. Oppositely, observing the FB models these amplifications were considerably larger (~2.7). In the case of XZ frame, which for the sake of brevity is not shown herein, the recorded amplifications of the accelerations were even larger.

The time-history diagrams of the base shear for the YZ frame subjected to selected accelerogram (Banja Luka) are shown in Fig. 8. Comparing the response of the FB and BI models, the maximum values of the base shear are very similar. However, it should be noted that while the base shear in FB model is decreased (after the first 5 seconds) in the case of the BI models the amplitudes are still quite high. Moreover, the time-histories are considerably distinguished in the frequencies.

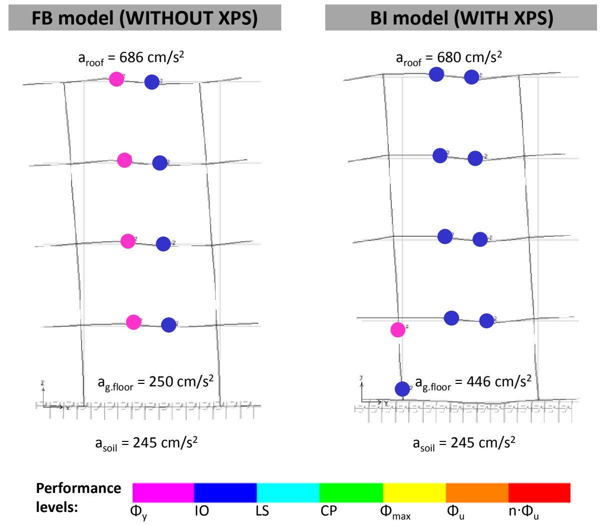

Fig. 7 The typical damage patterns and the average absolute maximum accelerations of the frame YZ: XPS400 d = 24 cm, foundation slab 30 cm, soil A, a_g = 0.25g

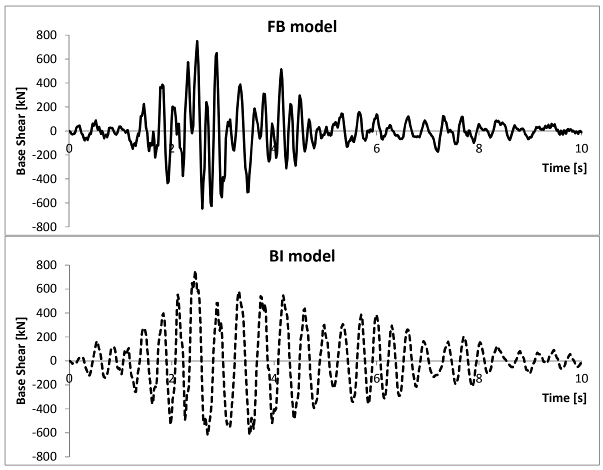

Fig. 8 The time-history of the base shear for YZ frame subjected to synthesized accel. Banja Luka: XPS400 d = 24 cm, foundation slab 30 cm, soil A, a_g = 0.25g

As said previously (see Fig. 4 and the comments given in section A of this chapter) the response of the building is significantly affected by the stiffness of the foundation slab, what was particularly evident when the 3-bay (XZ) frame was taken into investigation. The corresponding results obtained by the nonlinear dynamic analyses can be found in Fig. 9, where the typical damage patterns of the FB and BI frame XZ with fully rigid and real (30 cm) foundation slab are presented together with the average maximum horizontal (X) and vertical (Z) displacements at the roof and ground floor level. It can be seen that the obtained damage patterns are quite similar if the results of models with rigid and real foundation slab are compared. Similarly as already noticed from Fig. 7 (frame YZ), the BI frames XZ exhibited much more damage in comparison to the corresponding FB frames. Due to the flexibility of the foundation slab, the models on real foundation slab – if compared to the models on fully rigid foundation slab – are more susceptible to larger compressive deformations (negative values of D_z) of the XPS layer on one edge and/or uplifts (positive values of D_z) on the opposite foundation slab edge. It should be mentioned that the presented D_z includes the compressive deformation of the XPS layer as well as of the deformation of the soil bellow. In our particular case (considering firm soil A) the contribution of the soil to the total displacement D_z was relatively small (~10 %). Thus, around 90% of the presented D_z occurred in the XPS layer. Comparing the response of the BI XZ frames with real (30 cm) and fully rigid foundation slab the vertical displacements of the foundation slab edge were increased by factor 3 and more. It should be noted that the obtained horizontal displacements at the level of ground floor ($D_{x,\ g.floor}$) were very small. The reason lies in the large horizontal stiffness of the XPS layer which is the consequence of XPS material shear characteristics [10] and its large area (the XPS was considered beneath the whole area of the foundation slab). Here it should be mentioned that for the buildings founded on the XPS the main contribution to the changes of the buildings' dynamic characteristics in general has the XPS vertical (not shear) deformability, which causes the rocking effects resulted in local uplifts of the foundation slab. Such response is more pronounced in the case of slender structures (i.e. those with a higher height to shorter floor plan dimension ratio). In our analyzed case the shear strength of the XPS layer has been never exceeded. Although the obtained XPS compressive stresses or deformations resulted in higher demand/capacity ratios, they have never reached the XPS yield values (2.1% of the thickness of the XPS400 layer).

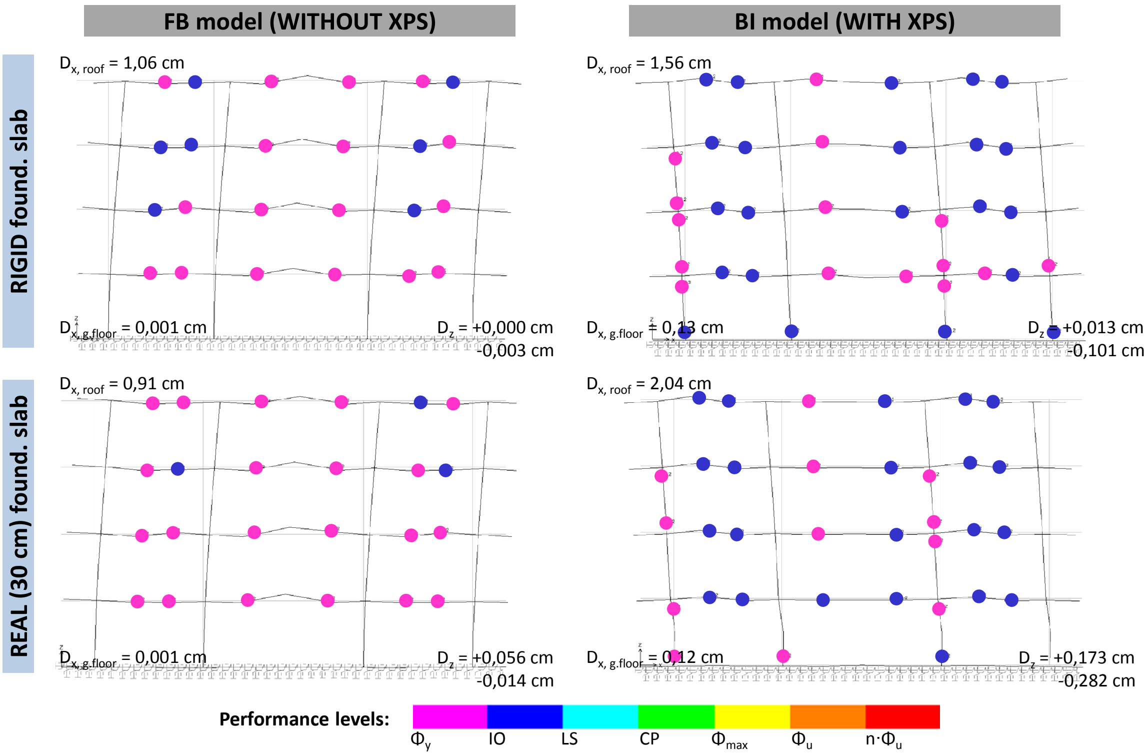

Fig. 9 The typical damage patterns and the average maximum displacements of the frame XZ with fully rigid and real (30 cm) foundation slab: XPS400 $d = 24$ cm, soil A, $a_g = 0.25g$

In order to investigate the effects of the XPS class on the response of BI models, both frames (XZ and YZ) were analyzed on XPS400 and XPS700. Both of them were considered with the same thickness (d = 24 cm). For this purpose only the models with real (30 cm) foundation slab were taken into investigation. In this particular case, the XPS class proved to not have considerable effect on the structural overall response. The fundamental periods of the frames on stiffer XPS class (XPS700) were only slightly smaller if compared to the frames on softer XPS: 0.231 s and 0.254 s for the XZ and YZ frame, respectively (the comparison with the values presented in Table I should be done). Similarly, the obtained damage patterns, displacements and storey drifts were in general decreased if the XPS700 was applied. For example, the roof horizontal displacement of the frame XZ took value equal to ~70 % of the corresponding displacement in the case of applying the XPS400 (in both cases the structures were assumed to be founded on firm soil A).

TABLE III
RESPONSE OF THE FRAME YZ AT TWO DIFFERENT EARTHQUAKE INTENSITIES

	$D_{y,\,roof}$ [cm]		$D_{z,g,floor}$ [cm]	
Seismic intensity	FB	BI	FB	BI
$a_g = 0.25$g	1.13	2.09	0.017	-0.357
$a_g = 0.35$g	1.86	4.22	0.019	-0.449

Besides the seismic intensity a_g = 0.25g the frame YZ was analyzed also under stronger earthquake event (a_g = 0.35g). For this purpose the previously used synthetized seven ground motion records were scaled by factor 1.4 and applied to the YZ frame with actual (30 cm) foundation slab on XPS400. The average maximum response of the frame YZ is presented in Table III. For vertical displacements of the foundation slab only the compressive deformations (negative values of D_z) are presented. As can be seen from the table a higher seismic intensity induce up to two-times larger lateral roof displacements (D_y), while the increase in vertical displacements at the edge of the foundation slab is much smaller (up to 25%). The standard deviations of the calculated $D_{y,\,roof}$ were up to 35% in the case of a_g = 0.25g and up to 50% in the case of a_g = 0.35g. As can be seen from Fig. 10 a higher seismic intensity caused a more critical damage pattern.

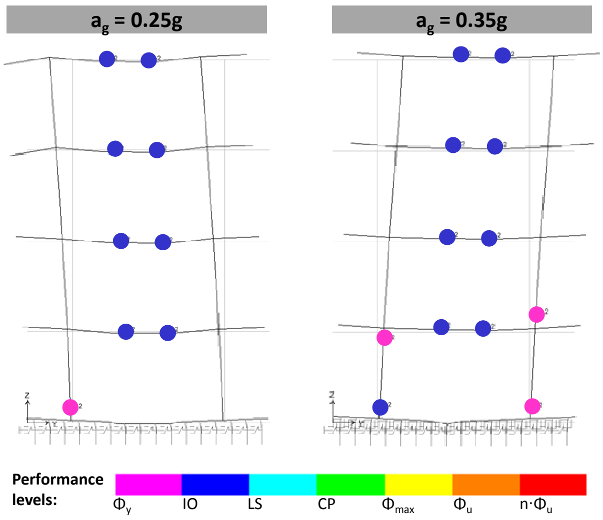

Fig. 10 Damage patterns of the BI frame YZ subjected to synthesized accel. Banja Luka at two different seismic intensities: XPS400 d = 24 cm, foundation slab 30 cm, soil A

V. CONCLUSION

Based on the results obtained, the following conclusions have been made, which should, however, be limited to the analyzed medium-height symmetric RC frame buildings and base-isolated with a layer of extruded polystyrene (XPS):

1) Comparing the seismic response of FB and BI models (structures without and with TI layer beneath the foundation slab) generally in many cases of conventional buildings no significant difference was observed. However, in certain specific building structures – as in the case presented in the study – the amplifications in the response could be up to 2-times. Such response amplifications might result in exceedance of the structural or the TI layer capacity values.
2) The main contribution to the changes in the dynamic characteristics of the buildings founded on the XPS layer in general has the XPS vertical deformability, which causes the rocking effects resulted in local uplifts of the foundation slab. Such response is more pronounced in the case of slender structures.
3) The incorporation of soil-structure interaction (SSI) has been recognized that it may have a significant impact on the dynamic system response.
4) The BI/FB response amplifications (structures with vs. structures without XPS beneath the foundation slab) of the analyzed structures obtain their maximum values in case of structures lying on firm soil.
5) The analysis of the foundation slab stiffness effect has shown that the superstructure as well as the XPS response is substantially affected by the stiffness of the foundation slab (especially in the case of multi-bay frames).
6) In the presented study the columns exhibited some nonlinear deformations only in the case of BI structures (with XPS beneath the foundation slab). In the case of FB structures (without XPS beneath the foundation slab) no damage in columns was recorded.
7) In the study simplified analysis of two characteristic 2D frames has been utilized. It should be noted, that in the case of real complex 3D building model the stresses and deformations of the XPS layer at the corners of the foundations could be increased.

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