Koce Todorov[1]

SEISMIC PERFORMANCE OF MASONRY INFILLED FRAMES WITH OPEN FIRST STOREY

Abstract: Reinforced concrete frames with masonry infill are often used structural systems throughout the world, especially in developing countries and countries around Mediterranean region. Irregular distribution of infill in plane and along building height can lead to series of unfavorable effects (torsion effects, dangerous collapse mechanisms, soft or weak story, variations in the vibration period, etc.). In order to investigate the influence of irregular distribution of masonry infill to the seismic performance of code designed reinforced concrete frames, an extensive nonlinear dynamic analysis was performed. The results from performed analyses shows that infill can have significant influence on the global structural behavior. In general, presence of infill increases the capacity and reduce the ductility of considered structures.

Key words: incremental dynamic analysis; masonry infill; open ground floor

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[1] Associated Professor, Ss. Cyril and Methodius University in Skopje, Faculty of Civil Engineering Skopje, Blvd. “Partizanski odredi”, No.24, Skopje, Republic of North Macedonia, todorov@gf.ukim.edu.mk

1. INTRODUCTION

The evaluation of the seismic performance of existing and newly designed structures is a relatively complex and multidisciplinary process. It includes elements of engineering seismology and soil dynamics, necessary to define the level of seismic hazard and the expected characteristics of input ground motion, elements of the dynamics of structures, for determination of the structural response, as well as elements of the structural mechanics, necessary for the accurately inclusion of the effects of material nonlinearity in the response of the systems under consideration, [2].

Having in mind the expected structural behaviour to the permanent or seismic action, it would be logical to select appropriate analysis methods that can predict the structural behaviour with a high degree of confidentiality. If linear behaviour can be successfully predicted using linear methods for analysis, then it is expected that a nonlinear response should be determined with nonlinear analysis methods. Despite these expectations and considerable efforts for the conceptual transformation of earthquake engineering that have been made in the last twenty years, the generally accepted methods for seismic analysis are based on linear approximations. And these approximations are not so wrong, if a certain number of effects are included in the process of analysis or design. Masonry infill, the influence of cracks in reinforced concrete elements, the contribution of effective slab width, soil - structure interaction, etc. are some of the parameters which can significantly change the desired seismic response of the analysed structures.

Reinforced concrete frames with masonry infill are often used structural systems in construction practice worldwide. In this type of structures, the external and internal walls made of different materials, usually of ceramic blocks and bricks, are built as infill panels between reinforced concrete structural elements. Masonry infill is characterized with significant strength and stiffness and it can greatly alter the response of structures exposed to dynamic loads. The infill panels increase the structural stiffness, strength and damping and act as a first line of defence in seismic activity, reducing the ductility demand and consequent damage of structural elements. However irregular distribution of infill in plane and along building height can lead to series of unfavourable effects (torsion effects, dangerous collapse mechanisms, soft or weak storey, variations in the vibration period, etc.). That is why different opinions about the influence of the infill on the seismic behaviour of the structures can be found in the scientific and expert community.

The presence of infill significantly changes the mechanism for lateral load redistribution. Thus, the predominant frame system, in which the elements are exposed to bending, is transformed into a predominant truss system whose elements are generally exposed to axial action [14]. Although this is an undeniable fact, according to the usual design practice, the interaction between the infill and the frame structure is most often neglected. This can lead to significant errors in determining the stiffness, bearing capacity and ductility of the analysed structure, which can especially be emphasized in reinforced concrete frames with discontinuity in the distribution of the masonry infill to the height of the building. Such structures in the literature are known as building with a weak floor (the strength on the lower floor is less than the strength of the upper floors), or structures with flexible or soft floor (the horizontal stiffness on the considered floor is lower than the stiffness of the upper floors). Although from a structural point of view these systems are unfavourable, for architectural or commercial reasons, they are quite attractive and exploited, especially in the central city cores and in densely populated urban areas. The open free space of such facilities, which is usually located on the first floor, is used as a corridor or as a space for accommodation in parking lots, shops, administration, etc., Figure 1.

Figure 1 – Reinforced concrete buildings with open first floor

The influence of the infill on the seismic performance of reinforced concrete structures designed according to different national regulations has been the subject of a significant number of researches available in the literature, Fardis and Panagiotakos [8], Fardis et al. [7], Kappos et al. [10], Kappos and Ellul [11], Dolsek and Fajfar [6], Decanini at al. [5], Korkmaz et al. [12], Ghalehnovi and Shahraki [9], Manfredi et al. [13] etc.

2. DESCRIPTION OF ANALYSED STRUCTURES

In order to analyse the influence of the masonry infill on the behaviour of reinforced concrete frames that have been designed without taking into account the influence of the infill, a series of nonlinear static and dynamic analyses was conducted in this research. Nonlinear dynamic analyses were performed for bare and infilled 2D frames with an open floor exposed to 27 earthquake ground motions with different intensities and with different frequency content. All numerical analyses were performed by the Seismostruct v6.0 [15].

2.1. Design parameters

In order to cover structures with different dynamic characteristics, six reinforced concrete plane frames with different number of storeys (n=2, 3, 5, 7, 10 and 13), in the following text marked as frames R1, R2, R3, R4, R5 and R6, are generated in this investigation. All analysed structures are designed as three bay plane frames with a span of 5m and a constant storey height of 3m. The frames are designed to represent the exterior frame of a reinforced concrete spatial frame structure, Figure 2. After the distribution of the surface load on the frames in both orthogonal directions, the beams of analysed frames were exposed to uniform load of 25kN/m’ on the floors and 15kN/m’ on the roof. At the beam – column joints additional concentrated forces, which represent the influence of the beams in the longitudinal direction, was applied. For the purpose of dynamic analysis and according to the defined loads, the distribution of the mass by floors, equal to 70.9t at each floor, and 38.2t at the level of the roof, was determined. A schematic representation of distribution of the gravity loads and the storey mass of the analysed frames is presented at Figure 2.

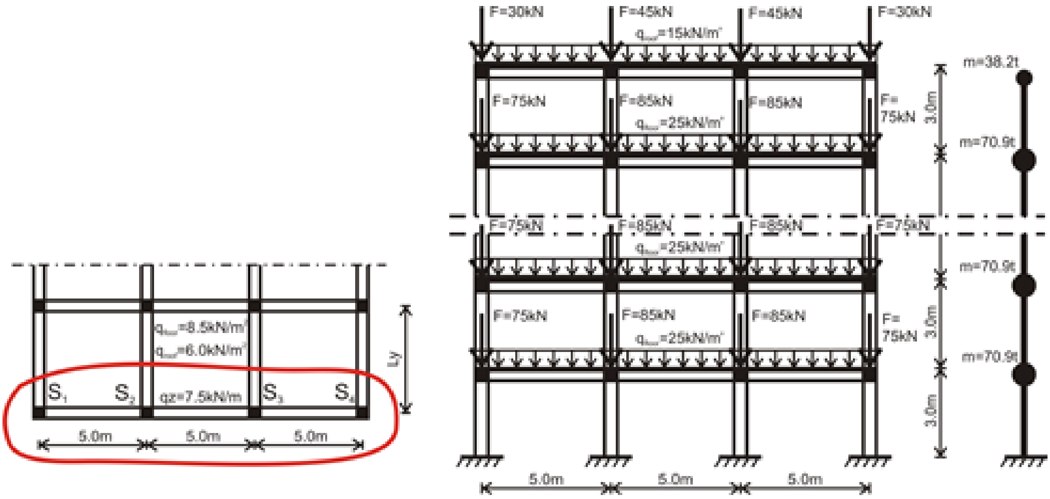

Figure 2 – Schematic representation of analysed frame structures

2.2. Adapted features of masonry infill

In order to observe the impact of the masonry infill on the seismic performance of the designed frames, all frames were additionally upgraded with the presence of masonry infill panels in all spans and storeys, except for the first one. Masonry infill was defined with two different strength and stiffness characteristics, namely week infill (WI) and strong infill (SI). The nonlinear behaviour of masonry infill was modelled with the equivalent diagonal strut, using the model developed by Crisafulli [4] and implemented in SeismoStruct by Smyrou at al. [16].

The weak infill is characterized by the compression strength in the diagonal direction f_m=0.8MPa and a thickness of 15 cm., while the strong infill has a compression strength of 1.2MPa and a thickness of 25cm. An initial modulus of elasticity equal to 1500f_m was adopted for both types of infill. The compression strength is reached at strains of 0.002, while complete degradation of the strength and stiffness of the infill occurs at ultimate dilatation of 1%. The secant modulus of elasticity, at the point of achievement of compression strength is equal to one third of the initial modulus of elasticity, E_{sec}=500f_m.

For defining the characteristics of the shear spring in the used material model, the initial shear bond strength of the infill τ_o=0.3MPa and coefficient of friction μ=0.5 were adopted. The maximum shear strength τ_{max} was limited to 0.6 MPa.

According to the adopted concept for modelling the masonry infill, a compression strut is characterising with variable area which reduction is a function of the reached axial strains. The initial area of compression strut is a product of the thickness of the infill and the width of the equivalent diagonal, for which a value of 20% of the length of the diagonal is adopted. When strains in infill reach the value ε_1=0.0005, the area of the diagonal begins to linearly decrease. At strain equal to 0.0063, its area is 50% of the initial. Schematic diagrams of stress – strain relationship for masonry infill, reduction of strut area and defined diagram lateral force – inter-storey drift for masonry infill panel are presented at Figure 3.

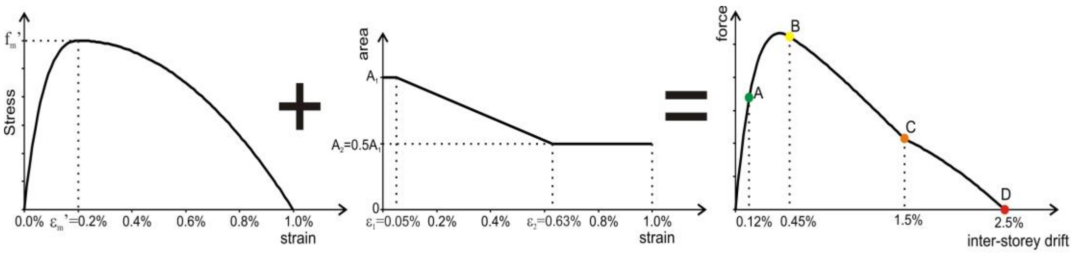

Figure 3 – Stress – strain relationship for masonry infill, reduction of strut area and defined diagram lateral force – inter-storey drift for masonry infill panel

The limit values of the adopted strains correspond to a certain level of damage in masonry infill and according to the Equation 1, can directly be connected with the achieved inter-storey drifts.

$$\varepsilon = \frac{\delta}{2}\sin 2\theta \qquad (1)$$

Strains at the beginning of the strut area reduction correspond to the state of cracks in the infill, which occur at interstorey drift in range from 0.05% to 0.15%. These range of interstorey drifts correspond to the operational seismic performance level. Strain at maximum strength in the infill correspond to interstorey drift from 0.4% to 0.5% and according to the performance level represent the end of the immediate occupancy state. The strains at the end of the diagonal area reduction correspond to interstorey drift of 1% to 2% and are the limit values of the life safety performance level. The ultimate strain in the infill which corresponds to interstorey drift in the range from 2-3% according the level of seismic performance represents the collapse prevention limit state. The boundary and the adopted values of strains for this research and corresponding interstorey drifts for different performance levels are given in Table 1.

Table 1- Performance level and corresponding boundary values of strain in masonry infill and interstorey drifts

Performance level	Range	Boundary values of strain	Adopted strain limit	Boundary values of interstorey drifts	Adopted values of interstorey drifts
Operational	0-A	εA 0.02-0.07%	0.05%	δA 0.05-0.15%	0.12%
Immediate occupancy	A-B	εB 0.1-0.3%	0.2%	δB 0.25-0.7%	0.45%
Life safety	B-C	εC 0.4-0.85%	0.63%	δC 1-2%	1.5%
Collapse prevention	C-D	εC 0.75-1.25%	1%	δD 2-3%	2.5%

3. SELECTION AND SCALING OF EARTHQUAKE GROUND MOTIONS

In the probabilistic approach for the assessment of seismic performance, the analysis of the structural behaviour usually is performed for the multiple earthquake scenarios, while the evaluation of seismic performance is carried out by statistical processing of the obtained results. This approach allows determination of the probability of damage for a certain earthquake scenario and also allows identification of the ground motion records that have the greatest influence on the behaviour of a certain structure.

Each ground motion is characterized by a few engineering parameters, among which the parameters that characterize the amplitudes and the frequency content of the earthquake are the most important. In order to select records with defined frequency content from the existing database of earthquake ground motions, a methodology for the identification of the dominant frequency domain in the acceleration spectrum was developed, Todorov [17]. The developed methodology is based on the transformation of the acceleration spectrum into a modified cumulative spectral intensity diagram, which enables easy identification of the boundary periods of the dominant frequency range (T_1 and T_2), the mean dominant period - T_m, and the mean amplification of the acceleration spectrum. The defined procedure was applied to 910 registrations from the PEER NGA database [3], of which 21 registrations, grouped into three groups of 7 records, Figure 4, were selected and used for further analyses.

The first group of earthquakes (EQ1) contains registrations of earthquakes with a predominantly frequency range of low periods. The boundary values of the dominant frequency for this group of earthquakes is in the range from (0.1sec. $<T_1<$0.15sec.) to (0.55sec.$<T_2<$0.6sec.). Thus, the width of the dominant frequency band, B_T, is limited to within the limits (0.4 $<B_T<$0.5 sec.), and the mean period T_m is defined in boundaries from 0.325sec to 0.375sec. This group of earthquakes should expose stiff structures to oscillate dominantly in the first mode, while more flexible structures to oscillate in higher mode shapes. A second group of earthquakes (EQ2) is a group of earthquakes with a predominantly frequency range of medium to high periods, (0.24sec.<T_1 <0.35sec.) and (1.1sec.<T_2 <1.5sec.). The width of the dominant frequency band in this group can range from 0.75 to 1.26 sec., while the median period can range from 0.725 to 0.91 sec. A third group of earthquakes (EQ3) is defined by the requirement that the width of the dominant frequency range be within the range of 1.8 to 2.6 sec., without imposing limits on the boundary values for the periods of the dominant frequency range.

The procedure for records selection is carried out in two steps. In the first step, according to the defined limit values of the dominant frequency ranges, 45 records from group 1, 34 from group 2 and 55 from group 3, that meet the set criteria, were selected. In the second step of selections, seven records with the smallest cumulative deviations were selected. The acceleration spectra of the selected records from each group, their mean spectrum, the standard deviation, and the mean spectrum ± one standard deviation are presented at Figure 4.

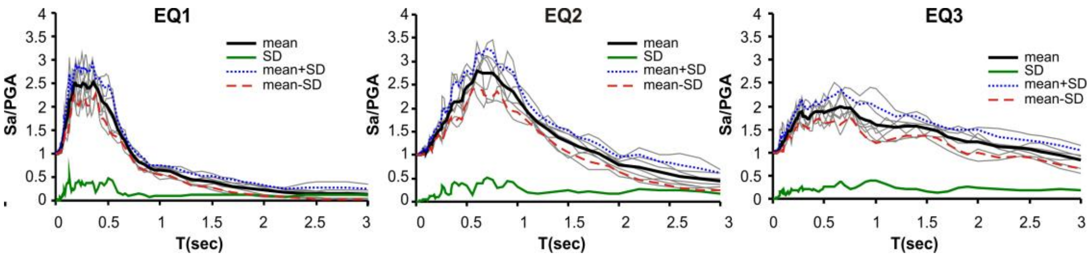

Figure 4 – Acceleration spectra of the selected records for analysis

To meet the requirements of incremental dynamic analysis the selected records were scaled to ten different amplitudes, based on scaling of pick ground acceleration. The records from the first group of earthquakes were scaled for pick ground acceleration in the range from 0.1g to 1.0g, while the records from the second and the third group of earthquakes for pick ground acceleration in the range from 0.05g to 0.5g

4. INCREMENTAL DYNAMIC ANALISIS

An incremental dynamic analysis (IDA) is one kind of parametric nonlinear dynamic analysis that provides a continuous display, from elastic behaviour via yielding to the state of collapse, of the considered structures exposed to seismic action. The concept of incremental dynamic analysis was first proposed by Bertero [1], but a detailed description of the method and development of a methodology for its practical application was provided by Vamvatsikos and Cornell [18]. In this analysis structural model is exposed to one or more acceleration records, each of them scaled at multiple intensity levels. Incremental dynamic analysis is a pushover analysis pendant, with the difference that in the pushover analysis results are obtained with the incremental increase of static load, while in the incremental dynamic analysis with the increases of the intensity of the input ground motion. The results of the incremental dynamic analysis are presented in the so-called IDA curves, which give the connection between a certain intensity measure (IM) and the behaviour of the structure expressed through a certain measure of damage (DM). In order to carry out this analysis, it is necessary to define confidential data for the behaviour of structural materials and elements exposed to cyclic loading over the limit of elasticity, to select and to scale the acceleration ground motion and to define a stable algorithm for solving the system of differential equations of motion.

3.1 Analysis of the obtained results

From the conducted nonlinear dynamical analyses, a large number of output data which present the global response of the analysed structures, as well as a large number of results that show the structural response at the local level are obtained.

Maximal top displacements, expressed as a percentage of the total height of the analysed frames, are noted at lower frames. By the increasing of number of stories, the maximal top drifts are reduced, which is particularly pronounced for the case of earthquakes with a dominant frequency range of low periods. Bare frames (BF) have a larger top displacements compared with the infilled frames, which is more emphasized at lower levels of peak ground acceleration.

Infilled frames with strong infill (SI) reach smaller values of maximal top displacement compared to the frames with weak infill (WI), which is more significant for lower levels of peak ground acceleration and for higher frames. The frequency content of the ground acceleration records is the most influential factor on the degree of reached top drifts. The peak top displacement at the masonry infilled frames with an open first floor, calculated as the mean value from the results obtained for peak ground acceleration of 1g, for records characterized by a dominant frequency range of low periods, are within the range of 0.5 % to 2% of the total height of the frames, Figure 5.

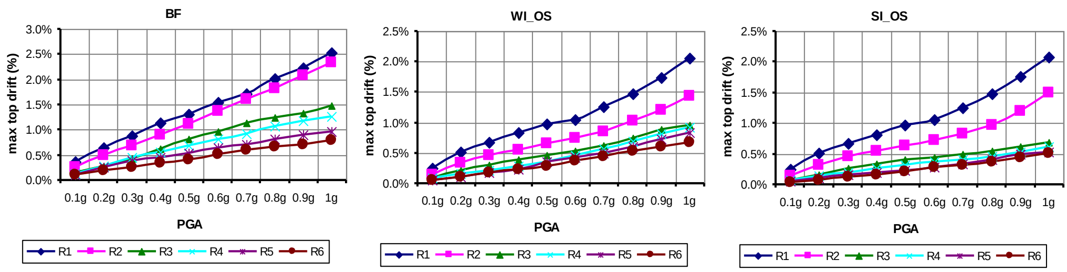

Figure 5 – Maximal top drift in function of pick ground acceleration for analysed frames exposed to first group of earthquakes

Similar values of the maximal top displacement, for the action of records with a predominantly frequency range of medium to high periods, as well as for registration with wide frequency regions, are obtained for pick ground accelerations which are 3 to 4 times smaller compared with the records with dominant frequency range at low periods.

Except for the influence on the level of maximal top displacement, masonry infill has a great influence on the distribution of displacements thru the height, as well as on the distribution and amplitudes of the maximal interstorey drifts.

At lower levels of peak ground acceleration, when the masonry infill is in the elastic domain of behaviour, it significantly stiffens the structure therefore reduces the maximal interstory drifts. At higher levels of seismic action, i.e. when the degradation of the stiffness and strength of the infill occurs, the interstorey drifts of the infilled frames increase significantly faster compared with the bare frames.

Strong infill has a favourable effect on the degree of maximal interstorey drifts to a certain level of peak ground acceleration. At higher level of PGA, the strongest infill leads to an undesirable failure mechanism, which increase the level of interstory drifts. Relationship between maximal interstorey drift and pick ground acceleration for analysed frames exposed to second group of earthquakes is presented at Figure 6.

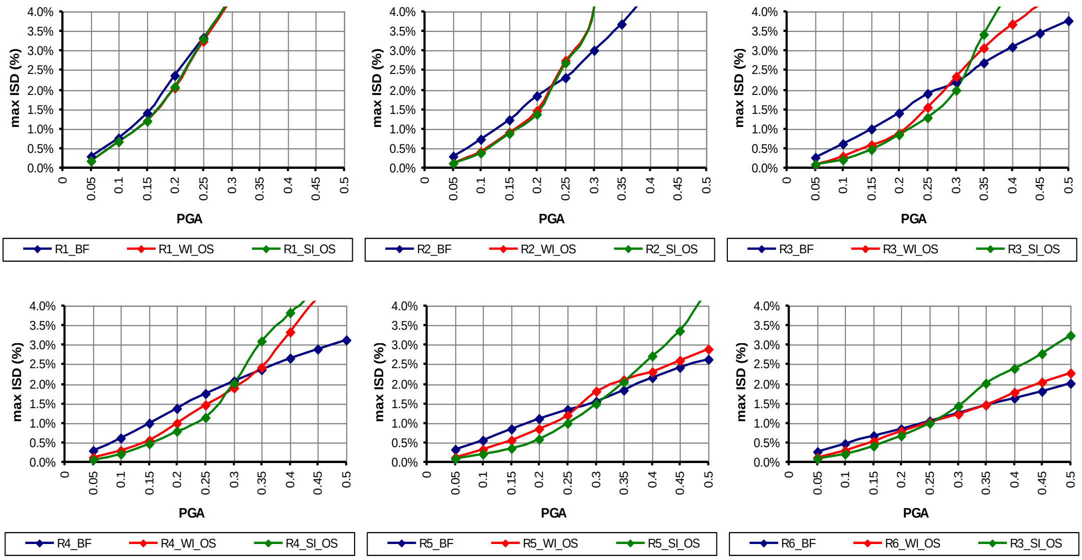

Figure 6 – Maximal interstorey drift in function of pick ground acceleration for analysed frames exposed to second group of earthquakes

Soft story mechanism of failure, expressed with the formation of plastic hinges at the ends of the columns on the first storey, where the infill is missing, is observed at the low frames with 2 and 3 storeys, regardless of the quality of the infill, as well as in the five storey frame with a strong infill, regardless of the characteristics of the ground motion. At the higher frames, the failure mechanism is represented by the formation of plastic hinges in the beams and columns on the first few floors.

3.2 Discrete levels of seismic performance

The distribution of interstorey drifts along the height is a direct indicator of the total damage degree that can occur within the frames for different earthquake scenarios. From the determined incremental curves, which present the relationship between the pick ground acceleration and maximal interstory drifts, the discrete seismic performance levels can be determined.

Depending on the achieved strains in the infill, which are related with the interstorey drifts, four limit states of seismic performance are defined: operational (δ_A=0.12%), immediate occupancy (δ_B=0.45%), life safety (δ_C = 1.5%) and near collapse with maximum interstory drift δ_D = 2.5%. It should be noted that the same limit values of the interstory drifts are used for all floors within the infilled frames, regardless of whether the extreme values appear at the level of the first open floor, or on any of the upper floors. In order to compare the results, the same limit values were used to define the limit states and for the bare frames.

For the action of first group of earthquakes, the most vulnerable for all levels of seismic performance are low rise frames. The operational limit state for this group of earthquakes is achieved for peak ground acceleration in range from 0.03g for frame R2 to 0.16g for frames R5 and R6 with weak infill or 0.2g for frame R6 with strong infill. The immediate occupancy limit state in lower frames is achieved at PGA from 0.1g to 0.12g, while in the higher frames, it increases to around 0.45g for frames with weak infill or 0.57g for frames with strong infill. The life safety limit state at lower frames is expected to be achieved for pick ground acceleration of about 0.4g, while in the higher frames this condition is not reached in the considered range of intensity. The near collapse limit state in frames with a strong infill is expected to be reached at peak ground acceleration in range from 0.7g for frames R1 and R2 to 1g for frame R4. In the case of lower frames with weak infill this limit state is reached for 2% to 4% higher peak ground acceleration compared with the frames with strong infill, while in the higher frames the achievement of this limit state is not registered. The relationship between peak ground acceleration, number of storey and achieved limit states, for the first set of earthquakes is presented in Figure 7.

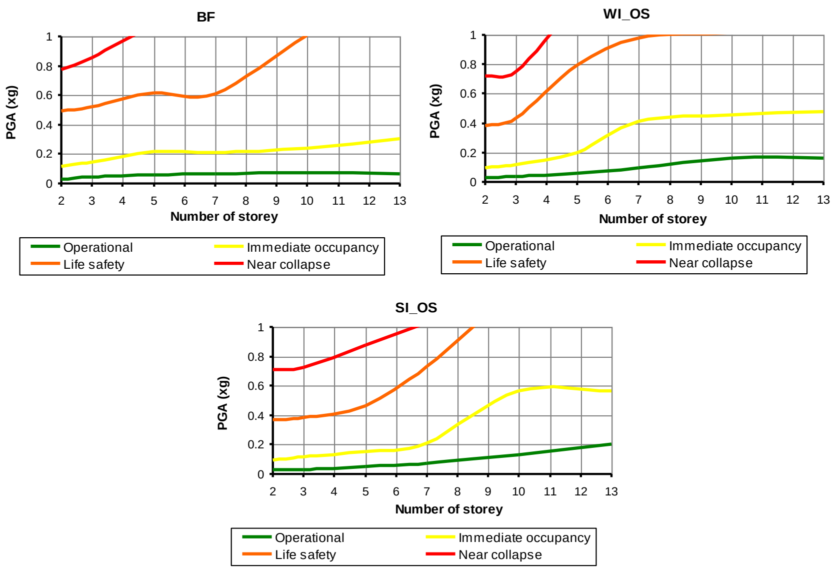

Figure 7 – Limit states of seismic performance for analysed frames exposed to first group of earthquakes

The achievement of the defined limit states in the considered infilled frames, for the action of earthquakes from the second or third group, differs considerably compared to the first group, which is more significant at the higher limit states, Figure 8 and Figure 9. The operational limit state is achieved for the PGA from 0.05g to 0.07g, for the action of the second group of records, up to a maximum value of 0.1g for the frame R5 exposed to the third set of records. The state of immediate occupancy, for frames up to 5 storey exposed to the second group of earthquakes, is reached for PGA in the limits from 0.08g to 0.15g, or from 0.1g to 0.2g for the action of earthquakes from the third group. At the higher frames with weak or strong infill, this condition is achieved at average PGA values from 0.13g to 0.16g for the second, or from 0.15g to 0.2g for the third set of records. The life safety limit state is achieved for peak ground acceleration from 0.17g to 0.35g. Lower values of intensity usually are related with the lower frames, where the formation of the soft storey mechanisms at the level of the first storey occurs. Soon after the achievement of life safety limit state, the performance level of near collapse is reached. This limit state is reached for peak ground acceleration in range from 0.22g to 0.42g, depending from the number of storey and from the characteristics of the ground acceleration.

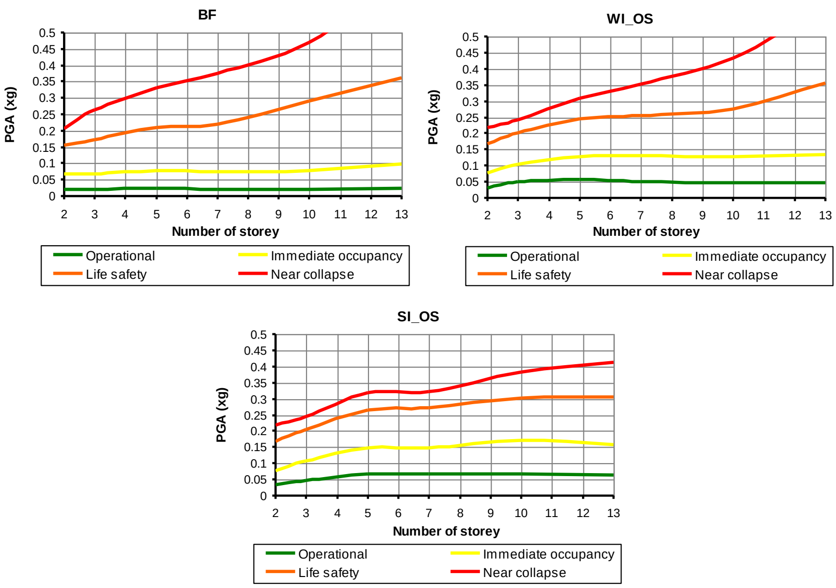

Figure 8 – Limit states of seismic performance for analysed frames exposed to second group of earthquakes

According to the achieved levels of seismic performance, it can be concluded that in order to provide the limit state of functionality, masonry infill, through the additionally added stiffness and strength, usually play a positive role, reducing the seismic demand. Due to the possibility of the appearance of a brittle failure in the infill, which can be reflected by the appearance of weak parts in the structure, it can be said that the masonry infilled frames are unreliable for providing of the higher limit states. In the lower frames with an open first floor, due to the presence of the infill on the upper floors, an undesirable soft storey mechanism usually occurs.

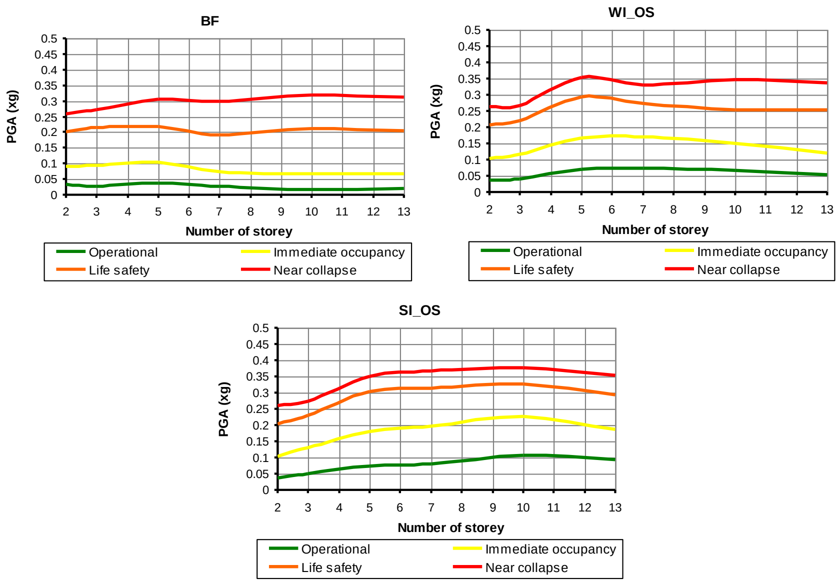

Figure 9 – Limit states of seismic performance for analysed frames exposed to third group of earthquakes

5. DEMAGE DISTRIBUTION INDEX

The maximal top drifts, represent a global measure of the degree of deformation of a structure. They may be close related with the degree of structural damage, however, the limit values of the total displacements which correspond to a certain degree of damage, depend on the distribution of the displacements by height. For a multi-storey structure, peak top displacement in the range of 0.2% from the total height may represent an insignificant degree of damage, if the inter-storey drifts are uniformly distributed along the height, or they can be an indicator of serious damage, if the inter-storey drifts are concentrated at one storey level.

The relationship between the maximal interstorey drifts (maxISD) and the maximal top drifts (maxTD) for the sixth analysed frames obtained with the individual records from the three groups of earthquakes scaled to ten levels of peak ground acceleration are presented at Figure 10. From the presented diagrams a nearly linear tendency of this relationship can be noticed. The largest deviations are observed at the bare frames and usually are result of earthquakes records from the first group.

In infilled frames R1 and R2, as well as the frame R3 with strong infill, there are almost no deviations from the linear relationship, which is due to the concentration of displacements at the level of the first storey. In the higher infilled frames, the ratio between the maximal top drift and maximal inter-storey drift is greater, indicating a more uniform distribution of the displacements in height. With increasing of the displacements, this ratio decreases, indicating a concentration of damages on separate floors.

For frames R1 and R2 with weak and strong infill the ratio maxTD - maxISD is almost identical, indicating a slight influence on the characteristics of the infill on the distribution of displacements. In the higher frames with a strong infill this ratio is lower, indicating the concentration of displacements at the level of one (in the case of frames R1, R2 and R3) or at the level of several floors (in frames R4, R5 and R5) of the considered structures.

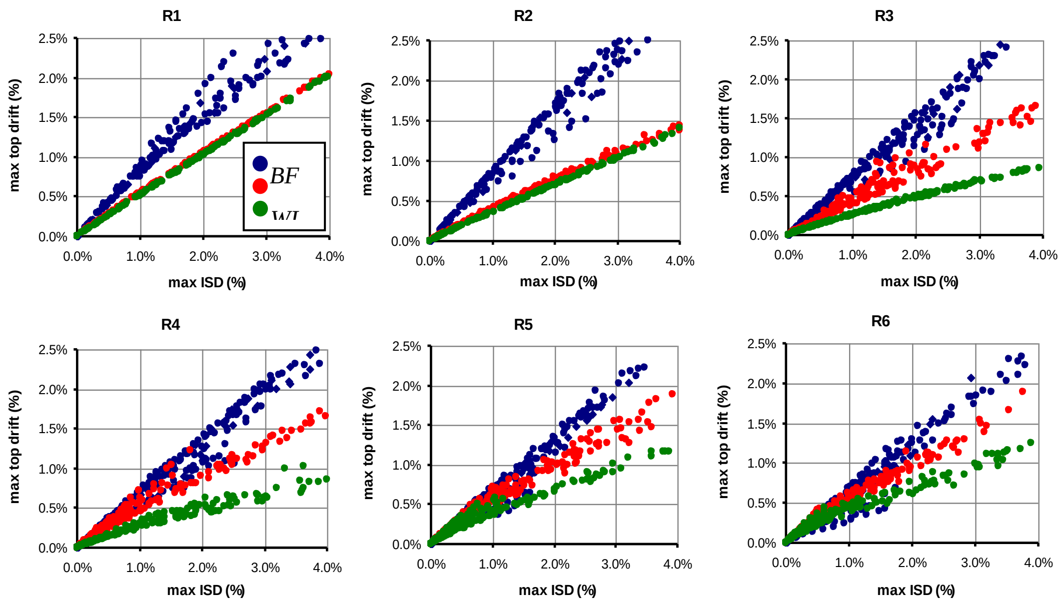

Figure 10 – Ratio between the maximal interstorey drifts (maxISD) and the maximal top drifts (maxTD) of the analysed frames

The ratio between maximal interstorey drift (maxISD) and maximal top drift (maxTD) can be a good indicator of damage distribution along the building height. In order to quantify this relationship, regardless of the number of stories (n), the parameter named as index for distribution of displacement at height (DDH) with the boundary values from 0 to 1 was defined, Equation 2. The change of the DDH index, in the function of the peak ground acceleration, for the frames R2 and R4, exposed on the individual records from the three sets of earthquakes is presented at Figure 11.

$$DDH = \frac{\frac{\max ISD}{\max TD} - 1}{n-1} \qquad (2)$$

For the same values of maxISD and maxTD, which implies a triangular distribution of displacement, or formation on beam sway mechanism, the value of this index is close to 0.

Higher values of this index (close to 1) indicate a concentration of interstorey drift on one storey level, which is characteristic for the formation of soft storey mechanism.

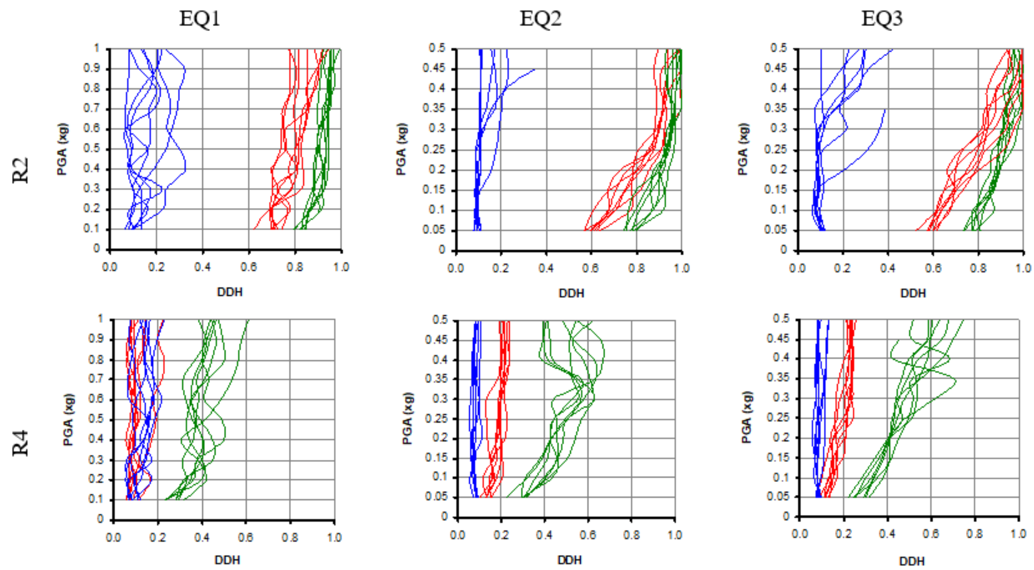

Figure 11 – DDH index in function of PGA for analyzed frames R2 and R4

6. CONCLUSIONS

The results from performed analyses on the infilled frame with open first storey shows that infill can have significant influence on the global structural behaviour. In general, presence of infill increases the capacity and reduce the ductility of considered structures. The selection and characteristics of ground motion records are one of the most influential factors that directly affect the quality of the obtained results from nonlinear dynamic analysis.

In the case of low rise buildings (n=2 and 3 storeys), the quality of infill does not have a remarkable effect to the structural behaviour. Compared with the behaviour of bare frame, the presence of infill is usually unfavourable for all levels of PGA, leading to the formation of soft storey mechanism. In the case of 5 and 7 storey buildings, the presence of infill reduces the seismic demand up to the PGA of 0.3g. Usually, strong infill corresponds to small interstorey drift demand, at low level of seismic hazard, compared with the weak infill. Soft storey mechanism has not been observed at high rise buildings. At this type of buildings distribution of damage depend on the mechanical characteristics of infill as well as the frequency content of input ground motion.

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