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INFLUENCE OF SEISMIC HAZARD AND IMPORTANCE CLASS ON THE BEHAVIOR OF REINFORCED CONCRETE STRUCTURAL ELEMENTS

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ABSTRACT

A significant step forward in structural engineering in our country is the implementation of European standards (Eurocodes) for structural design of buildings, which tend to be the only valid ones. For that reason, in this research it is decided the provisions of these standards to be applied to a real, reinforced concrete structure, which is a four – story residential building located in Skopje. Knowing that the region in which we live is extremely seismically active, special attention is paid to the seismic action and its influence on the behavior of reinforced concrete structural elements.

It is a numerical analysis that is realized by varying certain parameters that define the seismic action. Firstly, the seismic hazard is analyzed, which according to EN 1998-1 is described in terms of a single parameter, i.e. the value of peak ground acceleration a_{gR} (PGA). According to the seismic zone map for our country, its territory is divided into five seismic zones, so that PGA takes the following values: a_{gR}={0,1g; 0,15g; 0,2g; 0,25g; 0,3g}. By implementing an analysis for each of the listed values, it is realized how and to what extent the location affects the behavior of reinforced concrete structural elements. The second variable parameter is the importance factor that according to EN 1998-1 takes different values, depending the importance class in which the building is classified. For the purposes of this part of the research, these importance classes are analyzed: Class II (residential building), Class III (school) and Class IV (hospital).

It is expected that by increasing the variable parameters, the design values of the effects of actions in beams and columns also increase. As a consequence to that, the geometric reinforcement ratio increases, too. When PGA varies, the design values of bending moment in beams, and thus the reinforcement ratio increase by 2-3 times. At the column sections, this ratio reaches an increase of 40 % when PGA varies and 35 % when importance factor varies. Common feature for all column sections is the ductile failure, so that the strain in steel decreases up to 34 % in the sections where the ultimate strain in concrete is achieved, and the strain in concrete increases up to 94 % when the ultimate strain in steel is achieved.

Overall, it may be said that seismic action has a huge impact on the structural elements and the behavior of the structure at all. Therefore, it is necessary to pay special attention when designing buildings that are located in seismically active areas.

Keywords: Seismic hazard; Importance class; Geometric reinforcement ratio; Type of failure; Reinforced concrete structural elements.

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1. INTRODUCTION

In this paper, the behavior of reinforced concrete structure is analyzed. The emphasis is on the seismic action, in a way that certain parameters that define this action, are varied. It is important to say that the entire design process of the building is in accordance with the European standards, so called Structural Eurocodes, which tend to become the only valid ones in our country.

Through the presented analyses, a representation of the impact of seismic action on a real structure is created. The outcome of all analyses are the design values of internal forces and the corresponding area of reinforcement in specific structural elements, as well as the type of failure in the analyzed sections.

2. INITIAL MODEL

2.1. Description of the building

The investigated building in this paper is a multi-story reinforced concrete structure, which has 4 stories above the ground level and 1 basement story. The height of each story is 3 m, the ground floor is raised for 1,20 m above the terrain, so that the total height of the building above the terrain is 13,20 m. The height of the basement story is equal to 2,60 m and the dimensions in plan of this story are 23,60 x 23,40 m. The area of the other stories is smaller and it is equal to 23,60 x 13,80. The reinforced concrete frames in the direction of global axis X, are placed in the following spans: 5,60+4,40+5,0+4,40+4,00 m. The frames in the direction of global axis Y are placed in these spans: 5,20+4,55+4,10+4,55+5,20 m. The typical floor plan and the cross section of the building are shown in Fig. 1 and 2.

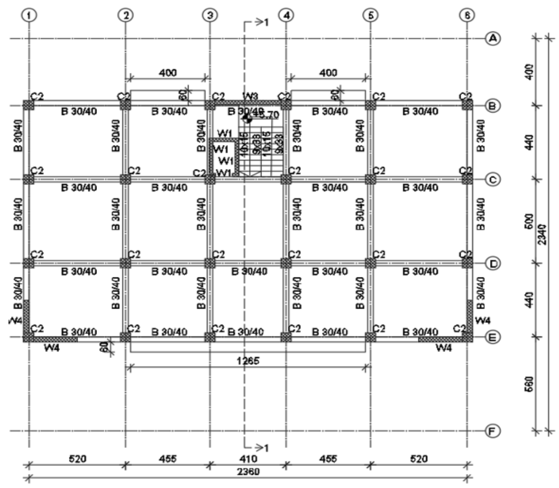


Fig. 1. Typical floor plan of the building

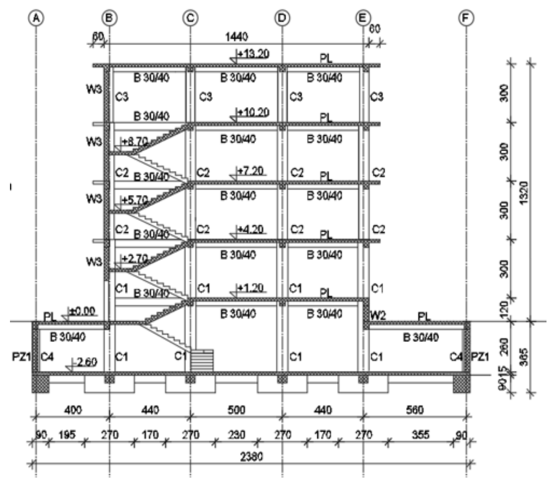


Fig. 2. Cross section of the building

The structural system consists of frames and walls. The cross section of all columns is square with side lengths of 60 cm in the basement and ground floor, 55 cm on the first and the second floor and 50 cm on the third floor. The beams are with rectangular cross section 30x40 cm. The slab is 15 cm thick. The vertical communication in the building is enabled through two-legged staircase with 15 cm thick slab and lift core which includes 20 cm thick walls. In the basement, there are peripheral walls, whose thickness is 30 cm. There are also several additional walls, whose role is to prevent the appearance of short column effect in the building and to reduce the eccentricity between the center of mass and the center of stiffness.

2.2. Structural model

For the whole static and dynamic analysis, the program Radimpex Software (Tower 6), which is based on the finite element method, was used. During the analysis, the provisions of MKS EN Standards (MKS EN 1990[3], MKS EN 1991[4][5][6], MKS EN 1992[7] and MKS EN 1998[8]) were applied. Columns and beams are modelled as line elements. Slabs and walls are modelled as surface finite elements, which are four node quadrilateral elements with 50 cm width. All elements are fully fixed at the foundation level -2,60 m. According to MKS EN 1998-1/4.3.1[8], the stiffness of the load bearing elements is evaluated taking into account the effect of cracking, in the way that the elastic flexural and shear stiffness properties are taken to be equal to one-half of the corresponding stiffness of the uncracked elements. For all load bearing elements, concrete C30/37 is used. The corresponding modulus of elasticity amounts to E_{cm}=33 GPa (MKS EN 1992/Table 3.1[7]). Steel S500 Class B is used, so that the characteristic strain at maximum load amounts to 5 %. The structure will be designed for ductility class DCM. 3D view and view in X and Y direction of the structural model are shown in Fig. 3 and 4.

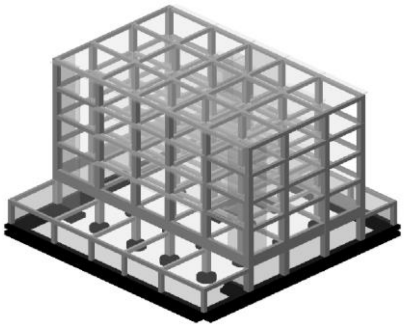


Fig. 3. Structural model – 3D

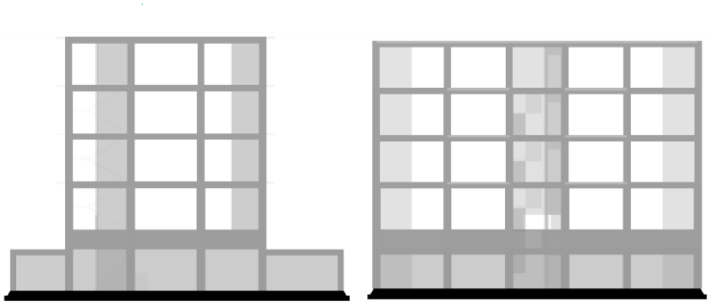


Fig. 4. Structural model – Y and X direction

2.3. Actions

The permanent vertical loads G are represented by the self-weight of the structure, which is taken into account automatically by the software, and additional permanent load. The additional permanent loads are precisely calculated and amount to 6,26 kN/m’ for facade walls, 6,21 kN/m’ for partition walls that separate the apartments, 3,40 kN/m^2 on all floor slabs, 0,70 kN/m^2 on roof slab, 4,20 kN/m^2 on stair slab and 2,00 kN/m^2 on the stair landings. When it is about the variable-live loads Q, the investigated building is a residential building, so that according to MKS EN 1991-1-1/Table 6.1 it belongs to category A. According to MKS EN 1991-1-1/Table 6.2[4] the variable-live load in terms of uniformly distributed load is 2,00 kN/m^2 on floors, 2,50 kN/m^2 on balconies and 2,00 kN/m^2 on stairs. The roof slab according to MKS EN 1991-1-1/Table 6.9 belongs to category H, so that the variable-live load is equal to 0,60 kN/m^2. The investigated building is located in Skopje, so the characteristic value of snow is equal to 0,83 kN/m^2 (MKS EN 1991-1-3:2012/NA:2020[12]) and the snow load on the roof is 0,67 kN/m^2. The input parameters for the calculation of wind action are: fundamental value of the basic wind velocity $v_{b,0}$=24,47 m/s according to MKS EN 1991-1-4:2012/NA:2020[13] for location Skopje, and terrain category IV (MKS EN 1991-1-4/Table 4.1). The calculated value of peak velocity pressure is equal to 0,507 kN/m^2.

2.4. Structural regularity

Criteria for regularity in plan

The criteria for regularity in plan are described in MKS EN 1998-1/4.2.3.2[8], and it limits the slenderness of the building, the structural eccentricity and the torsional radius. After the calculation for the specified parameters such as lateral stiffness, torsional stiffness, torsional radius, center of mass, center of stiffness and structural eccentricity, positive results are obtained, i.e. the investigated building is regular in plan. The obtained results and the implemented control of the analyzed parameters are shown in Table 1 and Table 2.

Criteria for regularity in elevation

After the conducted control for regularity in elevation, it is considered that the structure fulfills all requirements stated in MKS EN 1998-1/4.2.3.3[8], provided that only the upper part of the structure (above basement) is considered.

Table 1. Control of structural eccentricity

Floor	Level [m]	Torsional radius [m]		Eccentricity [m]		$0,3 \cdot r_x$ [m]	$0,3 \cdot r_y$ [m]	Control	
		$r_{x,i}$	$r_{y,i}$	$e_{ox,i}$	$e_{oy,i}$			$e_{ox} \leq 0,3 \cdot r_x$	$e_{oy} \leq 0,3 \cdot r_y$
Roof	+13,2	13,701	22,282	0,570	2,210	4,110	6,685	TRUE	TRUE
3	+10,2	15,140	18,532	0,560	2,370	4,542	5,560	TRUE	TRUE
2	+7,2	13,851	21,833	0,560	2,320	4,156	6,550	TRUE	TRUE
1	+4,2	15,643	17,880	0,550	2,30	4,693	5,364	TRUE	TRUE
Ground floor	+1,2	13,630	22,541	0,09	1,26	4,089	6,762	TRUE	TRUE

Table 2. Control of torsional radius

Floor	Level [m]	Torsional radius [m]		Radius of gyration [m]	Control	
		$r_{x,i}$	$r_{y,i}$	$l_{s,i}$	$r_{x,i} \geq l_{s,i}$	$r_{y,i} \geq l_{s,i}$
Roof	+13,2	13,701	22,282	8,312	TRUE	TRUE
3	+10,2	15,140	18,532	8,312	TRUE	TRUE
2	+7,2	13,851	21,833	8,312	TRUE	TRUE
1	+4,2	15,643	17,880	8,312	TRUE	TRUE
Ground floor	+1,2	13,630	22,541	8,129	TRUE	TRUE

2.5. Modal response spectrum analysis

Seismic action is taken into account through the implementation of modal response spectrum analysis, whereby it was performed independently for the ground excitation in two horizontal directions X and Y. For this purpose, design spectrum according to MKS EN 1998-1/3.2.2.5[8], is used. In doing so, spectrum Type 1 is chosen (MKS EN 1998-1/3.2.2.2). It is identified that the ground type, according to MKS EN 1998-1/Table 3.1, belongs to category B.

Periods, effective masses and modal shapes

In the modal response spectrum analysis 15 modes of vibration were taken into account and the sum of the effective modal masses amounts to 91,67 % of the total mass of the structure in direction X and 91,74 % in direction Y. In this way, the provision defined in MKS EN 1998-1/4.3.3.3.1[8], that this percentage has to be at least 90 %, is fulfilled. The three fundamental periods of vibration of the building amount to 0,31 s, 0,26 s and 0,21 s. The effective masses indicate that the first mode is predominantly translational in the Y direction, the second mode is translational in the X direction and the third mode is predominantly torsional. All three fundamental modes are shown in Fig. 5.

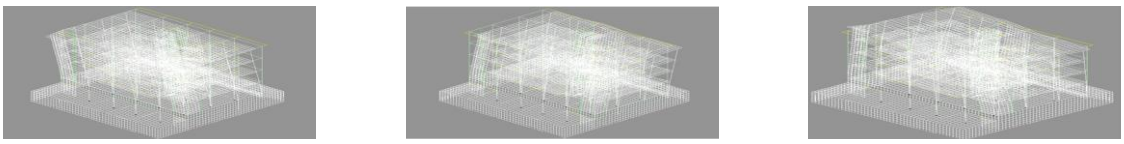


Fig.5. Mode 1 - translational in Y, Mode 2 - translational in X, Mode 3 - torsional

Behavior factor

Before calculating the behavior factor, it is necessary to determine the structural type of the building. After appropriate analysis it is concluded that this building belongs to a wall-equivalent dual system, where the shear resistance of the walls at the building base is greater than 50 % (51,63% in direction X and 57,83 % in direction Y). For this type of building the value of α_u/α_1 according to MKS EN 1998-1/5.2.2[8] amounts to 1,2, so that the value for q_o according to MKS EN 1998-1/Table 5.1[8] is equal to $3 \cdot 1{,}2=3{,}6$ for ductility class DCM. The factor k_w is equal to 1,0, therefore the behavior factor in both direction is equal to the basic value of the behavior factor $q=q_o=3{,}6$.

Peak ground acceleration and Design response spectrum

Design ground acceleration on type A ground a_g, which is one of the factor for defining the design response spectrum, is calculated as a product of reference peak ground acceleration on type A ground a_{gR} and importance factor γ_I. In fact, these two parameters have a crucial importance in this research, i.e. by varying their values, their influence on the behavior of reinforced concrete structural elements is analyzed. In the initial model the building is located in Skopje, so according to Seismic zones map (Fig. 6), peak ground acceleration a_{gR}=0,25g. In the same model, it is about a residential building, that belongs to importance class II – ordinary buildings (MKS EN 1998-1/Table 4.3[8]) and its value for importance factor γ_I=1,0. Finally, the value of design ground acceleration a_g=0,25g. The defined design response spectrum is shown in Fig. 7.

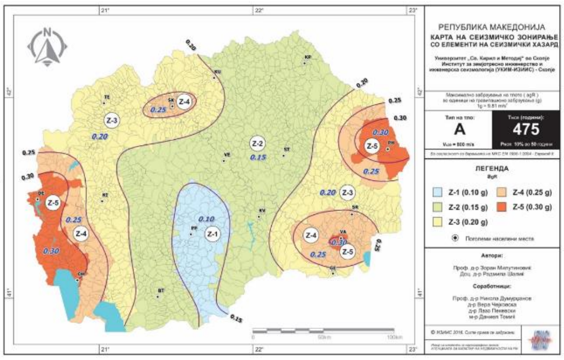


Fig. 6. Seismic zones map

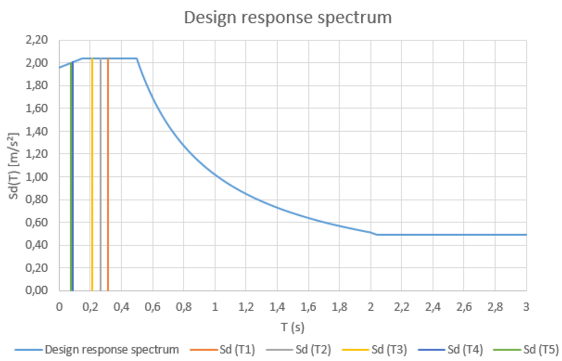


Fig. 7. Design response spectrum – initial model

Design and detailing of reinforced concrete structural elements

After the implementation of static and seismic analysis and generation of combinations of actions according to MKS EN 1990/6.4.3[3], the design values of the effect of actions in load bearing elements were obtained. The columns and beams are fully designed in bending and in shear, after checking the provisions listed in MKS EN 1992[7] and MKS EN 1998[8].

The design values for bending moments in beams and the design values for axial forces in columns are obtained through the analysis. Shear forces in beams are determined in accordance with the capacity design rule (MKS EN 1998-1/5.4.2.2[8]). It is specific that the design values for bending moments in columns are obtained either through the analysis, or through the capacity design requirement (MKS EN 1998-1/4.4.2.3[8]).

For the design process, there are selected three characteristic columns, that are the most loaded, and the frames in X and Y direction that they are part of: column C-3 (frame C in direction X and frame 3 in direction Y), column C-5 (frame C in direction X and frame 5 in direction Y) and column E-6 (frame E in direction X and frame 6 in direction Y).

When designing the columns in bending, there is a specific occurrence. Namely, when using bending moments obtained with analysis, all column sections have total longitudinal reinforcement ratio equal to the prescribed minimum ρ_l=1 % (MKS EN 1998-1/5.4.3.2.2[8]). But these values are not authoritative, because all analyzed sections, except the section at the level 1,2 at column C-3, are designed with values for bending moment obtained according to the capacity design rule (Table 3). In fact, they are on average more than 4 times larger. This results with an increase of the longitudinal reinforcement ratio, as follows: column C-3 → at level +10,2 m ρ_l=2,19 %, at level +7,2 m ρ_l=1,66 %, at level +4,2 m ρ_l=1,25 %; column C-5 → at level +10,2 m ρ_l=1,25 %; column E-6 → at level +10,2 m ρ_l=1,25 %. Changes occur on the higher floors, because in those sections the axial forces are lower, so they are generally exposed to biaxial bending. The biggest difference occurs in column C-3 at level +10,20 m, so these results are shown in Fig. 8. The increased values of bending moments in the lower levels do not affect the value of longitudinal reinforcement ratio, since those sections are mostly compressed, so ρ_l=1 %.

The presented conclusions require the need to emphasize the enormous importance of implementing the capacity design rule when designing columns. Such a design principle allows obtaining the hierarchy of resistance of the structural elements, necessary for ensuring the intended configuration of plastic hinges and for avoiding brittle failure modes.

When analyzing the failure modes in columns, it is concluded that all the sections have ductile failure. Namely, failure in sections occurs either by reaching the characteristic strain at maximum force in the reinforcement ε_{uk}=50 ‰, or by reaching ultimate compressive strain in the concrete ε_{cu}=3,5 ‰ and meanwhile the reinforcement is yielding. Hence follows the conclusion that another requirement for designing ductile reinforced concrete structural elements is fulfilled.

Table 3. Design bending moments and longitudinal reinforcement ratio in column C-3

Column C-3					
Level [m]		+1,20	+4,20	+7,20	+10,20
$M_{3\text{-}3,Ed}$ [kNm]	Analysis	95,89	75,88	65,30	36,23
	Capacity rule	87,24	87,24	100,36	100,36
$M_{2\text{-}2,Ed}$ [kNm]	Analysis	49,57	38,56	36,38	25,47
	Capacity rule	108,39	160,73	160,73	160,73
N_{ed} [kN]	Analysis	1337,97	823,40	380,34	91,97
A_s [ϕ]	/	12ϕ20	12ϕ20	16ϕ20	16ϕ20+4ϕ12
A_s [cm^2]	/	37,70	37,70	50,27	54,79
ρ_l [‰]	/	1,05	1,25	1,66	2,19

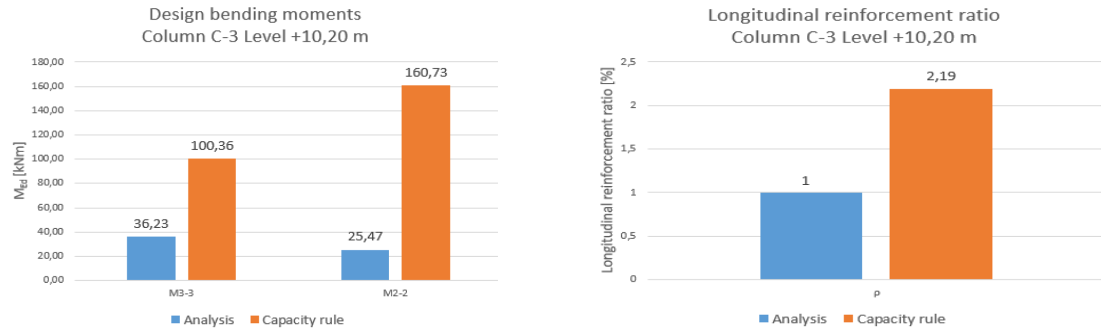


Fig. 8. Design bending moments and longitudinal reinforcement ratio in column C-3 level +10,20 m obtained through analysis and capacity rule

3. INFLUENCE OF SEISMIC HAZARD ON THE BEHAVIOR OF REINFORCED CONCRETE STRUCTURAL ELEMENTS

Seismic hazard according to MKS EN 1998 is defined by the value of reference peak ground acceleration on type A ground - a_{gR}. In compliance with Fig. 6, the territory of our country is divided into five seismic zones, where a_{gR}={0,10g; 0,15g; 0,20g; 0,25g; 0,30g}. The purpose of this part of the research is to vary this parameter, i.e. to conduct 4 more analyses, in the same way as it was shown in the Initial model chapter. It means that the building changes its location in each analysis (Skopje, Prilep, Kavadarci, Tetovo, Debar). It is important to say that in all analyses the importance class does not change, i.e. the importance factor has a constant value equal to 1,0. All other parameters defined in section 1.5 do not vary, too. The list of conducted analyses is shown in Table 4.

Table 4. List of conducted analysis when varying seismic hazard

Analysis	Seismic zone	Location	a_{gR}	Type of building	Importance class	γ_I	a_g
1 (Initial model)	Z-4	Skopje	0,25g	Residential building	II	1,0	0,25g
2	Z-1	Prilep	0,10g	Residential building	II	1,0	0,10g
3	Z-2	Kavadarci	0,15g	Residential building	II	1,0	0,15g
4	Z-3	Tetovo	0,20g	Residential building	II	1,0	0,20g
5	Z-5	Debar	0,30g	Residential building	II	1,0	0,30g

3.1. Influence of seismic hazard on the behavior of reinforced concrete beams

By varying the seismic hazard, changes in the design values of bending moments are noticed in the beams of RX-E, RY-3 and RY-6 frames. In fact, in those beam sections, the relevant bending combination includes a seismic load case, which is a direct cause of the resulting changes. The reason why seismicity has a dominant influence in those sections is the existence of reinforced concrete walls in the mentioned frames. Namely, in frame RX-E there are two walls W4, in frame RY-3 there is a wall W1 and in frame RY-6 there is a wall W4 (Fig.1). All of them affect the increase of bending stiffness in the corresponding direction, and this results in the attraction of greater seismic forces, whereby their value becomes dominant. As a consequence, the increase in bending moments affects the increase in the area of longitudinal reinforcement.

Opposite conclusion follows in the direction of the RY-5 and RX-C frames, where there is no existence of reinforced concrete walls, that would increase the stiffness. In fact, in the frame RX-C there is the wall W1, but its elevator core door openings reduce the bending stiffness. In these beam sections the relevant bending combination does not include a seismic load case, so the change of seismic hazard does not affect the design values of bending moments. This means that the area of longitudinal reinforcement has an immutable value.

Frame RX-E: Positive bending moments in each analysis have a mutual increase of 15-30%, i.e. on average positive bending moments obtained at a_{gR}=0,3g are 2,3 times greater than those obtained at a_{gR}=0,1g. Negative bending moments in each analysis have a mutual increase of 20-40%, i.e. on average negative bending moments obtained at a_{gR}=0,3g are 2,8 times greater than those obtained at a_{gR}=0,1g. As a consequence, the area of reinforcement obtained in the analysis where a_{gR}=0,3g is on average 2,2 times greater than the area when a_{gR}=0,1g.

Frame RY-3: Positive bending moments in each analysis have a mutual increase of 20-50%, i.e. on average positive bending moments obtained at a_{gR}=0,3g are 3 times greater than those obtained at a_{gR}=0,1g. As a result, the area of reinforcement obtained in the analysis where a_{gR}=0,3g is on average 2 times greater than the area when a_{gR}=0,1g.

Frame RY-6: Positive bending moments in each analysis have a mutual increase of 20-50%, i.e. on average positive bending moments obtained at a_{gR}=0,3g are 3 times greater than those obtained at a_{gR}=0,1g. For that cause, the area of reinforcement obtained in the analysis where a_{gR}=0,3g is on average 2,7 times greater than the area when a_{gR}=0,1g.

Overall, the design values of bending moments in beams and the corresponding area of reinforcement, when varying the reference peak ground acceleration from 0,1g to 0,3g, increase by 2-3 times.

3.2. Influence of seismic hazard on the behavior of reinforced concrete columns

The variation of seismic hazard, does not affect the design values of bending moments in column C-5. This is a consequence of the second conclusion in 2.1, i.e. the area of reinforcement in the beams, which are part of RX-C and RY-5 frames, is constant.

The average 30 % increase in bending moments in column C-3 results with 40% increase of geometric reinforcement ratio at the +7,20 m level, when comparing the values from the analyses a_{gR}=0,1g (ρ_l=1,25 %) and a_{gR}=0,3g (ρ_l=1,75 %). The greatest mutual increase of 33 % occurs when changing from a_{gR}=0,2g to a_{gR}=0,25g.

Because of the increase of longitudinal reinforcement in beams in frames RX-E and RY-6, there are changes in the geometric reinforced ratio in the sections of column E-6. In fact, the greatest impact is located at +10,20 m level, where the area of reinforcement increases for 50 %, comparing the results when a_{gR}=0,1g (A_L=25,13 cm^2) and a_{gR}=0,3g (A_L=37,70 cm^2).

The results for design bending moments and geometric reinforced ratio, when varying seismic hazard, for both sections are shown in Fig. 9.

Common feature for all column sections is the ductile failure, so that the strain in steel decreases up to 24 % in the section where the ultimate strain in concrete is achieved. This happens at level +7,20 m in column C-3, where ε_b=3,5 ‰ and ε_a varies from 24,077 ‰ (a_{gR}=0,1g) to 18,266 ‰ (a_{gR}=0,3g). When the ultimate strain in steel is achieved, the strain in concrete increases up to 94 %. This happens at level +4,20 m in column E-6, where ε_a=50 ‰ and ε_b varies from 0,986 ‰ (a_{gR}=0,1g) to 1,913 ‰ (a_{gR}=0,3g). In some sections, for different level of seismic hazard, different material reaches the ultimate strain. A characteristic example is the section at level +1,20 in column E-6, where ε_b varies from 1,921 ‰ (a_{gR}=0,1g) to 3,5 ‰ (a_{gR}=0,3g), i.e. 82 % increase, and ε_a changes from 50 ‰ (a_{gR}=0,1g) to 40,107 ‰ (a_{gR}=0,3g), i.e. 25 % decrease.

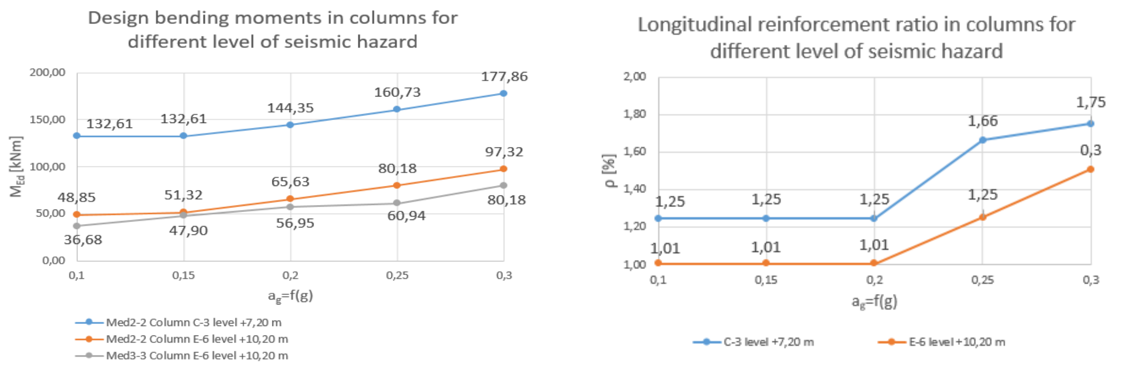


Fig. 9. Design bending moments and geometric reinforced ratio in specific sections in columns C-3 and E-6 for different level of seismic hazard

4. INFLUENCE OF IMPORTANCE CLASS ON THE BEHAVIOR OF REINFORCED CONCRETE STRUCTURAL ELEMENTS

In this part of the research, the influence of importance class on the behavior of structural elements is analyzed. It is implemented by varying the importance factor, which according to MKS EN 1998-1/4.2.5 takes different values depending on the importance class of the building, i.e. the consequences of its collapse for human life, and its importance for public safety in the immediate post-earthquake period.

In the sixth analysis a school is analyzed, i.e. the importance factor takes value of 1,2. In the last analysis a hospital is analyzed, which means that its integrity during earthquakes is of vital importance for civil protection (γ_I=1,4). The list of conducted analyses is shown in Table 5. It is important that the reference peak ground acceleration and all other parameters defined in section 1.5 remain unchanged.

Table 5. List of conducted analysis when varying importance class

Analysis	Seismic zone	Location	a_{gR}	Type of building	Importance class	γ_I	a_g
1 (Initial model)	Z-4	Skopje	0,25·g	Residential building	II	1,0	0,25·g
6	Z-4	Skopje	0,25·g	School	III	1,2	0,30·g
7	Z-4	Skopje	0,25·g	Hospital	IV	1,4	0,35·g

4.1. Influence of importance class on the behavior of reinforced concrete beams

By varying the importance class, changes occur only in the beams that are part of RX-E, RY-3 and RY-6 frames. It is a consequence of the facts explained in Chapter 2.1

Frame RX-E: Positive bending moments increase on average for 40 %, and negative bending moments for 35 %, when importance class changes from 1,0 to 1,4. As a result, the area of longitudinal reinforcement in beams is on average 30 % greater.

Frame RY-3: Positive bending moments increase on average for 42 %, when importance class changes from 1,0 to 1,4. As a result, the area of longitudinal reinforcement in beams is on average 15 % greater.

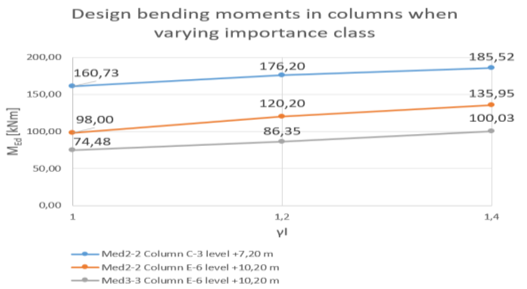
Frame RY-6: Positive bending moments increase on average for 41 %, when importance class changes from 1,0 to 1,4. As a result, the area of longitudinal reinforcement in beams is on average 17 % greater.

Overall, the design values of bending moments in beams increase on average 40% and the corresponding area of reinforcement increases on average 20 %, when varying the importance factor from 1,0 to 1,4.

4.2. Influence of importance class on the behavior of reinforced concrete columns

The variation of importance factor, does not affect the design values of bending moments and geometric reinforcement ratio in Column C-5. The reasons for this phenomenon are explained earlier in 2.1.

The average 15 % increase in bending moments in column C-3 results with 15 % increase of geometric reinforcement ratio at the +7,20 m level, when comparing the values from the analyses γ_I=1,0 (ρ_l=1,66 %) and γ_I=1,4 (ρ_l=1,90 %). Because of the increase of longitudinal reinforcement in beams in frames RX-E and RY-6, there are changes in the geometric reinforced ratio in the sections of column E-6. In fact, the greatest impact is located at +10,20 m level, where the area of reinforcement increases for 35 %, comparing the results when γ_I=1,0 (A_L=31,29 cm^2) and γ_I=1,4 (A_L=42,22 cm^2). The results for design bending moments and geometric reinforced ratio, when varying importance class, for both sections are shown in Fig. 10.



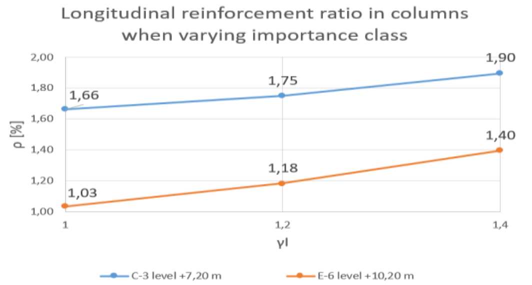


Fig. 10. Design bending moments and geometric reinforced ratio in specific sections in columns C-3 and E-6 when varying importance class

Common feature for all column sections is the ductile failure, so that the strain in steel decreases up to 34 % in the section where the ultimate strain in concrete is achieved. This happens at level +10,20 m in column C-5, where ε_b=3,5 ‰ and ε_a varies from 34,659 ‰ (γ_I=1,0) to 22,734 ‰ (γ_I=1,4). When the ultimate strain in steel is achieved, the strain in concrete increases up to 82 %. This happens at level +1,20 m in column C-3, where ε_a=50 ‰ and ε_b varies from 1,483 ‰ (γ_I=1,0) to 2,694 ‰ (γ_I=1,4). In some sections, for different level of importance class, different material reaches the ultimate strain. A characteristic example is the section at level +7,20 in column C-3, where ε_b varies from 1,829 ‰ (γ_I=1,0) to 3,5 ‰ (γ_I=1,4), i.e. 91 % increase, and ε_a changes from 50 ‰ (γ_I=1,0) to 32,156 ‰ (γ_I=1,4), i.e. 55 % decrease. The values of strain in steel and concrete in the specified sections, when varying important class, are shown in Fig. 11.

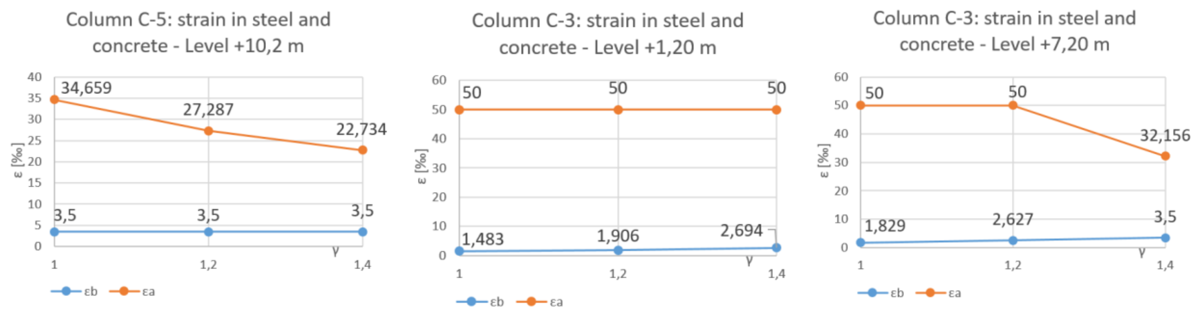


Fig. 11. Strain in steel and concrete in specific sections when varying importance class

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