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

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At the core of every project designed by structural engineers, lies the requirement that the structure has to maintain its integrity under different actions, among which the most specific is the seismic action. The Republic of North Macedonia is situated in region which is seismically very active, so we need to pay special attention to the seismic action and its influence on the behavior of reinforced concrete structural elements.

In this paper, parameter study and numerical analysis are performed by varying the seismic hazard of certain reinforced concrete building. According to the seismic zone map for our country, its territory is divided into five seismic zones. The variable parameter for each zone is the peak ground acceleration a_{gR} (PGA), which according to EN 1998-1 is used to define the seismic hazard. This parameter 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 seismic hazard affects the behavior of reinforced concrete structural elements.

It is expected that by increasing the value of PGA, the design values of the effects of actions in beams and columns will 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.

Overall, it may be said that the seismic hazard 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, peak ground acceleration, geometric reinforcement ratio, type of failure, reinforced concrete structural elements

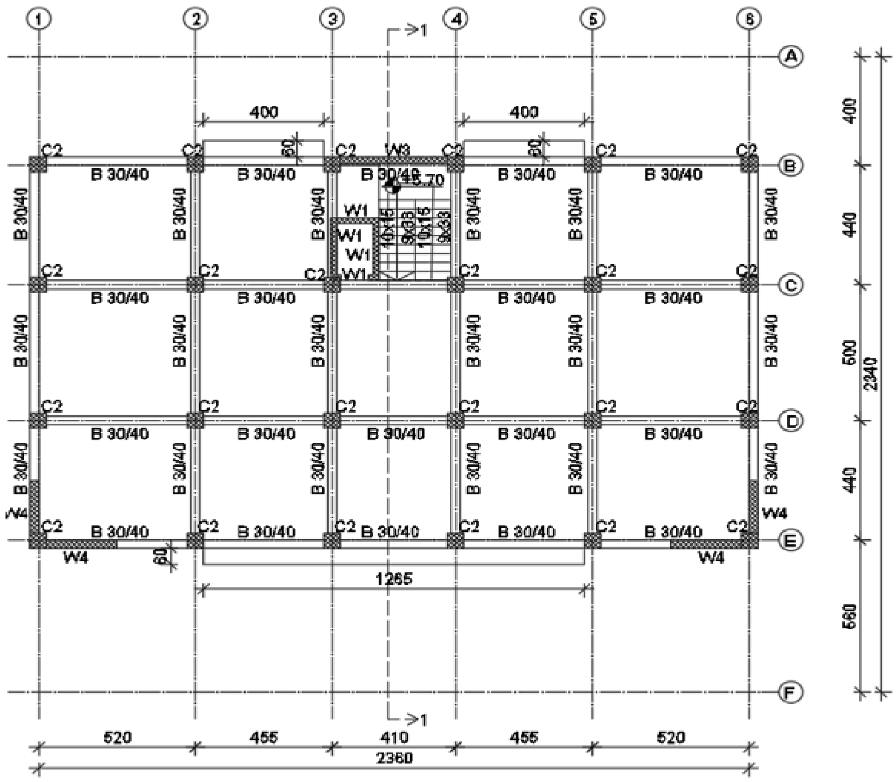

Figure 1. Typical floor plan of the building

1. INTRODUCTION

In this paper, the behaviour of reinforced concrete structure is analysed. The emphasis is on the seismic action, in a way that a parameter that defines this action, is 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 analysed sections.

2. INITIAL MODEL

2.1 DESCRIPTION OF THE BUILDING

The analyzed building in this paper is a multi-story reinforced concrete structure, which has 4 stories above the ground level and 1 basement. The height of each story is 3 m, the ground floor is raised for 1,20m above the terrain, so that the

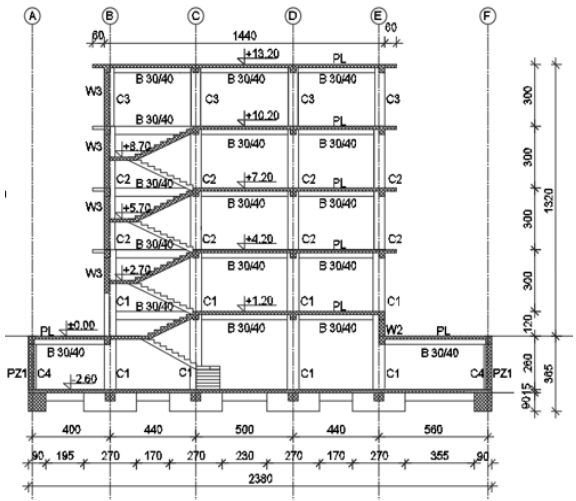

Figure 2. Cross section of the building

total height of the building above the terrain is 13,20m. The height of the basement is equal to 2,60m and the dimensions in plan of this level are 23,60 x 23,40m. The area of the other stories is smaller, and it is equal to 23,60 x 13,80m. The spans of the reinforced concrete frames in the direction of global axis X, are as follows: 5,60+4,40+5,0+4,40+4,00m. The spans of the frames in the direction of global axis Y are as follows: 5,20+4,55+4,10+4,55+5,20m. The typical floor plan and the cross section of the building are shown in Fig.1 and Fig.2.

The structural system consists of frames and walls. The columns are with square cross section with side lengths of 60cm in the basement and ground floor, 55cm on the first and second floor and 50cm on the third floor. The beams are with rectangular cross section 30x40 cm. The slab is 15cm thick. The vertical communication in the building is enabled through two-legged staircase with 15cm thick slab and lift core which includes 20cm thick walls. In the basement, there are peripheral walls, whose thickness is 30cm. 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 50cm 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 is 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 is 5%. The structure is designed for ductility class DCM. 3D view and view in X and Y direction of the structural model is shown in Fig.3 and Fig.4.

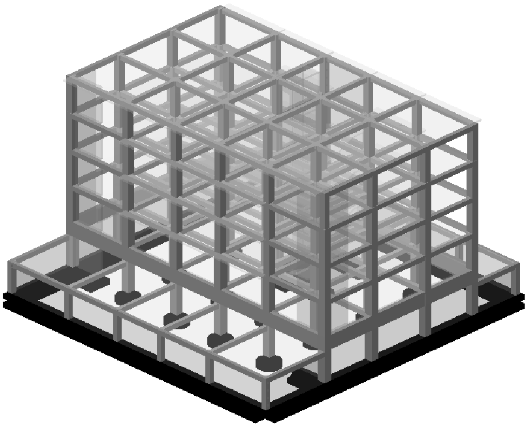

Figure 3. Structural model – 3D view

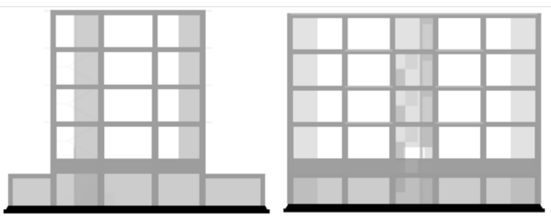

Figure 4. Structural model – Y and X direction

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 as 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. The analyzed building is a residential building, so 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 Q, as 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 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

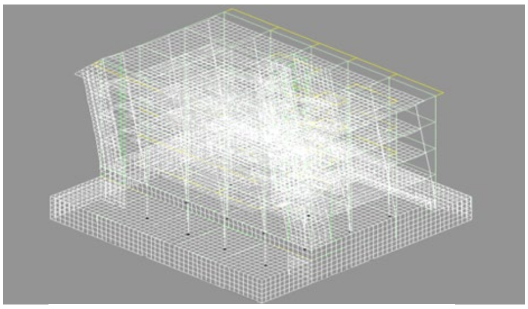

Figure 5. Mode 1 – translational in Y

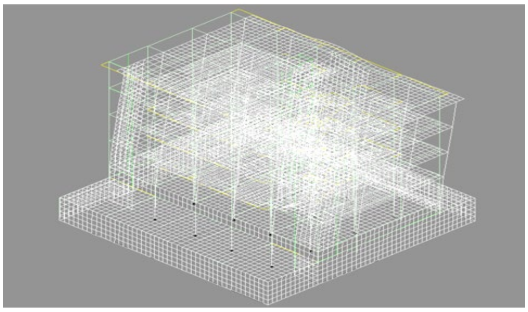

Figure 6. Mode 2 – translational in X

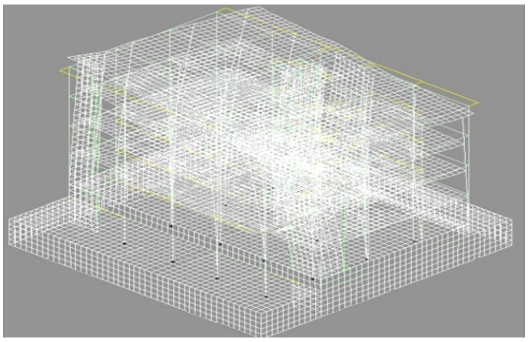

Figure 7. Mode 3 – torsional

obtained, i.e. the investigated building is regular in plan.

Criteria for regularity in elevation

After the conducted control for regularity in elevation, it is concluded 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.

2.5 MODAL RESPONSE SPECTRUM ANALYSIS

Seismic action is taken into account through the implementation of modal response spectrum analysis, whereby it was analyzed 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 is 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 are 0,31s, 0,26s and 0,21s. 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. 6 and Fig. 7.

Behaviour 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 factors 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 the initial model the building is located in Skopje, so according to Seismic zones map (Fig. 8), peak ground acceleration a_{gR} = 0,25g. Since the analyzed building is a residential building, it 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

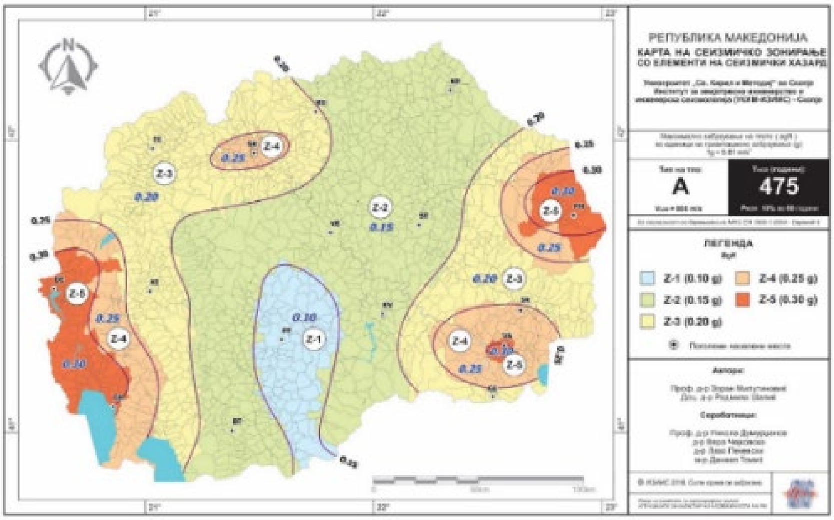

Figure 8. Seismic zones map

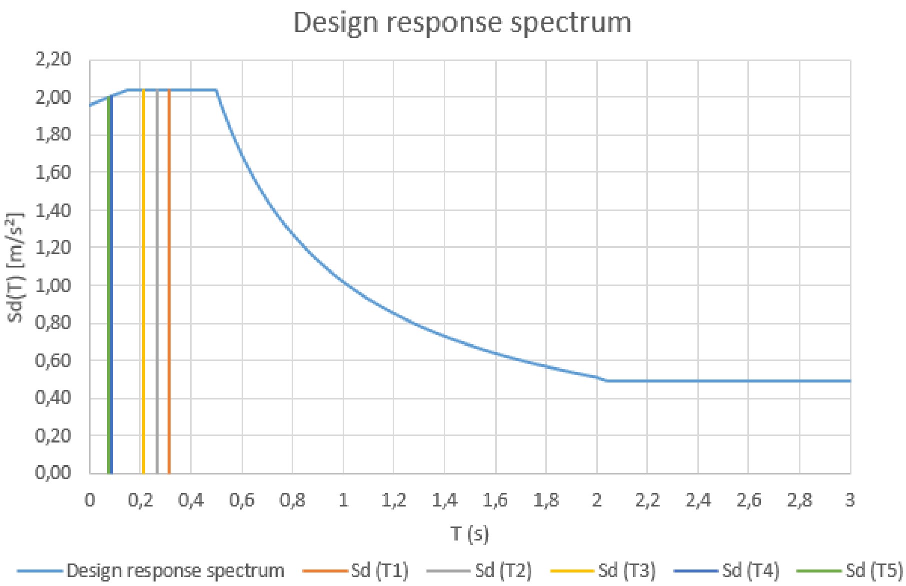

Figure 9. Design response spectrum – initial model

acceleration a_g=0,25g. The defined design response spectrum is shown in Fig. 9.

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].

For analysis and comparison of the results, three characteristic most loaded columns are selected, 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).

3. INFLUENCE OF THE 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. 8, 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 chapter 2 of this paper. It means that the building will change 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 1.

Table 1. List of conducted analysis when varying the location, i.e. seismic hazard

Analysis	Seismic zone	Location	a_{gR}
1 (Initial model)	Z-4	Skopje	0,25g
2	Z-1	Prilep	0,10g
3	Z-2	Kavadarci	0,15g
4	Z-3	Tetovo	0,20g
5	Z-5	Debar	0,30g

3.1 INFLUENCE OF THE SEISMIC HAZARD ON THE BEHAVIOR OF REINFORCED CONCRETE BEAMS

By varying the location, i.e. 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}=0,1 g.

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}=0,3 g are 3 times greater than those obtained at a_{gR}=0,1 g. As a result, the area of reinforcement obtained in the analysis where a_{gR}=0,3 g is on average 2 times greater than the area when a_{gR}=0,1 g.

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}=0,3 g are 3 times greater than those obtained at a_{gR}=0,1 g. 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}=0,1 g.

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

3.2 INFLUENCE OF THE SEISMIC HAZARD ON THE BEHAVIOR OF REINFORCED CONCRETE COLUMNS

The variation of the location, i.e. seismic hazard, does not affect the design values of bending moments in column C-5. This is a consequence of the second conclusion in 3.1, i.e. the area of reinforcement in the beams, which are part of RX-C and RY-5 frames, is constant.

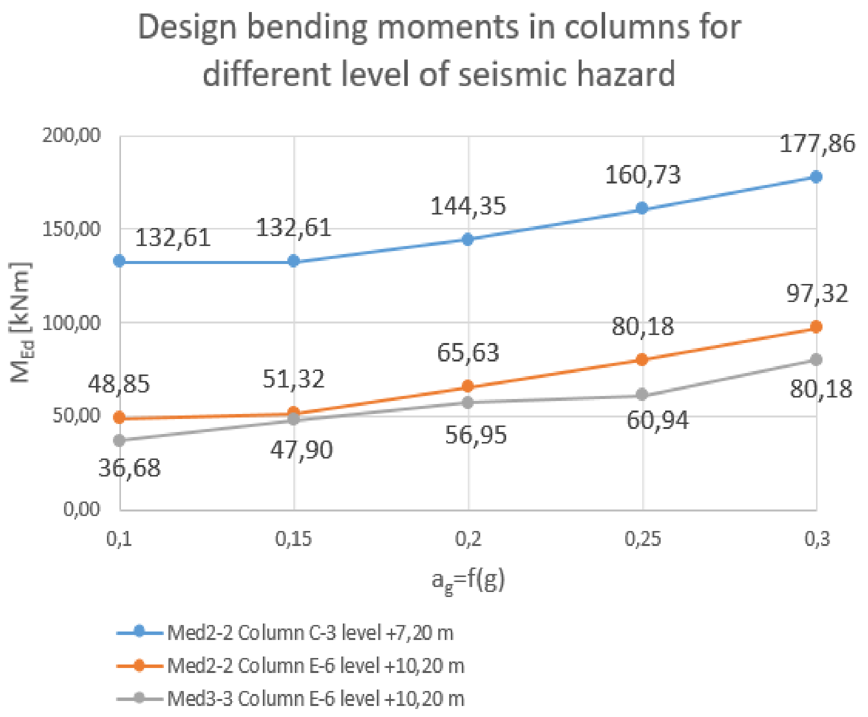

Figure 10. Design bending moments in specific sections in columns C-3 and E-6 for different level of seismic hazard

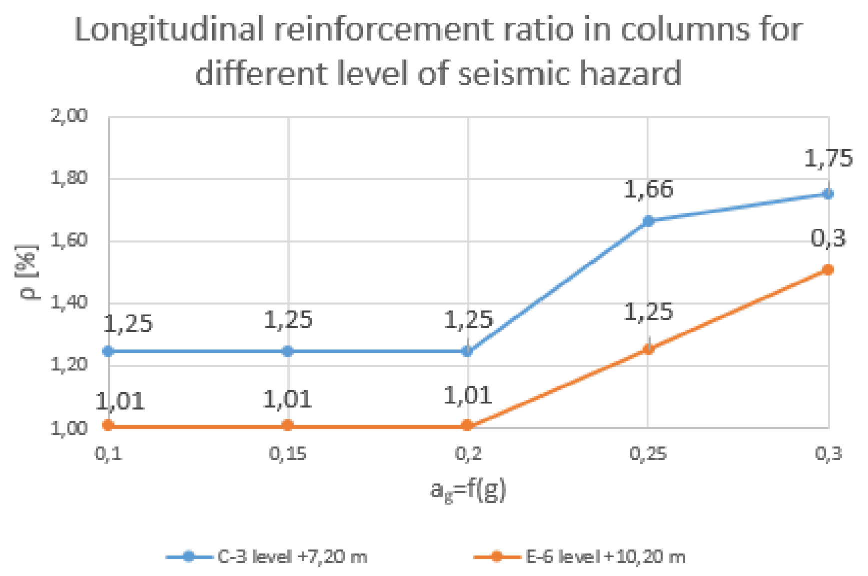

Figure 11. Geometric reinforced ratio in specific sections in columns C-3 and E-6 for different level of seismic hazard

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}=0,1 g (ρ_l=1,25%) and a_{gR}=0,3 g (ρ_l=1,75%). The greatest mutual increase of 33% occurs when changing from a_{gR}=0,2 g to a_{gR}=0,25 g.

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 influence is at +10,2 m level, where the area of reinforcement increases for 50 %, comparing the results when a_{gR}=0,1 g (A_L=25,13 cm^2) and a_{gR}=0,3 g (A_L=37,70cm^2).

The results for design bending moments and geometric reinforced ratio, when varying seismic hazard, for both sections are shown in Fig. 10 and Fig. 11.

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}=0,1 g) to 18,266‰ (a_{gR}=0,3 g). When the ultimate strain in steel is achieved, the strain in concrete increases up to 94%. This happens at level +4,20m in column E-6, where ε_a=50‰ and ε_b varies from 0,986‰ (a_{gR}=0,1 g) to 1,913‰ (a_{gR}=0,3 g). 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}=0,1 g) to 3,5‰ (a_{gR}=0,3 g), i.e. 82% increase, and ε_a changes from 50‰ (a_{gR}=0,1 g) to 40,107‰ (a_{gR}=0,3 g), i.e. 25% decrease.

REFERENCES

[1] Atanasovski S. 1992: Armiranobetonski konstrukcii I. December 1990. Faculty of Civil Engineering – Skopje. Skopje. p. 480.

[2] Atanasovski S. 2007: Betonski konstrukcii. Faculty of Civil Engineering – Skopje. Skopje. p. 311.

[3] EN1990: Eurocode – Basis of structural design, April 2022.

[4] EN1991-1-1: Eurocode 1: Actions on structures – Part 1 – 1: General actions – Densities, self-weight, imposed loads for building, April 2022.

[5] EN1991-1-3: Eurocode 1: Actions on structures – Part 1 – 3: General actions – Snow loads, July 2003.

[6] EN 1991-1-4: Eurocode 1: Actions on structures – Part 1 – 4: General actions – Wind actions, April 2005.

[7] EN 1992-1-1: Eurocode 2: Design of concrete structures – Part 1 – 1: General rules and rules for buildings, December 2004.

[8] EN 1998-1: Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings, December 2004.

[9] Lazarov Lj., Todorov K. 8. Praktichen primer – Analiza na 8. Katna armiranobetonska konstrukcija. p. 55.

[10] Markovski G., Docevska-Jovanova M., Janev D., Stojkovska E. 2023: Armiran beton spored Evrokod 2. February 2023. Faculty of Civil Engineering – Skopje. Skopje. p. 343.

[11] MKS EN 1990: 2012/HA:2012

[12] MKS EN 1991 – 1 – 3: 2012/HA:2020

[13] MKS EN 1991 – 1 – 4:2012/HA:2020

[14] MKS EN 1998 – 1/HA:2020