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THE INFLUENCE OF SECTION SIDES RATIO OF RECTANGULAR COLUMN ON SEISMIC RESPONSE OF RC BUILDING

Igor Gjorgjiev (1), Aleksandar Zhurovski (2), Borjan Petreski (3)

(1) *Prof., Ss. Cyril and Methodius University in Skopje, Institute of Earthquake Engineering and Engineering Seismology, Skopje, Republic of North Macedonia, igorg@iziis.ukim.edu.mk*
(2) *PhD Student, Ss. Cyril and Methodius University in Skopje, Institute of Earthquake Engineering and Engineering Seismology, Skopje, Republic of North Macedonia, zurovski@iziis.ukim.edu.mk*
(2) *PhD Student, Ss. Cyril and Methodius University in Skopje, Institute of Earthquake Engineering and Engineering Seismology, Skopje, Republic of North Macedonia, borjan@iziis.ukim.edu.mk*

Abstract

The response of structures exposed to earthquakes is mainly defined by the stiffness, capacity and ductility of the structural elements which is governed by their dimensions and the material properties. The buildings' seismic performance is essential for their earthquake resilience especially beyond safety point defined by the building code. The main philosophy of modern codes is to increase the earthquake resiliency of the buildings. Unfortunately, Macedonia is one of the last European countries where designing the structures according to outdated codes is allowed. The older national seismic codes have not been modified in view of the implementation of the most recent knowledge. In this sense, there are no strict limits to prevent the design of rectangular columns with an unsuitable ratio of the section sides. Mainly, due to the need for greater open space and flat interior walls, the columns are designed as rectangular where the lower section side is oriented along a larger span. This results in decrease of the global structural capacity in that direction affecting its response under earthquake excitation. In order to investigate the influence of the ratio of section sides of a rectangular column on seismic performance of building structures, an existing RC frame structure was chosen for analysis. The results of the performed seismic assessment of the selected structure by nonlinear static analysis, emphasize the importance of choosing square shaped columns or rectangular columns with appropriate arrangement in plan even for low-rise buildings. Based on the analysis' results, we conclude that designers need to pay more attention when choosing columns' cross section dimensions and orientation to achieve an acceptable building resilience against earthquakes.

Keywords: nonlinear static analysis; RC buildings; column cross-section; global response; element behavior

1. Introduction

The whole territory of N. Macedonia is in a seismically prone region where earthquakes with intensity between VII and IX according to European intensity scale can occur. Since Macedonia is located in the central Balkan Peninsula, a region often in the history exposed by the devastating nature of strong earthquakes [1], there have been several earthquakes in recent history that have resulted in considerable economic damage and social disruption. The building stock in the country is generally characterized by two types of existing buildings. The first type includes the buildings constructed prior to 1969 which do not meet the current seismic design standards. The second type covers newly build structures which are designed according to different seismic design standards and have different seismic capacity.

Because the buildings' seismic performance is essential for their earthquake resilience, especially beyond safety point defined by the building code, it is crucial to make a proper arrangement of the columns during building design. The response of structures exposed to earthquakes is mainly defined by the stiffness, strength, and ductility of the structural elements, resulting in varying seismic capacity. Therefore, the seismic capacity of characteristic existing buildings should be evaluated.

Unfortunately, N. Macedonia is one of the last European countries where the design of structures according to outdated codes is allowed. The national seismic codes have not been modified yet, in view of the implementation of the most recent advancements in the earthquake engineering field. In this

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sense, there are no strict limits for design of beam-columns joints for satisfying the moment capacity and achieving an acceptable level of ductility through 'strong column/weak beam' principle. Nowadays, due to the architectural demand for more open space, the columns are designed as rectangular where the shorter cross-section side is oriented along a larger span. This results in decrease of the column capacity in that direction, which affects the global response of the structure under earthquake excitation. In order to investigate the influence of the ratio of section sides of a rectangular column on the seismic performance of building structures, an existing RC frame structure designed according to the Macedonian seismic codes of practice was chosen for analysis. The analyzed structure is a three-story residential building which is regular in plan and elevation.

The analysis of the structure involves applying nonlinear static analysis to the designed structure and evaluating the performance through a pushover curve. In order to demonstrate the differences between the design of the structure following Macedonian codes of practice and more sophisticated capacity-based design procedure, pushover curves comparison is performed. Initially, the structure is analyzed as designed, with the existing column arrangement and direction – Model 3. Then, three models (Model 1, 2 and 4) based on the designed structure, but with different column arrangements are developed. Subsequently, pushover curves are calculated for each model and the comparison of the results is performed.

2. Analysis Procedure

Recently, there has been an increase in the use of nonlinear static analyses, in contrast to the ductility-modified spectra analyses. The nonlinear static analysis, also called pushover analysis, perfectly defines the deformation curve of a structure and shows its capacity through a pushover curve. This method is considered a step forward from the use of linear analysis and ductility-modified response spectra because it is based on a more accurate estimate of the distributed yielding within a structure, rather than an assumed, uniform ductility [2]. According to the NEHRP classification [3], the nonlinear response analysis is performed for many reasons including the designing of new buildings and assessing the performance of new and existing buildings. Additionally, a number of codes and guidelines have indicated it as a preferred assessment procedure [4].

For the evaluation of the performance of the investigated structural configurations, static pushover analyses were performed. A predefined set of lateral forces was applied to the structures stepwise, as a function of their masses and first mode shape vectors. At each step of the analyses, the yielding of beams or columns was accounted for by updating their stiffness matrices. The pushover analyses were performed until the structures reached collapse.

Considering the outdated Macedonian seismic design code, the performance levels for the assessment of the structures were obtained from more advanced contemporary codes of practice. For example, the Italian National Code defines 4 limit states: Operational, Damage Control, Life Safety and Collapse Prevention [5]. Based on FEMA 273 recommendations [6] for concrete frames, drift limit indicators are provided following the structure's response to different performance levels. FEMA 273 categorizes the drift limitations in three classes: 1) immediate occupancy, represented with 1% transient and negligible residual drift, 2) life-safety, exhibiting maximum of 2% transient and 1% residual drifts, and 3) collapse prevention, limited with 4% of transient and residual drifts. Therefore, in this study, a combination of the codes of practice can be observed as the FEMA 273 performance assessment limits are used for concrete frames designed following the Macedonian code of practice.

3. Case Study

3.1 Structural description of the building

The residential building that is the subject of this research is an RC frame structure. Due to the need for greater open space and flat interior walls, the columns were designed as rectangular, with shorter section side oriented along a larger span. The shape of the building in the plan is rectangular, and can be classified as regular. There are three floors with a story height of 2.9m. The spans in x-direction are 6.4

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and 5.65m, while the spans in y-direction are 3.4 and 4.5m. The beams are rectangular with dimensions 25/40cm and 25/45cm, while all columns are with dimension 25/45cm. This structural configuration is in accordance with the analytical model 3.

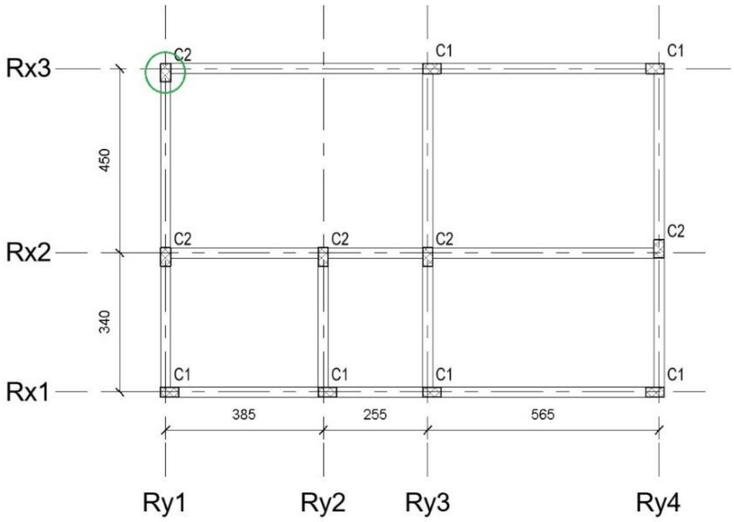

Figure 1. Layout of the building (Model 3).

Four structural analytical models were used to study the effect of column cross-section orientation on structural seismic capacity. The need for the layout given in Figure 1 is due to an architectural demand. Some columns are oriented with shorter section side along the longer span. In Model 1, the shorter section side of all columns is oriented along direction Y (C1 - 45/25 cm), while in Model 2, the shorter section side of all columns is oriented along direction X (C2 - 25/45 cm). The third analytical model represents a mixed arrangement of columns C1 and C2 (Figure 1) as it was designed. The last model - Model 4, is a structure where the cross-section of all columns is square (C4 - 35x35cm).

3.2 Numerical modeling

Perform3D software for nonlinear analysis and design was used to model and analyze the structure [7]. It offers several different types of elements for use, depending on the modeling requirements. Each element consists of numerous basic components that are used to simulate the nonlinear behavior of all types of structural elements. For this case study, the basic components used for the modelling are as follows:

- Inelastic non-buckling steel material was defined for the reinforcement according to the provisions given in the national standard. The stress-strain relationship of the reinforcement was modelled as trilinear, with strength loss (at point L), without cyclic stiffness degradation. The model curve is shown in Figure 2 (left).
- A macro model with Mander stress-strain relationship is most frequently used to describe the working condition of confined concrete in uniaxial compression [8], which is related to section shape and the configuration condition of the stirrup. By the program, the model is transferred in the standard force-deformation (F-D) relationship. Hence, for the design quality of concrete, a concrete model curve was computed according to the Mander model, mean value of concrete's strength and elastic modulus (Figure 2 - right).

The steel material properties are characterized with yield strength of f_y=400MPa, yield strain of ε_y=2‰, ultimate strength of f_u=500MPa and the ultimate strain of ε_u =25‰. The concrete material properties are obtained from the Mander stress-strain model for confined concrete. Therefore, the characteristic values are f_{co}'=30MPa, f_{cc}'=35MPa, ε_{co}=2‰, ε_{co}=3.5‰ and ε_{cu}=17.5‰ (the ultimate compressive strain ε_{cu} is obtained at first hoop fracture).

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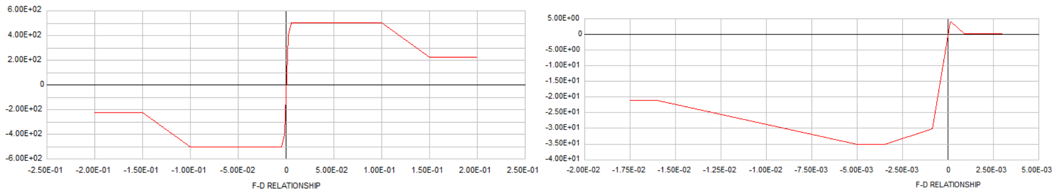

Figure 2. Material models for reinforcement (left) and confined concrete (right).

The columns were modelled using inelastic fiber sections that were assigned at the corresponding lengths of the elements to simulate the plastic hinge region, with the rest of the element remaining elastic. Concentrated (lumped) plasticity approach was used for the beams using inelastic moment hinges, rotation type. Beam-column joints were considered as rigid elements.

The deformation capacities for the components or the so-called performance acceptance criteria corresponding to different building performance levels are generally defined in terms of plastic rotation capacities. In the present study, plastic rotation capacities for Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) levels given in ASCE-41[9] were used for the column and beam elements (Table 1). In the analysis, only one criterium was used for performance-based evaluation. Namely, the performance of the structure was evaluated based on the flexural behavior of the columns only. It was assumed that the flexural behavior of the columns governs the structure's behavior, and the columns define the structure's failure mechanism. For the analysis, the rotation capacities in columns were used as referent. The damaged state of the structure corresponded to reaching the specified level of rotation in columns.

Table 1 – Rotation capacities for the structural elements used in the analysis

Level	Column Rotation Capacities (rad)	Beam Rotation Capacities (rad)	Color
IO	0,005	0,005	
LS	0,025	0,025	
CP	0,035	0,05	

3.3 Evaluation of the seismic performance

The moment-curvature relationships of a column section are highly important to assess the ductility of the element, the amount of the possible redistribution of stresses and its resistance against dynamic loading. The tool developed by Bentz and Collins [10] is used to calculate the moment-curvature relationship of the three implemented column cross sections. Moment-curvature relationships of the column cross-sections were calculated along the local x-axis for columns C1(45x25cm), C2(25x45cm) and C4(35x35cm) for predefined axial force from the gravity loads (Figure 3).

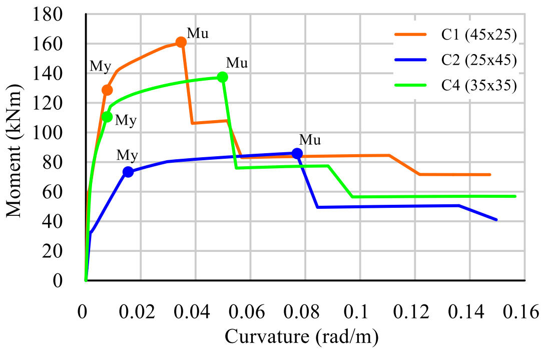

Figure 3. Moment-curvature relationships of the column cross-sections for axial force from gravity loads.

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As seen from Figure 3, the column C1 has the largest moment capacity, whereas the column C2 has the lowest moment capacity. The yield moment M_y for column C1 is 134kNm for C2 is 76kNm and for C4 is 112kNm. The ultimate moment M_u for column C1 is 167kNm for C2 is 92kNm and for C4 is 141kNm.

The calculated curvatures at yield and ultimate moments of the columns for three different levels of axial force are given in Table 2. The curvature ductility of all concrete columns is calculated accordingly. It is defined as the ratio of ultimate curvature to yield curvature and it presents a metric for warning of failure. The ductility for design gravity axial load for C1 is 5.05, while for C2 and C3 it is 6.73. The columns C2 and C4 have larger ductility than column C1. Therefore, the square column C4 shows balanced behavior regarding the ductility and moment capacity along both axes compared to the rectangular columns investigated. Table 2 consists of two additional axial load levels that show the variations in axial load in the column due to the pushover analysis.

Table 2 – Rotation capacities for the structural elements used in the analysis

Column	**N = 380kN**			**N = 300kN**			**N = 550kN**		
	φ_y [rad/m]	**φ_u [rad/m]**	**μ_φ**	**φ_y [rad/m]**	**φ_u [rad/m]**	**μ_φ**	**φ_y [rad/m]**	**φ_u [rad/m]**	**μ_φ**
C1 (45x25cm)	0.00767	0.03878	5.05	0.00767	0.03525	4.59	0.00844	0.03878	4.59
C2 (25x45cm)	0.01141	0.07678	6.73	0.01141	0.07678	6.73	0.01256	0.07678	6.12
C4 (35x35cm)	0.00741	0.04986	6.73	0.00741	0.04986	6.73	0.00815	0.04986	6.12

The results obtained from the nonlinear static analysis in terms of Pushover curves (Base shear vs. Top Drift) for the four models in both x and y orthogonal directions are shown in Figure 4. On the diagrams, several top drift levels arranged in regions (cyan, orange, red) are marked. The lines that lie in these regions were set by following the response of the structure under the increasing displacement in the analysis and the nonlinear response of the columns, as they are governing the stability of the building structure. The acceptance criteria directly follow the rotation capacities given in table 1. Top drift when the structure reaches IO, LS and CP damage states (acceptance criteria) is marked.

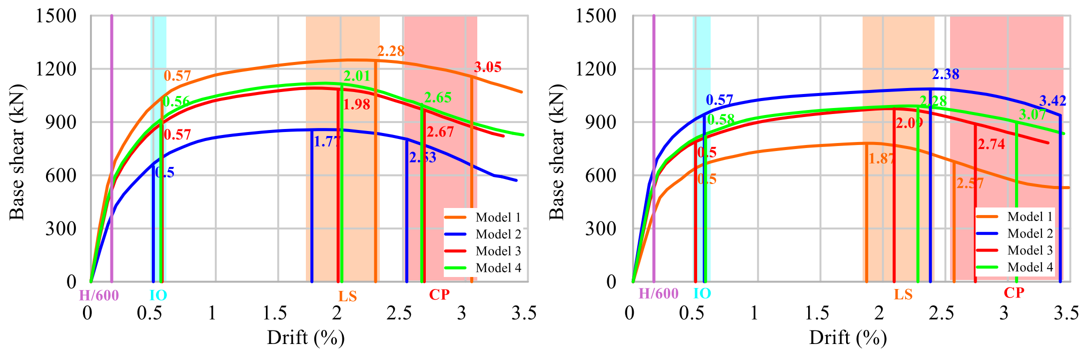

Figure 4. Pushover curves for Models 1, 2, 3 and 4 in x-direction (left) and y-direction (right).

It is observed from Figure 4 that the base shear capacity is highly dependent on the column cross-section orientation in plan. Therefore, Model 1 shows highest base shear capacity in x-direction (the direction of the longer column cross-section side), while Model 2 shows highest base shear capacity in y-direction. It is noted that Models' 3 and 4 base shear capacity is similar and demonstrates uniform capacity in both directions. This is a demonstration of balanced choice of column cross-section orientation for Model 3 and accordingly good choice of rectangular column cross-sections for Model 4. On the other hand, Models' 1 and 2 base shear capacity in both orthogonal directions shows differences of more than 30%.

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Regarding the Macedonian code of practice and its total horizontal deflection limitations (H/600, where H is the total height of the structure), it is observed that the IO performance criteria are reached at greater drift level for every model. This demonstrates the stringency of the Macedonian seismic code of practice since the total drift limitation is at 1/3 of the earliest plastic rotation capacity of a column as per the IO performance criterium.

Another parameter investigated is the design base shear according to the Macedonian seismic code. The structures are designed with a total base shear load of 350kN, for which the previously mentioned total drift limitation is assessed. This study demonstrates that the drift of the lowest capacity models in each direction is closest to the H/600 limitation, while Models 3 and 4 exhibit lower drifts for the same base shear load.

The performance criteria levels also show varying response of the models. For instance, the IO performance level region is quite narrow, showing similar results for each model with the highest difference being 16%. The LS and CP regions, on the other hand, demonstrate greater variation in the models' behaviour. They show wider drift regions associated with the plastic rotation capacities of the columns for LS and CP performance levels. The width of the LS and CP regions along with the narrowness of the IO performance level region indicate varying global structural ductility of the models investigated.

Figure 5 contains the damage state of the structure and the performance of the columns when the IO damage state is reached. It should be noted that the color when a column reaches certain rotation capacity limit is given in table 1 and it corresponds to the colors given in the diagrams and figures.

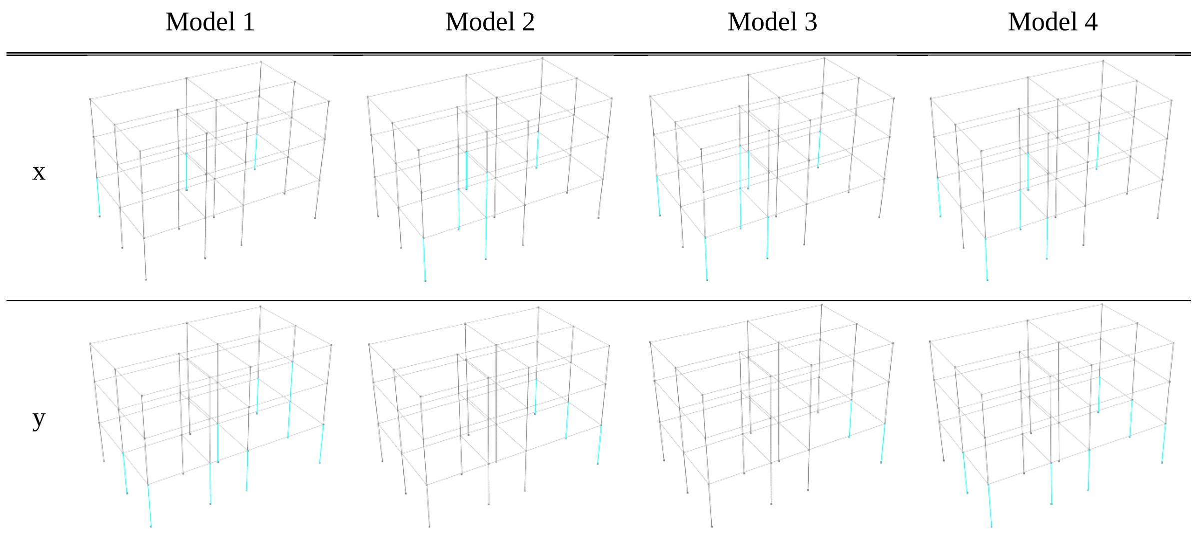

Figure 5. IO damage states in x and y direction for Models 1, 2, 3 and 4.

Figure 5 demonstrates some similarities between the investigated models. Previously, it was established that for IO performance level, the response of the four models is showing similar results. Thus, figure 5 additionally shows that IO is reached only for the ground floor columns for every model. However, the pattern for each model is different and governed by the column cross-section orientation and span length. The three-dimensional analysis additionally allows for different redistribution of loads for each model and affects the varying plastic rotation demand pattern.

After the evaluation of the global response of the four models, the study focuses on the response of a selected column (green circled in Figure 1). The response of the selected column is investigated in order to discuss the influence of the choice of different type of column on its response under the pushover loading. Figure 6 presents two graphs depicting the relationship between the ground floor interstorey drift and bending moment of the selected column for each direction. The graphs contain four curves that correspond to the four models and the appropriate regions of yielding moment for the selected

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column. The yielding moment regions are defined by the boundary values of calculated yielding moment for different levels of axial force. The graphs also contain two limit states defined in the national regulations, namely h/150=0.667% (allowable interstorey drift for design earthquake i.e. moderate nonlinear deformation in the structure), and h/350=0.2857% (allowable interstorey drift for linear behavior of the structure).

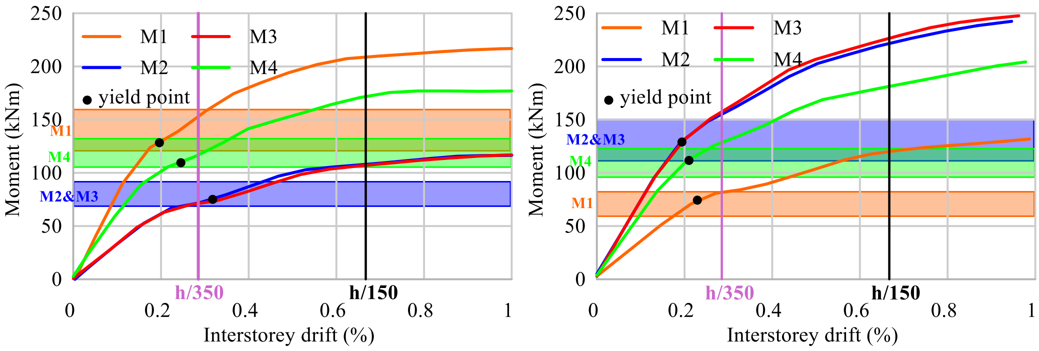

Figure 6. Moment in column vs. Interstorey drift in x–direction (left) and y–direction (right).

Generally, the results from Figure 6 demonstrate that when the ground level reaches the linear allowable interstorey drift limit, the selected column is already yielding. The column in Models 2 and 3 in x-direction reaches the yielding moment after reaching the allowed interstorey drift, which can be considered as favorable response. It is also important to note that the model with a square shaped column (M4) gives certain level of balanced behavior. The column reaches yielding in both orthogonal directions before the ground floor reaches linear interstorey drift limit. The hatched regions represent the yielding moment boundaries for varying axial force levels in the column caused by horizontal loading. The yielding moment associated with greater axial force in x-direction for all four models occurs after the code's linear interstorey drift limit. In the case for y-direction, all models experience yielding before linear interstorey drift limit (H/350).

Figure 7 contains two graphs showing the relationship between total base shear and bending moment in selected column (7 - left - pushover in X direction, 7 - right - pushover in Y direction). Four curves (M1, M2, M3 & M4) corresponding to the four models and the yielding moment regions of the column are shown. The yielding moment regions for each model (Model 1, 2, 3 & 4) are defined by the boundary values of calculated yielding moment for different levels of axial force, same as above (Figure 6). The results in x-direction showed that the yielding moment for M2 occurs at the lowest base shear, while for the other models, the yielding moment occurs at similar base shear. Considering y-direction, the yielding moment for M1 occurs at the lowest base shear. For the other models, there is a slight difference in base shear at which the yielding moment occurs. The yielding moment for M3 and M4 occurs at a similar base shear force in the x and y directions, indicating that both structural configurations behave similarly. Also, it can be noticed that the yielding moment for M3 is lower than the yielding moment for M4 for the same base shear force.

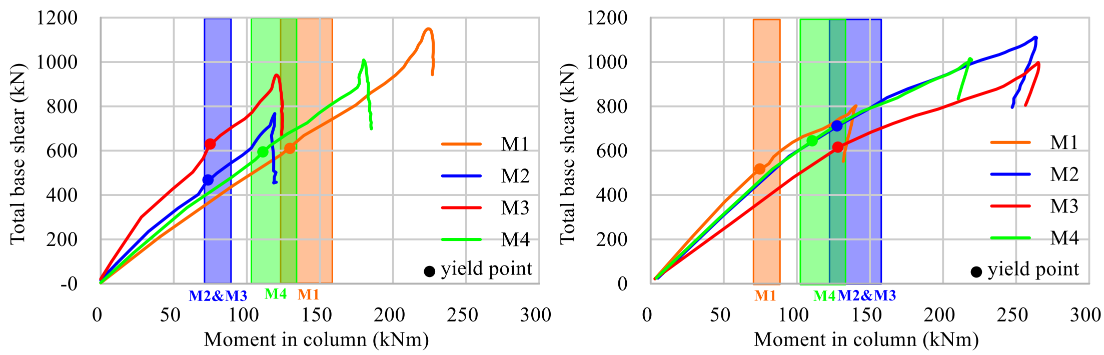

Figure 7. Moment in column vs. Base shear in x–direction (left) and y–direction (right).

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The results of the performed seismic assessment of four structural configurations by nonlinear static analysis, emphasize the importance of choosing the appropriate column arrangement in plan even for low-rise buildings. Therefore, the designers need to pay more attention when choosing columns' cross section dimensions and their orientation to achieve acceptable building resilience against earthquakes. The investigated influence of the column section orientation on seismic performance of building structures showed that the model with squared column cross-section demonstrated the most balanced behavior in both directions.

4. Conclusions

A comparison arising from an ongoing problem in the structural engineering field was portrayed in this study. Using a more sophisticated nonlinear static analysis for the assessment of the structural response of low rise reinforced concrete residential buildings, several cases were examined.

The results of the performed seismic assessment of the selected structures by nonlinear static analysis, emphasize the importance of choosing square shape columns or rectangular columns with appropriate arrangement even for low-rise buildings. When choosing rectangular shaped columns with relatively small dimension of a single cross-section side, greater attention should be paid to the layout and distribution of columns in plan, in order to provide sufficient bearing capacity and deformability of the structure in both orthogonal directions. Although the structure's global behavior can be optimized with these measures, it remains questionable if the rectangular shaped columns provide sufficient resilience against earthquakes. In any case, the square shaped columns also demonstrated more optimal balanced element behavior. Based on the analysis results, we conclude that designers need to pay more attention when choosing columns' cross section dimensions in order to achieve acceptable building resilience against earthquakes. This also points out that although the national codes do not explicitly describe the behavior at a local level, following these regulations leads to good design.

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