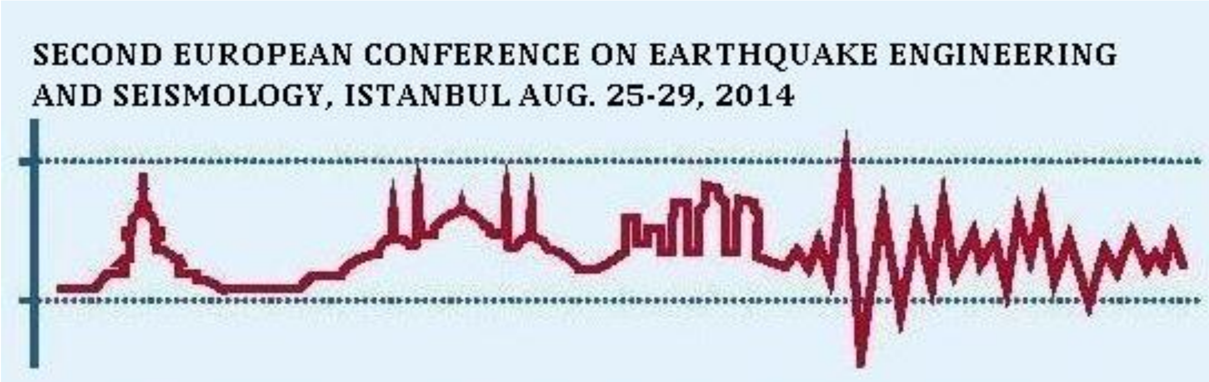

BEHAVIOUR OF A STRUCTURE DEPENDING ON VARIATION OF STIFFNESS OF STRUCTURAL ELEMENTS IN ACCORDANCE WITH THE RECOMMENDATIONS GIVEN IN DIFFERENT VALID REGULATIONS

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ABSTRACT

In the process of analysis and design of reinforced concrete structures in accordance permanent technical regulations it is not anticipated reduction of rigidity in structural members due to the appearance of cracks when the construction is in nonlinear area. To get an insight into the effect of stiffness of the structural elements upon behavior of a structure, a five-storey reinforced concrete structure has been designed in accordance with the recommendations given in different valid regulations.

The considered structure is almost symmetric at plan, whereat the main asymmetry is caused by the staircases and the openings in their plates. The structure is a five storey one in order to avoid the effect of the higher mode shapes on distribution of the static quantities. Modeling and analysis of this structure has been done by use of the Tower 6 software package whereat the dimensions of the structural elements and the loads have been identical in all the cases. The key fact for the differences in the results obtained from the analysis has been the variation of stiffness of the structural elements. In addition to this, there are also other factors among which the most important are the level of life load taken in the modal analysis, the mode of obtaining the maximum seismic force and its distribution, the coefficients for the individual groups of loads when they are combined and other. The stiffness of the elements was changed in accordance with the recommendations in the next regulations:

a) Macedonian regulations - real dimensions, no reduction of stiffness
b) Eurocodes - reduction amounts to 50% for all cross-sections
c) ASCE/SEI41 - stiffness was reduced to 30% for the beams, while for the columns, reduction of the stiffness was made depending on the extent they are used
d) fictional case – stiffnes of the beams was reduced to 25%, and 80% for the columns

The results obtained from each of the four analyses have been considered and compared and they contain changes in vibration periods of a structure, appearance of mode shapes, displacements, values of static effects and their re-distribution, steel savings and others. The results from these comparisans will bi widely explained in the paper.

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INTRODUCTION

The construction is a five-storey reinforced concrete frame with between storey height of 3m and almost symmetric basis. Analysis of loads and loading is based on the architectural project and they are identical for all cases.

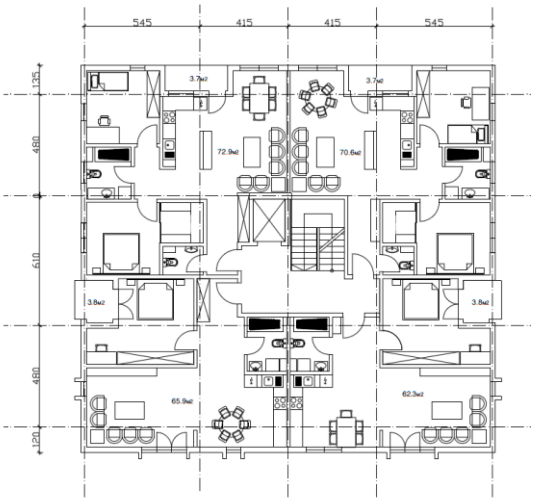

Figure 1. Basis from architectural project

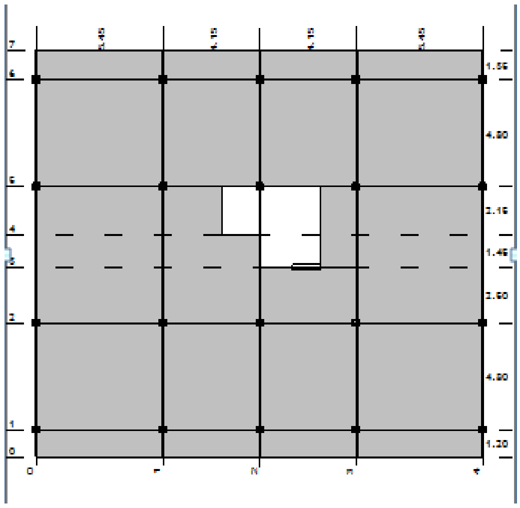

Figure 2. Basis from Tower6

The modeling was done in the software package Tower6. The dimensions of the structural elements in the 3D model who is fixed in the basis are also identical in all considered cases wherein the dimensions of the pillars decreases in height as the dimensions of the beams are the same for all spans. Staircase and openings for them in the slabs are also modeled and are the main reason for unsymmetrical behavior of the construction.

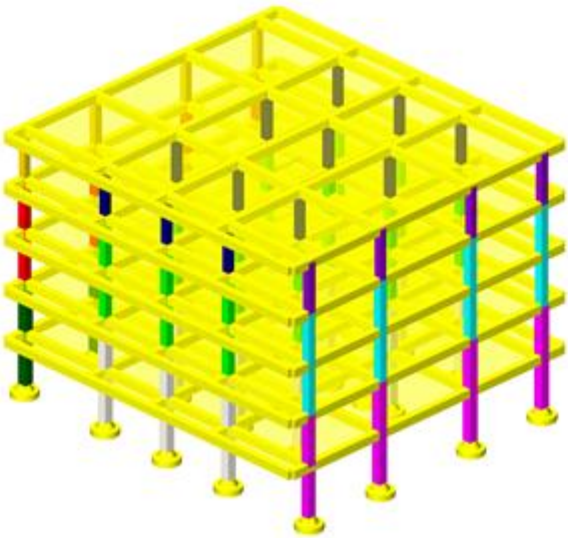

Figure 3. 3D model from Tower6

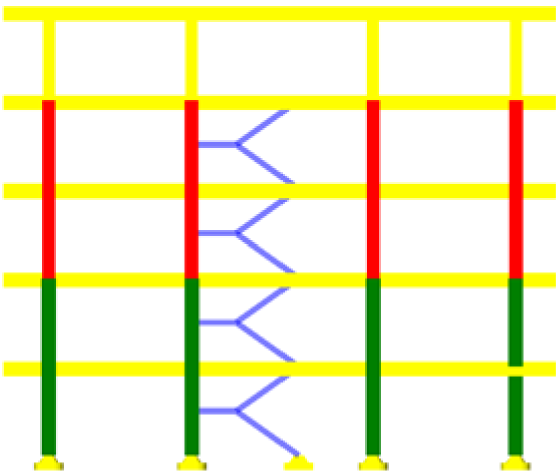

Figure 4. 2D view from the left side

MACEDONIAN REGULATION

The analysis and dimensioning was done according to the recommendations in the Macedonian valid regulation. The first differences is found during the preparation of modal analysis where only 50% of life load is active, as well despite the knowledge of appearance of cracks when construction will enter nonlinear area still there is no recommendation for reduction of the strength of structural members. The results of the analysis are as follows:

No	T [s]	f [Hz]
1	0.6225	1.6065
2	0.6023	1.6603
3	0.5880	1.7006

Figure 5. Periods and frequencies of oscillation

In observation of mode shapes could cleary see the unsymetric behavior of the structure which was expected, especially in the higher mode shapes. This problem was easily overcome by the use of reinforced concrete shear walls in the corners of the structure.

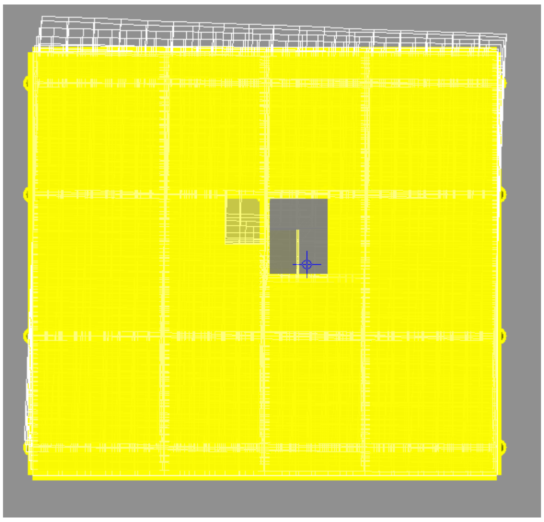

Figure 6. First mode shape from top view

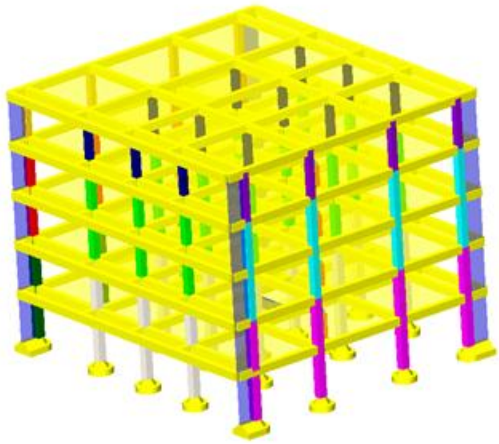

Figure 7. 3D model with shear walls

Although the model with reinforced shear walls behaves better and in every day practice would be better choice, for the purpose of understanding the behaviour of the structure depending on variation of stiffness in stuructural elements for further processing was chosen the frist model.

Seismic loading was performed by the metod of equivalent static load where the total seismic force is the product of coefficient K and the total weight of the structure. Coefficient K is a product of four basic coefficients Ko-depends of the object category, Ks-depends of the seismic intensity, Kp-depends of ductility of the structure and damping ratio, Kd-depends of frist natural period and the category of the ground. The placement of the total seismic force in the both orthogonal directions is the same and its made by the following formula:

$$S_i = S\frac{G_i \cdot H_i}{\sum_{i=1}^{n} G_i \cdot H_i}$$

Level	Z [m]	S [kN]
Rooftop	15.00	707.08
Forth floor	12.00	649.22
Between floor 4	10.50	9.62
Thrid floor	9.00	491.25
Between floor 3	7.50	6.81
Second floor	6.00	329.88
Between floor 2	4.50	4.18
First floor	3.00	166.13
Between floor1	1.50	1.39
Ground floor	0.00	0.05
	Σ=	2365.6

Figure 8. Seismic force placement by floors

Combinations of loads is made by the recommendations on the Macedonian valid regulations which differs a lot than the ones in EC because off the nonexistenting coefficient of safety for materials in Macedonia regulations.

In this paper will be shown only the distribution of bending moments for one frame and it will be made a comparison of the ratio of peaks in beams compared with the ones in columns.

Table 1. Bending moments distribution amd beam/columns ratio

Floors	Beams (KNm)	Columns (KNm)	Ratio
I	284.1	258.2	1.1/1
II	293.5	224.1	1.3/1
III	294	177.2	1.65/1
IV	194.4	152.3	1.27/1
V	110.3	108.3	1/1

From this chart clearly can be seen that the beams receive far more bigger moments than columns especially in the middle floors of the structure. Such behavior of the structure is quite inconvenient and causes excessive dimensioning of beams and occurrence of weak columns that will ultimately lead to appearance of plastic hinge in columns and collapse of the structure.

Control of displacement is performed only for top displacement of 2.25cm which is less than allowed by the regulations in this case h/600=2.5cm

The result from the dimensiong and will be compared in the next chapters.

EUROCODE

In the Eurocode regulations for the modal analysis there is a recommendation that only 15% of the live load should be active which represents reduction of total weight of the structure of 131.5t (5.4%). Another recommendations is that if there is no other research on the level of strength of structural elements in nonlinear area will be then they should be model with 50% of there real strength. Natural periods are now bigger because of the lower stiffnes of the elements.

No	T [s]	f [Hz]
1	0.8593	1.1637
2	0.8328	1.2008
3	0.8130	1.2300

Figure 9. Periods and frequencies of oscillation EC

The seismic force is now calculated by EC-standards so the parameters are selected so they can be closest to those from Macedonian regulations with the following placement of seismic forces.

Level	Z [m]	All mode shapes		
		Px [kN]	Py [kN]	Pz [kN]
Rooftop	15.00	531.74	146.26	0.24
Forth Floor	12.00	738.10	210.78	-0.17
Between floor 4	10.50	11.02	2.94	-0.28
Third floor	9.00	650.79	182.58	1.37
Between floor 3	7.50	13.50	4.28	0.78
Second floor	6.00	1024.5	301.41	-1.03
Between floor 2	4.50	15.44	4.31	-0.14
First floor	3.00	788.55	230.96	-1.84
Between floor 1	1.50	5.05	0.75	-1.63
Ground floor	0.00	0.08	0.03	-0.02
	Σ=	3778.8	1084.3	-2.72

Figure 10. Seismic force Sx+0.3Sy

Level	Z [m]	All mode shapes		
		Px [kN]	Py [kN]	Pz [kN]
Rooftop	15.00	157.04	499.41	0.32
Forth Floor	12.00	217.73	708.45	0.45
Between floor 4	10.50	3.29	10.44	-0.52
Third floor	9.00	194.58	622.48	-0.31
Between floor 3	7.50	4.00	13.15	1.46
Second floor	6.00	304.96	1002.2	-0.71
Between floor 2	4.50	4.51	15.15	-0.69
First floor	3.00	233.61	778.79	1.33
Between floor 1	1.50	1.46	5.30	-3.37
Ground floor	0.00	0.03	0.33	-0.43
	Σ=	1121.2	3655.7	-2.46

Figure 11. Seismic force Sy+0.3Sx

The seismic force obtained only from the first mode shape is bigger than the total according to Macedonian regulations, and now the maximum force is located on the second floor.

The distribution of bending moments in this case is much better and more realistic compared to the one from Macedonian regulations.

Table 2. Bending moments distribution EC compared to MK

Floors	Beams (KNm)	Columns (KNm)	Ratio EC	Ratio MK
I	250.8	246.6	1/1	1.1/1
II	252.8	208.1	1.2/1	1.3/1
III	214	172.4	1.25/1	1.65/1
IV	167.2	137.77	1.21/1	1.27/1
V	94.52	98.33	1/1	1/1

Top displacements and interstory drifts are now bigger but still in the allowed range which means this system can be used in practice.

Dimensioning is done with concrete C25 brand that responds to MB30 from Macedonian regulations and also is used same type of reinforcement.

If we compare the reinforcement used in one frame and assume that this relationship will continue in all of the frames from the construction will get the following results.

Table 3. Comparison of needed reinforcement MK and EC regulations

	One frame (Kg)	Whole sturcture (Kg)
MK regulations	3824.95	34586.55
Eurocode	3470.84	31237.56
Difference	372.11	3348.99

By using the Eurocodes approximately 3350kg or 3.35t of reinforcement was saved only for the frames of whole structure, which means for less money we got more realistic results and better behaviour.

ASCE/SEI 41

U.S. regulation suggests rigidity of the beams to be reduced to 30% while the columns that reduce dependes on the level of usage. For columns where strain capacity is used by 10% or less stiffnes should be lowerd to 30%, while usage of 50% or more rigidity to reaches 70%. For all values between linear dependence applys.

Difference of 0.1 sec appeared in natural periods of oscillation, but this doesn't show large differences in the total seismic force and its placement..

No	T [s]	f [Hz]
1	0.9663	1.0349
2	0.9428	1.0606
3	0.9149	1.0930

Figure 12. Periods and frequencies of oscillation

Level	Z [m]	All mode shapes		
		Px [kN]	Py [kN]	Pz [kN]
Rooftop	15.00	439.73	119.63	0.24
Forth Floor	12.00	644.92	183.18	-0.14
Between floor 4	10.50	9.74	2.59	-0.26
Third floor	9.00	594.21	166.23	1.78
Between floor 3	7.50	13.13	4.25	0.93
Second floor	6.00	1028.6	303.64	-1.55
Between floor 2	4.50	14.98	4.08	-0.26
First floor	3.00	713.50	209.56	-1.86
Between floor 1	1.50	4.44	0.71	-1.60
Ground floor	0.00	0.07	0.02	-0.02
	Σ=	3463.3	993.90	-2.75

Figure 13. Seismic force Sx+0.3Sy

Level	Z [m]	All mode shapes		
		Px [kN]	Py [kN]	Pz [kN]
Rooftop	15.00	129.33	411.21	0.19
Forth Floor	12.00	189.03	620.96	0.29
Between floor 4	10.50	2.92	9.21	-0.45
Third floor	9.00	177.72	566.56	-0.65
Between floor 3	7.50	3.90	12.85	1.70
Second floor	6.00	306.15	1008.3	-1.19
Between floor 2	4.50	4.39	14.65	-1.06
First floor	3.00	210.46	710.19	1.47
Between floor 1	1.50	1.27	4.78	-3.11
Ground floor	0.00	0.03	0.28	-0.37
	Σ=	1025.2	3359.0	-3.18

Figure 14. Seismic force Sy+0.3Sx

Beam/column ratio of the peak values of beanding moments are even better now, but problem appears with increase of top displacement which exceeds the allowed. This would increase the dimensions of the structural elements that would significantly change his analysis.

Table 4. Comparison of needed reinforcement MK and ASCE/SEI 41 regulations

	One frame (Kg)	Whole sturcture (Kg)
MK regulations	3824.95	34586.55
ASCE/SEI 41	3286.99	29582.1
Difference	555.96	5004.45

Savings are bigger now but that's not relevant because of the need for changing of dimensions of the structural elements.

FICTIONAL CASE

This case represents event when all plastic hinges will appear in the beams while columns have very small cracks. Inspiration for this analysis is the experiment shown when adoption of ASCE/SEI 41 was made. In this experiment examination was made on 221 columns and the results were shown on a graphic. Approximation that was made in this graph is pretty conservative so I decided to do a trial which would include realistic extreme values that may occur in a construction.

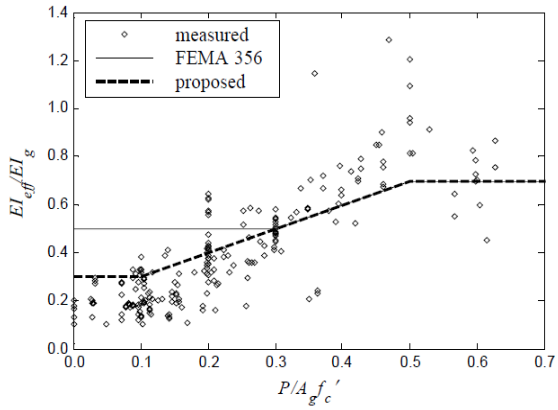

Figure 15. Graphic from the experiment FEMA

The extremes selected are 25% of the total stiffnes for beams and 80% for columns. The results of modal and static analysis are almost identical with the ones from ASCE/SEI 41. The biggest difference appears in the interstory drift that is far over the limit. That would mean that anyother comparison its irrelevant.

Because of the size of this Paper a number of other comparisons were not mentioned that I hope will have space to display in the following

CONCLUSION

The one sure conclusion that can be drawn on the safe side is that the occurrence of cracks in the concrete and thereby the reduction of stiffness of the cross-section is a realistic problem that should always be taken into account in design of a structure. The remaining comparisons cannot be referred to as conclusions since analysis of only one type of structure with omission of some factors and performance of certain approximations was at stake, but still by proportioning this structure according to the Macedonian regulations, greater static effects and hence greater reinforcement were obtained, but this did not mean a safer structure but, on the contrary, not achieving the required mechanism of behavior of the structure in the nonlinear range. In the next cases, when reduction of stiffness was taken into account, more realistic insight into the behavior of the structure was obtained so that we were able to perform its dimensioning better and with a smaller amount of reinforcement. Perhaps the best method is ASCE/SEI41 where each element is subjected to a certain reduction depending on the extent of its use but the application of this method leads in this to large displacements whose reduction should be solved in the most rational way.

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