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Structural optimization of a reinforced concrete building

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ABSTRACT : *In this study, a residential building has been designed multiple times, firstly with the initial configuration and later optimized based on different theories. Each of the optimized designs was created based on a specific idea of structural optimization. Eurocodes have been used as the design code for the models. In order to compare the designs, CSI ETABS commercial software has been used. In the design of the models, some initial conditions such as the soil type and the seismicity of the area were specified the same for the models in order for the optimization to be intended for the conditions of the real building. The main objective was to compare different design strategies and optimize to achieve the most suitable case.*

Keywords *–reinforced concrete design, structural optimization, FEM analysis, ETABS*

I. INTRODUCTION

Most of the residential structures built in Albania after 1990s are reinforced concrete structures. The planned service life for this type of building is around 50 years. The structural design of reinforced concrete buildings generally consists on finding the proper arrangement between geometrical shapes of the frame elements and materials (concrete and steel in this case). As the structure will be subjected to wide variety of loads and act predictably under different situations, the basic considerations for a structural design are: the load combinations, the soil type, the seismicity of the area and the properties of the materials. Design codes have been established to help engineers to create structures which comply with all the requirements.

In order to improve the design, optimization techniques are used. The main purpose of optimization techniques is to obtain the maximum benefit out of the resources available. To achieve an optimized design it is necessary to minimize the usage of materials and maximize the performance of the structure in various predicted situations. Most of the procedures used to achieve an optimized design consist on trying different cross sections, materials and arrangements in order to achieve the limit state parameters predicted by design codes. In cases when multiple design arrangements satisfy the code parameters, the design with the highest performance is considered as a better design compared to the others.

The performance of the building is clearly defined on various design codes as the behavior of a structure under different loading conditions. For the analysis of the models Eurocode 2 [1] was used to specify the different actions and conditions on the building.

Since the structural design of residential buildings takes a lot of time and requires a high accuracy in calculations, it is a common practice to use structural analysis software. In this study, ETABS2015 [2] has been used to analyze the models and to display various output parameters. The details of the reinforcement are an integral part of the design of any reinforced concrete structure, therefore the reinforcement details for the typical beams columns and slabs of the optimized structure generated have been designed.

II. MATERIALS AND METHODS

2.1 Structural Optimization

The idea of creating an improved design over the original one date back to 1600s when Leonardo da Vinci created multiple small scale structures and compared their performance. Structural optimization as a term and its importance was noted by Cohn and Dinovitzer in 1994, who showed the gap between structural theories and practical application on structural design and the importance of a good initial design [3].Sarma and Adeli, in 1998, pointed out that the cost of reinforced concrete structures depended on the cost of concrete, reinforcement and formwork and the combination of different materials and geometry arrangements would produce an optimized structure [4]. However, this study focused only on the economic analysis of the structure and not the optimization of its performance. Later with the development of computers and programming the field of structural optimization was separated into two main branches, the traditional approach and the heuristic approach. The traditional approach means an implementation of trial and error techniques and comparison between the initial and the optimized structure. Fletcher, 2001[5] and Hernandez and Fontan 2002 [6] were the first to start development into this approach. The heuristic approach followed the computerized path with the development of multiple algorithms such as genetic algorithms, bee colonies, threshold accepting, stimulated annealing etc.[7-10].

As it is pointed out by Tang, 2011 performance based structural optimization is divided into 3 categories: size optimization, shape optimization and topology optimization [11]. Size optimization consists on the size of the member being the only type of variable in the structure. Shape optimization consists on the variety of shapes to be used for different structural elements. Topology optimization consists on finding the optimal layout of the structure. The initial ideas for finding the optimal layout came from Maxwell, 1890 [12] and Michell, 1904, [13] who developed the layout optimization theory for thin bar structures such as trusses. Later, the optimal layout theory was developed by Prager and Rozvany, 1977, [14]. This theory was based on Mitchell's optimization for trusses but instead it focused on the optimization of grid-like structures. The bubble-method proposed by Eschenauer et al., (1994) [15] introduced the removal of non-functional material by placing holes or bubbles in order to create a new type of structure. Xie and Steven, 1993, [16] proposed the evolutionary structural optimization (ESO) method which implied the gradual removal of the elements until an optimized design had been achieved. Nibbling ESO was a part of this theory which allowed only the removal of external boundary elements similar to a worm which nibbles the leaf. Later, the bi-directional evolutionary structural optimization (BESO) by Querin, 1998 [17] which modified the original ESO by allowing also element addition where was necessary. Over the last years several approaches have been made in the improvement of ESO and BESO methods of structural optimization.

2.2 Case study

The case study building is an-eight-story residential building located in Fier, Albania and was constructed in 2009.The first floor is intended for commercial purposes while the other stories are intended for residential purposes. The floor area is 382.4 m^2 for each of the floors. The building is characterized by the irregular shapes of the balconies (Fig.1).

Figure 1: Case study building.

The slab type used on the structure is ribbed slab with a thickness of 300 mm. The thickness of the concrete layer is 50 mm and the remaining 250 mm is composed of polystyrene. The stem width at top is 500 mm, the stem width at bottom is 100mm and the rib spacing is 400 mm. Two types of beams were used for the structure each with different sections. The perimeter beam section 250 x 650 mm and the internal beams 250x 300 mm. The concrete grade used for the slabs and the beams is C20/25 concrete. The columns are of 7 different types of sections out of which 4 are columns with rectangular sections (900 x 250; 550 x 250, 650 x 250 and 800 x 250 mm). There are two L-shaped column sections (800 x 800 x 250; 900 x 900 x 250 mm). The last type of column section is a T-section 800 x 800 x 250 mm. The concrete grade used for the columns is C25/30 concrete. The partition brick walls used for the building were 120 mm for the internal partitions, 200 mm for the balconies and 250 mm for the external walls. For the lintels on the doors and windows, C20/25 secondary beams with a section of 125 mm x 250 mm were used.

In the consideration of earthquake loads, the response spectrum has to be defined. This required various input data such as the ground acceleration, ground type and the behavior factor. This data is different for each area, and specific studies for Albania were required. The seismic hazard assessment studies by Aliaj et al. (2004) [18], Duni et al., 2010, [19] and Galasso et al., 2013, [20] were considered on the earthquake considerations during the analysis of the models in this study.

2.3 Modeling

In this study, the structure was modelled using ETABS 2015 commercial software. Since ETABS is based on Finite Element Method FEM all the shell elements have to be divided into smaller elements. This is called the meshing process and it can be done manually or automatically in ETABS. Figure 24 shows the automatic mesh settings for floors with 1m approximate mesh size.

2.3.1 Loading considerations

Dead and live loads were assigned to the shell elements of the structure according to Eurocode 1 EN1991-1-1:2002 [21]. Since this is a domestic/residential building it is included under category A of Eurocode 1. The following values to be assigned were obtained from Table 6.2 on Eurocode 1.

- Dead Loads: Gk = 2.0 kN/m^2 (includes the floor tiles, finishes and screed concrete)
- Live Loads:
  - Floors: Qk = 2.0 kN/m^2
  - Stairs: Qk = 4.0 kN/m^2
  - Balconies: Qk = 3.0 kN/m^2

Earthquake loads were also taken into consideration in the form of response spectrum with a PGA = 0.3 g and a behavior factor 4.0 inferred from the soil type “D” “deposits of loose-to-medium cohesionless soil” based on EN 1998 [22].

In order to obtain the moment envelope for the load cases different combinations of these cases are specified based on Eurocode 1 [21] were defined:

- Combination 1: 1.25 Qk + 1.5 Gk
- Combination 2: 1.25 Qk + 1.05 Gk
- Combination 3: 1.35 Qk + 1.5 Gk
- Combination 4: 1.35 Qk + 1.05 Gk
- Combination 5: 1.35 Qk + 1.5 Gk + 0.7 EQx + 1.0 EQy
- Combination 6: 1.35 Qk + 1.5 Gk + 1.0 EQx + 0.7 EQy

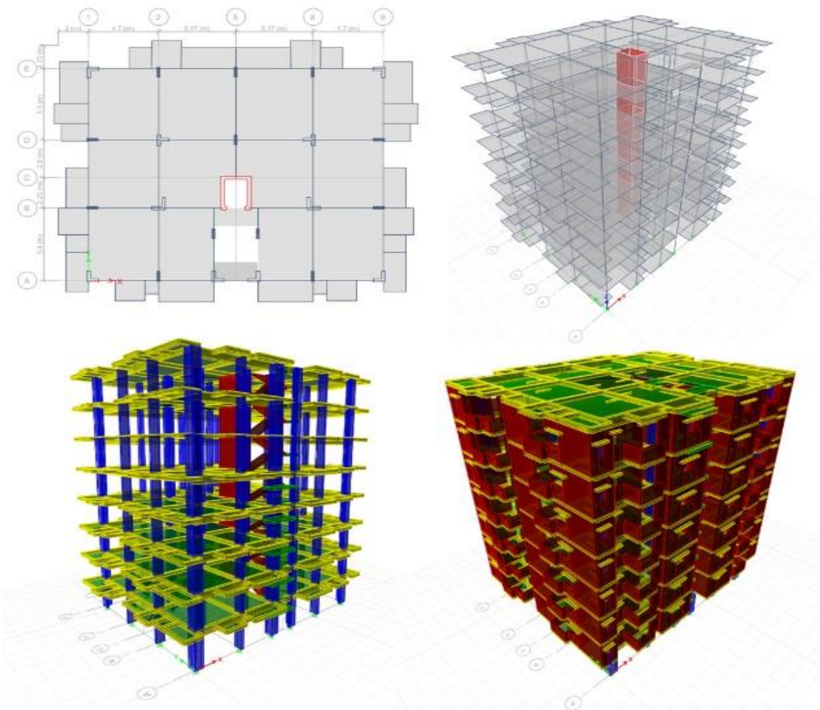

Figure 2: Stages of modelling of the structure in ETABS.

2.4 Analysis Approach

Analysis approach consists of comparing four different models of the same layout, subjected to the same type of loading conditions but having different cross-sectional characteristics, and material strengths. The comparison was made by comparing the maximum values of shell stresses and modal displacement values. The shell stresses are generated by the FEM algorithms incorporated into ETABS and are displayed as scalar field representation with varying colors representing the variation of stresses on shell elements.

The modal analysis uses superposition to linearly add the sinusoidal oscillations in order to obtain the deformations which happen on the building during the earthquake. The technique considers both spatial and time changes evaluating both displacements and periods/frequencies. The period of the building is the time required for a full cycle of displacement until the building returns to its original position.

2.4.1 Model 1
The first model was the same as the original structure with the same materials specified on section 2.2.

2.4.2 Model 2
In the second model the concrete grade used for the elements of the frame was changed. For each element, the concrete grade was increased by one scale based on EC2. For the columns and the staircase C30/37 concrete was assigned. For the beams and the slabs C25/30 was assigned. The second model was modeled to inspect the changes in behavior of the structure with the change of the material type.

2.4.3 Model 3
In the third model the section geometry of the columns was changed. All the columns of the building were substituted with circular columns with a diameter of 400 mm and 500 mm (only for the composite column). The vertical supporting area of the columns per floor was significantly increased from 58750 cm^2 to 126228 cm^2. The third model was created to show the optimization of the structure using a different type of section geometry.

2.4.4 Model 4
For the fourth model, the slab type was changed from ribbed slab into monolithic slab with the same slab thickness 300 mm. This was done in order to increase the weight of the structure and provide it with a lower center of gravity.

III. RESULTS AND DISCUSSION

For the shell stresses, the comparisons were made for the 4 models on 3 different load combinations. The load combinations taken into consideration were:

- Combination 3: 1.35 Qk + 1.5 Gk
- Combination 5: 1.35 Qk + 1.5 Gk + 0.7 EQx + 1.0 EQy
- Combination 6: 1.35 Qk + 1.5 Gk + 1.0 EQx + 0.7 EQy

The following shell stresses were extracted from ETABS in order to be used as variables for comparison of the four models: S11, S12, S13, S22, S23, Smax, SmaxV, Smin and SVM. The scalar field representations for the abovementioned shell stresses are shown in the following figures in the abovementioned order. Figs. 3-6 show stress distribution examples using various load combinations. The values of the stress are presented in detail in the tables below.

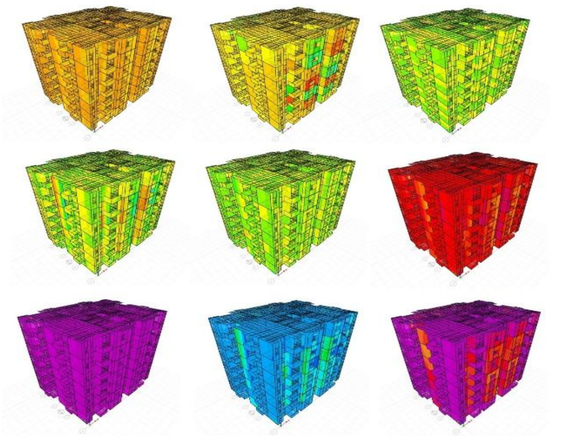

Figure 3: Shell stresses: model 1; load combination 3

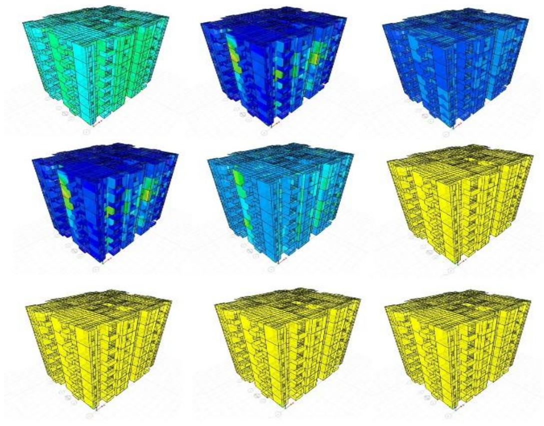

Figure 4: Shell stresses: model 1; load combination 5

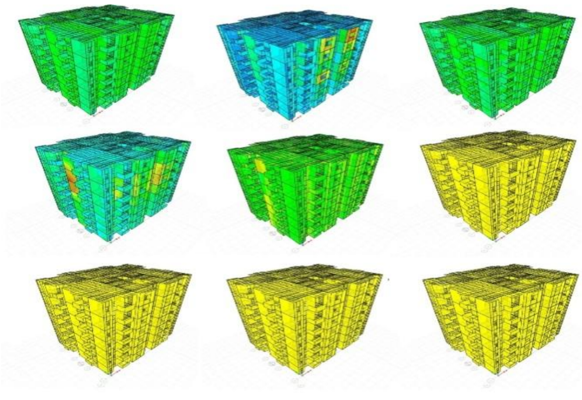

Figure 5: Shell stresses: model 2; load combination 5

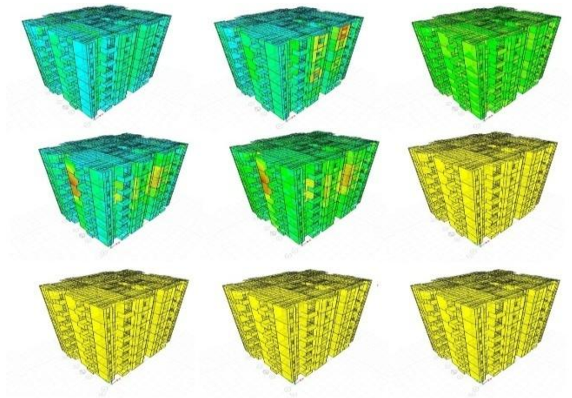

Figure 6: Shell stresses: model 2; load combination 6

The comparison between shell stresses of different models for the same load combination 1.35Qk + 1.5Gk is shown in Table 1. As it can be seen from the table, Model 2 is superior among them, as all the values are lower.

Table 1: Comparison of shell stresses on load combination 1.35 Qk + 1.5 Gk

Load Case: $1.35Q_k + 1.5G_k$					
		Model 1	**Model 2**	**Model 3**	**Model 4**
S11 (MPa)	Max	17.68	12.422	10.368	6.541
	Min	-12.351	-11.751	-12.491	-7.066
S12 (MPa)	Max	4.599	1.719	2.194	4.738
	Min	-4.609	-2.212	-1.772	-3.112
S13 (MPa)	Max	5.257	5.903	6.159	3.344
	Min	-5.603	-4.922	-4.11	-3.855
S22 (MPa)	Max	8.826	4.093	4.957	7.151
	Min	-9.428	-8.016	-8.694	-7.122
S23 (MPa)	Max	0.213	0.176	0.213	1.883
	Min	-0.311	-0.264	-0.253	-3.011
Smax (MPa)	Max	11.713	12.422	12.491	7.186
	Min	-1.79	-0.71	-0.881	-0.731
SMaxV (MPa)	Max	5.607	5.904	6.16	3.901
	Min	0	0	0	0
Smin (MPa)	Max	1.686	0.652	0.52	0.56
	Min	-17.713	-12.422	-12.491	-7.186
SVM (MPa)	Max	17.527	12.324	12.218	8.236
	Min	-4.2E-05	-5.1E-05	-2.29E-05	-8.4E-05

In Table 2, it is presented the comparison of the main stress values under combination 5. As it can be seen from the table, Model 3 is superior, followed by Model 2.

Table 2: Comparison of shell stresses on load combination 1.35 Qk + 1.5 Gk+ 0.7 EQx + 1.0 EQy

Load Case: $1.35Q_k + 1.5\ G_k + 0.7\ EQ_x + 1.0\ EQ_y$					
		Model 1	**Model 2**	**Model 3**	**Model 4**
S11 (MPa)	Max	23.51	9.653	10.526	5.809
	Min	-108.769	-34.027	-16.786	-8.803
S12 (MPa)	Max	2.509	1.297	1.145	1.868
	Min	-35.011	-6.89	-0.216	-9.663
S13 (MPa)	Max	3.261	4.266	3.571	2.752
	Min	-34.645	-10.347	-5.528	-7.911
S22 (MPa)	Max	5.035	5.422	4.221	9.871
	Min	-83.947	-18.949	-9.534	-14.822
S23 (MPa)	Max	0.384	0.278	0.162	2.204
	Min	-1.542	-0.42	-0.332	-4.005

For the modal analysis, five modes were taken into consideration. The maximum and minimum displacements and the periods were generated by ETABS for each mode. The modal analysis comparison between the models is displayed on Table 3.

Table 3: Comparison of modal displacements between the models.

		Model 1	**Model 2**	**Model 3**	**Model 4**
Mode 1	U_{max}(mm)	0.002089	0.00328	0.01932	0.002407
	U_{min}(mm)	-0.01832	-0.01892	-0.00228	-0.01963
	T(s)	1.82	0.344	0.069	0.514
Mode 2	U_{max}(mm)	0.02353	0.03536	0.03095	0.0239
	U_{min}(mm)	-0.031	-0.02301	-0.02421	-0.03127
	T(s)	0.105	0.093	0.108	0.08
Mode 3	U_{max}(mm)	0.7	0.1	0.8	0.8
	U_{min}(mm)	-0.8	-0.04236	-0.7	-0.7
	T(s)	0.024	0.037	0.024	0.024
Mode 4	U_{max}(mm)	0.01951	0.8	0.03048	0.0298
	U_{min}(mm)	-0.0302	-0.7	-0.01961	-0.0189
	T(s)	0.014	0.024	0.014	0.014
Mode 5	U_{max}(mm)	0.03193	0.01923	0.1	0.1
	U_{min}(mm)	-0.03328	-0.03003	-0.1	-0.1
	T(s)	0.012	0.014	0.007	0.009

IV. CONCLUSION

As it can be inferred from the comparisons, none of the models is superior to the others in every direction. While intending to optimize the structure for a specific behavior there might be “side effects” meaning that other properties might change to unexpected values.

The results obtained from the analysis of Model 2 show that an increase in the concrete grade optimizes the structure when subjected to dead and live loads in all the stress values. However after observing shell stresses and modal displacements having considered the earthquake loads, some of the values show no significant optimizations and some of them even show deterioration.

From the third model it is shown that while circular columns and the increase of column area per floor show no visible optimization on dead and live loading conditions, they decrease the shell stresses significantly when the building is subjected to earthquake loads therefore optimizing the structure. However since the beams which transmitted the loads to the columns had a smaller section compared to the column, a check on the connection between the beams and the column is required.

From the fourth model it is concluded that strength is not the only important parameter in structural design but weight is also needed to be taken into consideration. A lower weight especially on the upper floors contributes significantly in lowering the center of gravity for the entire structure as well as decreasing the stresses on many elements of the structure.

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