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Case Study: Designing a 40 Storey High Office Building Using Two Variants, With Regular Concrete Columns and With Compound Ultra-High Performance Concrete Columns and Regular Concrete Columns

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Abstract

The problem of having less and less space and of shortening distances results in urban agglomerations on small surfaces of land. Therefore, it is obvious that the future is that of very high, slender buildings, with unusual and daring shapes. These structures incorporate new and ever more efficient materials, faster lifts, more and more complex design methods and special execution technologies. Globally, there a real competition between developed economies for obtaining the record of having the tallest building on earth. Because columns are an essential component of these high structures, the addressed subject is of global interest. The design of a 40 storey office building is being proposed, using two variants, namely having two different types of columns: simple section – regular columns made of regular concrete (RC) class C 35/45 and compound section – columns with the core made of ultra-high performance concrete (UHPC) and an outer shell made of regular concrete (RC) also class C 35/45. The differences in economy and section between the two proposed variants will be analysed.

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Keywords: ultra-high performance concrete columns; regular concrete columns; compound columns, structural design, loads evaluation.

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1. Introduction

When dealing with tall structures, the quality control and the technology used to execute them is extremely difficult to manage and control. Let's imagine the following scenario: what happens if, 28 days after the concrete casting, one of the columns from the 2nd floor does not correspond to the resistance class designed for it and the execution of the structure has already reached the 6th floor. What can be done in this case? There are several possible solutions to this problem, but whatever the chosen solution is, it entails huge extra costs. As an alternative to avoiding such situations the following solution is being presented: columns with compound section. In the case of this solution, a part of the column is prefabricated (the core) in a concrete plant having a rigorous quality control, and the other part (the shell) is executed traditionally on the building site [1]. In this way the quality control is greatly enhanced and the chances of having an unforeseen situation like the one described above are considerably reduced. The design of a 40 storey office building whose columns will be computed in two different ways is being presented next. The building was divided into 4 zones per section height, see Fig. 1b and 1c.

2. The design of the 40 storey high building

2.1. Design data

Location of the building: Cluj – Napoca County. Wind zone: $v_{b,0}$ = 27 m/s, Urban zone – soil category no. IV, according to [2]. Seismic zone: a_g = 0.10g, T_c = 0.70s, T_B = 0.14s, T_D = 3.00s, β_0 = 2.50, according to [3] and [4]. The characteristics of the building are the following: office building with a tube structural system composed of a central core made of 60 cm thick reinforced concrete walls and concrete frames with perimeter columns. Number of floors: Ground floor (GF) + 40 Floors (FL). Current height of a floor is: H_{floor} = 3.00 m. For current level layout and section features: see Fig. 1.

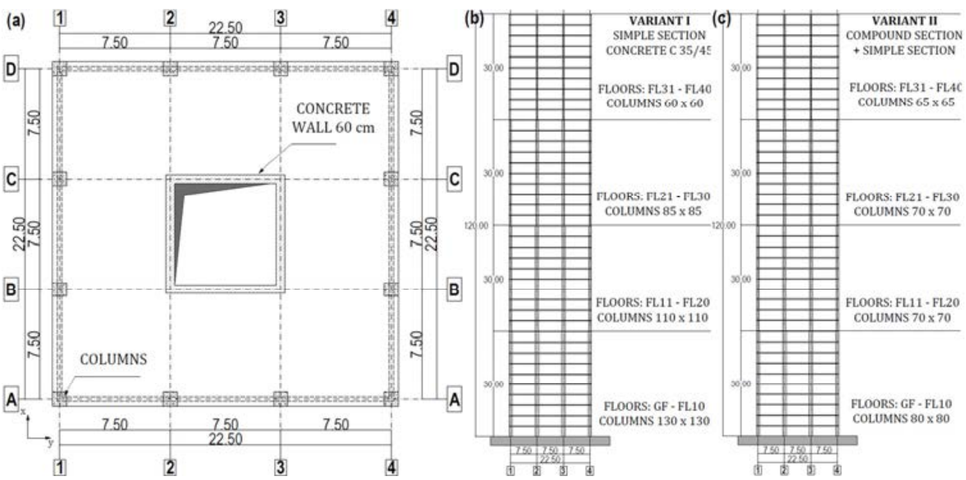

Fig. 1. (a) Level layout; (b) Section in variant 1, (c) Section in variant 2

Variant I: columns with regular concrete (RC) sections to all floors, see Fig. 1b.

Variant II: columns with compound section until the 20th floor and from the 21st floor all the columns were made of regular concrete (RC), see Fig. 1c.

The transverse simple section of the RC columns had a reinforcement casing made of steel BST 500 and C 35/45 class concrete, see Fig 2a. The transverse compound section had a core made of UHPC, concrete class C 130, a reinforcement casing made of steel BST 500 and an outer shell made of concrete class C 35/45 see Fig. 2b. The UHPC core is prefabricated in a concrete plant and the C 35/45 shell is executed on the building site, thereby the quality control is improved.

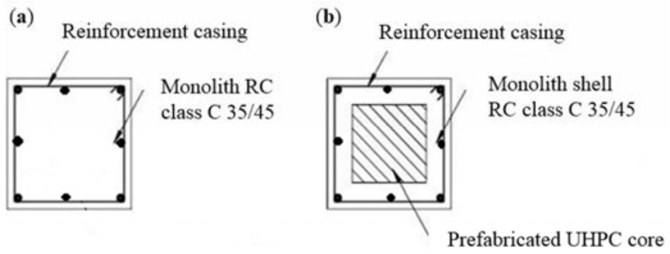

Fig. 2. (a) Simple section (no UHPC core); (b) Compound section (with UHPC core)

2.2. Preliminary data

The method of design proposed by the SR EN's uses partial safety factors. The following values were considered:

a. Design values of material strengths:

- for concrete: $f_{cd} = \alpha_{cc} f_{ck}/\gamma_c$ where according to [5] the recommended value for α_{cc} is 1.00 and for γ_c = 1.5;
- for reinforcement: $f_{yd} = f_{yk}/\gamma_s$ where $\gamma_s = 1.15$ according to [5].

b. Design values for actions will be calculated according to the general expression suggested by [6]:

$$F_d = \gamma_f \cdot \psi \cdot F_k, \quad (1)$$

where F_k is the characteristic value of the action. For the B category (office areas) according to National Annex from [6], service load safety factors are: $\psi_0 = 0.7$; $\psi_1 = 0.5$; $\psi_2 = 0.3$; for snow loads: $\psi_0 = 0.7$; $\psi_1 = 0.5$; $\psi_2 = 0.4$; for wind loads: $\psi_0 = 0.7$; $\psi_1 = 0.2$; $\psi_2 = 0.0$.

For partial safety coefficient of permanent load we have two design situations in which $\gamma_G = 1.35$ for permanent and transitory design situations and $\gamma_G = 1.00$ for accidental or seismic design situations.

2.3. Load evaluation

Permanent gravity and quasi-permanent loads: with characteristic values, where detailed in Table 1.

Table 1. Loads for different elements

Elements	Measurement unit	Load
External wall 3.0 m in height	[kN/m]	7.02
Current slab	[kN/m^2]	6.60
Terrace slab	[kN/m^2]	8.02
Wall 1.5 m in height	[kN/m]	11.5

Service loads: according to [7], characteristic service load for office buildings is $q_k = 3.0\ kN/m^2$. The [7] code allows the consideration of the partition walls as a supplement (between 0.5 ÷ 1.2 kN/m^2) for the characteristic service load distributed per unit area; consequently, for a current slab we have the following expression:

$$q_{k.total} = 3.0\frac{kN}{m^2} + 0.5\frac{kN}{m^2} = 3.5\frac{kN}{m^2} \tag{2}$$

The roof terrace was classified in category H ("Roofs inaccessible, except normal maintenance and repairs"); the characteristic value of the service load for the roof terrace was considered $q_{terrace}$=0.75 kN/m^2 thus respecting the recommended range (q_k can be selected between 0.00 and 1.00 kN/m^2).

Load from snow calculated according to [7], the expression to determine the load from snow is:

$$s = \mu_i \cdot C_e \cdot C_t \cdot s_k = 0.8 \cdot 1.00 \cdot 1.00 \cdot 1.50 = 1.20\frac{kN}{m^2} \tag{3}$$

Load from wind: calculated according to [2] the relation to determine the wind's reference speed is given by the following expression:

$$v_b = c_{dir} \cdot c_{season} \cdot v_{b,0} = 1.00 \cdot 1.00 \cdot 27\frac{m}{s} = 27\frac{m}{s} \tag{4}$$

Average wind speed varies with height and is given by expression:

$$v_m(z) = c_r(z) \cdot c_0(z) \cdot v_b = k_r \cdot ln\frac{z}{z_0} \cdot c_0(z) \cdot v_b = 0.19 \cdot \left(\frac{z_0}{z_{0,II}}\right)^{0.07} \cdot c_0(z) \cdot v_b \tag{5}$$

The recommended value for standard orographic factor ($c_0(z)$) is 1.0. Roughness length z_0 was assigned the value corresponding to the category no. IV, namely z_0 = 1.0 m.

The value obtained from the calculation for v_m at the maximum building height (123.00 m) was: $v_m(10\,m) = 30.446\,m/s$.

Wind turbulence intensity at the height z, defined as the ratio between the standard deviation of the turbulence and the mean wind speed given by equation (6) explained:

$$I_v(z) = \frac{\sigma_v}{v_m(z)} = \frac{k_I}{c_0(z) \cdot ln(z/z_0)} \tag{6}$$

For z = 123.00 m it has the value $I_v(123\,m) = 0.208$.

The peak pressure of the relative wind speed given by equation (7) and it had at z = 123.00 m the value:

$$q_p(z) = [1 + 7 \cdot I_v(z)] \cdot \frac{1}{2} \cdot \rho \cdot v_m^2(z) = 1.422\frac{kN}{m^2} \tag{7}$$

The outside pressure of the wind is given by expression (8), where c_{pe} is the exposure coefficient:

$$w_e(z) = q_p(z_e) \cdot c_{pe} \tag{8}$$

With regard to the modelling of wind action, the building was divided into three zones: a lower zone of height b = 22.5 m, an upper zone at the top of the building down to a height b = 22.5 m and the intermediate region divided into strips equal to the height of one floor. Thus, we have two sets of relevant values, related to the lower part or to the upper part), so deducting intermediate values being extremely easy. The results are summarized in Table 2.

Table 2. Results systematization

Situation	Areas [kPa]			
	A	B	C	D
Inferior zone	-1.707	-1.138	1.138	-0.995

Superior zone	-1.607	-1.071	1.071	-0.911

Load from seismic action: calculated according to [3], which is an extension of [4]. The seismic load, in compliance with the provisions of [6], was considered with less stringent requirements, its action was shaped as for a normal building, analysing the comparative savings made by using the core made of UHPC. Pre-dimensioning the structural elements of the building

Slab pre-dimensioning: studied building slabs are square, with a side of 7.50 m. Its thickness must comply with the condition $h_{slab,required} = l / 40 = 7500 / 40 = 188$ mm. Since the minimum conditions of fire resistance, structural strength and horizontal wall, the effective thickness considered was: $h_{slab} = 200$ mm.

Beam pre-dimensioning: based on spans, the code [5] defines the following limits for simply supported beam height: $h_{beam,min} = l / 15 \leq h_{effective} \geq h_{beam.max} = l / 12$. Since the minimum conditions of fire resistance, construction, technological and technical are satisfied, the following values will be considered: $h_{beam} = 550$ mm, $b_{beam} = 300$ mm.

Columns pre-dimensioning: in both cases (simple section and compound section) requires preliminary analysis of efforts and hence preliminary calculation of the size of sections in Ultimate Limit State (ULS). For reasons of simplicity of calculation and technology, the sections of the columns was kept for each series of 10 floors. Gravity loads (pointy) were summarized in Table 3.

Table 3. Loads in ULS

Floor	Simple section	Compound section
GF	30724 [kN]	21700 [kN]
FL11	21140 [kN]	14900 [kN]
FL21	13115 [kN]	9200 [kN]
FL31	6119 [kN]	4160 [kN]

a) Section pre-dimensioning for the simple concrete section variant

The area needed for a simple reinforced concrete section was calculated in ULS according to equation (9). By considering that the concrete section is a square the side of the column should be obtained as follows:

$$A_{nec} = \frac{N_{Ed,SLU}}{0.8\, f_{cd}} \; and \; h_{eff} = b_{eff} = \sqrt{A_{nec}} \tag{9}$$

To compute in the “special combination” – seismic combination, the norm [3] limits the value of the unitary compression stress to:

$$\nu_d = \frac{\sigma_{eff}}{f_{cd}} = 0.40 \tag{10}$$

The stress of the equivalent section was calculated with the formula:

$$\sigma_{eff,ech} = \frac{N_{Ed,GS}}{A_{ef}} \tag{11}$$

It results:

$$A_{nec,GS} = \frac{N_{Ed,GS}}{\nu_d \cdot f_{cd}} \; and \; h_{eff} = b_{eff} = \sqrt{A_{nec,GS}} \tag{12}$$

The sides needed for simple concrete sections will be considered the maximum between the two values for the two limit states.

b) Section pre-dimensioning for the compound section variant

For dimensioning the compound section of the column a constant ratio between RC area and UHPC area was considered. In order to compute in ULS it has been chosen to find an equivalent strength for the compound section according to (13) related to the proposed curve from [9]:

$$f_{cd,ech} = \frac{f_{cd,BO} \cdot A_{BO} + \frac{2}{3} f_{cd,BUIP} \cdot A_{BUIP}}{A_{BO} + A_{BUIP}} \tag{13}$$

The relation of the equation (12) applies to stress – strain diagrams described in [9]. The design elasticity moduli for each material of the compound section were calculated in [9]. From the dimensioning relation (14) the required area of the column with compound section in ULS was obtained:

$$A_{nec,SLU} = \frac{N_{Ed,SLU}}{0.8 \cdot f_{cd,ech}} \tag{14}$$

The remaining steps are similar to those described in calculating the simple sections.

2.4. Structural analysis

The structural analysis and the static model of the office building, see Fig. 3, was performed considering the characteristics of the material and section dimensions referred in [10].

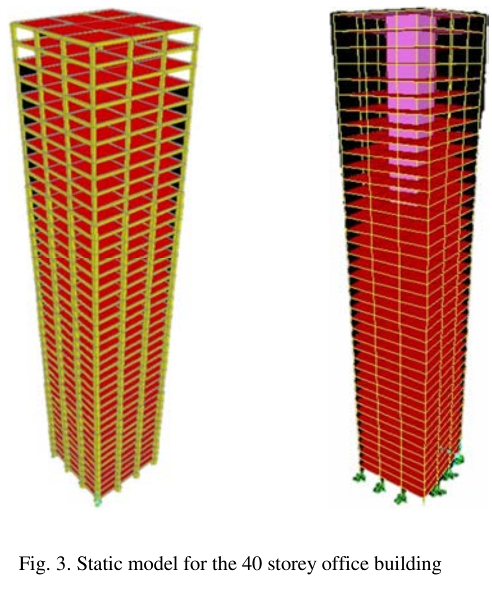

Fig. 3. Static model for the 40 storey office building

Columns communicate with beams by rigid joints; linear elements that communicate through a node deforms and behaves in solidarity. A generic displacement of the joint (rotation, translation) causes in the two types of elements one stress in proportion to the stiffness of each element and the induced movement of the node.

The stress $N - M_x - M_Y$ for each column were extracted separately considering four unfavourable situations as follows: Situation no. 1: N_{max}, $M_{x,af}$, $M_{y,af}$; Situation no. 2: N_{af}, $M_{x,max}$, $M_{y,af}$; Situation no. 3: N_{af}, $M_{x,af}$, $M_{y,max}$; Situation no. 4: N_{min}, $M_{x,af}$, $M_{y,af}$.

2.5. Reinforcing and checking the columns in both variants

The reinforcement and the check of the columns was made after processing the data obtained from structural analysis program (SAP 2000), for each column separately. A VBA routine (Visual Basic for Applications) was developed in order to test the validity of inequality (15) for each end of each column, in every combination involved in structural analysis. The determination of the effective amount of reinforcement involves an iterative process of testing and incrementing the area of the reinforcement, starting from the value of the minimum percentage of reinforcement (0.8% for ductility class "M"). At each step the validity of the following inequality must be checked according to [8]:

$$\left(\frac{M_{Ed,x}}{M_{Rd}}\right)^{\alpha_n} + \left(\frac{M_{Ed,y}}{M_{Rd}}\right)^{\alpha_n} \leq 1 \tag{15}$$

where $M_{Ed,x}$ = design bending moment in the direction X; $M_{Ed,y}$ = design bearing moment in the direction Y; M_{Rd} = capable bearing moment for the section in pure bending; α_n = coefficient obtained from linear interpolation, which represents the level of axial strain of the element. After processing the results of the values obtained, for both types of sections (simple and compound), they were summarized in Table 4.

Table 4. Summary of sections reductions and material economy

	Simple section				Compound Section			
Level	Side [cm]	Reinforcement [-]	Bearing moment [kNm]	Axial load [kN]	Side [cm]	Reinforcement [-]	Bearing moment [kNm]	Axial load [kN]
GF	130	$44\phi20$	1454.48	43723.12	80	$28\phi16$	283.41	21576.99
FL11	110	$40\phi18$	910.03	31392.18	70	$20\phi18$	308.99	17143.85
FL21	85	$24\phi18$	443.27	18753.64	70	$28\phi18$	408.96	13644.52
FL31	60	$40\phi22$	382.21	10759.38	65	$24\phi20$	405.02	12198.22

3. Conclusions

The reduction of the transversal section was between 32.18% and 62.13% for the columns belonging to the first 30 storeys. In the case of the variant of the compound section, due to the fact that the sections from the lower levels are more slender, the section of the columns from the last 10 storeys gets increased values of the bending efforts, which implies a greater reinforcement, therefore increasing the concrete section with 17.36% more than in the case of the simple section, see Table 5.

The analysis confirmed as in other cases, [12] and [13] that using UHPC, and the compound section in the case of high-rise buildings, allows structural engineers to design elements with transversal structures comparative to those of buildings made of Regular Concrete (RC). By reducing the section of the elements, a space economy is being obtained, which allows a more efficient organization of the total usable space. For example, using High Performance Concrete (HPC) for the columns at the Richmond – Adelaide building from Ontario, Canada, allowed the architects to increase the underground parking by ~30%, [14]. The economic advantages of using compound sections come from decreasing the sections used and implicitly reducing the quantity of reinforcement being used. According to [15] increasing the strength of the used concrete from 28 MPa to 83 MPa leads to a significant decrease of the quantity of longitudinal reinforcement which needs to be used up to ~67%. Using the compound section allows the structural engineers to maintain constant the section of the column for several storeys by reusing the same types of formworks. Corroborating the advantages mentioned beforehand, a significant economy can be made, the costs per square meter being considerably diminished.

By using the option with columns with compound sections, they have a total weight (sum of the two types of concrete and reinforcement associated with an average specific weight, for both sections of 2,500.00 kg/m^3) for the entire structure of 61.28 tons due to 119.48 tons in the simple section version. Therefore a reduction in mass of 48.71% is obtained by using compound columns versus using simple section columns. A direct consequence of the decrease of the structures mass is the decrease of the seismic load.

The results demonstrate that the building was designed to meet the design criteria in Service Limit State (SLS), Deformation Limit State (DLS) and Ultimate Limit State (ULS).

Table 5. Summarize of the section reduction and material economy

Level	Simple section		Compound Section		Section reduction
	[cm]	[cm^2]	[cm]	[cm^2]	[%]
GF	130	16900	80	6400	+62.13
FL11	110	12100	70	4900	+59.50
FL21	85	7225	70	4900	+32.18
FL31	60	3600	65	4225	-17.36

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