Seismic performance of a double core RC structure

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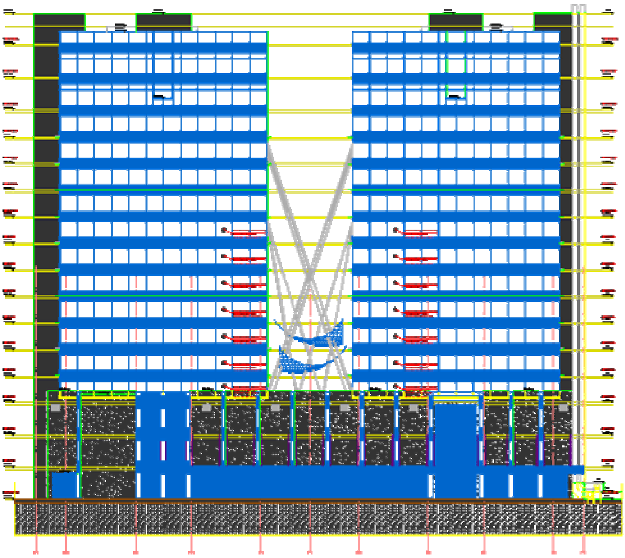
Abstract
The paper presents the conception and the seismic performance analysis of the double core RC structure for the new Transylvanian Business Centre in Sibiu, Romania. The height regime of the building is ground floor plus fifteen stories (the building is 50 m height), with two more underground levels. The earthquake resistant structure consists in two reinforced concrete cores, regular developed on the entire height of the building, and shear walls placed at the ground, first and second floors, more wide in the horizontal plane. The bearing structure is ensured by a flat slab structure. Seismic performance is assessed on the ground of European provisions for medium ductility members. The following parameters are considered: structural material properties, reinforcing systems and reinforcing ratios, confinement of concrete, insurance of the plastic potential zones, control of the axial force intensity, shear and twisting resistance and rigidity, story drifts limitation, redistribution of the internal forces and ductility in the plastic potential zones. The response spectra analysis procedure was used to approximate the dynamic response of the structural model. Modal results were combined through the Complete Quadratic Combination. Specific non-linear analyses were made through a numerical approach.

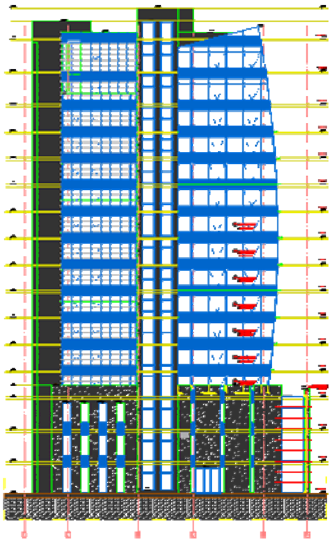
Keywords: structural conception, earthquake design, core structures, seismic performance, reinforced concrete.

1. Introduction
Initially designed to accommodate commercial and dwelling facilities, the building of the new Transylvanian Business Centre from Sibiu-Romania, shown in Figure 1, was modified to a full business facility during construction (see Figure 2). The building has two under ground floors for parking, and sixteen levels above ground. The total built area is 21,550 m^2. The ground floor, the first and second floors are monotonic developed on vertical covering a surface of 1,467 m^2, while from the third floor two symmetrical towers are developed on thirteen stories. The bearing structure consists in thick post-tensioned flat

slabs, stiffened by two cores, placed to the centre of each tower. Up to the third story, a transversal supplementary shear wall was introduced in the symmetry centre of the building, and the cores are stiffened by transversal and longitudinal lamellas.



a. main façade

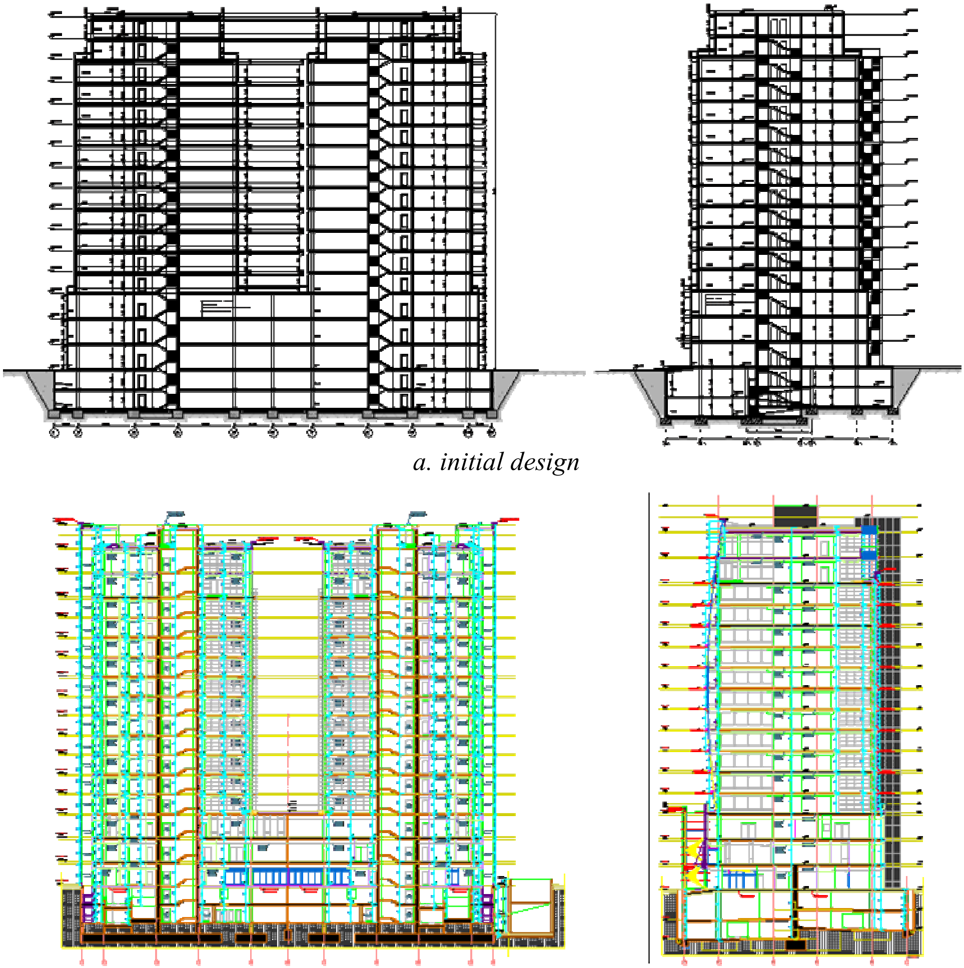


b. lateral façade

Figure 1: Views of the final design of the Transylvanian Business Centre

Figure 2: The building under construction

The infrastructure consists in a combination of mats and footings under thick RC boundary walls, columns and cores. The initial design presumed a withdraw of fourteen and fifteen decks at each tower, and a light planar building envelope (Figure 3.a). The second function presumed an extended surface of these floors and a curvilinear curtain wall on the main façade (Figure 3.b). Therefore, to preserve the safety of the foundation, the structure of these stories was modified to a light one, made of steel frames and composite decks. In order to improve the robustness of the structure to accidental actions, the columns were confined by bi-directional carbon fiber sheets at the ground and second floors. Sprayed concrete and steel nets confine the cores at the underground and ground floors too.



a. initial design

b. final design

Figure 3: Longitudinal and transversal cross-sections of the building

2. Structural conception

The building, framed in the second importance class, is placed on an area with the design ground acceleration a_g=0.16g and the elastic response spectrum presented in Figure 4. The primary structure is framed in the medium ductility class. On the ground given by a very restrictive actual code of practice for seismic design in Romania [1], based on many indirect structural design provisions, and the more permissible approach ensured by Eurocode 8 [2] (i.e., European norm fully applicable in Romania starting with 2010), presuming a more consistent direct approach in structural design, the primary (i.e., earthquake resistant structure) and secondary structures were separately considered.

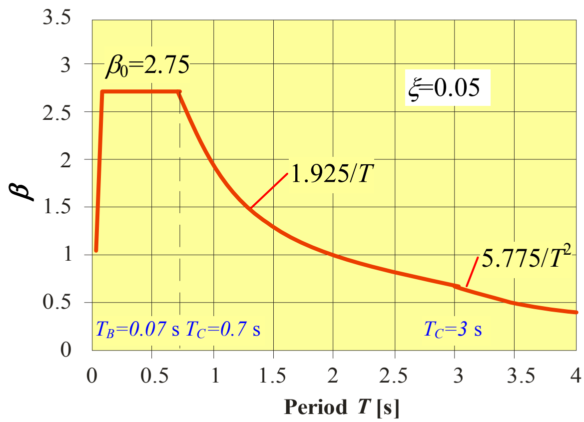


Figure 4: Elastic response spectrum for the ground (5% damping)

Due to the relatively reduced number of stories, a flexible solution was chosen, as shown in Figure 5. The primary structure of the building consists in two RC cores, developed on sixteen levels and two more rigid underground floors. The cores are stiffened by an inner RC staircase. Lateral stiffening was supplemented up to the third floor by a central transversal shear wall, and RC lamellas connected to the cores at the ground floor. From the forth floor, the cores become central cores in two towers developed up to the +51.84 m in elevation. The thickness of the cores is 550 mm from the ground floor (E0) to the forth story (E4), 450 mm between the fifth story (E5) and the eleventh story (E11), and 400 mm starting with the twelve story (E12). For all primary structural elements the design strength class of concrete is C 30/37, and the reinforcing high ductility steel (i.e., ribbed rebars) has f_{yk}=500 MPa, f_{uk}=550 MPa, f_{uk}/f_{yk}=1.1 and ε_{uk}=10 %.

The secondary structure (i.e., bearing structure) was conceived to ensure the maximum effective height. Therefore, the slab system consists in 300 mm thick post-tensioned flat slabs for the three extended floors above the ground, and 210 mm thick post-tensioned flat slabs for the two towers (see Figures 6.a and 6.b) up to the thirteen story. Boundary columns are disposed at 7.88 m and 6.24 m between axes and their cross-section varies gradually from 500×500 mm to 400×400 mm at the corners, and from 600×600 mm to

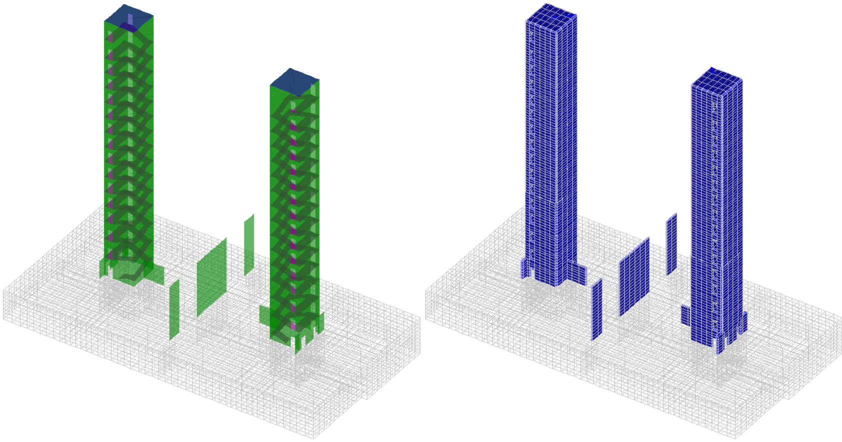
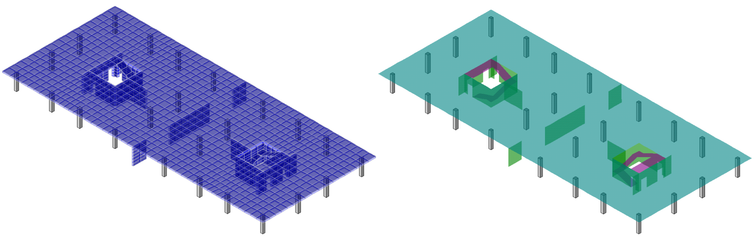
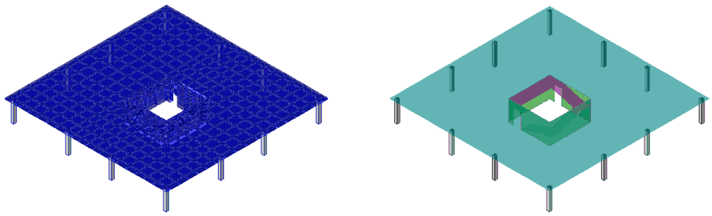


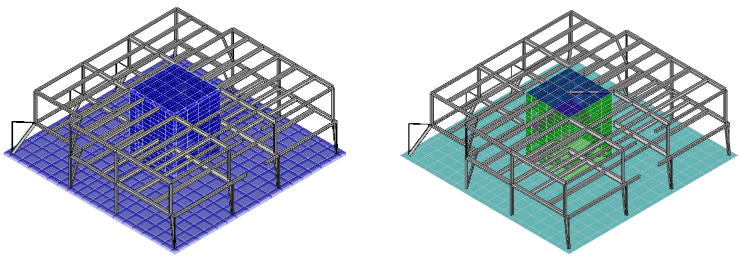
Figure 5: Model of the primary structure



a. ground floor



b. typical story above third floor, up to the thirteen floor



c. stories fourteen and fifteen

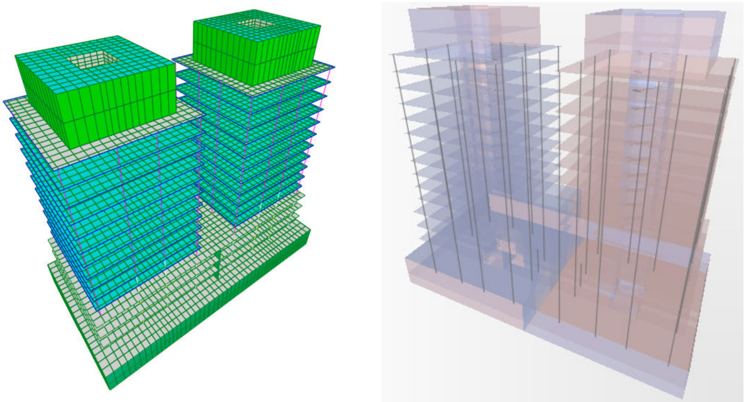
Figure 6: Floor models of the secondary structure

400×400 mm at the margins. Concrete strength class is the same C 30/37. The bearing structure of stories fourteen and fifteen was modified in order to comply with the new architectural design. Light steel frames sustain a slab system made of steel beams and a composite 13 cm deck, as shown in Figure 6.c.

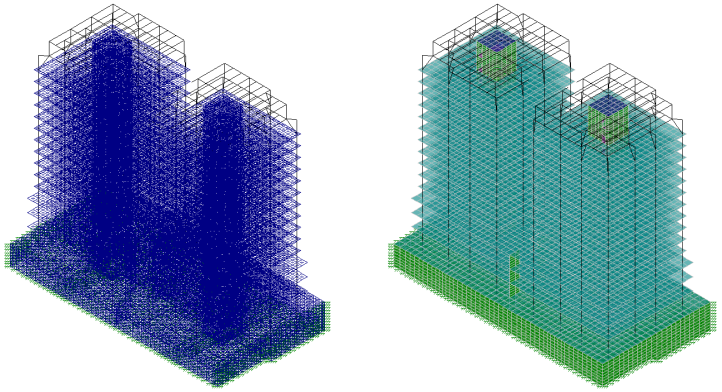
3. Structural analysis

3.1 Finite element modeling

Finite element models (Figure 7) were assembled by mixed interpolation (i.e., the in-plane and transverse shear strain components are derived independently) 4 joint quadrilateral elements (Bathe [3]), with five nodal degrees. Linear elements were used to model the columns.



a. initial model



b. final model

Figure 7: Complete FE models

Rigidity of the elements was considered as follows:

- Due to the weak connections between the flat slabs and the columns, these elements were considered with 40 % of the secant stiffness;

- The two cores and the shear walls were considered with 50 % of the secant stiffness, in order to take account by the cracking influence.

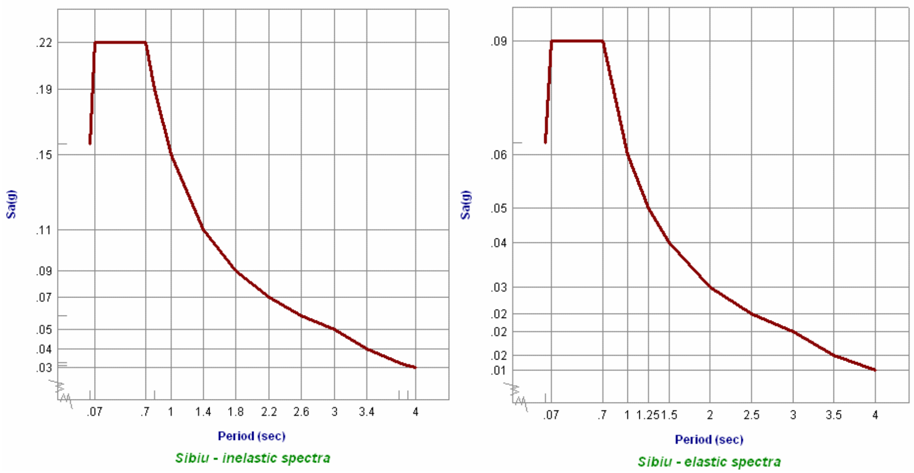
3.2 Dynamic analysis

Modal analyses were performed through the Response Spectra method, considering a damping ratio of 5 %. Design responses spectra for Ultimate Limit States (ULS) and Serviceability Limit States (SLS) are shown in Figure 7. Modal combinations were done through the Complete Quadratic Combination (i.e., CQC) method. Additional eccentricity and the rigid diaphragm effect were considered on each floors plane. Thirty independent vibration modes were considered, mass participation factors being more than 90 % on each principal direction and in horizontal plane rotation. The action effects due to the combination of the horizontal components of the seismic action were computed using the two following combinations proposed by Eurocode 8 [2]:

$$E_{Edx} \text{"+"} 0.30 E_{Edy} \quad (1)$$

$$E_{Edy} \text{"+"} 0.30 E_{Edx} \quad (2)$$

where "+" presumes "to be combined with", E_{Edx} are the action effects due to the application of the seismic action along the strong axis, and E_{Edy} are the action effects due to the application of the same seismic action along the weak axis.

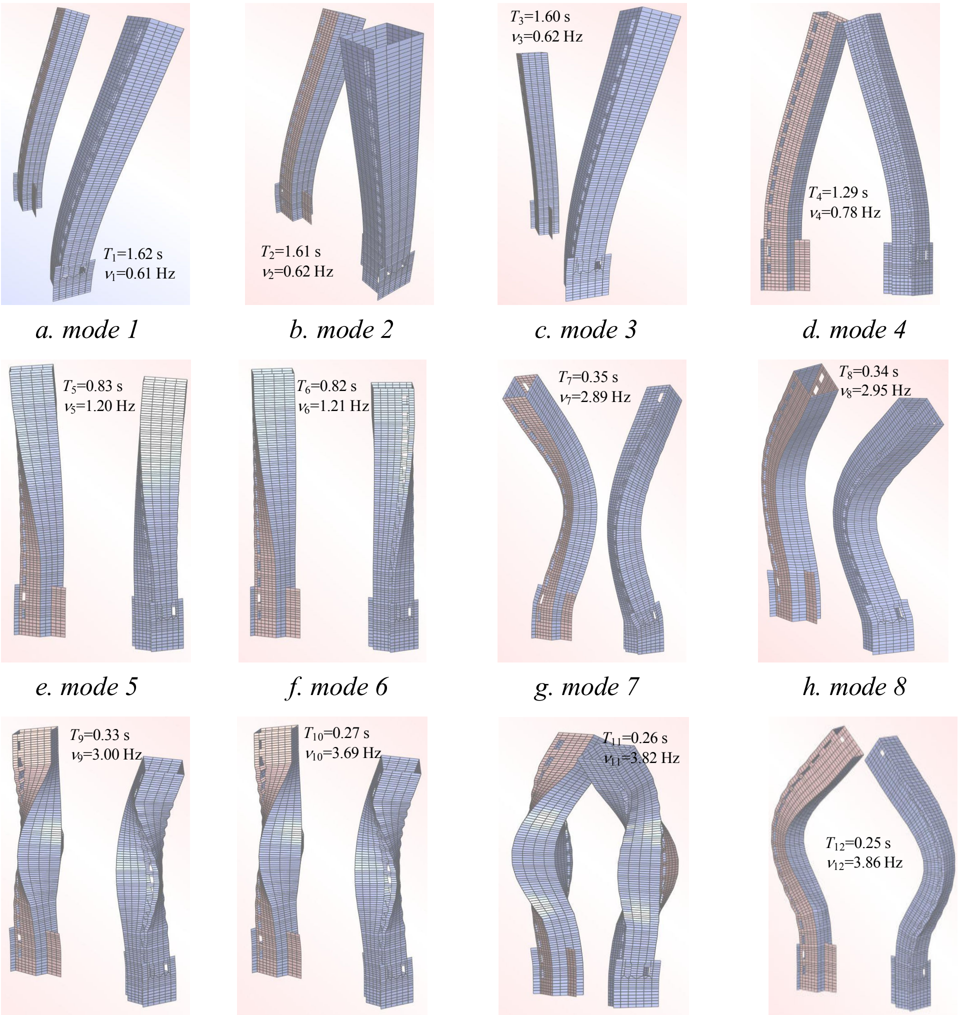


a. design spectrum for ULS (behavior factor=2)

b. design spectrum for SLS (recurrence factor = 0.4)

Figure 7: Responses spectrums

Figure 8 presents the most significant vibration modes resulted from the modal analysis. While the principal modes, with the most significant mass participation have vibration periods on the descendent branch of the design spectrum, superior bending modes and rotational modes have vibration periods that correspond to the maximum response spectrum. In this respect, the two regular hollow cores provide the adequate solution to ensure the required rigidity to sustain the twisting effects and to sustain the tangential stresses if the coupling beams have an elastic behavior.



a. mode 1 *b. mode 2* *c. mode 3* *d. mode 4*

e. mode 5 *f. mode 6* *g. mode 7* *h. mode 8*

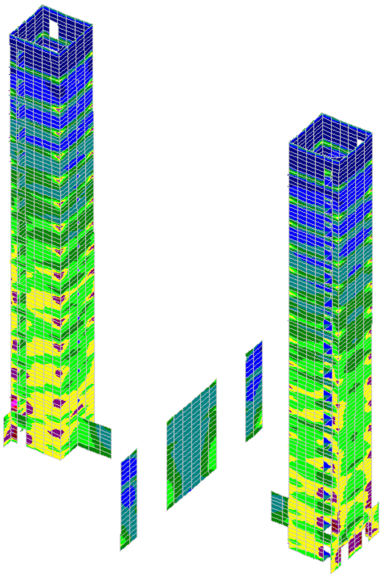
i. mode 9 *j. mode 10* *k. mode 11* *l. mode 12*

Figure 8: Significant modal results

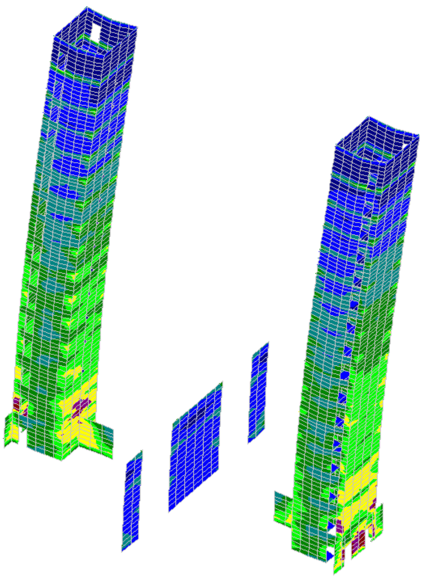
3.3 Seismic performance

Figure 9 shows the variation of the Von Mises stress resulted from the structural analysis. The normalized axial force in the cores is beyond 0.4 in all cross-sections of the primary elements. In order to ensure the cantilever behavior of the cores, an elastic response was given to the coupling beams, which as shown in Figure 9, is reasonable.

Figure 10 presents the reinforcement ratios on the height of the cores. The critical region was directed to stories E2 and E3, which were provided with less longitudinal reinforcement in order to ensure the local ductility. Another plastic potential zone was created at the storey E10, to adapt the post-elastic response to higher vibration modes.



a. dominant mode on weak direction



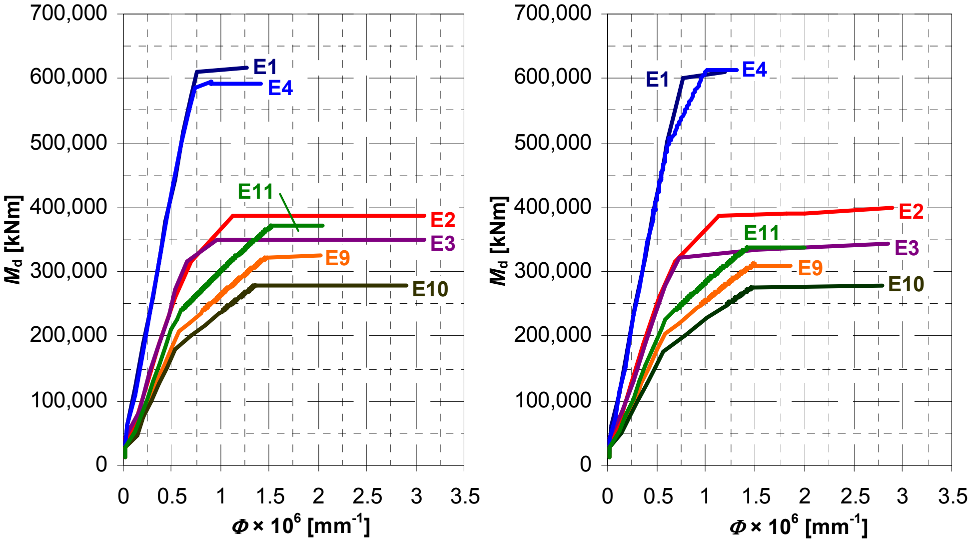
b. dominant mode on strong direction

Figure 9: Von Mises stress in the primary structure

Floor	ρ_l	ρ_w	ρ_l/ρ_w	Floor	ρ_l	ρ_w	ρ_l/ρ_w
E0	0.0448	0.0228	0.51	E8	0.0229	0.0179	0.78
E1	0.0448	0.0228	0.51	E9	0.0229	0.0179	0.78
E2	0.0224	0.0228	1.02	E10	0.0199	0.0179	0.90
E3	0.0224	0.0228	1.02	E11	0.0269	0.0179	0.67
E4	0.0448	0.0228	0.51	E12	0.0269	0.0179	0.67
E5	0.0308	0.0179	0.58	E13	0.0269	0.0179	0.67
E6	0.0308	0.0179	0.58	E14	0.0269	0.0179	0.67
E7	0.0229	0.0179	0.78	E15	0.0269	0.0179	0.67

Figure 10: Reinforcing ratios of the cores

Figure 11 shows the design moment-curvature diagrams in the vicinity of the critical regions obtained through non-linear analyses performed with the procedure given by Mircea and Petrovay [4], and Figure 12 shows the envelope of the design bending moments that results from structural analysis. The ability of the structure for internal redistribution of the bending moments is well emphasized.



a. weak axis *b. strong axis*

Figure 11: Design moment-curvature relations in the critical regions

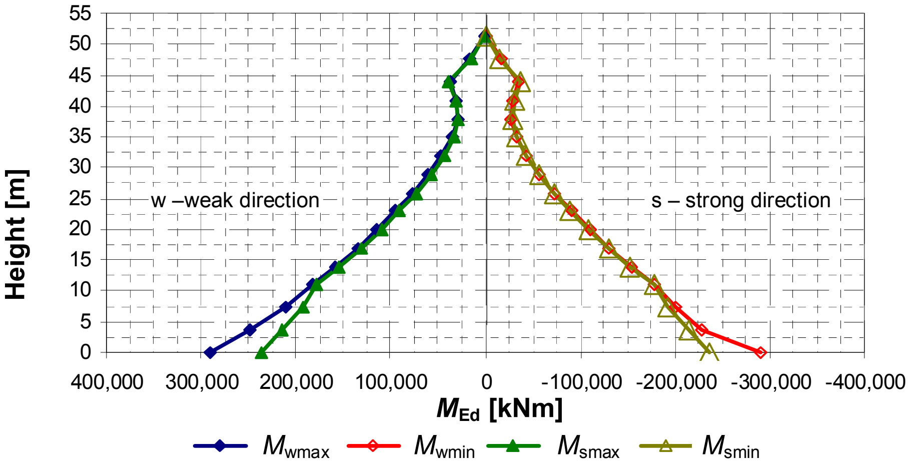


Figure 12: Envelope of the design bending moments

Because T_1=1.62 s > T_c=0.7 s, the minimum ductility factor in the critical regions is given by the relation:

$$\mu_\theta = 2q\frac{M_{\mathrm{Ed}}}{M_{\mathrm{Rd}}} - 1 \quad (3)$$

where q=2 is the behavior factor, M_{Ed} is the design value of the bending moment, and M_{Rd} is the resistant bending moment.

Figures 13 and 14 present the values of the curvatures and ductility factors in the critical regions (i.e., plastic potential regions). In the terms of the displacement states, structural performance is shown in Figures 15 and 16.

Floor	Φ_y	Φ_u	Φ_u/Φ_y		μ_θ
E2	1.12×10^{-6}	3.08×10^{-6}	2.75	>	1.17
E3	0.97×10^{-6}	3.10×10^{-6}	3.20	>	1.08
E10	1.36×10^{-6}	2.20×10^{-6}	1.62	>	0

Figure 13: Curvatures and ductility factors on the weak direction

Floor	Φ_y	Φ_u	Φ_u/Φ_y		μ_θ
E2	1.13×10^{-6}	2.90×10^{-6}	2.56	>	1.21
E3	0.72×10^{-6}	2.85×10^{-6}	3.96	>	1.39
E10	1.47×10^{-6}	2.78×10^{-6}	1.89	>	0

Figure 14: Curvatures and ductility factors on the strong direction

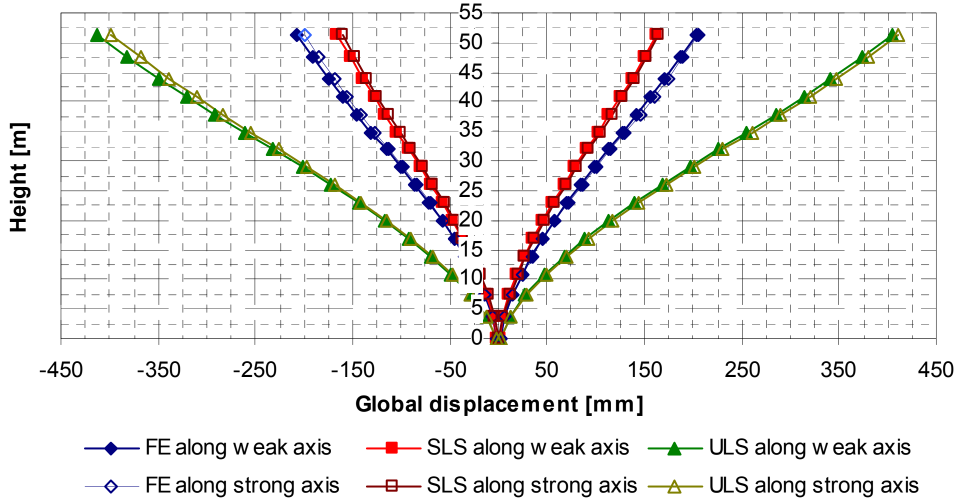


Figure 15: Envelope of the cores deflection

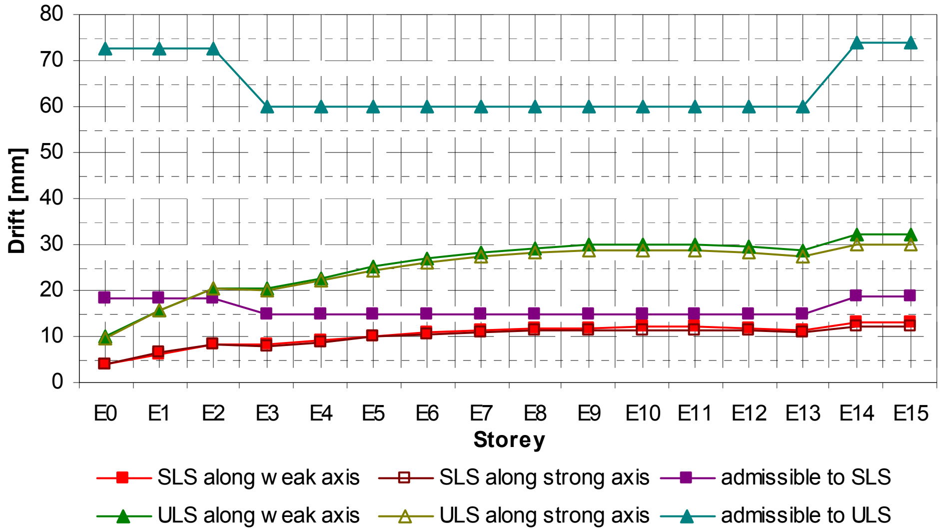


Figure 16: Storey drifts and admissible values

4. Conclusions

As shown above, core structures are very efficient also in seismic design, despite the so called torsional sensitivity. The hollow section provides the adequate torsional rigidity and resistance through an effective distribution of the shear stresses. Thus, the structural system is able to reach all performance parameters. Structural design of the double core structure presented in this paper proves that this solution is an optimal one for medium tall buildings.

Finally, it should be noticed that reinforced concrete remains the same valuable material, able to ensure all functional requirements and aesthetic conditions.

References

[1] P 100-1/2006, *Seismic design code: Design provisions for buildings* (*Cod de proiectare seismică: Prevederi de proiectare pentru cladiri*).

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

[3] Bathe K.J., *Finite Element Procedures*, Prentice-Hall, 1996.

[4] Mircea C. and Petrovay G., Virtual Bitmaps Application in Sectional Analysis of Composite Members, in *IASS 2004. Shell and Spatial Structures from Models to Realization*, Motro R (ed.), Editions de l'Esperou, 2004, 272-273.