Numerical studies on steel-concrete composite structures

G Urian¹, A Haupt-Karp²,
¹ Technical University of Cluj-Napoca, Faculty of Civil Engineering, Structures Department, 15 Constantin Daicoviciu street, Cluj-Napoca, Romania,
² S.C. Romsoft Comimpex S.R.L., 6 Portile de Fier street, Cluj-Napoca, Romania

E-mail: gabrielurian@yahoo.com

Abstract. The paper focuses on the seismic performances of steel-concrete composite structures made with fully encased steel-concrete composite columns and steel beams. An important objective is to study the influence of the structural steel ratio on the behaviour of composite columns. To meet the proposed objectives, a numerical study was developed on composite structures. Some comments, design recommendations and an economical study conclude the paper.

1. Introduction
The paper concentrates on the seismic performances of steel-concrete composite frames, with emphasis on the behaviour of composite columns. Concrete encased hot-rolled steel sections are still interesting for researchers from all over the world, being studied intensively for more than three decades. In order to improve design recommendations of current norms researchers are still developing extensive experimental tests, the latest in Singapore and China [1], especially on columns made with high strength materials [2]. Based on experimental tests, numerical models are implemented in different types of calculus programs (commercial or specially developed) in order to perform a large variation of sections, types of embedded profiles and quality of materials.

To achieve the proposed objectives a numerical study on composite structures was developed. The structures were designed with fully encased composite columns and steel beams. The analysis included five types of similar structures regarding the floor plan, but different heights. The studied structures had twelve, ten, eight, six and two levels. The columns were designed using three structural steel ratios: low, medium and high. The loads taken into consideration were identical for all types of structures. To investigate the seismic performances two types of analysis were performed: pushover and dynamic time history, based on the numerical model developed [3,4]. Beside the seismic performances of the structures, an economic study was conducted on the chosen frames.

2. Numerical model
The numerical model used was developed in 2013 in FineLg, a finite element program implemented at Liège University, ArGenCo department. The calibration and validation of the numerical model [3,4] was made using six experimental tests taken from the international literature on fully embedded steel-concrete composite columns.

2.1. Numerical model
The finite element used was a classic beam element (Bernoulli) for concrete plane frames with steel reinforcement and embedded beams with three nodes, as shown in figure 1, without taking into account the shear force effect. The total number of degrees of freedom corresponds to: one relative translational degree of freedom for the node situated at the mid-length of the beam element and one rotational and two translational degrees of freedom for each two nodes located at beam element ends and, as shown in figure 1. Nodes 1 and 3 have three degrees of freedom (u, v, θ), and node 2 has a single degree of freedom u, which allows to consider a possible relative displacement between steel and concrete.
This type of element does not allow the involvement of the local buckling phenomenon of the section. Because this analysis is two-dimensional, the bending phenomenon applied outside the section plane, such as torsional bending, is not taken into account in this analysis. It is considered a perfect connection between steel and concrete.

The materials behaviour model used are presented in figure 2 for concrete and figure 3 for steel.

2.2. Calibration and certification of the numerical model
The calibration and certification of the numerical model was made using six experimental tests taken from the international literature on fully encased steel-concrete composite columns. The experimentally tested columns had different types of concrete or steel, different structural steel or reinforcing steel ratios. The columns were tested (both monotonic and cyclic) to constant axial force and lateral forces. The subject of the paper is not the detailed presentation of the experimental tests used for the calibration and certification of the numerical model, but on the study of the seismic performances of the structures with composite columns, based on the case study developed with the numerical model validated using these experimental tests. Figure 4 presents the types of cross sections of the columns used for calibration and certification of the numerical model. The experimental programs used for calibration and certification of the numerical model were developed at Technical University of Cluj-Napoca, in Romania [5,6], at NCU, Chung-Li, in Taiwan [7], at UC, San Diego, California, in USA [8,9], at CTU, Hsinchu, in China [10], and at Lakehead University, Thunder Bay, Ontario, in Canada [11].

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Figure 1. Finite element

Figure 2. Material behaviour model for concrete

Figure 3. Material behaviour model for steel

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The main characteristics of test specimens are as follows: length of columns between: 0.80 ÷ 4.00 m, cross section of columns: between 160x180 ÷ 600x600 mm, characteristic compressive cylinder strength of concrete at 28 days: between 22.50 ÷ 92.30 MPa, yield strength of structural steel: between 293.40 ÷ 442.00 MPa, yield strength of longitudinal reinforcement: between 330 ÷ 634 MPa. The main parameters of test specimens are: structural steel ratio between: 0.16 ÷ 0.471 and slenderness ratio between: 0.32 ÷ 1.16.

The validation of the numerical model was performed by comparison of experimental force-displacement curves with numerical ones, as showed in figure 5 and figure 6.

With red colour are presented the experimental curves and with blue colour the numerically obtained ones. The difference between experimental and numerical values was between 0÷15 percent with a mean value of 5% [3,4]. For exemplification, in figure 5 is presented the comparison with a cyclically tested column in China [10] and in figure 6 with a monotonically tested column in Canada [11].
3. Case study
The developed case study [3] included five similar types of composite frames, with the same floor plan as showed in figure 7 and with twelve, ten, eight, six and two levels, as showed in figure 8 for the six levels structure. The composite frames had five openings of 6 meters in longitudinal direction and two openings of 7 meters in transversal direction.
The level height was identical for all levels, 3.20 m. The considered loads were identical for all levels: permanent load 6.50 kN/m² and live load 3.00 kN/m².

![Figure 8. Transversal frame for six levels structure](image)

The chosen seismic zone was the one corresponding to a peak ground acceleration of 0.40 g and a corner period of 1.6 s. After a preliminary design analysis performed with a commercial software were chosen IPE 550 profiles for the steel beams and the following materials for columns: C40/50 concrete, S355 steel for the structural steel profiles embedded in concrete and S500 steel for the longitudinal reinforcement. For each type of structure were chosen three types of composite columns. The difference between the columns was the structural steel ratio: low, medium and high. In table 1 are presented all the characteristics of the composite columns used in the analysis. The names of the structures have the following meaning: for example, 6Fb: 6 represents the number of levels, F comes from floors and the last number (b) represents the structural steel ratio chosen (a for low, b for medium and c for high). The columns of the twelve, ten and eight level structures were also varied in height, as showed in table 1.

### Table 1. Characteristics of case study structures

| Structure | Level | Column section (mm x mm) | Embedded profile | Longitudinal reinforcement | Structural steel ratio (δ) |
|-----------|-------|--------------------------|------------------|----------------------------|---------------------------|
| 2Fa       | all   | 390 x 400                | HE 200 A         | 16016                      | 0.288                     |
| 2Fb       | all   | 350 x 360                | HE 140 M         | 16014                      | 0.439                     |
| 2Fc       | all   | 350 x 360                | 160 x 150 x 18 x 28 | 16014                      | 0.506                     |
| 6Fa       | all   | 500 x 590                | HE 400 A         | 14022                      | 0.320                     |
| 6Fb       | all   | 490 x 510                | HE 260 M         | 14020                      | 0.543                     |
| 6Fc       | all   | 450 x 460                | 260 x 250 x 25 x 40 | 14018                      | 0.610                     |
In the preliminary design stage, all recommendations from P100-1/2013 [12] were considered. The ductility class chosen was DCH, with a q behaviour factor equal to 6.5 for the transversal frames and 4 for the longitudinal ones.

After the preliminary design stage, using the FineLg program, based on the numerical model presented at chapter 2.1., to study the seismic performances of the chosen frames two types of analysis were performed: pushover and dynamic time history. In the pushover analysis, the seismic forces had a triangular distribution. The time history analysis was performed using three artificial accelerograms, in accordance with P100-1/2013 [12] and one real (Vrancea 1977). The followed parameters were: the evolution of inter-storey drift at all levels, the global pushover curve and rotation capacity. In addition, the q behaviour factor was evaluated for all analysed frames.

![Figure 9. Pushover curves for 2Fa structure [3]](image-url)
For exemplification, in figure 9 are presented the pushover curves for 2Fa structure and in figure 10 the evolution of inter-storey drift at all levels for the same structure.

With a green vertical line is marked the inter-storey drift limitation of 0.0075h/ν, where ν represents the reduction factor that takes into account the lower return period of the seismic action associated with the damage limitation requirement and h is the height on structure. The 0.0075 value corresponds to buildings having non-structural elements with high dissipation capacity, attached to the structure, according to the seismic norm P100/1-2013 [12]. The yellow line represents the inter-storey drift limitation of 2.5%, FEMA 356-2000 [13] criteria for Life Safety.

The pushover analysis results for all studied composite structures are presented centralized in table 2. Table 2 presents the displacement (d_c) and corresponding force (F_b) for 0.0075h criteria [12], 2.5% drift limitation according to FEMA 356-2000 [13] and the values at concrete failure, when \( \epsilon_{cte2} \) reaches 3.5‰ value. The last column presents the corresponding force when \( \theta_p \) reaches 35 mrad value, where \( \theta_p \) represents the rotation capacity of the plastic hinge region. The six and two floor structures did not achieve a minimum rotation capacity of the plastic hinge region of 35 mrad, necessary to design the structure in DCH ductility class, as can be seen in table 2. From the eight floor, analysed frames reached a superior rotation capacity of the plastic hinge region, 37 mrad for 8Fa structure to 69 mrad for 12Fc structure.

Table 2. Pushover analysis results

| Structure | 0.0075h/ν | 2.50% | Concrete failure | 35 mrad corresponding force (kN) |
|-----------|-----------|-------|------------------|----------------------------------|
|           | F_b (kN)  | d_c (m) | F_b (kN) | d_c (m) | F_b (kN) | d_c (m) |
| 2Fa       | 676       | 0.046  | 804      | 0.112   | 934      | 0.067   | - |
| 2Fb       | 610       | 0.049  | 804      | 0.121   | 858      | 0.082   | - |
| 2Fc       | 616       | 0.050  | 891      | 0.131   | 891      | 0.113   | - |
| 6Fa       | 744       | 0.112  | 1443     | 0.307   | 1480     | 0.369   | - |
| 6Fb       | 733       | 0.115  | 1267     | 0.314   | 1351     | 0.374   | - |
Table 3 presents the \( q \) behaviour factor obtained in the pushover analysis and dynamic one, using artificial accelerograms, according to P100/1-2013 [12] and real ones (Vrancea 1977).

### Table 3. \( q \) behaviour factor results

| Structure | \( q_{\text{max}} \) Pushover | \( q_{\text{max}} \) P100-1-2013 | \( q_{\text{max}} \) Vrancea |
|-----------|-------------------------------|-------------------------------|-----------------|
| 2Fa       | 4.00                          | 4.40                          | 4.20            |
| 2Fb       | 4.20                          | 4.50                          | 4.30            |
| 2Fc       | 4.50                          | 4.75                          | 4.35            |
| 6Fa       | 5.00                          | 5.25                          | 5.10            |
| 6Fb       | 6.10                          | 6.10                          | 5.98            |
| 6Fc       | 6.30                          | 6.30                          | 6.20            |
| 8Fa       | 5.30                          | 5.30                          | 5.20            |
| 8Fb       | 6.60                          | 6.40                          | 6.30            |
| 8Fc       | 7.20                          | 7.40                          | 7.10            |
| 10Fa      | 5.80                          | 6.10                          | 5.95            |
| 10Fb      | 6.40                          | 6.45                          | 6.35            |
| 10Fc      | 7.00                          | 7.20                          | 7.00            |
| 12Fa      | 5.60                          | 5.45                          | 5.20            |
| 12Fb      | 6.50                          | 6.60                          | 6.40            |
| 12Fc      | 7.20                          | 7.10                          | 6.95            |

Based on the performed analysis it is recommended that low level structures (with one up to six-seven levels) to be designed in medium ductility class. Structures with more than eight levels can be designed in medium or high ductility class, depending on the architectural and/or structural restrictions. Higher structural steel ratio leads to an important increase of structure ductility, more pronounced from low to medium than from medium to high. Given the financial importance, the structural analysis was completed with an economical study of the frames for an optimal choice of structural steel ratio. In ‘Table 4’ is presented the cost of each type of designed column per 1 linear meter of element. The final price was obtained by summing the costs of all materials (structural steel, concrete and reinforcement), formwork and labour. The prices were calculated based on average offers received from local suppliers. The most economical solution is offered by choosing a low structural steel ratio. Up to eight floors, the price difference between columns with a low structural steel ratio and medium steel ratio is about 15%. This difference decreases substantially at higher structures up to about 5%. In comparison with low structural steel ratio, a medium one offers an important increase of structural ductility and slimmer cross-sections, so the 5% cost difference is considered acceptable. The values of the structural steel ratio for the chosen structures were for low: between 0.209÷0.32, with a mean value of 0.276, for medium: between 0.349÷0.543, with a mean value of 0.403 and for high: between 0.506÷0.610, with a mean value of 0.562.
Table 4. Costs of case study columns – per 1 linear meter of column

| Structure | Level | Concrete price/1m (euro) | Formwork price/1m (euro) | Reinforcement price/1m (euro) | Steel price/1m (euro) | Column price/1m (euro) |
|-----------|-------|--------------------------|--------------------------|-------------------------------|-----------------------|------------------------|
| 2Fa       | all   | 15.5                     | 12.6                     | 27.5                          | 57.6                  | 113                    |
| 2Fb       | all   | 12.5                     | 11.3                     | 21.1                          | 86.0                  | 131                    |
| 2Fc       | all   | 12.5                     | 11.3                     | 21.1                          | 109.8                 | 155                    |
| 6Fa       | all   | 29.3                     | 17.3                     | 45.5                          | 170.2                 | 262                    |
| 6Lb       | all   | 24.8                     | 15.9                     | 37.6                          | 234.2                 | 312                    |
| 6Fc       | all   | 20.5                     | 14.5                     | 30.5                          | 261.8                 | 327                    |
| 8Fa       | 1÷4   | 46.4                     | 22.6                     | 83.9                          | 145.7                 | 299                    |
|           | 5÷8   | 34.6                     | 18.9                     | 53.7                          | 125.8                 | 233                    |
| 8Fb       | 1÷4   | 39.7                     | 20.5                     | 65.0                          | 190.6                 | 316                    |
|           | 5÷8   | 29.4                     | 17.3                     | 53.7                          | 152.5                 | 253                    |
| 8Fc       | 1÷4   | 29.4                     | 17.3                     | 43.0                          | 337.6                 | 427                    |
|           | 5÷8   | 22.9                     | 15.3                     | 34.8                          | 234.2                 | 307                    |
| 10Fa      | 1÷5   | 48.6                     | 23.5                     | 83.9                          | 204.2                 | 360                    |
|           | 6÷10  | 33.2                     | 18.6                     | 53.7                          | 125.8                 | 231                    |
| 10Fb      | 1÷5   | 41.7                     | 21.3                     | 65.0                          | 258.7                 | 387                    |
|           | 6÷10  | 27.3                     | 16.7                     | 43.5                          | 152.5                 | 240                    |
| 10Fc      | 1÷5   | 34.4                     | 18.9                     | 52.0                          | 358.0                 | 463                    |
|           | 6÷10  | 22.9                     | 15.3                     | 34.8                          | 234.2                 | 307                    |
| 12Fa      | 1÷4   | 119.1                    | 41.3                     | 206.3                         | 302.2                 | 669                    |
|           | 5÷8   | 79.4                     | 33.4                     | 157.9                         | 204.2                 | 475                    |
|           | 9÷12  | 33.2                     | 18.6                     | 53.7                          | 125.8                 | 231                    |
| 12Fb      | 1÷4   | 85.2                     | 34.5                     | 136.9                         | 427.5                 | 684                    |
|           | 5÷8   | 61.9                     | 27.3                     | 109.1                         | 328.1                 | 526                    |
|           | 9÷12  | 27.3                     | 16.7                     | 43.5                          | 152.5                 | 240                    |
| 12Fc      | 1÷4   | 59.4                     | 26.5                     | 92.3                          | 634.4                 | 813                    |
|           | 5÷8   | 43.9                     | 21.8                     | 71.5                          | 543.2                 | 680                    |
|           | 9÷12  | 22.9                     | 15.3                     | 34.8                          | 234.2                 | 307                    |

4. Conclusions
Composite frames made with steel beams and fully encased steel-concrete composite columns can be an efficient solution for buildings situated in medium and high seismicity zones. From the case study developed some notable conclusions can be drawn: small structures (up to 6-7 levels) are recommended to be designed in medium ductility class; for higher structures a medium or high ductility class can be adopted, the solution chosen being optimized from different points of view: cross-section dimensions, necessary rotation capacity, costs, etc. When considering only the economical point of view the structures with low steel ratio offered the best results, but considering the 5% (for tall buildings) difference in using low or medium steel ratio it is recommended a medium one when designing a composite column.

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