Behavior of Fully Encased Steel-Concrete Composite Columns Subjected to Monotonic and Cyclic Loading

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Abstract

The paper describes experimental aspects for composite steel-concrete columns, with steel encased profile. In composite columns with HSC case, even if elastic displacement $v_y$ decreases, the value of the lateral force $H_y$ corresponding to the $v_y$ displacement and the maximum lateral loading $H_{max}$ indicates a significant increase. Failure modes were different, characterized by sudden and violent concessions due to cracking developments through aggregate in columns with HSC, while columns with NSC shows a “slow” failure mode characterized by gradual decline of bearing capacity with the growth of the displacements. It is well known that the high strength concrete is more susceptible to fragile failure than the normal concrete, so it is, in a way, the presumed result. On the other hand, from the graphics and parameters analysis we can conclude that the columns with HSC have a higher energy absorption capacity, which can recommend this solution to the construction in seismic areas, even the failure mode was brittle. In structural terms, composite columns with concrete class C70/85 provide obvious better performances to structures, having significant increases to almost all analyzed parameters.

The solution of fully encased composite column is a competitive solution for seismic and non-seismic zones, due to the excellent seismic performances (resulted from the presented experimental tests) and also because of improved fire protection. The results obtained on the columns made with high strength concrete showed improved performances, especially resistance. Due to the brittle fracture of the high strength concrete more experimental and numerical research must still be made.

Keywords: steel-concrete composite column, bearing capacity, monotonic and cyclic loading.

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1. Introduction

The composite members are used in construction from the necessity of protecting the steel member from fire and corrosion. The benefits brought to the columns yield and stability by using concrete to encase the steel profile haven’t been took account of until the 50’s. The tests made by Faber [1] revealed the fact that using a higher concrete quality the member can be evaluate as a composite member. More recently, with the advent of modern composite frame construction in high rise buildings, engineers developed new methods to take advantage of the stiffening and strengthening effects of concrete and reinforcing bars on the bearing capacity of steel-concrete composite member. The developments related to high-strength concrete and seismic design motivate the review of composite column behaviour and current design provisions.

Using the composite leading to larger openings, reducing the height levels and provides a better lateral stiffness. Under large-magnitude seismic events, concrete shells crack and lower the flexural stiffness of composite beam-columns. Nevertheless, the steel core acts as a back-up system in providing the shear strength and the required ductility to prevent brittle failure modes. Eurocode 4 [2] provides a relative simple method to design composite columns. The EC4 Simplify Method has some restrictions related to cross section, it applies only for bi-symmetric form of the cross section of the steel shape, and for the concrete class, restricted to maximum C50/60 [1-29].

2. Experimental programs

The first two experimental programs used for validation were developed in the Structures Department, at Faculty of Civil Engineering, Technical University of Cluj-Napoca, Romania, year 2000 and 2011. The third program was developed at National Central University in 2008, Taiwan.

3. Experimental program developed at UTC-N, Romania, 2000

The experimental program realized by Cristina Campian, 2000, at Technical University of Cluj-Napoca, Romania, included 12 tests (3 monotonic and 9 cyclic) on fully encased steel-concrete composite columns. All columns had the same cross-section and were grouped according to their length. The elements were made with a Romanian steel section I12 (which is quasi similar to IPE 120 section) fully covered with reinforced concrete including 4 φ10 longitudinal bars as shown in figure 2. In table 1 are presented some characteristics of the tested specimens.
The failure of all tested columns was governed by the plastic hinge formation at column base (see Fig. 2).

4. Experimental program developed at UTC-N, Romania, 2011

The test were carried out in Technical University of Cluj Napoca laboratories, and they are focused on the increasing the concrete class effects. Therefore the mechanical behavior and seismic resistance of the 8 specimens of concrete encased composite columns Fig. 3 were observed. Seismic resistance of columns was tested on full-scale specimens subjected to cyclic loading and a constant axial force. The cross-sectional area of composite columns was 220 mm×170 mm and height varying from 2 m long, for 4 specimens to 3 m long for the rest of them, resulting different slenderness.

The steel profile used was an IPN 120, being classified as a S275 steel grade after the test for traction, and it was encased in high strength concrete (HSC) C70/85. The longitudinal reinforced are 4 PC52 (BST500) bars of ø12 diameter. The confining hoops have a ø 8 diameter with spacing of 100 mm on the critical zone (600 mm) and 200 mm spacing on the rest of the column length, Fig. 3. The specimens were tested combining several parameters as
magnitude of axial force N which was chosen to correspond to a design compression rate of 7% from $N_{pl,Rd}$, respectively 14% from $N_{pl,Rd}$, to simulate static load on the column, where $N_{pl,Rd}$ was the nominal compressive strength of the composite column. The $N_{pl,Rd}$ value was calculated in accordance with the design rules of EC4 Simplified Method except the concrete provisions.

The horizontal load is applied on the column at the distance representing the middle height of the story. This load is cyclically applied in positive and negative range of value by two 80 kN actuators placed in the right and in the left side of the free end to simulate the horizontal seismic force on the column Fig. 5. The column is considered embedded on the base and free of restraints on the other side. To ensure a suitable full restraining at the column base, the elements were ended by a sudden cross-section enlargement acting as a foundation (the flexural stiffness ratio between the element and the so-realized foundation was about 1/5).

Fig. 5. The setup for the cyclic loading test

Fig. 6. The setup for the cyclic loading test
Similar experimental tests have been held in our department in 2000 [3] but the used concrete was a normal concrete (NC) C20/25. This was the start point for furthermore researches in order to be able to make a full comparison and parameters analysis. The testing procedure in both cases was the one recommended by ECCS for characterizing the behavior of steel elements assessing to seismic action (ECCS-TWG 1.3, 1986 – using a monotonic test to calibrate the cyclic tests) [4].

5. Monotonic loading test

The monotonic test according to ECCS Procedure was meant to be done for deduction from the recorded F-e curve, the conventional limit of elastic range F_y^+ and the corresponding displacement e_y^+. So value of elastic displacement resulted e_y^+ = 30 mm

![Fig. 7. Failure mode](image)

6. Monotonic and Cyclic loading test

The lateral displacements were imposed to the column as is shown in Fig 7. The levels of the displacement increased starting with one cycle of e_y^+/4, 2e_y^+/4, 3e_y^+/4, e_y^+ intervals, continuing with three cycles on the 2e_y^+, 4e_y^+ intervals. The stiffness of the columns decreased with increase of the loading cycles. The first cracks in the concrete occurred by the direction of confining hoops. After that the concrete was expelled the longitudinal bars have been revealed. At that stage a big percentage of the bearing capacity of the concrete section was lost, and we could took into account just the concrete section located between the longitudinal bars and the steel shape.
The critical moment is achieved when the longitudinal bars are buckling and all the concrete is expelled from the steel profile. In this stage the loading is carried out only by steel profile. The test usually ends when the member fails under the effect of the compressive force. The failure modes of the columns are shown in Fig. 10 corresponding to plastic hinges developed at the base of the column.

7. Tests results and discussions

In the present paper, only one series of tests, the one of 3 m length specimens, was fully described. Based on these results the comparison with columns using normal concrete was made. The results are listed in Tab. 2 including various values of axial forces and the mode of applying lateral force, monotone (M) or cyclic (C).

Table 2. Values for axial force N and lateral force H

| Specimen | Loading type | Axial force N [kN] | Max. lateral force H [kN] |
|----------|--------------|--------------------|--------------------------|
|          |              |                    |                          |
On the one hand the monotonic tests revealed that elastic displacement $e_+^y$ decreases with a range of 16% for the specimen with high strength concrete versus the ones with normal concrete and, on the other hand the lateral force achieve to a maximum values with an 28% higher than the specimen with NC. The shape of the F-e curve is quite different. The F-e curve of the columns with HSC linearity was noticed and a more pronounced tilt of the ascending zone caused by the superior stiffness. The descending branch the curve tends to plumb, as it can bee seen in Fig. 6, it was decreasing sharply in comparison with columns using NC curve.

Table 3. Values for elastic displacement related lateral force H

|                  | Columns with High Strength Concrete | Columns with Normal Concrete |
|------------------|-------------------------------------|----------------------------|
| Elastic Displ. [mm] | 30                                  | 35                         |
| Max. Lateral Force (daN) | 2445                                | 1740                       |

As it can be seen in the hysteretic diagram in Fig. 11 and Fig. 12 the behaviour of the specimens subjected to a different level of compression force is quite different. On the one hand, the analysis made on $2e_+^y$ cycles, reveals the fact that the maximum lateral force increase with a rate of 10% for specimen subjected to 400 kN axial force, on the other hand when we increase the displacement to $4e_+^y$ cycles, the specimen S4-3C collapse under this loading effect, without increasing lateral force. The column S2-3C is able to take-over the loading related to the displacement of three $4e_+^y$ cycles and further more.

![Fig. 11. Hysteresis loops of the specimen S2-3C subjected to cyclic loading and axial loading of 200 kN](image1)

![Fig. 12. Hysteresis loops of the specimen S4-3C subjected to cyclic loading and axial loading of 400 kN](image2)

The comparison between columns using high strength concrete C70/85 and columns using normal concrete involve the analysis of the average area of 3 cycles of $2e_+^y$ displacement, respectively the average area of 3 cycles of $4e_+^y$ displacement. The graphics are presented in Fig. 13 and Fig. 14 respectively Fig. 15 and Fig. 16. With areas obtained from the cycles we can calculate the absorbed energy ratios. Eq. (1) and Eq. (2)
\[ \eta^+_i = \frac{A^+_i}{F^+_y \cdot (e^+_y + e^+_e + e^-_e - e^-_y)} \]  

\[ \eta^-_i = \frac{A^-_i}{F^-_y \cdot (e^-_i + e^-_e + e^+_e - e^+_y)} \]  

Fig. 13. Comparison average values between S2-3C and SIII-2 for three 2e+y cycles  
Fig. 14. Comparison average values between S2-3C and SIII-2 for three 4e+y cycles  

Fig. 15. Comparison average values between S3-3C and SIII-3 for three 2e+y cycles  
Fig. 16. Comparison average values between S3-3C and SIII-3 for three 4e+y cycles  

Comparison results reveals that the absorbed energy ratio for three 2 e_+ y cycles in column using high concrete
case is 140% higher than the energy ratio for three \(2 e_y\) cycles of column using normal concrete, as for three \(4 e_y\) cycles absorbed energy ratio is 22% higher. The values are presented in Tab. 4.

| Specimen   | Absorbed Energy ratios | 2\(e_y\) cycles | 4\(e_y\) cycles |
|------------|------------------------|------------------|-----------------|
|            | \(\eta_i\)             | \(\eta_i\)       | \(\eta_i\)      | \(\eta_i\)      |
| S2-3C(HSC) | 0.174                  | 0.176            | 0.41            | 0.42            |
| S3-3C(HSC) | 0.20                   | 0.22             | 0.31            | 0.38            |
| SIII-2(NC) | 0.067                  | 0.12             | 0.36            | 0.26            |
| SIII-3(NC) | 0.087                  | 0.055            | 0.31            | 0.31            |

The failure mode was similar for all tested specimens. In comparison with the columns made with normal concrete, the failure of the columns made with high strength concrete was violent and brittle.

Experimental program developed at NCU, Chung-Li, Taiwan, 2008.

8. Experimental program developed at NCU, Chung-Li, Taiwan, 2008

The experimental study made by H. L. Hsu, F. J. Jan and J. L. Juang, 2008, was developed at the Department of Civil Engineering, National Central University, Chung-Li, Taiwan. All tested columns had the same cross-section, 370 mm x 370 mm, with six different embedded profiles (Tab. 5). The type of loading and direction are presented also in Tab. 5.

| Column type | Embedded profile | Loading direction | Loading type        |
|-------------|------------------|-------------------|---------------------|
| YAM         | H100x100x6x8     |                   |                      |
| YBM         | H150x100x6x9     |                   |                      |
| YDM         | H200x100x5.5x8   |                   |                      |
| YCM         | H150x150x7x10    |                   |                      |
| YEM         | H200x150x6x9     |                   |                      |

weak-axis bending

strong-axis bending

Fig. 17. Cross-section of the tested specimens and failure mode

Table 5. Characteristics of tested specimens

Table 5. Absorbed energy ratios
Identical reinforcement were used in all specimens, 4 Ø20 as longitudinal reinforcement and Ø 9.525 stirrups. The stirrup spacing was 100 mm within the confined zones and 150 mm in the non-confined zones. Yield strength for the structural steel, longitudinal bars and stirrups were 314 MPa, 543 MPa and 586 MPa respectively. The concrete compressive strength, determined from cylinder tests was 38 MPa.

The member performances were governed by plastic hinge formation, as shown in Fig. 17.

9. Behavior of fully encased steel-concrete composite columns

The general procedure accepted for experimental testing of steel elements in EU is "Recommended Testing Procedure for Assessing the Behavior of Structural Steel Elements under Cyclic Loads". One method to determine the properties and behavior of the steel-concrete composite columns is to evaluate the ECCS recommended parameters.

| Specimen | Identified | Axial loading + strong- | cyclic |
|----------|------------|-------------------------|--------|
| YFM      | H200x200x8x12 |
| XAC00    | H100x100x6x8  |
| XBC00    | H150x100x6x9  |
| XDC00    | H200x100x5.5x8|
| XCC00    | H150x150x7x10 |
| XEC00    | H200x150x6x9  |
| XFC00    | H200x200x8x12 |
This procedure was developed for steel elements, because of the similarity was adapted for the composite steel-concrete section as well. The referred parameters allow us to study some mechanical characteristics of the composite steel-concrete columns, with embedded steel profile, such as: ductility of the element, load bearing capacity, stiffness of the element or absorbed energy capacity. All presented parameters are defined as a ratio from the cyclic loading and the monotonic reference test (in absence of the monotonic test the elastic limit is determined from first cycle). In Fig. 18 are compared some ECCS parameters determined from experimental results. The presented parameters were determined for specimen 3 and 7 from the experimental program presented at 2.4. With NC was noted the results for specimen 3, column with normal concrete and with HSC the results for specimen 7, made with high strength concrete.

The plastic evolution of the columns is characterized by its ductility, partial or full. The evolution of the partial ductility of the two columns is presented in Fig. 18/a. This parameter represents the ratio between the maximum displacement (positive or negative) at cycle $i$ and the elastic limit. The partial ductility increases from one cycle to the other. The partial ductility doesn’t take into account the instant degradation in half of cycle, while the full ductility does. Fig. 18/b presents the evolution of the full ductility parameter for the analyzed columns. The full ductility has the same behavior as partial one, increasing by one cycle to another. All values obtained on experimental tested columns dictate that fully embedded composite columns are ductile elements in a structure. Fig. 18/c presents the full ductility ratios for all tested columns. The behavior is similar; it increases by one cycle to another. The ductility represents a conclusive method to establish the energy dissipation capacity of a structural system. The safety of a structure increases when the energy dissipation is through a large number of plastic hinges. A parameter who offers information about the column degradation during the cycles is the ratio between the maximum force at cycle $i$ and the reference force. It can be seen in Fig. 18/d that after reaching the maximum force, the resistance ratio decreases by one cycle to another.

A parameter who offers information about the column degradation during the cycles is the ratio between the maximum force at cycle $i$ and the reference force (the resistance ratio). A more pronounced degradation is observed on columns made with high strength concrete. The absorbed energy ratio parameter describes better the cyclic behavior of the tested elements. The curve slope increases for all analyzed sections (see Fig. 18/e), so the stored energy increases by one cycle to another. To further evaluate member performance under earthquake excitation, the energy dissipation of member tested were compared. Energy dissipation was evaluated using the cumulative area bounded by the hysteretic curves (see Fig. 18/f). The steel-concrete composite columns are a competitive solution for seismic zones, due to the many advantages (economic sections, fire protection, etc.) and to the energy dissipation capacity.
10. Conclusions

The paper describes experimental aspects for composite steel-concrete columns, with steel encased profile. The following conclusions can be highlighted, within the limitation of the current research.

In composite columns with HSC case, even if elastic displacement $v_y$ decreases, the value of the lateral force $H_y$ corresponding to the $v_y$ displacement and the maximum lateral loading $H_{\text{max}}$ indicates a significant increase.

Failure modes were different, characterized by sudden and violent concessions due to cracking developments through aggregate in columns with HSC, while columns with NSC shows a “slow” failure mode characterized by gradual decline of bearing capacity with the growth of the displacements. It is well known that the high strength concrete is more susceptible to fragile failure than the normal concrete, so it is, in a way, the presumed result.

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The solution of fully encased composite column is a competitive solution for seismic and non-seismic zones, due to the excellent seismic performances (resulted from the presented experimental tests) and also because of improved fire protection. The results obtained on the columns made with high strength concrete showed improved performances, especially resistance. Due to the brittle fracture of the high strength concrete more experimental and numerical research must still be made.

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