Seismic Behaviour of Composite Steel Fibre Reinforced Concrete Shear Walls

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Abstract. In this paper is presented an experimental study conducted at the “Politehnica” University of Timisoara, Romania. This study provides results from a comprehensive experimental investigation on the behaviour of composite steel fibre reinforced concrete shear walls (CSFRCW) with partially or totally encased profiles. Two experimental composite steel fibre reinforced concrete walls (CSFRCW) and, as a reference specimen, a typical reinforced concrete shear wall (RCW), (without structural reinforcement), were fabricated and tested under constant vertical load and quasi-static reversed cyclic lateral loads, in displacement control. The tests were performed until failure. The tested specimens were designed as 1:3 scale steel-concrete composite elements, representing a three storeys and one bay element from the base of a lateral resisting system made by shear walls. Configuration/arrangement of steel profiles in cross section were varied within the specimens. The main objective of this research consisted in identifying innovative solutions for composite steel-concrete shear walls with enhanced performance, as steel fibre reinforced concrete which was used in order to replace traditional reinforced concrete. A first conclusion was that replacing traditional reinforcement with steel fibre changes the failure mode of the elements, as from a flexural mode, in case of element RCW, to a shear failure mode for CSFRCW. The maximum lateral force had almost similar values but test results indicated an improvement in cracking response, and a decrease in ductility. The addition of steel fibres in the concrete mixture can lead to an increase of the initial cracking force, and can change the sudden opening of a crack in a more stable process.

1. Introduction

Reinforcing concrete with steel bars is a very common practice in construction but in recent years an increased interest in development of concrete technology lead to one of the major progresses which is Fibre Reinforced Concrete (FRC). Many studies show that this composite material has many advantages over the traditional reinforced concrete (RC). Among all kinds of fibres which can be used as concrete reinforcement, Steel Fibres are the most popular ones. Experimental investigations have been made on small scale elements such as beams, [1, 2], beam-column joints, [3, 4], or on steel fibre reinforced concrete (SFRC) samples to determine how these fibres can influence the mechanical properties of the composite material [5]. These investigations imply that the use of FRC may be an alternative solution to increase the shear capacity and damage tolerance capacity.

Steel fibres can also be used in ultra-high performance concrete, [6, 7], and self-compacting concrete mixture, [8, 9], to improve the mechanical characteristics of materials and the bearing capacity of the structural elements. Although most studies were developed by analysing small scale elements, there are also studies regarding the performances of wall elements, [10, 11], subjected to seismic actions, as the
need to improve the bearing capacity of the structural elements and to reduce the amount of work in the execution of high-rise buildings.

The main objective of this research project is to investigate the possibility to replace totally or partially the traditional rebar placed in the field of composite walls with steel fibres and evaluate the main parameters of the behaviour: stiffness, strength, ductility and make comparisons with common RC walls and Composite Shear walls with steel encased profiles already tested in another experimental program, [12, 13].

2. Experimental program

An experimental program was developed at the Politehnica University of Timisoara, Romania, with the purpose of studying the behavior of Composite Steel Fibre Reinforced Concrete shear Walls (CSFRCW) subjected to combined axial and quasi-static lateral cyclic loads.

2.1. Experimental specimens

Through this program quasi-static cyclic tests were carried out on six 1:3 scale elements. Five specimens are fibre reinforced concrete shear walls with encased steel profiles, while one is the control reinforced concrete wall. The variables of the experimental program are related to the steel encased elements shape, position and encasement level, and the fibre contribution ratio. They were designed using the principles from the existing codes: Eurocode 2, [14], Eurocode 4, [15] and Eurocode 8, [16] applied to composite steel-concrete elements. The steel fibre contribution ratio is determined according to the Technical Regulation from 21/04/2003, [17], the maximum dosage of steel fibre is 55 kg/m³. In this paper are presented the results obtained for the fourth and fifth Composite Steel Fibre Reinforced Concrete shear Wall specimen (CSFRCW-4 and CSFRCW-5) and for the reference Reinforced Concrete wall (RCW-6). The design details of the two types of CSFRCW are presented in figure 1 and for the RCW wall in figure 2.

![Figure 1. Details of experimental CSFRCW-4, 5 [mm]](image-url)
The experimental specimens have 3000 mm height, 1000 mm width and 100 mm in thickness, and represent a three storeys and one bay element from the base of a lateral resisting system made by shear walls. The wall panels were embedded in a heavily reinforced concrete foundation with 1500 mm length, 400 mm height and 350 mm width. The structural steel profiles were connected with the concrete web by headed shear stud connectors with d=13 mm diameter and h=75 mm length placed every 150 mm. The reinforcements from the web panel consists of Ø8/10 mm vertical bars and Ø8/75 mm horizontal stirrups. Both the steel profiles and the reinforcements were embedded into the reinforced concrete foundation block to ensure anchorage.

2.2. Material properties

To determine the compressive and tensile strength of the concrete, 6 samples concrete cubes and 3 samples of prismatic elements were provided for either steel fibre concrete or simple concrete. After testing, the concrete samples we obtained a strength class (C30/37) higher than the concrete design strength class (C20/25) used in the design process. The results of the tests obtained at the age of 28 days are presented in table 1, figure 3. The normalization of the concrete strength is done to the strength of element RCW-6. Compressive strengths of the resulting concrete are slightly different from one element to another. In the case of tensile strength, the addition of steel fibres in the concrete mixture lead to an increase in strength with 50%. For disperse reinforcement were used 60 x 0.8 mm hooked steel fibres with a tensile strength min. 1100 N/mm² and modulus of elasticity of 210x10³ N/mm². Structural reinforcements design quality is Bst500S. The structural steel profiles (S355 JR OL52-2k) were manufactured by welding the steel plates. The mixing formula of the concrete used to fabricate the specimens is the following: Cement 320 kg; Water: 170 kg/m³; Sand: 725 kg; Aggregate: 1087 kg; Filler: 70 kg; Additive: 3.5 l.

| Specimen label | Number of samples (compression) | \( f_{cm} \) [N/mm²] | Number of samples (tension) | \( f_{ctm} \) [N/mm²] |
|----------------|---------------------------------|----------------------|-----------------------------|------------------------|
| CSFRCW-4       | 6                               | 43.06                | 3                           | 4.9                    |
| CSFRCW-5       | 6                               | 40.44                | 3                           | 5.1                    |
| RCW-6          | 6                               | 47.29                | 3                           | 3.38                   |

Table 1. Material properties of concrete
2.3. Test set-up and loading procedure

The experimental test was performed in the laboratory of Civil Engineering Department, at “Politehnica” University of Timisoara. The specimens were tested under constant vertical load and quasi-static reversed cyclic lateral loads. The lateral loads were applied alternatively from left and right. The test specimen was placed in the same plane as the loading frame, figure 4, and was anchored with steel bolts into the laboratory reaction floor.

The loading frame consist of two steel braced frames, placed symmetrically. Initially, a constant vertical load of 100kN was applied to the specimen and was maintained constant during the test. The horizontal forces were applied at 400 mm below the top of the elements, providing thus sufficient anchorage length above the load application level for the reinforcing bars and steel profiles. A transverse brace system was used in order to avoid any out of plane displacements of the specimens.

The recommended ECCS short testing procedure, [18], was used for the cyclic tests. The tests were performed in displacement control model. Minimum four cycles were performed before the elastic limit of the element was reached. After the elastic limit $\Delta y \geq 20$mm, three cycles were performed at each displacement level. The horizontal forces were applied under controlled cyclic displacements, until the strength of the specimens decreased to 85 % of the peak horizontal load.

In order to monitor the behaviour of the experimental specimen, pressure transducers, displacements transducers (D) and strain gauges (G) glued on the reinforcements bars and on the steel profiles, were used.

3. Experimental results and comparative study

3.1. General behaviour and failure modes of CSFRCW

The behaviour mode of a reinforced concrete wall subjected to in-plane lateral loading can be referred to as either flexural or shear. The most common situation is regarded as flexural-shear failure represented by the appearance of first horizontal cracks at the edges and then slopping toward the compressed toe of the web panel, a behaviour better described by element RCW-6, figure 5. Contrary to this failure mode, in the present experimental study, wall specimens CSFRCW-4 and CSFRCW-5 presented a shear mode failure with the damage of the concrete and steel profile yielding. The behaviour aspects of the elements during the test are presented in table 2 at every displacement level. Before the elastic limit is reached only diffused cracks are formed, after this loading stage existing cracks develop towards failure. Also for a better representation of the relation force (P) – displacement (Δ) at each loading cycle a hysteretic curve was generated for each experimental specimen, figure 6.
Figure 4. General view of the test set-up

Figure 5. Representation of crack distribution at every displacement level for RCW-6

Table 2. Response characteristics of the specimens

| Displacement stage Δ (mm) | CSFRCW-4 | CSFRCW-5 | RCW-6 |
|---------------------------|----------|----------|-------|
| Δ < 5mm                   | No visible cracks appeared | No visible cracks appeared | Diffuse horizontal and inclined cracks appeared |
| 5 < Δ < 10mm              | No visible cracks appeared | Diffuse inclined cracks appeared | Inclined cracks developed along element and new cracks appear |
| 10 < Δ < 15mm             | Diffuse inclined cracks appeared | Inclined cracks developed along element and new cracks appear | Failure: Cracks developed towards failure. Crushing of compressed concrete in one direction. |
| 1 ≤ Δ < 20mm              | Failure: Cracks developed towards failure. Crushing of compressed concrete in one direction. | Cracks developed towards failure | Failure: Cracks developed towards failure. Crushing of compressed concrete in one direction |
| Δ > 20mm                  | Failure | Failure | Failure |
| Δ > 40mm                  | Failure | Failure | Failure |
3.2. Cracking analysis

During the experimental tests, crack development at the surface of the concrete was monitored. The behaviour of tested elements is presented in figure 7 and figure 5 for each displacement level followed by the cracking distribution during the test. Almost all cracks started developing at a distance of 15 cm form the edge of the wall, corresponding to the internal face of the steel profile, as the more rigid areas, the edges of the concrete web panel reinforced with steel profile, vertical bars and stirrups, separate from the core of the wall.

Compared with RCW-6, in which case first horizontal cracks appeared at the edges at a drift of 10mm, in the situation of the other two elements failure starts with diffused inclined cracks, which appeared in the tensioned zone. These cracks are caused by the stresses transfer from the steel profile to the concrete. For element CSFRCW-4 the first crack appeared approximately at 15 cm from the bottom line during the cycle 0.75*Δy, corresponding to a 15mm drift. In the case of wall CSFRCW-5 the first crack appeared at 40 cm from the bottom line at a drift of 15 mm, during the cycle 0.75*Δy. During the next loading step, reaching Δy, new horizontal cracks and the extension of the existing ones were observed. As loading cycles continued, diagonal cracks developed towards failure, in cycle 2*Δy.

The collapse of the element occurred with the development of the existing inclined cracks, vertical reinforcements and steel profile yielding and crushing of concrete in compressed zone for element CSFRW-4. For specimen CSFRCW-5, with partially steel encased profile, yielding occurred first in steel profile at a drift value of 22.25mm and only in the next cycle in vertical reinforcement placed near the steel profile at the extremity of the element at a drift of 24.14mm. The maximum strain values recorded in structural steel and vertical bars is 4.43‰ and 4.23‰ respectively. In the case of CSFRW-
4, with totally encased steel profile, yielding occurred first in vertical reinforcements at a drift value of 33.7mm before in was recorded in structural steel at a drift value of 35.72mm. The maximum strain values recorded in structural steel and vertical bars is 3.99‰ and 4.11‰ respectively. In none of both cases yield strain was not reached in stirrups.

**Figure 7.** Representation of crack distribution at every displacement level for CSFRCW-4, 5

### 4. Discussion

The main objective of this research project is to investigate the possibility to replace totally or partially the traditional rebar, placed in the field of composite walls, with steel fibres and evaluate the main parameters of the behaviour: stiffness, strength, ductility. The results obtained from this program are compared with the results from another research program developed previously at the Politehnica University Timisoara, [12, 13]. The respective program studied six structural Composite Steel Reinforcement Concrete Walls (CSRCW), without steel fibres. In order to provide an accurate comparison, the geometrical dimensions and the cross section are similar in both studies. Taking into account the same manner of testing, this section of the current paper proposes a comparative study for two tested elements from each of the two research programs (CSFRCW-4 and CSFRCW-5 from the current program and CSRCW-4 and CSRCW-5 from the previous one, as mentioned above).

A first comparison can be made regarding the failure mode of the elements by analysing the crack pattern, figure 8. For the two specimens from previous study, we can observe first cracks starting horizontally from the edge and then sloping toward the compressed zone in the web. This is a common situation and it can be regarded as flexural-shear behaviour. In the current study, both CSFRCW-4 and CSFRCW-5, showed a shear mode failure as diagonal cracks were predominant on the surface of the web panel which, after the elastic limit, developed rapidly towards failure.

The damage process can be expressed in terms of forces and displacements, as there can be identified four main stages, for all tested specimens, denoted as follows: initial cracking, element yielding, limit stage and failure stage. In table 3 the correspondence between the forces and the displacements at each characteristic point from the tests is presented.

It can be observed that in comparison with elements CSRCW-4, 5, until the elastic limit is reached, for elements CSFRCW-4, 5 initial cracking occurs at a force corresponding to a higher displacement level. The addition of steel fibres in the concrete mix lead to an increase of the initial cracking force and
the sudden opening of cracks changed in a more stable process. After the elastic limit stage, existing cracks develop leading to a brittle failure of the steel fiber reinforced concrete. Although the values of the maximum lateral force are with only 30% smaller, as failure stage was reached at a smaller displacement level, CSFRCW-4, 5 present a low displacement ductility, 2-3 times smaller, figure 9. The differences between the behavior characteristics of the specimens are clearly visible by generating an envelope curve that describes the major hysteretic characteristics under cyclic loading, figure 10.

Table 3. Forces and displacements at characteristics stages

| Specimen    | Initial cracking | Element yielding | Limit stage | Failure stage |
|-------------|------------------|------------------|------------|--------------|
|             | $P_{cr}$ [kN]    | $\Delta_{cr}$ [mm] | $P_{y}$ [kN] | $\Delta_{y}$ [mm] | $P_{max}$ [kN] | $\Delta_{max}$ [mm] | $P_{85\%}$ [kN] | $\Delta_{85\%}$ [mm] |
| CSRCW-4     | 94.6             | 7.56             | 238.6      | 26.4          | 324.8          | 117.8            | 275.4          | 137.2          |
| CSFRCW-4    | 136.6            | 15               | 195.8      | 21.84         | 244            | 39.96            | 174.6          | 40.95          |
| CSRCW-5     | 84.0             | 5.00             | 258.3      | 26.3          | 357.3          | 115.1            | 303.7          | 135.2          |
| CSFRCW-5    | 145.2            | 15               | 180.4      | 20.06         | 245.6          | 34.24            | 210.8          | 33.47          |

Figure 8. Comparative representation of crack pattern at failure stage

Figure 9. Left: Load bearing capacity of the specimens; Right: Displacement ductility
5. Conclusions
The work presented in this paper regards an experimental study which provides results from a comprehensive experimental investigation on the behaviour of two composite steel fibre reinforced concrete shear walls (CSFRCW) with partially or totally steel encased profiles. Based on the performed study, the following conclusions can be formulated related to the seismic behaviour of CSFRCW-4, 5.

- The addition of steel fibres in the concrete mixture can lead to an increase of the initial cracking force and changing the sudden opening of cracks in a more stable process.
- Because of totally replacing the traditional reinforcement, from the core of the web panel, with steel fibres, considering the same value of steel ratio, the resulted failure mode is a shear-based one with diagonal cracks, yielding of vertical reinforcements and structural steel profile, and crushing of concrete in the compressed zone.
- Having the steel profile, from the edges of the web panel, only partially embedded in the concrete wall, influence the yield strain measurement. Compared with CSFRCW-4, with totally encased steel profile, in element CSFRCW-5 yield strain was recorded first in structural steel, before vertical reinforcements.
- Although the obtained results in terms of load capacity are slightly smaller, failure occurred at a smaller displacement level denoting a brittle failure with a significantly smaller displacement ductility, in comparison with the composite steel reinforced concrete walls tested previously, CSRCW-4, 5.
- Although for both CSFRCW-4 and CSFRCW-5 first cracks appeared at the same loading stage, when failure stage was reached, the first one recorded a slightly smaller lateral force but a higher value of displacement.

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References
[1] Halit Cenan Mertol, Eray Baran, Hussain Jibril Bello, “Flexural behaviour of lightly and heavily reinforced steel fibre concrete beams”, Construction and Building Materials, vol. 98, pp. 185-193, 2015.
[2] Fasheng Zhang, Yining Ding, Jing Xu, Yulin Zhang, Weiqing Zhu, Yunxing Shi, “Shear strength prediction for steel fibre reinforced concrete beams without stirrups”, Engineering Structures, vol. 127, pp. 101-116, 2016.
[3] R. Siva Chidambaram, Pankaj Agarwal, “Seismic behaviour of hybrid fibre reinforced concrete beams with steel fibre”, Construction and Building Materials, vol. 117, pp. 66-77, 2016.
cementitious composite beam-column joints, *Materials and Design*, vol. 86, pp. 771-781, 2015  
[4] Constanze Roehm, Saptarshi Sasmal, Balthasar Novák, Ramanjaneyulu Karusala, “Numerical simulation for seismic performance evaluation of fibre reinforced concrete beam-column sub-assemblages”, *Engineering Structures*, vol. 91, pp. 182-196, 2015.  
[5] Facundo Isla, Gonzalo Ruano, Bibiana Luccioni, “Analysis of steel fibres pull-out. Experimental study”, *Construction and Building Materials*, vol. 100, pp. 183-193, 2015.  
[6] Luigi Biolzi, Sara Cattaneo, “Response of steel fibre reinforced high strength concrete beams: Experimental and code predictions”, *Cement and Concrete Composites*, vol. 77, pp. 1-13, 2017  
[7] Doo-Yeol Yoo, Young-Soo Yoon, “Structural performance of ultra-high-performance concrete beams with different steel fibres”, *Engineering Structures*, vol. 102, pp. 409-423, 2015.  
[8] Luigi Biolzi, Sara Cattaneo, Franco Mola, “Bending-shear response of self-consolidating and high-performance reinforced concrete beams”, *Engineering Structures*, vol. 59, pp. 399-410, 2014.  
[9] Alireza Khaloo, Elias Molaei Raisi, Payam Hosseini, Hamidreza Tahsiri, “Mechanical performance of self-compacting concrete reinforced with steel fibres”, *Construction and Building Materials*, vol. 51, pp. 179-186, 2014.  
[10] Carrillo, J., Alcocer, S. M. & Pincheira, J., “Shaking Table Tests of steel Fibre Reinforced concrete Walls for Housing”, 15th World Conference on Earthquake Engineering, Lisbon, 2012.  
[11] Joshua S. Pugh, Laura N. Lowes, Dawn E. Lehman, “Nonlinear line-element modeling of flexural reinforced concrete walls”, *Engineering Structures*, vol. 104, pp. 174-192, 2015.  
[12] Dan, D., Fabian, A. & Stoian, V., “Theoretical and experimental study on composite steel–concrete shear walls with vertical steel encased profiles”, *Journal of Constructional Steel Research*, vol. 67, pp. 800-813, 2011.  
[13] Dan, D., Fabian, A. & Stoian, V., “Nonlinear behaviour of composite shear walls with vertical steel encased profiles”, *Engineering Structures*, vol. 33, pp. 2794-2804, 2011.  
[14] EN 1992-1-1. Eurocode 2: Design of concrete structures, part 1-1, general rules and rules for buildings.  
[15] EN 1994-1-1. Eurocode 4: Design of composite steel and concrete structures, part 1-1, general rules and rules for buildings.  
[16] EN 1998-1. Eurocode 8: Design of structures for earthquake resistance.  
[17] Technical Regulation “Guide to establishing the criteria for performances and composition for steel fibre reinforced concrete (in Romanian), Official Monitor, 1, No.575bis, 12/08/2003.  
[18] ECCS. Recommended testing procedure for assessing the behaviour of structural steel elements under cyclic loads, European Convention for Constructional Steelwork, 1999.