Assessment of the Performance of Steel Fibre Reinforced Self-Compacting Concrete in Elevated Slabs

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Abstract

The reinforcement mechanisms at the cross section level assured by fibres bridging the cracks in steel fibre reinforced self-compacting concrete (SFRSCC) can be significantly amplified at structural level when the SFRSCC is applied in structures with high support redundancy, such is the case of elevated slab systems. To evaluate the potentialities of SFRSCC as the fundamental material of elevated slab systems, a ¼ scale SFRSCC prototype of a residential building was designed, built and tested. The extensive experimental program includes material tests for characterizing the relevant properties of SFRSCC, as well as structural tests for assessing the performance of the prototype at serviceability and ultimate limit conditions. Three distinct approaches where adopted to derive the constitutive laws of the SFRSCC in tension that were used in finite element material nonlinear analysis to evaluate the reliability of these approaches in the prediction of the load carrying capacity of the prototype.

*Keywords***:** Steel fibre reinforced self-compacting concrete (SFRSCC), Elevated SFRSCC slab, Post-cracking constitutive laws, Finite element analysis.

1. Introduction

The continuous improvements of the technology of steel fibre reinforced concrete (SFRC), and the better knowledge on the characterization of the mechanical behaviour of this composite are contributing to enlarge the field applications of SFRC. When using relatively high content of fibres (1 to 1.5% in volume) of large aspect ratio (65 to 80), SFRC of relatively high post-crack residual tensile strength can be developed, capable of constituting the fundamental material of structural elements with load carrying capacity much higher than its cracking load [1].

By taking advantages of the statically indeterminate character of slabs supported on piles or columns, and considering the benefits from the high post-cracking residual strength of SFRC, the use of this composite has recently been explored for the construction of this type of structural system [2, 3], which is generally designated by elevated steel fibre reinforced concrete slab (E-SFRC). This type of slab includes anti-progressive collapse reinforcement formed by steel bars placed in the bottom of the slab in both alignments of the columns [4].

Due to lack of conventional smeared bar reinforcement, the load carrying capacity of E-SFRC slabs is directly dependent on the post-cracking behaviour of the SFRC. Therefore, to evaluate accurately the behaviour of the E-SFRC slab, it is fundamental to derive the constitutive law of the SFRC that best characterize the post-cracking response of this composite. The stress versus crack width relationship $(\sigma_t - w)$ is the most appropriate constitutive law to simulate the post-cracking behaviour of strain softening materials, like the SFRC commonly used in E-SFRC slabs [5]. The $\sigma_t - w$ response of SFRC is conventionally derived from direct tensile tests [6], or is indirectly determined from three-point notched beam bending tests [7] by executing an inverse analysis with the force-deflection results obtained in these tests [8]. Nevertheless, the direct application of the post-cracking response of the SFRC determined from the abovementioned material tests for the design of E-SFRC slabs still deserves further research. In fact, the post-cracking response of the SFRC is noticeably influenced by the number and orientation of fibres crossing the cracks [9], which are, in turn, affected by the shape and geometry of the element, casting process and rheological properties of this composite [10]. Moreover, cracking nature of slabs is different from tensile and bending tests conducted on small prismatic specimens, since the distribution and orientation of fibres might be not representative of slab applications. Therefore the internal stress redistribution occurred in slab elements is not correctly captured by executing tests with this specimens.

Since steel fibres are heaviest than the other concrete constituents, a tendency for an increase of steel fibre

percentage from the top to the bottom surface of the slab is currently reported, mainly when vibration conditions are applied during the cast operations [11]. In an attempt of avoiding this situation, and to decrease the period of time of the construction process, a steel fibre reinforced self-compacting concrete (SFRSCC) was developed and used to build a ¼ scale prototype of residential building. Apart the suppression of the vibration in the technology of SFRSCC, with the consequent economic and environmental benefits, the distribution of fibres can be more homogeneous along the slab's cross section [12, 13], resulting E-SFRSCC slabs of larger load carrying capacity. In the present research three approaches are adopted for deriving the $\sigma_t - w$ of the SFRSCC including direct tensile tests (DTT), inverse analysis (IA) using the force versus deflection responses determined in three-point notched beam bending tests, and by using the recommendations of the *fib* Model Code 2010 [7] (hereafter abbreviated by MC-2010). The predictive performance of these three approaches is assessed by the numerical simulation of the tested prototype. The tests executed in the prototype were also used to appraise the performance of this material and structural system for serviceability and ultimate limit conditions.

2. Experimental program

2.1 Geometry and construction of the prototype

In Fig. 1 is represented the built prototype, which is a ¼ scale construction representative of a residential building. It is composed of a SFRSCC slab supported on soil and a E-SFRSCC slab supported on columns, both slabs of 3700×2100 mm dimensions in plant and without any type of conventional reinforcement. The E-SFRSCC slab has a constant thickness of 75 mm, formed by the six continuous panels of 1200×1000 mm dimensions in plant. This slab is supported on twelve SFRSCC columns of square cross section of 100 mm edge, with 1000 mm height, spaced at 1200 mm and 1000 mm in *x* and *y* direction, respectively. Each column is reinforced longitudinally with four steel bars of 6 mm diameter, positioned in the corners of the section by means of two steel hoops of 6 mm diameter located at the extremities of the column in order to assure a concrete cover thickness of 10 mm for the longitudinal bars. The geometry of the SFRSCC slab supported on soil was optimized in preliminary analysis to avoid the formation and localization of premature macro-cracks in the vicinity of the columns. For this purpose, this slab is formed by an orthogonal grid of 100 mm thickness in both alignments of the columns, a zone of 200 mm width where the thickness has a smooth linear variation from the grid to the internal panels of 30 mm thickness (see Fig. 1).

2.2 Composition and rheological properties of SFRSCC

The method adopted in the present work for preparing SFRSCC is based on the three following steps [14]: i) the proportions of the constituent materials of the paste are defined; ii) the proportions of each aggregate on the final solid skeleton are determined; iii) the paste and solid skeleton are mixed in different proportions until self-compacting requirements are assured in terms of spread ability, correct flow velocity, filling ability, blockage and segregation resistance. Cement CEM I 42.5R, fly ash, limestone filler, superplasticizer SikaViscocrete 3005, water, three types of aggregates (fine and coarse river sand of maximum dimension of 0.6 mm and 4.8 mm, respectively, and crushed granite 5-13 mm), and hooked ends steel fibres were the used materials. The fibres had a length (l_f) of 35 mm, a diameter (d_f) of 0.5 mm, an aspect ratio (l_f/d_f) of 70 mm and, according to the supplier, a yield stress of about 1300 MPa. The SFRSCC composition is indicated in Table 1, and the self-compacting requisites of this composite were evaluated by executing the slump cone and the L-Box test according to the EFNARC recommendations [15]. The total spread, s, measured with the slump cone was 740 mm, and the H_2/H_1 (blocking ratio) parameter of the L-Box test was 0.8.

To build the entire prototype, three batches of the same SFRSCC composition were prepared. For each batch, three cylinder specimens of 150 mm diameter and 300 mm height, and two beams of $600\times150\times150$ mm³ were cast for the characterization of the compressive and flexural behaviour of the developed SFRSCC, respectively.

2.3 Compressive behaviour of SFRSCC

By performing compression tests with the cylinder specimens according to EN 206-1 recommendations [16], an average compressive strength (f_{cm}) of 63.63 MPa was obtained at 28 days. By applying the recommendations of the MC-2010 [7], this f_{cm} was used to determine the modulus of elasticity $(E_c = 39.84 \text{ GPa})$ and the average tensile strength $(f_{ctm} = 4.23 \text{ MPa})$ of the SFRSCC.

2.4 Characterization of the post-cracking behaviour of SFRSCC

2.4.1 MC-2010 recommendations

The post-cracking behaviour of the SFRSCC was assessed by performing six notched beam bending tests according to the recommendations of the MC-2010 [7] (Fig. 2).

Due to relatively high content of steel fibres used in the SFRSCC $\left(V_f = 1.1\%\right)$, and in attempt to restrict, as much as possible, the crack localization to the ligament over the notch tip, a depth of 60 mm was adopted for the notch. The envelope curves corresponding to the relationship between the applied force $\left(F\right)$ and the crack mouth opening displacement (*CMOD*) are depicted in Fig. 3.

From the *F-CMOD* average curve, the average flexural residual strength parameters ($f_{Ri,m}$) corresponding to $CMOD_i = 0.5$, 1.5, 2.5, and 3.5 mm were determined by applying the following equation [7]:

$$f_{Ri,m} = \frac{3F_{i,m}l}{2bh_{sp}^2}$$ (1)

where $F_{i,m}$ is the average force corresponding to $CMOD_i$, $l = 500$ mm is the span length of the specimen, and $b = 150$ mm and $h_{sp} = 90$ mm are, respectively, the width and the height of ligament over the notch. In Table 2 is included the f_{Rim}, as well as the corresponding characteristic values ($f_{Ri,k}$) determined by applying the following equation [17]:

$$f_{Ri,k} = f_{Ri,m} - k\,s_d$$ (2)

where k is a dimensionless parameter considered 2.18 for the six specimens tested, and s_d is the corresponding standard deviation, also indicated in Table 2. Since relatively high dispersion of results was obtained, the characteristic values are considerably lower than the corresponding average values. For design and quality control of the SFRSCC applied in structural applications, series of tests composed of at least nine specimens is recommended [18].

In Fig. 4 is schematized the tensile behaviour recommended by MC-2010 [7] for FRC. In this diagram $f_{Fts,k}$ and $f_{Ftu,k}$ are determined by applying the following equations [7]:

$$f_{Fts,k} = 0.45 f_{R1,k}$$ (3)

$$f_{Ftu,k} = f_{Fts,k} - \frac{w_{max}}{CMOD_3}(f_{Fts,k} - 0.5f_{R3,k} + 0.2f_{R1,k}) \geq 0 \tag{4}$$

where w_{max} =2.5 mm is the considered maximum crack opening in structural design. In the diagram of Fig. 4, $\varepsilon_{cr} \left(= f_{ct,k} / E_c\right)$ is the strain corresponding to the tensile strength $\left(f_{ct,k}\right)$, and w_s and w_u are the crack width corresponding to $f_{Fts,k}$ and $f_{Ftu,k}$, respectively:

$$w_s = CMOD_1 \tag{5}$$

$$w_u = \min\left(0.02 l_{cs}, 2.5\,\text{mm}\right) \tag{6}$$

where l_{cs} is the structural characteristic length (in mm), which is considered equal to the thickness of the E-SFRSCC slab's cross section (=75 mm), in accordance with the recommendations of the MC-2010 for FRC members without conventional flexural reinforcement [7].

Fig. 5 represents the tensile response of the SFRSCC applied in the prototype, following the recommendations of the MC-2010 and considering the results included in Table 2.

2.4.2 Direct tensile tests

From the half parts of each notched beam tested in bending, four prismatic specimens of 250×150×75 mm³ were extracted according to the scheme represented in Fig. 6a, in order to execute direct tensile tests (DTT) for evaluating the tensile behaviour of the developed SFRSCC. The adopted extraction procedure has assured that DTT specimens were not affected by any type of damage occurred during the notched beam bending test of the "*mother*" element. To promote crack localization at middle length of the DTT specimens, two lateral notches of 5 mm width and 25 mm depth were executed in the two shortest lateral sides at this plane, leading to a net area of 100×75 mm² (Fig. 6a), which is considered the fracture surface for the evaluation of the tensile stress.

The DTT specimens were directly glued on extra loading platens of the testing rig by using an apparatus developed for this purpose, and adopting high strength epoxy resin. The tensile tests were executed in a fatigue servo-hydraulic equipment of 1000 kN capacity, equipped for this experimental program with a load cell of 50 kN capacity. The DTT was carried out on ten samples, under displacement control at a displacement rate of 5 μm/s by considering the average signal of four linear variable displacement transducers (LVDT) with a gauge length of 30 mm, mounted on the edges of the specimen (Fig. 6b).

The tensile stress was obtained by dividing the applied tensile force by the net area of the notched plane (100×75 mm^2). The envelope and the average curve of the stress versus crack width relationships $(\sigma_t - w)$ obtained in the DTT tests are represented in Fig. 7a, where the crack width is the average of the values measured by the four LVDTs installed in each specimen. The average $\sigma_t - w$ curve is approximated by a quadrilinear diagram in Fig. 7b by adopting equal fracture energy (area behind the $\sigma_t - w$ curve) for the experimental curve and approximated diagram.

2.4.3 Inverse analysis

In the third approach the stress-crack width relationship $(\sigma_t - w)$ of the SFRSCC was obtained from inverse analysis (IA). The IA consists of evaluating the values of the $(\sigma_t - w)_i$ points that define the $\sigma_t - w$ diagram, by minimizing the deviation between the experimental and the numerical force – midspan deflection $(F - \delta)$ curves corresponding to the three-point notched SFRSCC beam bending tests described in section 2.4.1. The error err_δ is calculated as:

$$err_\delta = \left| A_{F-\delta}^{\mathrm{exp}} - A_{F-\delta}^{num} \right| \Big/ A_{F-\delta}^{\mathrm{exp}} \tag{7}$$

where $A_{F-\delta}^{\mathrm{exp}}$ and $A_{F-\delta}^{num}$ are the areas below the experimental and the numerical $F - \delta$ curves, respectively, up to a certain deflection δ. The analysis was performed with FEMIX V4.0 computational software [19], which is based on the finite element method (FEM). The notched beam used in the three-point bending test was simulated by the finite element mesh represented in Fig. 8, composed of plane stress quadrilateral elements of four nodes, with Gauss-Legendre (G-L) integration scheme of 2×2 integration points (IP), while the opening of the crack over the notch was simulated by interface elements of six nodes with Gauss-Lobatto integration scheme of 1×3 IP. A detailed description of the IA can be found elsewhere [20].

The IA was carried out by using the average force-deflection curve of the six three-point notched SFRSCC beam bending tests (three series of two specimens). This curve was obtained by assuming a *t*-student distribution:

$$F_m = F_{m,s} - t_{10}\, s_d \big/ \sqrt{n} \tag{8}$$

where $F_{m,s}$ is the mean force registered in the experimental tests for a certain deflection, s_d is the standard deviation, and t_{10} is the "student" distribution equal to 1.48 for the six tested samples $(n=6)$ and 80% confidence level. In Fig. 9a the force-midspan deflection response of the notched beams resulted from IA is compared with the experimental one, while the corresponding $\sigma_t - w$ diagram is depicted in Fig. 9b.

2.5 Prototype test

2.5.1 Test setup

A two-phase test program was conducted on the prototype. In the first phase, by using load units composed of 21 cement bags with 1.0×1.2 m² bottom area (= 6 kN/m²) distinct levels of uniform distributed load were applied to some panels of the E-SFRSCC slab (Fig. 10a) to evaluate the deflection response of the loaded panels in the serviceability limit states. A typical load versus deflection relationship of the prototype is represented in Fig. 10b, where the central deflection of panels 1 and 3 has increased linearly up to a load of 36 kN/m² (positive values mean downward deflections). The applied uniform distributed load levels were maintained on the prototype until the panels' central deflection stabilized, in order to also investigate the long term response of the panels under the prescribed load levels. In Fig. 10c is represented the evolution of the central deflection of panel 1 and 3 under an applied load of 36 kN/m² up to about 500 hours. At the end of this loading phase no cracks were visible in the prototype. For the maximum prescribed distributed load of 39 kN/m² applied on panel 1 no signs of cracks were visible in the E-SFRSCC slab. This load level is more than 6 times higher the one considered to design the main structural components of a building for sport events of high occupancy [21].

In the second phase of the test program, the distributed load was replaced by a concentrated load, where a servo-hydraulic actuator of 200 kN capacity has applied a load distributed in a relatively small area (200×200 mm²) in the centre of panel 2 (see Fig. 1) by using a steel plate of 40 mm thickness. The test setup is represented in Fig. 11.

To apply the load to the slab, the actuator was installed in a reaction frame of steel profiles supported on the prototype by four vertical elements, forming a system in equilibrium. Four high strength steel bars of 15 mm diameter were used to connect the vertical elements of the reaction frame to the columns of the prototype adjacent to the tested panel. Each steel bar was introduced in a hole of 20 mm diameter and 250 mm depth, executed in the centre of the RC columns, and was bonded to the surrounding concrete by using high strength

epoxy resin (Fig. 11b). To avoid premature failure in this connection, the confinement system represented in Fig. 12, formed by steel profiles, was applied to the top extremity of the columns supporting the reaction frame. Moreover, to measure the vertical deflection of the slab, seven LVDTs were installed in a supporting system mounted under the panel to assure that only the deflection of the slab would be recorded by the LVDTs. The arrangement of the LVDTs is illustrated in Fig. 13.

2.5.2 Loading configuration

The quasi-point load applied to the panel was displacement controlled by using the LVDT internal to the actuator, and imposing a displacement rate of 10 μm/s. The loading scheme is composed of the four steps indicated in Fig. 14. The first three steps were composed of loading/unloading ramps up to a load level of 10, 40, and 70 kN, respectively. The last step (fourth) was formed by a loading ramp (also in displacement control) up to the attainment of a relatively high deflection in the post-peak branch of the force-central deflection response of the tested prototype (structural softening).

2.5.3 Results and discussion

In the first and second steps of loading no cracks were visible in the top and bottom surfaces of the E-SFRSCC slab. First cracks were detected in the third loading step in the bottom (centre of the panel) and top (contour of the panel) surfaces of the E-SFRSCC slab at a load level of 43 and 64 kN, respectively. The crack patterns at the maximum load level of the third loading step ($F_{\max,3}$ =70.52 kN) in the top and bottom surfaces of the E-SFRSCC loaded panel are schematically represented in Fig. 15.

The crack patterns at the maximum load ($F_{\max,4}$ =79.92 kN) and at the end of the test (δ_{pu} =26.63 mm) are illustrated in Figs. 16 and 17, respectively. As expected [22], up to the peak load an almost circular crack band with a radius approximately equal to the distance from panel centre to the adjacent RC columns was propagated in the top surface, while in the bottom surface radial cracks have irradiated from the loaded area towards the perimeter of the loaded panel.

When the test was interrupted, which occurred at a deflection of about 27 mm in the centre of the panel, cracks in the contour and in the central part of the panel were visible in the top surface. In the bottom surface four macro-cracks became well visible, irradiating from the loaded area to the central span of the edges of the panel, but with a certain inclination in relation to the main directions of the alignments of the slab's columns. The

cracks number 1 and 2 (Fig. 17) seem to have crossed totally the slab's cross section. The widening of these flexural failure cracks was followed by the closing of the remaining cracks. The force versus central deflection relationship of panel 2 is depicted in Fig. 18, while the most relevant results are indicated in Table 3, where $F_{\max,i}$ is the maximum applied load in the ith loading step, $\delta_{cp}^{F_{\max,i}}$ is the corresponding deflection at the centre of the panel, $\delta_{cp}^{F_i=0}$ is the deflection at complete unloading of the ith loading step, and $k_{\sec,i}/k_{\sec,1}$ is the ratio between the secant stiffness at the ith loading step and the initial stiffness:

$$k_{\sec,i}/k_{\sec,1} = \left(F_{\max,i}/\delta_{cp}^{F_{\max,i}}\right)\bigg/\left(F_{\max,1}/\delta_{cp}^{F_{\max,1}}\right) \tag{9}$$

The relatively large number of cracks formed during the loading process justifies the ductile response of this structural system, where after the peak load, the load has decreased smoothly, demonstrating a very high post-cracking load carrying capacity in the structural softening phase up to a deflection that was almost $L/45$, where L is the maximum span length of the slab (=1200 mm). At the deflection when the test was interrupted (about 27 mm, which is 3.6 times the deflection at peak load), the recorded load was 48.28 kN, which is 60% of the peak load.

3. FEM-based simulations of the prototype test

The stress-crack width diagrams simulating the post-cracking behaviour of the SFRSCC obtained by the approaches indicated in section 2.4 are represented in Fig. 19. In material nonlinear analysis with smeared crack constitutive models implemented under the framework of FEM, the stress-crack width was converted in a stress-strain diagram that simulates the fracture initiation and propagation, in mode I, of the smeared cracks formed in a certain integration point of the finite element by using the concept of crack band width $\left(l_b\right)$:

$$\varepsilon_{nn}^{cr} = w/l_b \tag{10}$$

where ε_{nn}^{cr} is the strain normal to the smeared cracks. In the present analysis the l_b was assumed equal to the square root of the area of the corresponding integration point in order to assure results independent of the mesh refinement [23].

By using the diagrams represented in Fig. 19 for modelling the fracture mode I initiation and propagation in the SFRSCC, the test carried out on the prototype was simulated by FEM. The objective of these simulations is to assess how reliable are the three above described approaches for deriving the fracture mode I constitutive law in

the context of material nonlinear analysis based on a smeared crack model implemented in FEMIX V4.0 [19], and described in detail in [24]. For this purpose the E-SFRSCC slab was discretized with 8 nodes finite elements composed of 5 layers of 15 mm thickness according to the Mindlin-shell theory described elsewhere [25]. A G-L integration scheme of 2×2 IP was used for these elements. The columns and the slab supported on soil were simulated by solid elements of 20 nodes with 2×2×2 G-L IP. To assure rotational compatibility between shell and solid elements in the E-SFRSCC slab/column connections, T cross arrangements of 3D Timoshenko beam elements were utilized with 1×3 IP and suitable flexural stiffness. The finite element mesh of the prototype is shown in Fig. 20.

In the performed FEM analysis, the soil was simulated by surface springs working only in compression and with a stiffness whose contribution is orthogonal to the surface of the slab in contact with the soil. The lateral surfaces of this slab were also assumed in contact with soil. Reaction module [26] values of 50 and 25 MPa/mm were assumed for the soil in contact with, respectively, the bottom and lateral surfaces of the slab on soil. A null contribution was assumed for the soil in tension in order to simulate eventual occurrences of loss of contact between soil and the indicated surfaces of the slab. The values for the reaction module adopted for the soil are representative of the type of soil giving support to the prototype. Preliminary simulations had indicated that the adopting values for the soil reaction modules in an interval acceptable for the local type of soil, the behaviour of the prototype does not change significantly. Preliminary analysis had also indicated that, up to a load level that introduces significant damage in the E-SFRSCC slab, no cracks have formed in the slab supported on soil and in the columns. This was confirmed in the experimental tests. Therefore, the columns and the slab on soil were assumed as presenting linear and elastic behaviour in order to decrease the computing time, but without compromising the modelling accuracy and do not affecting the credibility of the main conclusions to be extracted from this analysis.

The fracture mode I component of the crack constitutive law in the context of the multi-directional smeared crack approach is defined by the quadrilinear diagram represented in Fig. 21. In this diagram, G_f is the SFRSCC fracture energy and l_b is the crack band width. The values that define this diagram for the approaches investigated are included in Table 4.

The relationships between force and deflection at the centre of the loaded panel $\left(F - \delta_{cp} \right)$ obtained from FEM simulations for the three approaches are compared in Fig. 22 with the corresponding one registered

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experimentally in the four loading steps.

As Fig. 22 shows, the constitutive laws derived from the three adopted approaches provided similar results in terms of force-deflection response of the prototype in the numerical simulations. When compared to the test results, the force versus central deflection relationship of the panel predicted by the FEM analysis are very close to the ones recorded in the first and second step of the loading scheme in the experimental test (Figs. 22a and 22b), where the linear elastic response was dominant in the prototype. However, in the third step of loading, the FEM analysis predicted noticeably smaller central deflection at the maximum prescribed force ($F_{\max,3} = 70\,\text{kN}$), of about 70% of the one obtained in the test (Fig. 22c). As Fig. 22d shows, the FEM approaches have predicted a load carrying capacity of about 20% higher than the one registered experimentally in the fourth step of loading.

In samples utilized in DTT and three-point notched beam bending tests, fibres are predominantly oriented in the direction of the axis of this type of specimens, which is almost orthogonal to the failure crack. Therefore, fibre orientation and distribution are not representative of the real conditions existing in an E-SFRSCC slab. Tests on square or round panels can be more appropriate to characterize the post-cracking response and the toughness classification of FRC applied in laminar structures, such is the case of the E-SFRSCC slabs, since the formation of several cracks of different orientation in these panels mobilizes more representatively the fibre reinforcement mechanisms really occurred in this type of structures [15, 27]. However, to derive the $\sigma_t - w$ relationship from the force-deflection response obtained in these panel tests a sophisticated model is required, which is an ongoing research activity of the authors of the present work.

To evaluate the fracture mode I parameters of the crack constitutive law of the material applied in the E-SFRSCC slab, a last series of simulations were carried out by fitting, as best as possible, the experimental $F - \delta_{cp}$ relationship. This was executed by performing an inverse analysis of the prototype structural response (IAS) with the $F - \delta_{cp}$ recorded experimentally (Fig. 18). In Fig. 23 is compared the $F - \delta_{cp}$ obtained experimentally and applying the IAS. The quadrilinear diagram obtained from IAS approach is compared in Fig. 24 to the diagrams derived from the other three approaches, whose parameters are included in Table 4.

Fig. 24 and Table 4 indicate that the constitutive law that governs the fracture mode I of the SFRSCC applied in the E-SFRSCC slab has an abrupt stress decay up to 65% of the tensile strength at the initial phase of the crack formation process, followed by a smooth decay from 65% to 50% of the tensile strength up to about 90% of the

ultimate strain. The tensile strength and the fracture energy can be those obtained from IA (see Table 4). The shape of this strain softening diagram and the value of G_f are in agreement with the recommendations of Barros and Figueiras [28].

4. Conclusion

To appraise the potentialities of the post-cracking response of steel fibre reinforced self-compacting concrete (SFRSCC) as fundamental material for elevated slabs (E-SFRSCC), an experimental program was conducted on a ¼ scale prototype. This program included different uniformly distributed load levels, and a quasi-point load in order to assess the behaviour of this material/structural system at serviceability and ultimate limit state conditions. For serviceability limit state conditions no cracks were formed in the E-SFRSCC slab, and the maximum deflection is far below the code design requisites for this type of elements. Even increasing the uniformly distributed load up to 39 kN/m², no cracks were detected. This uniformly distributed load level is about 20 times higher the live load considered in the design of elevated slabs of single family-houses. For ultimate limit state conditions a quasi-point load was applied on the centre of a panel. The prototype demonstrated a noticeable post-peak residual strength with large deflection capacity in consequence of the formation of a diffuse crack pattern. At the deflection when the test was interrupted (3.6 times the deflection at peak load), the recorded load was 60% of the peak load.

The post-cracking response of the used SFRSCC was characterized in terms of stress-crack width $(\sigma_t - w)$ relationship by three different approaches: direct tensile tests, three-point notched beam bending tests according to the *fib* Model Code 2010 recommendations, and inverse analysis. The obtained $\sigma_t - w$ relationships were adopted in a FEM analysis carried out to simulate the behaviour of the prototype tested up to failure. By adopting in the FEM-based numerical simulations the $\sigma_t - w$ diagrams obtained from the three aforementioned approaches, a load carrying capacity higher than the one registered experimentally was predicted. Moreover, the notable ductile response of the prototype was not correctly captured by the FEM analysis, which indicates that the $\sigma_t - w$ diagrams obtained by the aforementioned approaches do not represent integrally the fracture initiation and propagation of the SFRSCC applied in the E-SFRSCC slab.

An inverse simulation was carried out to determine the fracture mode I parameters that best fit the experimental force-deflection response of the prototype. A fracture mode I diagram of almost trilinear configuration was

obtained, formed by a first and a third branches that simulate an abrupt stress decay, connected by a second branch that reproduces a smooth stress decay from $0.65f_{ct}$ to $0.5\ f_{ct}$ (f_{ct} is the stress at crack initiation). The shape of this diagram and the value of the fracture energy are in agreement with the recommendations of Barros and Figueiras [28].

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