Universidad Politécnica de Madrid

Escuela Técnica Superior de Ingenieros de Caminos,
Canales y Puertos

**EXPERIMENTAL AND NUMERICAL STUDY OF CEMENTITIOUS
MATERIALS SUBJECTED TO HIGH STRAIN RATE TENSILE LOADINGS**

Tesis doctoral

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Madrid 2017

Departamento de Ciencia de los Materiales

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LOS VOCALES

EL SECRETARIO

In the name of Allah, Most Gracious, Most Merciful

To my parents,
my wife and my
kids

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ABSTRACT

Over the last decades, many attacks involving explosive materials have taken part in sensitive building and structures in different parts of the world. These events have stimulated scientists to study and provide solutions for this problem from different standpoints. From the point of view of the structural engineers, the studies have focused on the behaviour of structural materials subjected to blast loading and on the development of new construction materials with enhanced mechanical properties when subjected to high strain rates.

According to the experience gained from this kind of events, many victims of these attacks were not caused by the direct impact of the explosion but by the so called progressive collapse. Previous researches have shown that the use of ductile materials may improve the structural behaviour against such progressive collapse. In this sense, self-compacting concrete reinforced with different steel fiber shapes can be considered a good candidate to improve structural behaviour against collapse due to its enhanced ductility. For these reasons this thesis focus on self-compacting concrete reinforced with two types of fibers, to investigate its properties under a wide range of loading rates.

Three different self-compacting fiber-reinforced concretes, named A, B, and C, having the same cementitious matrix but different quantity and types of fibers were produced. The types of fibers considered were straight and hooked-end types. 40 kg/m³ of straight fibers were used in the three mixes, while the amount of hooked-end fibers was 0, 20 and 60 kg/m³ for the concretes A, B, and C, respectively. These three concretes exhibited different mechanical behavior in all tests performed on them, even under quasi-static loading rates.

For the three concretes, three-point bending tests were conducted on prismatic notched beams with two different instruments, a servo-hydraulic testing machine and a drop-weight impact machine. The recommendations of the RILEM TC 162- TDF committee and the standard of EN 14651 were followed. With the servo-hydraulic machine two different loading rates were used: 2.20×10^{-3} mm/s (quasi-static) and 2.20×10^1 mm/s. The result showed an increase in peak load and fracture energy due to the increase in fibers content and in the loading rate.

About the tests conducted with the drop-weight impact machine, three different heights were used (40mm, 160mm and 360mm) and a fixed weight for the impactor (120.6 kg), producing therefore three different impact energies and velocities. The results of these tests demonstrated again that the higher the strain-rate, the higher the specific fracture energy for the three concretes. However, such increase of the specific fracture energy is lower as the quasi-static strength of the concrete is higher.

Besides the abovementioned laboratory tests, blast test were conducted on four slabs of each concrete type. To this end a steel structure with the capability of testing up to four slabs subjected to the same explosive load was used. For each concrete, one slab was subjected to an explosive load of 2.46kg of TNT equivalent, while three slabs were subjected to 3.18kg of TNT equivalent. In all cases the standoff distance of the explosive was kept fixed at 1.5m. The results indicated that increasing the amount of fiber prevents fragmentation of the specimens, leading to

ABSTRACT

a more distributed crack pattern. Besides this, these tests were used to validate the numerical model for fracture of concrete under high strain rates presented in this thesis.

In order to study the residual strength of the concretes, some of the slabs previously tested under explosive loading were retested under static loading to investigate the flexural load and to compare them with intact slabs. The results provide valuable experimental information concerning the behaviour of this kind of material against progressive collapse.

In the numerical field, a new approach for the simulation of fracture of fiber reinforced concretes under high strain rates has been presented. Such approach is based on the cohesive zone model (CZM) in conjunction with the strong discontinuity approach. The original contribution of this new approach consists on the definition of the dynamic increase factor (DIF) for the tensile strength as a function of the cohesive crack opening. This new approach has been implemented in the finite element commercial code LS-DYNA through a user subroutine.

The numerical model has been successfully applied to the experimental tests conducted in this thesis, with the only exception of the residual strength tests. The good agreement achieved between the numerical simulations and the experimental results supports the hypotheses made in this new model.

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State of art

1.1 Scope

In recent years, terrorist attacks in the whole world on important government installations have increased, increasing the need for direct and indirect ways to resist these attacks. These attacks have caused huge damages and losses in structures and human victims. The greatest danger will happen following these explosions by the progressive collapse inside the structure (Islam AKMA, 2008; JD; Luccioni BM, 2004; Thompson D). As seen from Fig. 1.1, in 1996 the eight-storey Building #131 structure of the Khobar Towers in Saudi Arabia was attacked by a truck bomb, killing 19 and injuring around 498 people. The truck was standing approximately (22 m) away from the building with at least (9,100 kg) of TNT detonation material, according to a later assessment of the Defense Special Weapons Agency.

(A)

(B)

Fig. 1.1 Alkhobar attacked ("https://en.wikipedia.org/wiki/Khobar_Towers_bombing," ; "<https://www.lawfareblog.com/captured-mastermind-behind-1996-khobar-towers-attack>,")

This attack created a huge distortion and subsequent progressive collapse because of the quantity of explosion material and the distance of the explosion which took place very close to the target. Increasing the distance from a large detonation center, the effects of the explosion will reduce whether in victims or asset losses ("Departments of the Army, the Navy, and the Air Force. Structures to resist the effects of accidental explosions TM 5-1300," 1990). Because of these events, researchers work to improve the behavior and characteristics of the material that could be subjected to sudden impacts, explosion or earthquakes. These days, steel-fiber reinforced concrete (SFRC) has been used widely for pavements, highways, airport runway overlays in civilian, military and industrial environments. It is considered a suitable material for structures developed to absorb and resist highly dynamic events such as impacts, earthquakes and explosions, and reduce both cracking and thickness (Garfield, Richins, Larson, Pantelides, & Blakeley, 2011; Mindess, Young, & Darwin, 2003) thanks to its good strength and durability (Okamura & Ouchi, 2003). Thus, the dynamic mechanical behavior of SFRC is receiving more and more attention (Nemkumar Banthia, Fischer, & Li, 2006; Caverzan, Cadoni, & Di Prisco, 2012; Daniel, Gopalaratnam, & Galinat, 1996; Tran, Tran, & Kim, 2015; Yoo, Yoon, & Banthia, 2015; Zhang, Elazim, Ruiz, & Yu, 2014). Different kind of fibers in a high-strength concrete matrix will produce High Performance Fiber Reinforced Concrete (HPFRC) with high post-cracking residual strength and strong mechanical properties. There are few studies on these materials under high strain rates as well as few simulations of them under finite element. Therefore, this thesis works with this material under wide loading rates to characterize HPFRC and to study its behavior under every condition. In the end, the simulation of the experimental tests and their results are presented.

In this thesis, three different concrete mixes have been designed and subjected to a wide range of loading rates from low loading rate (10^{-3} mm/s), medium loading rate (10^3 mm/s) to reach blast loading conditions. These concretes have the same self compacting concrete matrix with differences in the fibre densities, A, B and C types. The instruments used with these materials were a Servo-Hydraulic Machine, a Drop-Weight Impact Machine and a specially designed test bench for blast tests. The program used to simulate all types is LS-Dyna with same subroutine for all loading rates. The three mean points my thesis centered around, are the material characteristics of the three types depends on the wide range of loading rates (static and medium), the blast test and the subsequent behavior of the material, as well as same simulation related to all tests to replicate our results behavior.

The structure of this thesis has been divided in seven chapters. This first chapter presents the scope of our thesis and secondly all literature reviewed for the thesis.

In Chapter 2: The research Motivations and main objectives are presented.

In Chapter 3: Presents material and mechanical characteristics, contains the experimental program details, material and experimental devices used for static, quasi-static and medium loading rates with three point bending tests. These instruments were used to design the study of static and dynamic behavior of structural concrete samples. In the end, the results of the experiments obtained from the tests are shown.

In Chapter 4: Presents the experimental device that allows for testing up to three samples on each detonation. Also, the results of the tests obtained from the explosion tests are shown. In the end, we measure the residual strength for all slabs used and compare their results with intact slabs.

In Chapter 5: The thesis studied the numerical simulation which was used to mimic our experiment behaviors for every type of concrete under every loading rate.

In Chapter 6: presents the results of the numerical simulation and compare them with the experimental ones.

Lastly, the conclusion and future work proposals are presented.

1.2 Literature review

1.2.1 Fiber reinforced concrete

1.2.1.1 Self-compacting concrete (SCC)

The basic material in this composite was Self-Compacting Concrete (SCC). This mix defined by (Okamura & Ouchi, 2003) as concrete which, depending on its weight, is able to run in the interior of any mold, and casting it around the supported rods and other obstacles, and letting it set and dry (Okamura & Ouchi, 2003; Pająk & Ponikiewski, 2013). The first evaluation of SCC took place in Japan in 1988 to build durable concrete structures, as well as to improve the quality of the construction process, and to increase its economic, social and environmental benefits over traditional vibrated concrete construction (Goodier, 2003). In spite of this improvement, hardened SCC is still as brittle as normal concrete and offers little resistance to crack growth. The addition of a ductile material like steel fibers will improve the post-peak parameters of SCC but still needs to be developed (Pająk & Ponikiewski, 2013).

1.2.1.2 Development SCC with steel fibers

The long process of development to create a modern SFRC probably started in California In 1874 when A. Bernard strengthened concrete by adding steel parts. In 1910 Porter tried applying short wire to concrete to increase its homogeneity and reinforced it with thick wire. In 1918 in France, Alfsen worked to increase the tensile strength of concrete by modifying concrete with long steel fibers. The first person who mentioned the influence of coarseness of the surface of fibers onto their adhesiveness to matrix was Alfsen. He was also concerned with the problem of anchorage of fibers. A lot of researchers after the first patents have paid attention to the shape and probable applications of SFRC like G.C. Martin, who focused on the production of SFRC pipes in his patent from 1927(Maidl, 1995). In 1986 the ultra-high performance fiber reinforced concrete (UHPFRC) started to be developed in Denmark (Bache, 1987).

There are many factors that affect the efficiency of the reinforcement. The post-peak behavior of FRC and its mechanical properties are directly related to the characteristics of the concrete matrix (SCC), and also their type, quality, geometry, volume fraction, orientation and mechanical properties of the engineered steel fibers. This will depend on tensile strength of the fiber, the compressive strength of the matrix and the fiber-matrix bond (A. E. Naaman & Reinhardt, 1996). The most popular types of steel fibers used in research and industry are: traditional straight, hooked, crimped, with deformed ends (coned, with end paddles or end buttons) and with deformed wire either indented, etched or with roughened surface (Katzner & Domski, 2012). Also, the statistics presented by the largest fiber producers indicate that around 67 % of fiber sold consists of the hooked type, whereas straight fiber only represents around

9%, fiber with deformed wire around 9%, crimped fiber around 8% and others fiber types only 7% (Katzner, 2006).

The cracking strength of the flexural behavior in a SFRC is divided into two parts: pre and post-cracking. Pre-crack is primarily affected by the strength of the matrix (Katzner & Domski, 2012). On the other hand, the post cracking is dependent on the length of the fibers and their bond strength, as well as the mechanical bond between the fiber and the matrix, which is increased with the deformed shape of hooked end steel fibers (A. Naaman & Reinhardt, 2006; Tran et al., 2015). It is widely acknowledged that, steel fibers are most effective once the brittle matrix has started to crack (Pająk & Ponikiewski, 2013).

There are two possible behaviors that can occur after the first crack appears in SFRC: conventional fiber reinforced concretes exhibit softening in their stress-strain curves, while HHPFRCCs exhibit strain hardening behavior, accompanied by multiple cracking and absorbing a considerable amount of energy, proportional to the area under the curve. This is shown in fig. 1.2, which details the difference between these two behaviors.

Fig. 1.2 Difference between behaviors of conventional and HPFRCC (A. Naaman & Reinhardt, 2006).

The fibers can be divided according to their length into micro and macro fibers. The fiber length classified as a micro fiber is mainly used to reduce the shrinkage cracks and inhibit the initiation and growth of micro-cracks. This allows the strain to remain localized and limits its effect on the post-peak part of the load-deflection curve. The fiber length classified as macro steel fibers mainly used to provide effective bridging stresses across the crack after the micro-cracks

coalesce into macro-cracks. This type of fibers which has an important influence on the post-peak strength, may abate further propagation of cracks, improve the toughness, ductility and strength (Balaguru & Shah, 1992; Bentur & Mindess, 2006; Ding, 2011; A. E. Naaman & Reinhardt, 1996).

The aspect ratio is the ratio of length over diameter or equivalent diameter, which will influence the workability and spacing of fibers in a fresh concrete mix. To improve its workability, the aspect ratio is generally less than 100. The results of (Pająk & Ponikiewski, 2013) for fibers with aspect ratio from 50 to 80, indicate that the SCC reinforced can be deflection-hardening (Torrijos, Barragán, & Zerbino, 2010) or deflection-softening as in (Katzner, 2006; Long, Lin, Chen, Zhang, & Wang, 2014; Miao, Chern, & Yang, 2003).

A fiber volume fraction V_f is the proportion of the fiber volume related to the entire volume of a fiber reinforced composite material (Derek, 1981). With an increase in V_f and efficiency, the behavior of a SFRC element improves significantly its behavior under load.

According to (Nemkumar Banthia & Sappakittipakorn, 2007), in hybrid Fiber Reinforced Concrete (FRC) *“there is positive interaction between the fibers and the resulting hybrid performance exceeds the sum of individual fiber performances. This phenomenon is often termed SYNERGY.”* As he mentioned many fiber combinations may provide this phenomenon as a hybrid based on:

Constitutive response: The stronger and stiffer fiber offers reasonable ultimate strength at first crack, while the flexibility of the second fiber improves its toughness and strain capacity in the post-crack zone.

Fiber dimensions: The smaller type of fiber bridges micro-cracks and therefore controls their spread and delays coalescence, leading to a higher tensile strength of the matrix. The larger one stops small size the propagation of macro-cracks and therefore substantially improves the fracture toughness of the matrix. Small size fibers (often called micro-fibers) delay crack coalescence in the cement paste and mortar phases and the resulting tensile strength will improve (NBAUOBC Banthia, Moncef, Chokri, & Sheng, 1995; S. P. Shah, 1992).

Fiber function: One type of fiber improves the initial properties such as ease of production and plastic shrinkage, while the second fiber provides enhanced mechanical properties.

In our thesis we used two types of fibers (straight and hooked end), which have differences in shape, strength and in length to study these phenomena which Banthia mentioned.

We also used notched specimens in our thesis. This is because the notched specimen absorbs less dissipated energy than that of the un-notched one due to the fact that the crack is located close to the notch plane. Also, the stability under test conditions of the notched specimen provides greater reliability (Zhang, ELAZIM, Ruiz, Sallam, & Chang, 2013).

1.2.2 Fiber reinforced concrete under dynamic loading

To establish a numerical program model that can recreate different rating rates, namely static, quasi static, impact and blast loading, the sensitivity of rating rate must be taken into consideration. In Fig. 1.3, some examples of expression strain rates defined as a derivative of strain correspond to time, can be seen.

Fig.1.3. Magnitude of strain rates expected for different loading cases (Bischoff & Perry, 1991).

The behavior of the material varies with changes in rating rates as observed by (Abrams, 1917). The dynamic increase factor (DIF), is the ratio of the dynamic to the static strength (Malvar, 1998). The experimental study of DIF on compression strength is greater than tension strength in case of plain concrete. Thus, it is possible to confirm the enhancement of the characteristics of the material in tensile strength thanks to the increase in strain rate.

1.2.2.1 Differences in flexural behavior of steel fiber reinforced self-compacting concrete

(Pająk & Ponikiewski, 2013) in their paper, do experiments to investigate the flexural behavior of SCC and compare it when it is reinforced with three different quantities of straight and hooked end steel fibers. They note that the compressive strength and the modulus of elasticity of concrete is not affected by the increase in steel fibers, but has clear effects on the residual tensile and flexural strengths, in the other words, its post-peak behavior. They use the bending tests on notched beams related to make their recommendation (Rilem, 2002b) and (EN, 2005b). The proportions of the composition mix which shown in table 1.1 are maintained and only the fiber content changes. The three different volume fractions of steel are 0.5%, 1.0%, 1.5% with short (S) and also with Hooked end (HE) fibers.

Table 1.1 Composition of SFR-SCC mix. (Pająk & Ponikiewski, 2013)

Material	CEM II/ B-V 32,5	Sand (0-2 mm)	Fine aggregate (2-8 mm)	Water (l/m3)	Steel Fibers	Super plasticizer	Stabilizer	W/C
Kg/m3	490	808	808	201	40, 80;120	17	2	0.42

Three specimens for each type and volume ratio of steel fibers were tested and compared with three plain SCC beams results. The loading rate used in this test of 0.2 mm/min and the end of deflection displacement was 5 mm.

Fig. 1.4 and Table 1.2 show the load–deflection curves derived for each type and all volume ratios of steel fibers. They note that the flexural behavior is highly affected by the type of steel fibers of the tested beams and that the higher the fiber content for both types of steel fibers the more the maximum load will increase. SCC reinforced with the hooked end steel fibers will result in a gradual decrease in load after reaching peak load while the straight steel fibers indicate a sudden load drop.

Table 1.2 Experimental results of peak and post-peak parameters SFR-SCC (Pająk & Ponikiewski, 2013).

Mix	Flexural tensile strength (MPa)	Maximum load deflection (mm)	$f_{eq,2}$ (Mpa)	$f_{eq,3}$ (Mpa)	$f_{R,1-CMOD}$ (Mpa)	$f_{R,2-CMOD}$ (Mpa)	$f_{R,3-CMOD}$ (Mpa)	$f_{R,3-CMOD}$ (Mpa)
0	2.45(23.4)	0.05	-	-	-	-	-	-
0.5_s	3.66(6.5)	0.05	1.72(14.2)	1.25(18.8)	2.09(14.3)	1.40(21.2)	1.00(20.4)	0.76(18.1)
1_s	4.27(8.0)	0.06	2.99(3.8)	2.25(3.0)	3.31(2.8)	2.48(3.9)	1.93(4.8)	1.45(14.6)
1.5_s	5.42(10.6)	0.11	4.62(13.5)	2.85(5.6)	4.85(7.3)	3.64(0.9)	2.70(2.1)	2.04(1.4)
0.5_HE	3.8(6.9)	0.15	3.38(12.7)	2.77(12.4)	3.61(11.4)	3.13(10.2)	2.60(14.7)	1.97(25.2)
1_HE*	6.14(1.5)	0.92	5.61(3.2)	5.21(0.8)	5.81(1.0)	5.65(3.1)	4.46(0.9)	3.55(6.2)
1.5_HE	8.31(0.9)	0.64	8.21(1.3)	6.91(3.1)	8.09(0.8)	7.58(1.8)	6.32(3.2)	5.09(9.7)

* Average of two specimens

Fig. 1.4 SCC reinforced with: (a) 0.5%; (b) 1.0%; and (c) 1.5% of volume fraction of straight and hooked end steel fibers (Pająk & Ponikiewski, 2013).

(Pająk & Ponikiewski, 2013) try to confirm their results by comparing them with other researchers which used (Rilem, 2002b) as their standard. There is a linear relationship correlations investigated by (J. Barros & Antunes, 2003; J. A. Barros, Cunha, Ribeiro, & Antunes, 2005), between post peak parameters and equivalent tensile strengths of the concrete. $f_{eq,3}$ resulted to be about 92% of the $f_{eq,2}$. Fig. 1.5-a presents the relationship between $f_{eq,2}$ and $f_{eq,3}$ obtained by the authors, excluding the results obtained for tests on beams reinforced with the highest volume fracture of straight steel fibers. $f_{eq,3}$ is only about 84% of the $f_{eq,2}$ close to Barros' calculations which means that, up to a deflection of δ_3 , the energy absorption capacity of the designed SFRC was approximately maintained. Barros also suggested a linear trend also exists between $f_{R,1}$ and $f_{R,4}$. They observed a linear correlation between $f_{R,1}$ and $f_{R,4}$, with $f_{R,4}$ being at about 78% of $f_{R,1}$. The scatter in the $f_{R,1} - f_{R,4}$ diagram is greater than in the $f_{eq,2} - f_{eq,3}$ relation. Barros conclude the result with that f_R parameters are more likely to be affected by

local irregularities in the $f - \delta$ curve (J. A. Barros et al., 2005). Pająk & Ponikiewski try to follow this relation results but unfortunately the residual flexural tensile strength $f_{R,4}$ as shown in Fig. 1.5-b was about only 52% of the $f_{R,1}$ which is different to the one obtained by Barros. In the end, as shown in Fig. 1.5-c there is no matching relation between $f_{eq,2}$ and $f_{R,1}$ with $f_{eq,3}$ and $f_{R,4}$. They conclude that the model proposed by Barros has not been confirmed for SFR-SCC, according to their results.

Pająk & Ponikiewski identify the fracture energy (parameter) as fundamental when applied by (Beygi, Kazemi, Nikbin, & Amiri, 2013). They noted that tensile strengths (maximum, residual and equivalent) are enhanced by increases in fiber content. Moreover, the fracture energy of the tested beams is increased and its relation with the fiber content for any given fiber type is linear.

This thesis follows the procedure done in this paper with expanding the loading rates.

Fig. 1.5 Relationship between equivalent some parameters (Pająk & Ponikiewski, 2013)

1.2.2.2 Impact loading rate using free load instrument

In addition to bearing quasi-static loads, concrete structures also may momentarily subjected to medium loading rates, caused by impacts from wind gusts, earthquakes or machine dynamics. To deal with these situations, researchers have developed several techniques to study the dynamic fracture behavior of concrete and concrete structures. Modified Charpy impact test, Split Hopkinson pressure bar test and drop weight impact test are examples of (Zhang, Ruiz, & Yu, 2008). (Caverzan et al., 2012) also confirm their techniques as variable for their studies, namely the dynamic fracture behavior of a HPFRC using a modified Hopkinson bar with notched cylinder specimens. There were tested less than 3 m/s loading rate.

There has been intensive investigation for many years about the behavior of concrete structures which have been subjected to dynamic loading specifically as a response to medium loading rates such as impact loads (N Banthia, Gupta, & Yan, 1999; Bindiganavile & Banthia, 2001; Bindiganavile, Banthia, & Aarup, 2002a; Dey, Bonakdar, & Mobasher, 2014; Wu, Oehlers, Rebentrost, Leach, & Whittaker, 2009; Yoo, Banthia, Kim, & Yoon, 2015; Yoo & Yoon, 2014). Despite the work of researchers in this field, there is still a lack of data and understanding of the conditions related to medium strain rates. Until now, it is still complex to deal with concrete under dynamic loads, because of the significant sensitivity of this material to strain loading rate (Bindiganavile et al., 2002a). Also, there is no a standard procedure to measure the fracture

energy (Bindiganavile & Banthia, 2006). Moreover, this situation is made more complex by different extraneous influences on impact testing, such as inertia (N. P. Banthia, 1987) and test machine effects (N Banthia & Bindiganavile, 2002). Upon comparison to quasi-static loading (Habel & Gauvreau, 2008) dynamic loading shows progressive increases in strength and fracture energy. Moreover, (Bentur & Mindess, 2006) reported that the strain rate sensitivity decreases as the strength of concrete increases. So, the strength gained under medium rates is higher for steel fiber reinforced concretes than in conventional ones (Maalej, Quek, & Zhang, 2005; A. E. Naaman & Gopalaratnam, 1983; Parant & Rossi, 2004).

Due to inertia forces, increasing the loading rate, provokes a change in the failure mode-I to mixed mode which homogenizes the material in the impact zone and forces damage (crack). In addition, if the movement of the crack is relatively quick, the force of inertia will stop further cracking (Ožbolt, Bošnjak, & Sola, 2013). Consequently, if a single crack splits into two different branches, the speed of the crack increases, resulting in a decrease of the stress intensity factor at the crack tip.

Test Performed by (N. P. Banthia, 1987)

The researchers in this paper used a drop-weight impact machine to perform an impact flexural loading with four types of concrete beams to calculate the peak bending load and the energy consumed to break the specimen or until the peak load was reached. These types of beams were normal-strength plain, high-strength plain, fiber-reinforced, and conventionally reinforced concrete. This machine has a frame of 3.5 m in height and sits on a $1.5 \times 1.5 \times 0.9$ m reinforced-concrete pedestal. A hammer used in this machine has a weight of 3.38 kN which is falling freely on the beam from height 2.4 m above the surface of the specimen. The hammer hits its goal and used a “tup” that contains strain gauges to measure the load. The beam is fixed to two supports anvils that used the strain gauges to read the reaction forces. This machine used a photocell with a source of light constrained by a metal strip that contains holes punched in 100 mm apart to trigger the data-acquisition system and to measure the acceleration of the hammer as shown in Fig. 1.6.

Fig. 1.6 Schematic diagram for impact machine and the photocell assembly (N. P. Banthia, 1987)

As mentioned by the researchers, the reading load by the tup is not the true bending load produced by the specimen because of existing of inertial response. That implies they need to subtract this load (inertial load) using accelerometers which were mounted on the surface beam,

from the tup load to obtain the true bending load. The kinetic energy lost by the hammer may be calculated using this equation:

$$\Delta E = \frac{1}{2} m_h \left[2a_h h - \left(\sqrt{2a_h h} - \frac{1}{m_k} \int P_t(t) dt \right)^2 \right] \quad \text{Eq. 1.1}$$

Where m_h , a_h , h and P_t are the mass, acceleration, the drop height of the hammer and the tup load respectively. Part of this energy will transfer to the beam and the remainder stays in the machine as elastic strain and vibrations. Because the last part of the energy cannot be determined, equation 1.1 is not valid to calculate the energy consumed in the beam from the hammer at any time (t). The important energy to identify is the bending energy (Fracture Energy) which is the area under the true bending load instead of the load point deflection.

$$E_b(t) = \int_0^t P_b(t) du_0 \quad \text{Eq. 1.2}$$

The load point deflection can be calculated by a double integration of the extrapolated acceleration at the load point.

$$u_0 = \int_0^t \int_0^t \ddot{u}_0(t) dt dt \quad \text{Eq. 1.3}$$

All results of the tests mentioned in table 1.3 are a comparison between the four concrete types used. The researchers conclude their work by stating that the energy absorbed under impact is greater than under static loading.

Table 1.3 Results of tests (N. P. Banthia, 1987)

	Static ¹ Peak Bending Load ² N	Fracture Energy ³ N.m	Impact ² Peak Bending Load ³ N	Fracture Energy ³ N.m
Normal-Strength Plane Concrete ⁵	6344 (306) ⁴	5.5 (1.5)	16932 (428)	90.1 (6.5)
High-Strength Plain Concrete ⁵	9720 (1809)	2.8 (0.6)	18760 (446)	74.9 (18.8)
Normal-Strength Polypropylene- Fiber-Reinforced Concrete ⁷	7302 (99)	14.0 (4.4)	17300 (821)	119.4 (8.1)
Normal-Strength Fiber-Reinforced Concrete ⁸	11500 (670)	44.8 (19)	24006 (1629)	237.6 (7.5)
Conventionally- Reinforced- Normal-Strength Concrete ⁹	22671 (3102)	442 ¹⁰ (45)	36664 (888)	880 ¹⁰ (300)

1 Static tests done at the cross head speed of 4.2×10^{-7} m/s

2 Height of hammer drop = 0.5 m

3 Average taken over six or more specimens

4 Numbers in parentheses are the standard deviations

5 Crushing strength = 42 MPa

6 Crushing strength = 82 MPa, 16 percent (by weight of cement) of microsilica

7 Fibrillated polypropylene fibers, 37-mm long, 0.5 percent by volume

8 Steel fibers with both ends hooked, 50-mm long, 1.5 percent by volume

9 Steel Area = 1.12 percent

10 Calculated up to a point when the load had dropped back to 113 of its peak value

Tests performed by (X.X. Zhang)

(X.X. Zhang) in their research worked on notched beams of steel fiber-reinforced concrete (SFRC). They used both a servo-hydraulic machine and a drop-weight impact device followed the RILEM recommendation (150×150 in cross section, 700 mm in length notch-depth ratio was around 1/6 and the span of 500 mm). The experiments were performed only on a single type of SFRC. Steel fiber was added as the reinforcement was 64.5 kg/m³, with only one type which was hooked-end (50 mm in length, 0.75 mm in diameter and 67 in aspect ratio, and tensile strength of 1900 MPa). The tests Followed ASTM C39 to measure compressive tests (see tables 1.4 and 1.5).

Table 1.4 Properties of the matrix material at age 56 days (Zhang et al., 2014)

	f_c (MPa)	E (GPa)	ν	ρ (kg/m ³)	f_t (MPa)	G_F (N/m)
<i>Mean</i>	76	32	0.18	2297	5.1	145
<i>Standard deviation</i>	3	1	0.01	6	0.7	18

Table 1.5 Properties of the SFRC at an age of 65 days(Zhang et al., 2014)

	f_c (MPa)	E (GPa)	ν	ρ (kg/m ³)
<i>Mean</i>	92	35	0.18	2438
<i>Standard deviation</i>	5	1	0.01	17

The impact hammer weight employed 120.6 kg dropped from different heights which were 40, 160 and 360 mm that correspond to impact velocities that were 8.85×10^2 mm/s, 1.77×10^3 mm/s and 2.66×10^3 mm/s respectively. Without the inertia effect on the beam, the reaction force measured at the supports has considered to be the equivalent bending load. Also, by assuming that all the kinetic energy is passed on to the first mode of vibration of the beam, an approximation of the beam response was determined at impact loading conditions. The mode of this condition is the same as the static loading rate but with increased flexural strength and reduced shear strength (G. Hughes & Beeby, 1982). For all tests they performed, a cut-off point was selected at the displacement δ_s (central deflection) of 2 mm which was used to measure fracture energy as recommended by (Recommendation, 1985) equation (1.2) with a specified deflection of the beam (2 mm). The comparison between load-displacement curves at all loading rates shown in Fig. 1.7, gave them confirmation in relation to the peak load increasing with increases in loading rates.

Fig. 1.7 Load-displacement curves at different loading rates (Zhang et al., 2014)

They noted that for the tests under drop heights 40 mm the peak of the reaction force is more than that of the impact one, whereas in the drop height of the hammer at 360 mm, the peak of the reaction force is lower than that of the impact one. On other hand, the beams failed under low loading rates have more branch cracks than under high loading rates. In addition, no broken fiber was seen, which means that all fibers were pulled out.

According to loading rates 8.85×10^2 mm/s, 1.77×10^3 mm/s and 2.66×10^3 mm/s, the time intervals between the start points of the impact force and the reaction force are 344, 250, and 248 ms respectively. That mean, when the loading rate was increased, we observed a decrease in the time delay.

The researchers noted that when the loading rates increased, there is an increase in peak load. On the other hand, the property is small under low loading rates, while it is marked under high loading rates. This relation can be shown using this equation in Eq. (1.4).

$$DIF_P = \frac{P_{max}}{P_{max}^S} = \frac{P_{max}}{53.3} = 1 + k \left(\frac{\dot{\delta}}{\dot{\delta}_0} \right)^n = 1 + (7.5 \times 10^{-4}) \left(\frac{\dot{\delta}}{\dot{\delta}_0} \right)^{1.01}, \text{ for } \dot{\delta} \text{ in mm/s} \quad \text{Eq. (1.4)}$$

Where P_{max}^S is the static peak load in kN, $\dot{\delta}$ is the loading rate in mm/s, $\dot{\delta}_0$ is set as 1 mm/s thus, those adjustment parameters k and n are without units.

In addition, the tendency of DIF of the fracture energy is controlled under low loading rates, while under high loading rates it is very high. There is also a prediction equation that can be used see Eq. (1.5) and Fig. 1.8.

$$DIF_{G_f} = \frac{G_F}{G_F^S} = \frac{G_F}{4565.9} = 1 + m \left(\frac{\dot{\delta}}{\dot{\delta}_0} \right)^r = 1 + (7.6 \times 10^{-6}) \left(\frac{\dot{\delta}}{\dot{\delta}_0} \right)^{1.54}, \text{ for } \dot{\delta} \text{ in mm/s} \quad \text{Eq. (1.5)}$$

Where G_F^S is the static fracture energy, 4565.9 N/m in our case. Coefficients m and r are adjusting parameters without units due to the fact that $\dot{\delta}_0$ is set as 1 mm/s as mentioned before. Table 1.6 shows detailed information about the experimental results which consider the lowest loading rate which is taken as the quasi-static loading condition.

Table 1.6 Experimental results at different loading rates (Zhang et al., 2014)

Experimental results at different loading rates.

Testing machine	H (mm)	$\dot{\delta}$ (mm/s)	P_{max} (kN)	DIF for P_{max}	G_F (N/m)	DIF for G_F	Number of fibres
Servo-hydraulic testing machine	—	3.33×10^{-3}	49.5 (11)	1	4432 (1022)	1	267 (59)
	—	0.1	55.4 (4)	1.119	4590 (566)	1.04	283 (56)
	—	3.33	55.5 (6)	1.121	4676 (455)	1.06	253 (46)
Drop-weight impact machine	40	8.85×10^2	91.6 (6)	1.85	—	—	266 (31)
	160	1.77×10^3	134.1 (6)	2.71	8089 (782)	1.83	264 (14)
	360	2.66×10^3	172.5 (11)	3.48	11,167 (1391)	2.52	248 (32)

Note: values in parentheses are standard deviations.

The researcher indicates the pullout behavior of a fiber is affected by the rate. Because no single broken fiber was found in the crack surface, the fracture behavior affected by the loading rate of SFRC is mainly related to the behavior of the mixture and pullout of fibers. Because of the presence of free water in voids and porous structures in the matrix which are attributed to viscous effects (Ruiz et al., 2011), the effect on the fracture of the SFRC is minor in the low rate range, whereas the rate effect is pronounced under impact loading rates.

Fig. 1.8 Loading rate dependence of the peak load (a) and the fracture energy (b) (Zhang et al., 2014).

1.2.2.3 Blast loading

As we mentioned before, concrete is a brittle material, which means it is very poor in absorption capacity and so the energy under such high strain-rate loadings causes several concerns (Yoo, Yoon, et al., 2015), that means it is a material sensitive to the strain rate (Bindiganavile, Banthia, & Aarup, 2002b). Therefore, many researchers (Heard, Basu, Nordendale, & Hoemann, 2011; C. Pantelides, Garfield, Richins, Larson, & Blakeley, 2012; Yusof et al., 2013) have performed their studies to enhance its energy absorption capacity under blast by using steel fibers reinforcements. Because steel fiber is one of the most common reinforcement materials and is widely used to improve its tensile performance (Yoo, Yoon, et al., 2015), we reached the conclusions of (Li, Wu, & Hao, 2015) which found that significantly the explosive resistance of structures could be increased by using the high strength concrete with steel fiber. On the other hand, fiber reinforced concrete behavior when subjected to the blast test (Astarlioglu & Krauthammer, 2014; Mao, Barnett, Begg, Schleyer, & Wight, 2014; C. P. Pantelides, Garfield, Richins, Larson, & Blakeley, 2014), rarely studied the strain rates effect. The researchers in (Millard, Molyneaux, Barnett, & Gao, 2010) conclude that the enhancement of strain rate is limited to Ultra high Performance Fiber Reinforced Concrete (UHPFRC). This type of experiments are limited because of security restrictions, and need special detonation materials,

scatter control difficulties and need a special place to perform them with very high safety control.

A bending failure mode type for concrete elements under static loading may switch to a shear failure when subjected to fast dynamic, impulsive loading (Johan Magnusson & Hallgren, 2003, 2004; Razaqpur, Tolba, & Contestabile, 2007; Toutlemonde, Rossi, Boulay, Gourraud, & Guedon, 1995). This behavior may be related to crack propagation velocity and inertia forces effects, and is relevant, because brittle material failure is more likely in shear behaviors.

Tests performed by (B.D. Ellis, 2014)

The authors casted four Ultra High Performance Fiber Reinforced Concrete (UHP-FRC) panels with geometries $1626 \times 864 \times 51$ mm without steel bar reinforcement. These slabs were subjected to impulse loads between 0.77 – 2.05 MPa. ms. The straight fiber volume fraction is 2 % with 14 mm long by 0.185 mm diameter. The result estimation concludes that the 0.97 to 1.47 MPa.ms reflected impulse is the fracture range of this material. The authors used the experimental result to validate a multi-scale model structure with a two length scales. These length scales are a multiple fiber length and a structure length. These were tests performed in Blast Load Simulator (BLS) located in Vicksburg, MS. The schematic of this test shown in Fig. 1.9.

Fig. 1.9 Schematic of blast load simulator (BLS) (B.D. Ellis, 2014)

The fracture created formatted fracture surfaces including protruding fibers in the four panels. The researchers conclude their works by indicating that the two length scales that compounded multi-scale model basis depend on the physical experiments. These length scales are liked by several relevant quantities which are the mean tensile strength standard deviation, the mean dissipated energy density and the standard deviation of the dissipated energy density. The three mean value experiments measured which the model validated are critical specific impulse, the fracture patterns and the displacement deflection of the panel. The factors of fiber geometry, packing, and volume fraction are the corner stone that help a model improve the resistance of a UHP-FRC slabs to a detonation load. Fig. 1.10 shows the slab fraction changed in time. This paper attempts to determine the lower and upper measurements of the reflected impulse for a UHPC panel without reinforcement rods. This study has done only for one type of concrete and ignored other types, so no cooperation took place.

Fig. 1.10 slab fraction with change in time (B.D. Ellis, 2014)

Tests performed by (S. J. Barnett 2010)

The authors of this paper concentrate on tests explosion for four Ultra High Performance – Fiber Reinforced Concrete (UHP-FRC) panels. The dimensions of the slabs used in this test were $3500 \times 1300 \times 100$ mm and conducted with 100 TNT equivalent explosion loads. The setup of the tests is shown in Fig. 1.11. Two of these slabs have conventional steel reinforcement with straight steel fibers (2 % in volume fraction) and 13 mm long. One of these slabs was located on a stand at a distance of a 9 m whereas the other one was located 7 m away. The results of the maximum displacement deflection for the two types were 110 and 210 mm respectively. There were a residual recovery in these two types only of 20 and 50 mm respectively; this phenomena occurs due to the existence of steel bars.

The two other slabs didn't have any reinforced bars, with the only differences in their fiber content. The fourth slab contained a mix of two different fibers with 4 % in volume fraction whereas the third type contains only one fiber type with 2 % volume fraction. Both of these two types stand-off with a 12 m displacement. The maximum deflections of the third and fourth slabs resulting from the test were 180 and 90 mm respectively.

Fig. 1.11 Setup of the tests (S. J. Barnett 2010)

The variables that affected this material are the type, the quantity of fiber reinforcement, the reinforcement of the steel bars and the stand-off distance of the slabs from the detonation source. All slabs cracked horizontally on their surface and all of them remained standing after the test. The authors conclude this test by stating that UHP-FRC have excellent properties to

resist explosion and may be utilized to save people and structures from sudden attacks. A significant increase of the performance of the slab can be achieved by increasing the quantity of fibers of the steel bars, which will make the structure more suitable to withstand against the greater attacks. The quantity of TNT equivalent was very high and the tests were in different time which weakens the real comparison between them.

Tests performed by (C. Wu, 2009)

The researchers in this work casted 6 slabs with $2000 \times 1000 \times 100$ mm in dimensions. Two of these slabs were conventional reinforced on both sides of tension and compression with a 12 mm diameter mesh. The major bending has spaced at 100 mm, whereas in the minor one at 200 mm as shown in Fig. 1.12. The characteristics of this material are summarized in compressive strength, tensile strength and young modulus and were 39.5 MPa, 8.2 MPa and 28.3 GPa. These slabs were used for controlling the specimen. Another two were retrofitted with fiber reinforced polymer (CFRP) on the compression side using two layers of CFRP plates with a depth of 2.8 mm. The last two slabs were cast using Ultra High Strength Concrete (UHSC); one of them without steel reinforcement and the other with the reinforcement as shown in Fig. 1.12. The quantity of the explosive charge of TNT equivalent material ranged between 1 to 20 kg, and the stand-off distance range between 1-3 m.

Fig. 1.12 Control specimen dimension (C. Wu, 2009)

The test has performed with the steel frame fixed to the slabs and an effective span of 1800 mm as shown in Fig. 1.13. One of control slabs and UHSC not reinforced with steel has subjected to the same quantity of explosive. The result was that more damage happened with the control slab. The RUH-PFC specimen was subjected to the highest quantity of explosive impulse (approximately 20 times greater) than the other to reflect its high energy absorption capacity. That means the performance of the RUH-PFC slab was superior compared to all types in the test.

Fig. 1.13 Support of the explosion charge (C. Wu, 2009)

The last two papers used more than one type of concrete but with only one shot explosion which implies that the comparison between them is not accurate.

Tests performed by (Morales Alonso et al., 2011)

(Morales Alonso et al., 2011) have performed their experimental work using a novel set-up under blast conditions that allows the simultaneous testing of up to four concrete samples which will make experimental scatter be greatly reduced see Fig. 1.14.

Fig. 1.14 Schematic view of the experimental set-up (Morales Alonso et al., 2011)

The slab dimensions were $500 \times 500 \times 80$ mm and used two different types of concrete, namely normal strength concrete (NSC) and high strength concrete (HSC). The dimensions of the specimens are shown in Fig. 1.15. The concrete specimens were supported on steel frame in open air detonation tests. This construction consisted of four vertical columns tied horizontally with beams that also supported the concrete slabs. The design of this device makes the slabs be placed at a height of 1.7 m over the ground which is greater than their distance to the explosive (1.5 m) to avoid shock reflections. The explosive materials hung in the center of the steel frame to ensure every slab received the same quantity of shock wave. In addition to the cost reduction, there has a significant reduction in explosive handling and time expenses.

Fig. 1.15 Test specimen geometry and reinforcement details (in mm) (Morales Alonso et al., 2011).

Steel mesh bars with a 6 mm diameter of steel grade B 500 S with of 150 mm space in each direction were used to reinforce the back side of the concrete. 5.712 kg of Goma 2 ECO, equivalent to a 5 kg TNT explosion was applied subjected to the 12 slabs (six of each concrete type). Three concrete slabs of the same concrete type plus an aluminum control plate were tested in each detonation. Under blast test on his paper there are three predictions related to the response of the slabs:

The slabs bent on their corners; bending, shear and mixed modes were caused a tensile failure of the concrete at the rear slab side; or the slabs broke away from their supports and fell from the beams to the center of the steel frame.

Table 1.7 Failure mode of concrete slabs (Morales Alonso et al., 2011).

Concrete type	Failure mode (number of specimens)		
	Bending	Shear	Mixed
NSC	1	5	0
HSC	2	2	2

Table 1.7 and Fig. 1.16 shows the results obtained in these detonation tests were it can be proved that the shear failure with circular-like cracks occurred mainly at the edge of the steel support plates. Concrete that failed under static loading switched from bending type to a shear failure when it was subjected to fast dynamic, impulsive loading, because the impact wave moving outward from impacted surface may cause shear failure before any noticeable bending occurs. In these paper results, resistance to blast load was influenced by tensile strength for both concrete types whereas compressive strength is thought to have a small effect. These results could also be made extensive to most structural elements subjected to a blast load, in which the forces acting are primarily tensile. We used in our thesis this procedure test in blast load to perform our tests.

Fig. 1.16 Tested specimens (Morales Alonso et al., 2011)

1.2.3 Residual strength

Blast-resistant structures must prevent progressive collapse because it is the main cause of injuries in buildings attacked by explosion (Oswald, 2005). On the other hand, residual load-carrying capacity of member plays a critical parameter in the structural post blast performance. Thus if the member has sufficient residual load-carrying capacity, the threat of progressive collapse on it can be minimized, (Li et al., 2015). The primary purpose of the residual tests done in this thesis was to collect data from benchmark problems for validating simulation methods and material models for blast events and has been carried out to improve the blast resistance for three HPFRC with different quantities of fiber steel by measuring residual strength for them.

Tests performed by (C. P. Pantelides et al., 2014)

In (C. P. Pantelides et al., 2014) two types of concrete was used; as shown in Table 1.8 thirteen reinforced were concrete panels and seven FRC panels constructed with the macro-synthetic fibers (polymer type with 50 mm in length) shown in Fig. 1.17. Moreover, the average compressive strength of the concrete was 51 MPa while that of FRC was 46 MPa. In addition, the average static tensile strength of concrete was 4.0 MPa and that of FRC 4.3 MPa. The internal reinforcement was made from steel and GFRP bars. The steel bars had a tensile strength of 420 MPa and their modulus was elasticity of 200 GPa. The Ø16 GFRP bars had a tensile strength of 717 MPa and an elastic modulus of 43 GPa and the tensile strength of the Ø10 GFRP bars was 758 MPa and the elastic modulus 41 GPa.

The blast test performed on these panels had a TNT equivalent of 6.2–15.6 kg. The researchers in this work tested concrete panels previously subjected to blast were retested under monotonic static load to find out their static post-blast load resistance. Also panels that had been not subjected to blast were tested under static load.

Table 1. 8 Description of panels and tests (C. P. Pantelides et al., 2014).

1.2 × 1.2 m panels		Thickness, designation, and reinforcement		
Type	Description	152 mm	254 mm	356 mm
A4	RC (Steel bars)	A4-6	A4-10	A4-14 ^b
		Ø 10 @ 305 mm	Ø 13 @ 305	Ø 16 @ 305
B4	FRC	B4-6	B4-10	B4-14 ^b
		No Rebar	No Rebar	No Rebar
C4	FRC + steel bars	C4-6 ^a	C4-10 ^a	C4-14 ^a
		Ø 10 @ 152 mm	Ø 13 @ 152	Ø 16 @ 152
D4	RC (GFRP bars)	D4-6 ^a	D4-10	D4-14 ^a
		Ø 10 @ 152 mm	Ø 16 @ 229	Ø 16 @ 152
E4	RC (Steel bars) + GFRP laminate	E4-6 ^a	E4-10 ^a	E4-14 ^a
		Ø 10 @ 305 mm	Ø 13 @ 305	Ø 16 @ 305
COND	RC (GFRP bars)	CON-1 ^a , CON-2, CON-3, CON-	N/A	N/A
		Ø 16 @ 305 mm		
CONB	FRC	CON-5	N/A	N/A
		No rebar		

RC = reinforced concrete; FRC = fiber reinforced concrete; GFRP = glass fiber reinforced polymer.

a Panels subjected to post-blast static load test.

b Panels tested only under static load test.

Fig. 1.17. Polypropylene macro-synthetic fibers (C. P. Pantelides et al., 2014)

The nine panels previously subjected to blast were subsequently tested under monotonic static load: C4-6, C4-10, C4-14, D4-6, D4-14, E4-6, E4-10, E4-14, and CON-1. It was noted that the cracking caused by the blast reduced the initial stiffness and when the crack was smaller, the panel would be stiffer during the post-blast tests. In the end, they conclude that panel Type C was the best under static post-blast load resistance, and had a higher post-blast stiffness, a higher ultimate deflection and a higher ultimate static force. All the results of the post-blast tests are shown in Fig. 1.18, 1.19 and 1.20. We followed in this thesis the way that the researchers mentioned in this literature used to measure the residual strength to compare them with the intact slabs.

Fig. 1.18. Post-blast performance of 152 mm thick panels (C. P. Pantelides et al., 2014)

Fig. 1.19. Post-blast performance of 254 mm thick panels (C. P. Pantelides et al., 2014).

Fig. 1.20. Post-blast performance of 356 mm thick panels (C. P. Pantelides et al., 2014).

1.2.4 Simulation of concrete elements

To minimize the quantity of samples tested, numerical methods can be helpful to easily predict the dynamic behavior. The significant benefit of numerical modeling is that the influence of individual parameters can be studied separately and in more detail. This advantage will produce better understanding of the relevant properties (e.g. tensile strength and fracture energy) and understand the influence of inertia which improves our understanding of its importance in high loading rates. Constitutive modeling which is the mathematical description that provides the material response in terms of stresses, at a given strain state is important to obtain the numerical result. In our thesis same numerical model will be presented under quasi-static, impact and blast loading. Because of the changes in the loading rate in our constitutive modeling, we need to take into account the materials strain rate sensitivity. The change in velocity subjected to the material is defined as the strain rate which is the derivative of the applied strain with respect to time. The behavior under tensile stresses is simpler than in compressive ones in material models for concrete. Under dynamic loading, numerical modeling was used especially for branching in the case of concrete failure with the finite element (FE) method. In the numerical modeling, we considered different constitutive laws and modeling techniques (Rabczuk and Belytschko 2004, Ožbolt et al. 2006, Larcher 2009, Pedersen 2010). Most of the researcher's studies usually concentrate on comparing differences in the mechanical behavior of the concrete under static and dynamic loading.

There are several analytical expressions proposed for the softening curve of concrete which required for their definition at least the values of the tensile strength and the fracture energy for their definition. Some examples of analytical expressions are the rectangular, the linear (Hillerborg, Modéer, & Petersson, 1976), the bilinear (Petersson, 1981), the exponential (Planas & Elices, 1986), the General Bilinear Fit (GBF) (Guinea, Planas, & Elices, 1994) Long Tail (ELT) (Planas & Elices, 1992). The analytical expression most frequently used by researchers for the simulation of concrete are bilinear softening curves, which provide a precise prediction of the softening behavior for concrete. In the thesis we will use DIF with the impact and explosion tests to predict the softening.

There are two different approaches to calculate for the DIF in the numerical simulations that would be suitable in our tests. The formula for the DIF was taken from:

$$DIF = \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s} \right)^{\delta} \quad \text{for } \dot{\epsilon} \leq 1 \text{ s}^{-1} \quad \text{Eq. (1.6)}$$

$$DIF = \beta \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s} \right)^{1/3} \quad \text{for } \dot{\epsilon} > 1 \text{ s}^{-1} \quad \text{Eq. (1.7)}$$

Where DIF = Dynamic increase factor ; $\dot{\epsilon}$ = Strain rate

$\dot{\epsilon}_s$ = Static strain rate (10^{-6} 1/s)

$$\text{Where } \log \beta = 6\delta - 2 \quad \text{Eq. (1.8)}$$

$$\text{And } \delta = 1/(1 + 8f_{cs}/f_{co}) \quad \text{Eq. (1.9)}$$

f_{cs} = Compressive strength and f_{co} = Reference strength (10MPa).

The first approach uses the same DIF for the tensile strength and for the specific fracture energy as reported in (Schuler, Mayrhofer, & Thoma, 2006; Zielinski & Reinhardt, 1982). The result is calculated by multiplying the whole softening curve by the DIF obtained for the tensile strength.

The second approach, according to the results achieved by (Weerheijm & Van Doormaal, 2007), where the DIF applied was calculated only with the tensile strength whereas the specific fracture energy used a similar value to the one measured under quasi-static conditions. This hypothesis was followed by (Oña, Morales-Alonso, Gálvez, Sánchez-Gálvez, & Cendón, 2016).

In our thesis, where the behavior of the dynamic increase factor for the tensile strength and for the specific fracture energy are not the same, the DIF for the specific fracture energy is different than the DIF for the tensile strength. This part will be shown in Chapter 5.

Tests performed by (C. P. Pantelides et al., 2014)

The numerical analysis of the detonation tests in (Morales Alonso et al., 2011) was carried out using LS-DYNA (LS-DYNA keyword user's, 2007), an explicit dynamic finite element software based on the central difference integration rule. It is mainly used to test impulsive loads and strong discontinuous responses. A total of 546000 elements containing concrete and support steel plates were meshed with $4 \times 4 \times 4$ mm one point integration solid (brick) elements. 1500 beam (truss) elements of 4 mm length related to steel rebar. The common nodes between solid and beam elements are normally steel bonded bond steel with concrete. This simulation numerical analysis is provided for concrete elements subjected to blast. An experiment that enables up to four concrete elements to be tested simultaneously under the same blast load was developed.

In order to compare more than one simulation, the researchers used two different material models of the concrete. Winfrith Concrete model (Broadhouse, 1995; Broadhouse & Attwood, 1993; Broadhouse & Neilson, 1987) was one of these material models.

Fig. 1.21 shows that a cracking pattern, predicted by Winfrith model on NSC and HSC, is mainly of a bending type, although around the supports a shear failure mode can be observed. Plastic compressive strain is presented on the front face of the slabs. In the front face there was a cross formed by disturbed elements as well as another cross formed by the cracks on the rear face beneath the crushed elements; these results give an incorrect prediction because the results obtained with the detonation test were different.

Fig. 1.21 Cracking pattern on slab back side and crushed elements on front face as predicted by Winfrith model (Morales Alonso et al., 2011).

When the researchers were dealing with the Brittle Damage model, they noted the shear failure was the dominant behavior with both types of concrete as shown in Fig. 1.22 which were true to the failure mode predicted in NSC. These results are closer to reality of the Winfrith model for NSC. Fig. 1.23 shows a comparison between an actual cracking pattern and a numerical one showing a Brittle Damage model.

Fig.1.22 Crack distribution on rear face of NSC (left) and HSC (right) slabs as predicted by the brittle damage model (Morales Alonso et al., 2011).

Fig.1.23 Details of cracking pattern on NSC slabs and prediction of brittle damage model (Morales Alonso et al., 2011).

1.2.4.1 New constitutive model

A new constitutive model for concrete under high strain rates in LSDYNA was built, because the slab failure in the experimental campaign was governed by the tensile failure of concrete. So, constitutive modeling of concrete follows the hypothesis supported by the Brittle Damage Model (BDM) which is appropriate to post-failure behavior of concrete and enhances our capability of tracking cracking paths. For finite elements, the explicit discontinuity embedded cohesive crack model was the selected approach for simulating cracks in concrete. With respect to the fracture mechanics theory, this model was developed, as an evolution of the Cohesive Crack Model with Strong Discontinuity Approach presented by (J. Planas and J.M. Sancho (Sancho, et al, 2007a, Sancho, et al, 2007b)).

The model can be represented as a linear elastic model with its failure only occurring under tension when a threshold value for the maximum principal stress is reached. A material property independent of geometry called a softening function describes the relation between the crack opening and the stress transferred. The decoupling of the displacement field into a continuous and a discontinuous part is a method to represent the Kinematics of strong discontinuity. Two material parameters which are the tensile strength (f_{ct}) and the fracture energy (G_F) of concrete govern the softening curve as shown in the expressions for the exponential softening curve:

$$f(\tilde{w}) = f_{ct} \cdot e^{\frac{f_{ct}}{G_F} \tilde{w}} \quad \text{Eq. (1.10)}$$

$$G_F = \int_0^\infty f(\tilde{w}) \cdot d\tilde{w} = \int_0^\infty f_{ct} \cdot e^{\frac{f_{ct}}{G_F} \tilde{w}} \cdot d\tilde{w} \quad \text{Eq. (1.11)}$$

1.2.4.2 Fracture energy

Although there is a limit in the study of the specific fracture energy, there are differences in the results of the specific fracture energy between researchers. When the strain rates are increased from moderate to high, some researchers point to negligible effect on the fracture energy (Birkimer & Lindeman, 1985; Weerheijm & Van Doormaal, 2007) whereas others point to the significant effects on it (Zielinski & Reinhardt, 1982). Moreover, while a sensitivity effect of fracture energy exists in the case of high strain rates, there is a doubt about the moderate one. So, there are some scientists who don't consider the fracture energy as a property of materials (Toutlemonde et al., 1995).

Motivations and main objectives of the study

In recent years the world there have been some events whether by natural or human made such as earthquake and explosions attacked. These disasters cost huge losses in infrastructures, facilities and in human being injured and deaths, especially when this event follows by progressive collapse. From this point, the researchers have worked to investigate the reasons for these effects and to improve the materials which will help to protect people and constructions. In spite of all these investigations by scientists, still there is a fear to understand the behavior of this material under high loading rates. So, the work is one of the projects contribute to produce composite cementations material which used hybrid Steel fibers to produce High Performance Fiber Reinforced Concrete (HPFRC) to increase the structural ductility and, subsequently, to improve the structural resilience. The main aim of this thesis is to investigate the quasi-static and dynamic fracture properties of concrete under impact loading rate and blast by combining the experiments and the simulation analysis, especially focusing the investigation on the dynamic fracture of concrete. This aim will contribute to improve knowledge on the dynamic fracture of concrete.

This aim can be summarized as follows:

1. Investigate the effect of different volume fraction of several fibers on fracture energy under several quasi static loading rates.
2. Investigate the effect of different volume fraction of several fibers on fracture energy under impact loading rates.
3. Investigate the effect of different volume fraction of several fibers on fracture energy under blast loading.

4. Compare between loading rates effects for every type of concrete.
5. Study the different behavior of concrete types under the blast load.
6. Determine the dynamic increase factor (DIF) of the flexural strength and fracture energy in the three bending point rates and predict the loading rate effects on every type of concrete.
7. Study the effect of the explosion on the residual strength of the different concrete types.
8. Simulate the numerical material model for every type of concrete under every loading rate to predict the behavior of these materials.

Material and mechanical characteristics

3.1 Introduction

This work details an experiment campaign using high-strength fiber reinforced concrete beams subjected to a wide range of loading rates from quasi static rates to medium loading rates. Three self-compacting high-strength concretes reinforced with steel fibers have been designed using the same concrete matrix but different types and quantity of fibers (low, medium and high). The mechanical characterization conducted later over the three concretes, that includes fracture energy measurements at medium strain rates, has confirmed the remarkably different softening behaviors achieved with the three mixtures designed. All experiment test results on steel fiber-reinforced concrete (SFRC) made by PACADAR Company in 10/4/2014 are presented in this chapter.

As will be described in detail later, the specimens and their tests were prepared according to various ASTM and RILEM standards. Cylindrical specimens were tested to characterized young modulus and compressive strength, and beam specimens were tested to investigate fracture behavior related to static, quasi static and impact loading (medium strain) rates. The static tests were carried out using a servo-hydraulic machine with two loading rates, 2.20×10^{-3} mm/s and 2.20×10^1 mm/s. Impact tests were carried out on instrumented drop-weight impact machines using a hammer of 120.6 kg in weight dropped from three heights: 40, 160 and 360 mm. This drop-weight impact machine was designed and constructed at the Laboratory of Materials and Structures at the University of Castilla-La Mancha in Ciudad Real to evaluate the mechanical behavior of the fiber reinforced concrete prismatic specimens various flexural strengths are determined from the Load-CMOD and Load-Deflection curves.

These tests were conducted to study the influence of loading rate on the peak load and crack propagation. Three-point bending tests were performed on notched beams.

3.2 Material and specimens

The mixing materials used in this thesis and the shapes of samples casted are clarified in this section.

3.2.1 Concrete mixes

Three different concrete mixes were made in an electric concrete mixer, namely A, B and C for all concrete types the production process was the same and the composition of the concrete matrix was also the same and is shown in table 3.1

Fig. 3.1 Coarse sand

Fig. 3.2 Fine sand.

Table 3.1 Mix composition details

Laboratory name	Mix proportions by weight
Cement	1
Silica fume	0.12
Filler Siliceous	0.35
Fine Sand (Fig. 3.1)	1.12
Coarse Sand (Fig. 3.2)	1.27
Water	0.38
super plasticizer	0.021

Two different types of fibers were used: straight short Bekaert OL 13/.20 fibers and hooked-ends-long Bekaert RC 80/30 BP fibers shown in Fig. 3.3. The fiber contents for each type of concrete are shown in Table 3.2.

Table 3.2 Quantity of fibers in SFRC types

		Straight short fiber (kg/m ³)	Hooked end long fiber (kg/m ³)	Volume ratio (%)
Type of Fiber	Length	(13mm)	(30mm)	-
	Diameter of Fiber	0.2 mm	0.38 mm	-
	Aspect Ratio	Bekaert OL 13/.20 (65)	Bekaert RC 80/30 BP (80)	-
Type of Concrete	A	40	-	0.51
	B	40	20	0.77
	C	40	60	1.23

Fig. 3.3 Steel fibers types

3.2.2 Workability

The workability of the fresh mixtures was determined from the slump flow measurements using a standard 6 liter Abrams cone according to (Bartos, Sonebi, & Tamimi, 2002) as shown in Fig. 3.4. The results of workability in fresh state for the three types were:

1. For the first one (A- Type) which was with 40 kg/m³ of straight fibers OL 13/0.20, the round slump flow resulted in 70×70 cm.
2. The slump flow in (B- Type) decreased to a value of 66 × 67 cm due to the addition to 40 kg/m³ OL 13/.20 with long hooked fibers 20 kg/m³ of long fibers RC 80/30 BP.
3. The last type (C- Type) which contains 40 kg/m³ OL 13/.20 with long hooked fibers 80 kg/m³ of long fibers RC 80/30 BP, resulted in a diameter of 57×57 cm.

Fig. 3.4 Slum flow test.

3.2.3 Specimens shapes

In each type of SFRC four big cylinders, seventeen prisms and six slabs were manufactured. The specimen dimension for compression tests were big 150 mm×300 mm cylinders. The dimension of the beam was 100 mm × 100 mm × 450 mm and the span was 333 mm (see Figs. 3.5, 3.6). The slabs will be discussed in depth in the next chapter. The specimens which have been tested under low loading rates have letter P with numbers from P1 to P9; and the specimens which have been tested under medium loading rates have letter P with numbers from P11 to P19.

Fig. 3.5 Steel molds for casting of all samples

Fig. 3.6 General view of casting concrete

Fig. 3.7 shows sample of prismatic beam and table 3.3 summarized all samples (in number) for all concrete types have been produced.

Fig. 3.7 Prismatic beam sample

Table 3.3 All samples used (in numbers) in thesis

Type	Cylinder specimen	Prismatic specimen					Slab		
	Compressive strength	Low loading rate		Medium loading rate			Blast		Low load
		2.20×10^{-3} mm/s	2.20×10^1 mm/s	40 mm Height	160 mm Height	360 mm Height	2.46 kg TNT	3.18 kg TNT	0.007 mm/s
A	4	4	4	4	4	4	1	3	2
B	4	4	4	4	4	4	1	3	2
C	4	4	4	4	4	4	1	3	2

3.3 Mechanical tests

In this section we will explain the different tests with their equipments used to perform them, and the results. Cylindrical samples and prism beams have been cast from the mixed concrete. Steel molds were used in all samples.

3.3.1 Compressive tests

A total of 24 cylindrical specimens have been casted (4 of each concrete type) for the uniaxial compressive strength tests.

3.3.1.1 Equipments used

Compressive tests following ASTM (C39-02, 2002) (which is analogous to EN 12390-3), on 150×300 mm (diameter $\times$ height) cylinders, were performed to obtain the compressive strength, the elastic modulus and the Poisson's ratio, as shown in Figs. 3.8 and 3.9. To perform the compressive tests, a servo-hydraulic testing machine, Servosis Model MES-300-(3000 kN) resolution 0.1 kN, maximum non-linearity of transducer: $\pm 0.58\%$ of full scale was used. The machine contains two steel plates exist to perform the compression guided by four columns, as shown in Fig. 3.8, Fig. 3.10. The tools and the necessary extensometer were adapted to the machine according to the characteristics of each type of test. This machine applies a compression force perpendicular to the base of the cylinder, in the cylinder axis direction, to achieve the specimen damage.

Fig. 3.8 Compressive test set up.

Fig. 3.9 Mechanism of modulus test

As for dynamometry that measures the load applied in the tests, the machine uses the following pressure sensors:

- HBM pressure sensor. 500B P8AP ± 3000 kN, with a sensitivity of 2 mV/V $\pm 2\%$ and a standard deviation of 0.3% of its characteristic curve.
- HBM pressure sensor. P8AP 50B ± 300 kN, with a sensitivity of 2 mV/V $\pm 2\%$ and a standard deviation of 0.3% of its characteristic curve.

Fig. 3.10 Experimental set-up for compressive tests (Servosis Machine of 3000 kN).

Extensometers: Deformation and length were measured with the following extensometers, as shown in Fig. 3.9:

- Two resistive extensometers type Solartron AX/I/S to measure the axial deformation for cylindrical specimens. These extensometers are calibrated to a distance of ± 1 mm, with a sensitivity of 213 mV /v /mm and a standard deviation of 0.15% of its characteristic curve. We have used them to measure the axial deformation of the modulus and

Poisson's ratio test. For that purpose, we have adapted them to the cylindrical specimens. The serial of both extensometers is averaged by the machine itself.

- Two inductive extensometers, type Solartron AX/I/S, are used to measure the radial deformation of cylindrical specimens with a deviation of 0.22% of its characteristic curve. These devices are calibrated to a distance of ± 1 mm and have a sensitivity of 213 mV/v/mm. We have used them to measure the radial deformation of the modulus and Poisson's ratio test, for which we have adapted them to the cylindrical specimens by fitting two metal rings fixed to the central part of the sample. Both extensometers signals are averaged by the machine itself.
- Data Acquisition: data acquisition and control of the machine has been carried out using Servosis PCD 1065-W software.

3.3.1.2 Results

The values of mechanical properties can be determined by cylinder specimens for every concrete type. Compressive tests were conducted on concrete mixes at an age around 7 months by using a servo-hydraulic testing machine. These tests were followed ASTM (C39-02, 2002) and ASTM (C469/C469M, 2014) standards with four cylinders of 150 mm $\times$ 300 mm (diameter $\times$ height) for each type of SFRC. As shown in Table 3.4, the characterizations taken from compression test on cylinder specimens which have been determined are the peak load, the compressive strength, the elastic modulus and the Poisson's ratio. From the table we can note that the addition of steel fibers does not change significantly the compressive strength and the modulus of elasticity (Pająk & Ponikiewski, 2013).

Appendix A shows all detailed recording sheet related to Compressive tests for all types. Every sheet contains this information:

- Specimen name
- Specimen age (data of batch and the date of test)
- Test Parameters: Control and velocity
- Specimen outline, weight and dimensions
- Mechanical properties, such as peak load (P_{max}), compressive strength (f_c), elastic modulus (E) and Poisson's ratio (ν), and density of the material (ρ).
- Load – Displacement curve

Table 3.4 Mechanical properties of SFRC.

	D (mm)	L (mm)	ρ (Kg/m ³)	E (MPa)	ν	P_{max} (kN)	f_c (MPa)
A 1	149.42	296.64	2,366.27	-	-	1,959.51	111.75
A 2	150.16	297.73	2,361.59	43.96	0.169	1,993.81	112.58
A 3	150.12	297.47	2,359.80	46.16	0.178	1,965.88	111.07
A 4	150.08	297.94	2,361.73	49.07	0.196	1,973.62	111.57
A (Mean)	(149.95)	(297.45)	(2,362.35)	(46.40)	(0.181)	(1,973.21)	(111.74)
S.D.	0.35	0.57	2.76	2.56	0.01	14.90	0.63
B 1	150.21	298.03	2,369.83	-	-	2,010.67	113.46
B 2	150.71	298.19	2,346.45	42.85	0.166	1,858.28	104.17
B 3	150.56	298.11	2,364.25	46.55	0.175	2,060.65	115.74
B 4	150.62	296.92	2,460.20	46.22	0.160	2,042.03	114.61
B (Mean)	(150.53)	(297.81)	(2,385.18)	(45.21)	(0.167)	(1,992.78)	(112.00)
S.D.	0.22	0.60	51.00	2.05	0.01	92.09	5.30
C 1	149.57	297.55	2,421.91	-	-	2,044.87	116.38
C 2	150.15	296.82	2,405.48	48.92	0.180	2,023.60	114.29
C 3	150.90	297.59	2,394.63	44.71	0.171	2,077.80	116.19
C 4	149.55	297.75	2,409.60	44.05	0.161	1,949.12	110.97
C (Mean)	(150.04)	(297.43)	(2,407.91)	(45.89)	(0.171)	(2,023.38)	(114.46)
S.D.	0.63	0.41	11.27	2.65	0.01	54.58	2.51

3.3.2 Three-point bending test under low loading rates

These tests were conducted to study the influence of loading rates on the fracture energy of concrete. Three-point bending tests were performed on notched beams. The weight of the standard notched beam (around 40 kg in weight) as indicated in the recommendations of the RILEM TC 162- TDF committee (Rilem, 2002a) and EN 14651 standard (EN, 2005a) is not easy to handle in the lab. For this reason, the small notched beam suggested by (Giaccio, Tobes, & Zerbino, 2008) to measure design parameters of fiber-reinforced concrete (FRC), was used for our prismatic specimens of cross section 100×100 mm. The results of the tests due to the post-peak parameters of these specimens as proved by the researcher weather equivalent or residual strength did not have any difference from the standard one considering a significance level of 10%. We used a factor matching this standard as discussed in reference (Giaccio et al., 2008). The initial notch-depth ratio a/ D was approximately $1/ 6$, and the span S was fixed at 333 mm. The notches were cut at mid span using a circular diamond cutting saw; they were 16.67 mm deep. The actual notch depths in the individual beams were measured just before the test Fig. 3.11.

Fig. 3.11 Schematic diagram of the specimen (units in mm)

For these tests we used a servo-hydraulic Servosis frame machine with 1000 kN capacity. All load-displacement curves were drawn. Curves stopped at 3 mm and the other continued more, so we applied our calculations up to 3 mm to make a fair comparison.

As mentioned, the three point bending in this test followed RILEM recommendations (Rilem, 2002a), so we used a factor to match this standard as discussed in reference (Giaccio et al., 2008). So, we depend on a factor Eq. (3.1):

$$Factor = \frac{150 \text{ mm (h in RILEM)}}{100 \text{ mm (h in our spacemen)}} = 1.5 \quad \text{Eq. (3.1)}$$

The tensile strength of the specimen in non-cracked central cross section: Limit of proportionality ($f_{ct,L}$), considering the maximum load F_L as the maximum load taken from CMOD between 0 and 0.05 mm, assuming that the stress distribution along the depth of the beam is linear.

Limit of proportionality F_L : it is the highest value of the load in the interval (δ or CMOD) of 0.05 mm with our factor 0.033 mm. Also, equivalent tensile strengths ($f_{eq,1}, f_{eq,2}$) are evaluated for two deflection limits: $\delta_2 = \delta_L + 0.65$ mm and $\delta_3 = \delta_L + 2.65$ mm, where δ_L the deflection corresponds to the highest value of load in the interval of 0.05 mm (δ_L).

The residual flexural tensile strength: Fictitious strength at the bottom of the crack, in the central cross section submitted to a load F_j ($j = 1, 2, 3$ and 4) where f_{R1} Corresponds to the load

at CMOD 0.5 mm, with our factor CMOD will be 0.33 mm, f_{R2} Corresponds to the load at CMOD 1.5 mm, with our factor CMOD will be 1 mm, f_{R3} Corresponds to the load at CMOD 2.5 mm, with our factor CMOD will be 1.67 mm and f_{R4} Corresponds to the load at CMOD 3.5 mm, with our factor CMOD will be 2.33 mm. These tensile strengths are calculated from the Eq. (3.2):

$$f = \frac{3FS}{2BD^2} \quad \text{Eq. (3.2)}$$

Where F is the corresponding F_L or F_j suggested by the RILEM recommendations (Rilem, 2002a); S is the span B and D are the width and depth of the central cross-section, respectively. Furthermore, the flexural tensile strength f_t corresponding to the peak load F_L , not suggested by the code, was also calculated. Fig. 3.12 shows the Stress - CMOD curve marking the points of flexural strengths corresponding to their CMOD values.

Fig. 3.12 Typical stress - CMOD curve.

3.3.2.1 Equipments used

In order to study the fracture behavior of the SFRC, three-point bending tests were conducted on prismatic specimens by using a Servo-hydraulic machine (INSTRON Company) coupled to a robust frame from (Servosis Company).

The three-point bending tests were performed, as shown in Fig. 3.13, which shows a photo of the experimental set-up. A rigid steel beam was attached to the floor of the laboratory and two supports were installed to sustain the slab over the beam. A loading bar was set over the specimen and joined to the actuator through a rigid profiled beam and some platens. The alignment of the supports and the loading line was checked when installing each specimen. The two supports were free to move in the axis of the frame.

Fig. 3.13 Photo of the experimental set-up of the servo-hydraulic machine.

To measure the loading point displacement, we use two LVDTs (linear variable differential transducers), model WA/50 mm, HBM Company, measuring range ± 25.0 mm, resolution 0.002 mm, as shown in Fig. 3.14 (<http://www.solartronmetrology.com/products/analogue-gauging/analogue-gauging-probe-ax.aspx>).

Fig. 3.14 LVDT (Linear Variable Deferential Transducers)

Dynamic Extensometer 13 mm GL ± 3.5 mm travel placed in horizontal position, were used for the crack mouth open displacement (CMOD) readings, Max. Strain 20% Full Scale Range ± 3.6 mm with Frequency Response 60 Hz Fig. 3.15.

Fig. 3.15 Dynamic extensometer to measure CMOD

An adaptor is a connector used to join parts or devices that have different sizes, designs, etc. which in our test are (LVDT and Dynamic Extensometer), that allowing them to be fitted or to work together Fig. 3.16.

Fig. 3.16 Adaptor

To measure the load, the machine that uses a load cell which is a transducer which converts a value of force into a proportional electrical signal (Dynamic Rating ± 250 kN, INSTRON Company, weight 23kg) was adopted Fig. 3.17.

Fig. 3.17 Load cell, LVDTs for measuring CMOD and loading point displacement

3.3.2.2 Results

In this tests we calculate fracture parameters, such as the fracture energy (G_F), and peak load (P_{MAX}), are shown which obtained by three point bending tests. The object is to check which batch produces less scatter results on the flexural residual strengths obtained from three point bending tests following RILEM recommendations (Rilem, 2002a). Moreover, in order to evaluate the mechanical behavior of the fiber reinforced concretes, various flexural strengths of prismatic specimens are determined from the Load CMOD curve.

Prisms (3 types) from the chosen suitable mix composition were tested, with four specimens at each loading rate velocity. Two loading rate quasi-static level (2.20×10^{-3} mm/s) and rate dependent (2.20×10^1 mm/s), were applied. Load versus Displacement relation at quasi-static and static loading rates for all specimens were shown in Figs. 3.18, 3.19, 3.20, 3.21, 3.22 and 3.23 for loading rates with all types A,B and C respectively using a servo-hydraulic machine.

Fig. 3.18 Load displacement relation under $2.20 \times 10^{-3} \text{ mm/s}$ (A type)

Fig. 3.19 Load displacement relation under $2.20 \times 10^{-3} \text{ mm/s}$ (B type)

Fig. 3.20 Load displacement relation under $2.20 \times 10^{-3} \text{ mm/s}$ (C type)

As we can note from these results related to $2.20 \times 10^{-3} \text{ mm/s}$ that show figures for steel concrete type (A) has more similarity in behavior before and after peak load. On the other hand, when we add more hooked end fibers in steel concrete type (B) and (C), we can note different behaviors in prismatic beams tests. The behaviors of the beams are due to the bond between the mixed material and steel fibers which depend on the orientation of the fibers and the volume fracture on the crack surface. Moreover, these figures proved that the increase of peak load is related the increase in fibers.

Fig. 3.21 Load displacement relation under $2.20 \times 10^1 \text{ mm/s}$ (A type)

Fig. 3.22 Load displacement relation under 2.20×10^1 mm/s (B type)

Fig. 3.23 Load displacement relation under 2.20×10^1 mm/s (C type)

From the test Figures related to 2.20×10^1 mm/s, we can note that the steel concrete types (A) and (B) have more similarities in behavior before and after peak load though steel fiber concrete type (B) contains hooked end fibers. On the other hand, we can note some different behaviors in prismatic beams tests in type (C), that contains more hooked end fibers. In the end, these figures prove that the peak of load is not only affected by the increase the number of fibers but also increase in the loading rate.

Table 3.5 shows peak load values in each case and also the number of fibers in the crack section value of total fracture energy is calculated from Eq. (3.3) which is more accurate than the one recommended by (Recommendation, 1985) taking into account the self weight of the beam.

$$G_f = \frac{W_0}{B(D-a)} + \frac{mg\left(1-\frac{L}{2s}\right)\delta_s}{B(D-a)} \quad \text{Eq. (3.3)}$$

Where W_0 , B , D , a , S , L , m , δ_s , and g are the area under the experimental load displacement curve, width, depth, notch, span, length, mass, specified deflection of the beam ($\delta_s = 3$ mm) and gravitational acceleration, respectively.

Table 3.5 Quasi-static and static loading rates all results obtained (Servosis frame machine)

PRISMAS	Loading Rate	Peak load kN	ft MPa	B	D	L	a	weghit kg	Energy Area Wo untill 3mm J	Gf1	Gf2	Gf Gf1+Gf2 J/m2	density kg/m3	Number of fibers
PA1	0.0022 mm/s	11.87	8.62	99.85	100.61	451.00	17.60	10.73	24.59	2966.74	12.28	2979.03	2367.84	337
PA2		12.60	8.82	101.93	100.73	451.00	17.05	10.78	29.34	3440.06	12.00	3452.06	2328.22	399
PA3		11.99	8.38	102.80	99.91	450.00	16.53	10.88	24.29	2834.13	12.11	2846.24	2355.14	349
PA4		11.37	8.02	101.36	100.75	450.00	17.18	10.81	25.26	2982.18	12.17	2994.35	2353.22	406
mean		11.96	8.46	101.48	100.50	450.50	17.09	10.80	25.87	3055.78	12.14	3067.92	2351.11	372.63
S.D.		0.50	0.34	1.24	0.40	0.58	0.44	0.07	2.35	264.67	0.12	264.59	16.58	34.93
PA6	22 mm/s	15.54	10.97	101.58	100.74	451.00	17.26	10.77	25.98	3063.84	12.06	3075.90	2334.05	326
PA7		16.04	11.28	102.49	100.00	450.00	16.73	10.85	33.74	3954.03	12.12	3966.15	2352.33	435
PA8		16.03	11.59	99.73	99.38	451.00	16.15	10.54	34.27	4129.03	12.06	4141.08	2358.54	381
PA9		15.20	10.69	101.87	99.51	452.00	16.01	10.77	33.33	3918.42	11.96	3930.38	2350.20	397
mean		15.70	11.13	101.42	99.91	451.00	16.54	10.73	31.83	3766.33	12.05	3778.38	2348.78	384.63
S.D.		0.41	0.39	1.19	0.62	0.82	0.58	0.13	3.92	477.29	0.07	477.28	10.44	45.06
PB1	0.0022 mm/s	25.26	17.53	101.43	102.23	449.00	18.00	10.85	51.11	5982.89	12.17	5995.06	2331.42	534
PB2		23.76	17.34	102.40	100.10	451.00	18.36	10.89	51.31	6129.85	12.34	6142.19	2354.82	605
PB3		17.63	12.32	101.35	101.58	452.00	17.60	10.87	33.86	3978.64	12.06	3990.70	2335.30	486
PB4		16.76	11.77	100.52	101.72	451.00	17.60	10.77	34.95	4133.70	12.09	4145.78	2335.18	451
mean		20.85	14.74	101.42	101.41	450.75	17.89	10.84	42.81	5056.27	12.17	5068.43	2339.18	518.75
S.D.		4.28	3.12	0.77	0.91	1.26	0.36	0.05	9.71	1158.11	0.13	1158.22	10.58	66.62
PB6	22 mm/s	21.61	15.24	101.14	100.08	450.00	16.39	10.69	42.10	4973.57	12.04	4985.61	2346.90	400
PB7		29.44	20.99	100.12	101.85	450.00	18.20	10.59	54.80	6543.97	12.06	6556.03	2307.61	549
PB8		29.14	20.27	101.91	102.64	449.00	18.71	10.96	55.14	6446.76	12.27	6459.03	2333.31	588
PB9		27.21	19.21	99.19	101.74	451.00	17.28	10.61	55.39	6612.41	12.02	6624.44	2331.87	611
mean		26.85	18.93	100.59	101.57	450.00	17.64	10.71	51.86	6144.18	12.10	6156.28	2329.92	536.63
S.D.		3.63	2.57	1.19	1.07	0.82	1.02	0.17	6.51	783.36	0.12	783.39	16.34	94.98
PC1	0.0022 mm/s	33.64	23.24	100.78	101.15	451.00	16.46	10.93	78.07	9146.98	12.16	9159.14	2377.84	720
PC2		39.57	27.37	99.89	101.68	450.00	16.65	11.01	100.56	11840.03	12.36	11852.39	2409.45	935
PC3		32.47	23.31	99.54	100.87	450.00	17.25	10.80	80.44	9664.55	12.37	9676.92	2390.74	784
PC4		39.54	27.79	100.73	100.51	451.00	16.51	10.98	100.89	11924.26	12.32	11936.58	2405.68	962
mean		36.31	25.43	100.23	101.05	450.50	16.72	10.93	89.99	10643.96	12.30	10656.26	2395.93	849.88
S.D.		3.78	2.49	0.62	0.49	0.58	0.37	0.09	12.43	1445.68	0.10	1445.73	14.51	117.10
PC6	22 mm/s	49.46	35.05	100.94	100.53	451.00	16.96	10.98	119.99	14224.45	12.36	14236.81	2400.19	827
PC7		35.15	25.07	100.53	100.30	451.00	16.84	10.82	74.24	8848.47	12.24	8860.71	2379.45	637
PC8		46.54	32.97	100.51	100.98	451.00	17.21	10.98	110.81	13161.55	12.38	13173.93	2399.28	808
PC9		44.76	32.04	100.02	100.75	451.00	17.23	10.93	103.07	12338.29	12.41	12350.70	2404.10	761
mean		43.98	31.28	100.50	100.64	451.00	17.06	10.93	102.03	12143.19	12.35	12155.54	2395.76	757.88
S.D.		6.20	4.33	0.37	0.29	0.00	0.19	0.08	19.77	2328.23	0.08	2328.29	11.07	85.60

From this table, the fracture energy was increased by increasing the fiber contents and by increasing the loading rate. Also, the number of fibers in the crack cross section was increased with an increase of fiber contents regardless of loading rates. In the end, it can be observed that the mix composition C type has both the best mechanical properties, and the highest residual flexural strength.

In section A (Appendix), a sheet with detailed recording for each of the flexural tests conducted for the SFRC can be found. This recording sheet consists of the following information:

- Specimen name
- Specimen age (date of batch and the test date)

- Test Parameters: Control and velocity
- Specimen outline, weight and dimensions
- Peak load (P_{max}), fracture energy up to 3 mm displacement (G_f), number of fibers in the fracture surface
- Photo of the crack side view and photo of the crack surface morphology
- Load – Displacement curve.

3.3.2.3 Correcting the initial zone of the curve $P - \delta$

For all the experimental curves, first we had to eliminate each of the initial non-linear section recorded by the data acquisition system. This non-linear part corresponds to the adaptation of the hinge and the compression plates of the machine on the specimen. Fig. 3.24 represents this initial section which is to be removed in the case of the curve $P - \delta$. This accommodation involves energy consumption by local damage that should not be reflected in the value of G_f . This initial zone is a curve with an increasing slope which is stabilized in a straight line as the test proceeds; the correction consists in taking the straight section and stabilizing it as an initial straight curve. Then, the zero abscissa entire record must be recorded by subtracting the value obtained in the initial intersection of the line with the horizontal axis, δ_0 , according to the Eq. (3.4):

$$\delta = \delta_m - \delta_0 \quad \text{Eq. (3.4)}$$

Where δ_m is the original value of the displacement registry.

Fig. 3.24 Correction of the initial zone of a nonlinear curve $P - \delta$ (Mohamed, 2013)

3.3.2.4 Method of obtaining fracture energy (G_f)

The method of obtaining G_f can be summarized as follows:

1. Obtaining the curve $P - \delta$ of the three-point bending test of a prism specimen.
3. Making the correction from zero load.
3. Correlating the initial zone of $P - \delta$.

4. Measuring the area under the experimental curve of $P-\delta$ up to 3 mm.
5. Determining the fracture energy G_f by using the Eq. (3.3).

3.3.2.5 Determination of elastic modulus by using three point bending test

The recommendation of RILEM TC 89- FMT (S. Shah, 1990) which is related to plain concrete, was tried for our prismatic beams (FRC). The elastic modulus is calculated from Eqs. (3.5) and (3.6).

$$E = 6SaV_\beta(\alpha)/(C_iBD^2) \quad \text{Eq. (3.5)}$$

$$V_\beta(\alpha) = V_\beta\left(\frac{a}{D}\right) = 0.8 - 1.7\alpha + 2.4\alpha^2 + \frac{0.66}{(1-\alpha)^2} + (-0.04 - 0.58\alpha + 1.47\alpha^2) - 2.04\alpha^3 \quad \text{Eq. (3.6)}$$

Where C_i is the initial compliance determined from Load-CMOD curve and $V_\beta(\alpha)$ is a dimensionless shape function depending on β and α is the relative notch depth the rest parameters of the beam is defined in Fig. 3.24.

The comparison of measurements of elastic modulus from compressive test on cylinders following ASTM (C469/C469M, 2014) standard and the three-point bending test following the RILEM recommendation are shown in Table 3.6.

Table 3.6 Comparison of Elastic Modulus.

Concrete type	Elastic modulus (GPA)		Relative error
	From Cylinder	From Beam	
A	46.4 (3)	44.8 (2)	3.4 %
B	45.2 (2)	42.9 (3)	5.1 %
C	45.9 (3)	44.0 (4)	4.1 %

Note: values in parentheses are standard deviations.

In spite of the fact that the RILEM recommendation is for plain concrete, it is still valid for FRC. The table results show a good agreement between the two methods and the relative error is less than 7%.

3.3.3 Three-point bending tests under medium loading rates (impact tests) ($10^2 - 10^3$ mm/s)

Three-point bending tests at medium loading rates were carried out on notched beams. The hammer heights, impact velocity and the energy produced in these types of tests are shown in detail in Table 3.7.

Table 3.7 Drop hammer height for impact tests.

Hammer height (mm)	Impact velocity (mm/s)	Energy produced (J)
40	8.85×10^2	47
160	1.75×10^3	189
360	3.65×10^3	425

3.3.3.1 Equipments used

To perform the dynamic test we used a Drop-Weight Impact Machine (DWIM). For all test heights the corrected acceleration, due to the friction of the hammer with the columns, was $9.7 \pm 0.2 \text{ m/s}^2$. For the data acquisition system, the sample rate was established at 1 MHz. The drop-weight impact machine schematic diagram is shown in Fig. 3.25. The hammer was guided by the two columns of height 3.70 m and pre-stressed against the upper and lower 1.2-m-thick slabs of the floor. These columns push the lower slab through a 95 mm thick steel plate to make the supports for the specimens can be tied. Also, there is a very thin neoprene layer of 0.5 mm between the steel plate and the concrete floor to distribute the stresses transmitted to the floor as uniformly as possible. In this way, the floor serves as a very stiff reaction frame. Moreover, it can disperse the energy transmitted through the specimen and insure there is little or negligible movement. Besides, the operation of the machine produces no damage or disturbance whatsoever to the structure of the building containing the Laboratory, since the strong floor is a completely independent structure.

Fig. 3.25 Schematic diagram of the Drop-Weight Impact Machine (Mohamed, 2013)

This instrument contains the frame supports two 90 mm diameter guiding columns as rails. A hammer is raised to a predetermined height by a hoist and chain system, and then allowed to fall freely to hit the samples that are fixed onto two supports, as shown in Fig. 3.26. The ability of this instrument can provide a 2595 mm free drop height above the support.

Fig. 3.26 The drop-weight impact machine (Mohamed, 2013).

The instruments used in this test are:

- Force sensors: A piezoelectric force sensor (made by PCB Company, New York, NY, USA), Model 204C used in this instrument between the hammer tip and the specimen to measure the impact force with measurement range up to 177.92 kN. Moreover, two force sensors model 203B between the support and the specimen can measure up to 89 kN, used in this instrument to measure the reaction force. The settings of the sensors were also provided by the PCB Company, and the accuracy of sensors is 0.7 N.
- Accelerometers: The accelerometers that were set up along the length of the beam are piezoelectric sensors. Their resonant frequency was greater than 70 kHz. With a resolution of 0.1 g, the accelerometers can read up to ± 1000 g (where g is the Earth's gravitational acceleration). The settings of the accelerometers were provided by the PCB Company.
- Magnetic strip and magnetic sensor: the detection of the movement of the hammer was achieved by a magnetic strip mounted on the column and a magnetic sensor fixed on the side of the hammer (Model MSK 5000; SIKO GmbH, Kirchzarten, Germany), its resolution 0.001 mm.
- Optical fiber photoelectric sensor: The sensor optical fiber photoelectric sensor (Model: OMRON E3x-NA; Omron Electronics, Sidney, Australia) emits a 24 V signal, that triggers the data acquisition system when a plate fixed on the hammer passes by the light detector mounted on the frame.
- Data acquisition system: The data acquisition system is made up of two Tektronix oscilloscopes (Model: TDS3014B; Tektronix, Beaverton, Oregon, USA), two signal conditioners (Model: 482A22; Tektronix), one HBMMGC plus (Hottinger Baldwin Messtechnik GmbH, Berlin, Germany) device and a computer, as shown in Fig. 3.27.

Fig. 3.27 Data Acquisition System (DAS).

3.3.3.2 Results

Four prism specimens for every type of SFRC were tested under each loading rate from 40mm height which tested only one prism specimen for every type of concrete. Load versus Displacement relation at these rates related to the impact heights 40, 160 and 360 mm were shown in Figs. 3.28, 3.30, 3.31, 3.32, 3.33, 3.34 and 35 with all types A,B and C respectively using a drop weight impact machine. The Load that we are providing in these plots has been measured on the reaction supports. These forces were detected by two force sensors located between the supports and the specimen.

- **40 mm Height (loading rate: 8.85×10^2 mm/s)**

Fig. 3.28 Load displacement relation under 40mm height for all concrete types.

All specimens didn't break from that impact load with height 40 mm (loading rate: 8.85×10^2 mm/s) as shown in Fig. 3.28. However, the measurements of the peak reaction forces increased with the increasing of the content of fibers in the mixture.

The impact energy was too small to fracture the beam as shown in table 3.8. There is a crack that appeared in every concrete type due to its strength. In Fig 3.29, the concrete type (A) had an obvious main crack along the ligament, whereas the concrete type (B) had a crack till half of the beam only. The last one which was concrete type (C) the crack was very small and equal to one third of the beam height.

(a) Concrete type A

(b) Concrete type B

(c) Concrete type C

Fig. 3.29 Failure modes of SFRC beams for the impact height 40 mm for the three types

- **160 mm height (loading rate: 1.75×10^3 mm/s)**

Fig. 3.30 shows an almost similar behavior for all the four prism beams. In PA12 which have the highest number of fibers in the fracture crack surfaces, the two neighboring peak loads which return to interact between impact and react loads prevent the specimen reaction behavior to be the highest peak load.

Fig. 3.30 Load displacement relation under 160mm height (A Type)

PB15 in Fig. 3.31 had the highest peak load that probably is due to the number of fibers in the fracture crack surfaces and also number of the effect fibers which is the highest comparing with others.

Fig. 3.31 Load displacement relation under 160mm height (B Type)

PC 12 in Fig. 3.32 was the only specimen broken in these tests of 160 mm height that is justified by to the quantity of fibers in the fracture crack surface which was lower than others; and the number of effect fibers also lowers than the average of other specimens by more than 37 %.

Fig. 3.32 Load displacement relation under 160mm height (C Type)

- **360 mm Height (loading rate: 2.65×10^3 mm/s)**

Fig. 3.33 Load displacement relation under 360mm height (A Type)

PA18 in Fig. 3.33 had a highest peak load that probably comes down to the number of fibers in the fracture crack surfaces which is the highest on comparison with others.

Fig. 3.34 Load displacement relation under 360mm height (B Type)

PB17 in Fig. 3.34 had a highest peak load that probably is due to the number of fibers in the fracture crack surfaces which is the highest comparing it with the others and also the number of the effect fibers was high. On the other hand PB 16 had the lowest peak load because it had a very low effect fiber in the fracture crack surfaces, although it did not have the lowest of total number of

Fig. 3.35 Load displacement relation under 360mm height (C Type)

Two prism beams PC16 and PC19 shown in Fig. 3.35 were broken in this test but the other two did not break the return to the quantity of total fibers which were very high in the two specimens, also the quantity of the effect fibers in the unbroken specimens are double the broken beams.

Table 3.8 shows peak load values in each case and also the number of fibers in the crack section. It clearly shows the increment of the fracture energy and the reaction force related to increasing the fiber quantity in the same type of concrete and in rating load. Value of total fracture energy is calculated by Eq. (3.3).

Table 3.8 Medium loading rates all results obtained (Drop weight impact machine, Hammer weight 120.6 kg).

PRISMAS	Loading Rate	Reaction Force	Impact Force	R	B	D	L	a	weghit	Energy Area Wo untill 3mm	Gf1	Gf2	Gf Gf1+Gf2	Number of fibers
	mm/s	kN		Mpa			mm		kg	J		J/m2		
PA11	885 Drop Hight 40 mm	28.57	35.21	19.64	101.75	101.08	451.00	16.57	10.88	34.49	4010.87	12.01	4022.88	331
PA12	1750 Drop Hight 160 mm	49.82	49.30	35.21	101.83	100.27	451.00	16.95	10.87	70.82	8347.69	12.16	8359.85	413
PA13		62.27	102.82	42.73	102.07	100.54	451.00	16.09	10.89	77.08	8942.30	11.99	8954.29	212
PA14		52.75	57.75	36.65	101.45	100.09	452.00	15.91	10.73	63.92	7484.71	11.87	7496.58	363
PA15		55.68	64.79	38.77	101.34	100.68	450.00	16.55	10.76	58.45	6855.56	12.04	6867.59	356
MEAN		55.13	68.66	38.34	101.67	100.39	451.00	16.37	10.81	67.57	7907.56	12.01	7919.58	336.00
S.D.		5.33	23.63	3.27	0.34	0.27	0.82	0.47	0.08	8.11	921.94	0.12	921.96	86.48
PA16	2650 Drop Hight 360 mm	60.81	111.27	41.80	101.89	101.70	451.00	17.24	10.89	66.44	7720.73	12.01	7732.74	327
PA17		58.61	77.46	41.49	101.24	100.82	451.00	17.33	10.78	68.61	8117.65	12.11	8129.76	339
PA18		79.12	122.54	54.94	102.13	100.60	451.00	16.67	10.91	82.06	9573.47	12.08	9585.55	409
PA19		66.67	73.24	46.26	101.42	101.04	450.00	16.79	10.86	76.40	8941.67	12.12	8953.79	304
MEAN		66.30	96.13	46.12	101.67	101.04	450.75	17.01	10.86	73.37	8588.38	12.08	8600.46	344.75
S.D.		9.20	24.49	6.27	0.41	0.47	0.50	0.33	0.06	7.20	830.58	0.05	830.61	45.23
PB11	885 Drop Hight 40 mm	46.89	43.51	32.07	101.60	102.19	452.00	17.41	10.85	16.46	1911.25	11.90	1923.15	543.00
PB12	1750 Drop Hight 160 mm	48.35	57.04	33.76	100.95	102.98	450.00	18.79	10.98	75.89	8930.22	12.32	8942.54	475
PB13		49.82	63.38	34.62	101.24	102.53	450.00	18.27	10.96	104.68	12271.69	12.25	12283.94	495
PB14		57.14	54.93	40.39	101.45	101.10	449.00	17.63	10.78	89.42	10560.17	12.19	10572.36	548
PB15		78.39	59.15	52.66	99.74	101.42	451.00	15.08	10.66	105.85	12292.49	11.75	12304.24	551
mean		58.43	58.63	40.36	100.84	102.01	450.00	17.44	10.85	93.96	11013.64	12.13	11025.77	517.25
S.D.		13.85	3.61	8.72	0.76	0.89	0.82	1.64	0.15	14.18	1608.77	0.26	1608.61	38.14
PB16	2650 Drop Hight 360 mm	71.79	76.06	50.44	99.78	102.16	450.00	17.75	10.84	102.30	12146.94	12.27	12159.20	473
PB17		91.58	112.68	66.07	99.69	100.98	450.00	17.64	10.66	118.11	14217.44	12.24	14229.67	556
PB18		84.98	147.88	59.79	100.00	101.62	450.00	17.36	10.68	117.80	13981.34	12.08	13993.43	458
PB19		83.52	83.10	56.94	101.88	101.72	449.00	16.91	10.78	128.27	14846.66	11.96	14858.61	528
mean		82.97	104.93	58.31	100.34	101.62	449.75	17.42	10.74	116.62	13798.09	12.14	13810.23	503.75
S.D.		8.23	32.74	6.49	1.04	0.49	0.50	0.37	0.08	10.71	1159.78	0.14	1159.67	46.03
PC11	885 Drop Hight 40 mm	66.68	52.93	46.46	101.08	100.63	454.00	16.41	11.09	35.43	4161.50	12.19	4173.69	886
PC12	1750 Drop Hight 160 mm	93.04	70.42	66.89	99.14	100.43	451.00	16.71	10.76	115.92	13968.20	12.31	13980.51	599
PC13		111.36	74.65	79.10	99.42	100.53	452.00	16.43	10.83	138.25	16534.24	12.24	16546.48	695
PC14		101.10	63.38	72.67	100.83	100.96	452.00	17.94	10.97	144.62	17276.13	12.38	17288.50	725
PC15		84.27	63.38	57.86	101.22	102.74	450.00	17.96	11.29	146.22	17040.61	12.55	17053.15	782
MEAN		97.44	67.96	69.13	100.15	101.16	451.25	17.26	10.96	136.25	16204.79	12.37	16217.16	700.25
S.D.		11.55	5.56	9.02	1.02	1.07	0.96	0.80	0.24	13.98	1522.85	0.13	1522.91	76.54
PC16	2650 Drop Hight 360 mm	138.46	118.31	96.44	100.64	100.28	451.00	15.87	10.85	177.70	20917.62	12.12	20929.75	762
PC17		157.51	122.54	110.53	100.05	99.97	452.00	15.62	10.86	243.61	28867.34	12.16	28879.50	939
PC18		170.70	112.68	118.70	99.72	100.69	451.00	15.81	10.92	253.26	29924.70	12.24	29936.94	910
PC19		122.34	119.72	87.59	100.60	99.54	452.00	16.26	10.81	167.76	20025.17	12.19	20037.36	791
MEAN		147.25	118.31	103.32	100.25	100.12	451.50	15.89	10.86	210.59	24933.71	12.18	24945.89	850.50
S.D.		21.23	4.14	13.94	0.44	0.48	0.58	0.27	0.04	44.07	5183.50	0.05	5183.53	87.07

Note: P11 prismatic w_0 stop less than 1 mm

In section A (Appendix), a detailed of recording sheet for each of the flexural tests conducted for the SFRC can be found. This recording sheet consists of the following information:

- Specimen name
- Specimen age (date of batch and the test date)
- Test Parameters: Control and velocity
- Specimen outline, weight and dimensions
- Peak load (P_{max}), fracture energy up to 3 mm displacement (G_F), number of fibers in the fracture surface
- Photo of the crack side view and photo of the crack surface morphology

- Load – Displacement curve.

3.3.3.3 Calculation of displacement

We follow (Zhang, Ruiz, Yu, & Tarifa, 2009) assumption which is summarized in “that the hammer displacement was essentially equal to the displacement of the specimen at the loading point, the velocity and the loading point displacement could be evaluated using the following equations. Eq. (3.7) shows the initial impact velocity of the hammer” as:

$$\dot{u}_h(0) = \sqrt{2aH} \quad \text{Eq. (3.7)}$$

Where (a) is the corrected gravitational acceleration ($9.7 \pm 0.2 \text{ m/s}^2$) and (H) stands for the hammer drop height. Also, the velocity of the hammer can be obtained during the impact process using Eq. (3.8).

$$\dot{u}_h(t) = \dot{u}_h(0) + \int_0^t \ddot{u}(t) dt \quad \text{Eq. (3.8)}$$

Where $\ddot{u}(t)$ is the acceleration recorded by the accelerometer attached to the hammer; then, the displacement of the hammer $u(t)$. The loading point displacement of the specimen can be calculated by Eq. (3.9).

$$u_h(t) = \int_0^t \left[\dot{u}_h(0) + \int_0^t \ddot{u}(t) dt \right] dt \quad \text{Eq. (3.9)}$$

From this equation which used in our work, the loading point displacement was obtained when the accelerometer hit the hammer. For this to be true, we assumed that the loading point displacement of the specimen was identical to that of the hammer. Both loading point displacement capture methods were used in our tests to verify their assumption (Zhang et al., 2009).

There is no acceptable standard under dynamic load conditions to measure the fracture energy. In our thesis, we follow the hypothesis developed by (N Banthia, Mindess, Bentur, & Pigeon, 1989) to measure the fracture energy consumed of the fracture surface of a prismatic specimen in a three point bending test.

Because the specimen was not completely broken during the test, the area under the load versus displacement at mid-span curve up to a specified deflection were used in the evaluation of the fracture energy (G_f).

3.4 Analysis of results

From results we concluded by different tests and with different concretes the researcher light on critical reasons effect on calculations.

3.4.1 Loading rate effect on SFRC mechanical properties

In this part of the research we studied the fracture behavior of steel fiber-reinforced concrete specimens at a wide range of loading rates. The main properties in this study are: Peak load (P_{max}) and fracture energy (G_f).

3.4.1.1 Loading rate Effect on the peak load (P_{max}) and DIF

Table 3.9 shows a sample of the beam test behaviors due to all loading rates in impact instrument test of every type of concretes. The relation in the figure between typical impact and reaction forces versus time curves. The time intervals between the starting points of the impact force and the reaction force for the concrete type A are 240, 172, and 168 μ s, and for type B are 232, 220 and 172 μ s, and for type C are 246, 228 and 228 μ s corresponding to loading rates 8.85×10^2 , 1.75×10^3 and 2.65×10^3 mm/s respectively. Thus, there is an apparent time delay. So, we can conclude that the time delay decreases with an increase in loading rate. Also, the time delay decreases with an increase of quantity of fibers.

Table 3.9 Typical impact and reaction forces versus time for all types of concrete under each impact loading rate.

Table 3.10 shown the comparison between impact and reaction forces versus displacement curves. The reaction force was shifted to the initial time of the impact force to compare between the slope of the impact and the reaction, and the peak load between them.

Table 3.10 Comparison of load versus displacement at all loading rates with every type of concrete.

For the tests under drop height 40 mm (loading rate: 8.85×10^2 mm/s), the reaction force slope was greater than the impact one for all types. The peak load for the impact force is greater for the lowest strength type (A) than the reaction force, whereas for the B type and C type the reaction force is greater in peak load. The difference between impact peak and reaction peak

loads increases with an increase in strength in the concrete type because the reaction peak loads increase in relation to the same trend.

Furthermore, for the tests under drop heights 160 mm (loading rate: 1.75×10^3 mm/s). For type (A) and (B) all beams were fractured and broken completely. For beams related to part (C) all beams were deformed and fractured clearly but not completely broken except PC12 which was broken completely. As shown in table 3.10 the slope for impact load was greater than the react one in the lowest concrete fibers contents. On other hand for the others the slopes were almost same for the two loads. The peak load for the impact force is greater for the lowest strength type (A) than the reaction force, whereas for C type the reaction force is greater in peak load and the (B) one has the behavior of peaks between these types.

For the test under drop heights 360 mm (loading rate: 2.65×10^3 mm/s). The slope of the impact load related to type (A) was quite larger than the slope of reaction load. On the other hand, the slope of the impact and react load in type (B) and the highest strength concrete (C) were almost same. Furthermore, the impact peak relations have the same trend as in the previous loading rate. The impact peak loads for type (A) are greater than the reaction load and for the type (C), the impact load was lower than the reaction load in peak. On the other hand, the (B) type has the results of peaks between (A) and (C) types.

We can note that there is a difference between these types of concrete in peak loads especially between the lowest and the highest strength ones. In the lowest one (Type A), all results give us the peak of impact load higher than the peak of the reaction force and the result of the C type was completely reversed. From my opinion that because of the increase in redundancies when the strength of the concrete is increased, the results show us that the reaction peak load is increasing more in concrete type (C) than the others.

Fig. 3.36 shows the comparison of the typical load displacement curves at different loading rates from 10^{-3} to 10^3 rates for each type of concretes. From the figures shown, we can see that the increase in loading rates leads to increase in the peak load. However, conclusive variation in the stiffness of the beam cannot be appreciated. This is because of the sensitivity of the elastic flexibility of the beam to the boundary conditions during the application of the concentrated load when the concentrated load is being applied, as mentioned by (Planas, Guinea, & Elices, 1994).

Fig. 3.36 Load-displacement curves at different loading rates.

The dynamic increase factor (DIF) is determined by the ratios of the flexural strength (R) and the fracture energy (G_F) to their corresponding quasi-static values for each type of SFRC due to the lowest loading rate (2.20×10^{-3} mm/s). All these values were shown in Table 3.11 for all the

experimental results, data in parenthesis are the standard deviation. (H) is the drop height of the hammer for the impact tests.

It can be observed from Fig. 3.37 and the table 3.11 that the flexural strength increases together with an increase in loading rates for each type of SFRC. From Fig. 3.37 we observed that the tendency is too small under low loading rates, while it is marked under medium loading rates instead. For instance, DIF for three different concretes is around 1.3, i.e., the enhancement of flexural strength is around 30% at loading rate 2.20×10^1 mm/s. However, under impact conditions, the rate effect is more pronounced. DIF reaches the range from 2.73 to 5.45. Clearly, the concrete less sensitive to the loading rate was the high performance SFRCs (PC and PB); for instance, at loading rate 2.66×10^3 , DIF of flexural strength for concrete PA is 5.45, while they are 3.94 and 4.06 for Concretes PB and PC, respectively. This study gives us similar results tendencies as (N Bantia et al., 1989) reported. Furthermore, a prediction for each concrete of the rate effect on the flexural strength is derived from the experimental results as shown in Eqs.(3.10), (3.11) and (3.12), the correlation coefficient is over 97%.

$$PA: DIF_R = 1 + k \left(\frac{\dot{\delta}}{\dot{\delta}_0} \right)^n = 1 + 0.048 \left(\frac{\dot{\delta}}{\dot{\delta}_0} \right)^{0.58}, \text{ for } \dot{\delta} \text{ in mm/s} \quad Eq. (3.10)$$

$$PB: DIF_R = 1 + k \left(\frac{\dot{\delta}}{\dot{\delta}_0} \right)^n = 1 + 0.0073 \left(\frac{\dot{\delta}}{\dot{\delta}_0} \right)^{0.77} \quad Eq. (3.11)$$

$$PC: DIF_R = 1 + k \left(\frac{\dot{\delta}}{\dot{\delta}_0} \right)^n = 1 + 0.0016 \left(\frac{\dot{\delta}}{\dot{\delta}_0} \right)^{0.95} \quad Eq. (3.12)$$

Where $\dot{\delta}$ is the loading rate in mm/s and $\dot{\delta}_0$ is set as 1 mm/s. Thus, the adjustment parameters k and n are without units. The equations are useful to predict efficiently the rate effect on the flexural strength and can assist when numerical simulations are performed. Since both steel and concrete are strain-rate sensitive materials, the pull-out behavior of fibers can be expected to be affected by the rate at which the pull-out occurs.

Fig. 3.37 Load-displacement curves at different loading rates.

Table 3.11 Experimental results at different loading rates

Concrete type	H (mm)	δ (mm/s)	P_{max} (kN)	R (MPa)	DIF for R	G_F (N/m)	DIF for G_F
PA	-	2.20×10^{-3}	11.96 (0.50)	8.46 (0.34)	1	3067.92 (264.59)	1
	-	2.20×10^1	15.70 (0.41)	11.13 (0.39)	1.32	3778.38 (477.28)	1.23
	160	1.77×10^3	55.13 (5.33)	38.34 (3.34)	4.52	7919.58 (921.96)	2.58
	360	2.66×10^3	66.30 (9.20)	46.12 (6.27)	5.45	8600.46 (830.61)	2.80
PB	-	2.20×10^{-3}	20.85 (4.28)	14.74 (3.12)	1	5068.43 (1158.22)	1
	-	2.20×10^1	26.85 (3.63)	18.93 (2.57)	1.28	6156.28 (783.39)	1.21
	160	1.77×10^3	58.43 (13.85)	40.36 (8.72)	2.86	11025.77 (1608.61)	2.18
	360	2.66×10^3	82.97 (8.23)	58.31 (6.49)	3.94	13810.23 (1159.67)	2.72
PC	-	2.20×10^{-3}	36.31 (3.78)	25.43 (2.49)	1	10656.26 (1445.73)	1
	-	2.20×10^1	43.98 (6.20)	31.28 (4.33)	1.23	12155.54 (2328.29)	1.14
	160	1.77×10^3	97.44 (11.55)	69.13 (9.02)	2.73	16217.16 (1522.91)	1.52
	360	2.66×10^3	147.25 (21.23)	103.32 (13.49)	4.06	24945.89 (5183.53)	2.34

Note: values in parentheses are standard deviations.

3.4.1.2 Loading rate effect on the fracture energy (G_f) and DIF

The loading rate effect on the fracture energy is also shown in Fig. 3.38, which had similar trend as that of the peak load, i.e., the tendency under low loading rate was moderate, while under medium loading rates it is dramatic. With the correlation coefficient at over 92% the following equations were used to represent this behavior; see Eqs 3.13, 3.14 and 3.15.

$$\text{PA: } DIF_{GF} = 1 + m \left(\frac{\dot{\delta}}{\dot{\delta}_0} \right)^r = 1 + 0.28 \left(\frac{\dot{\delta}}{\dot{\delta}_0} \right)^{0.24}, \text{ for } \delta \text{ in mm/s} \quad \text{Eq. (3.13)}$$

$$\text{PB: } DIF_{GF} = 1 + m \left(\frac{\dot{\delta}}{\dot{\delta}_0} \right)^r = 1 + 0.24 \left(\frac{\dot{\delta}}{\dot{\delta}_0} \right)^{0.25} \quad \text{Eq. (3.14)}$$

$$\text{PC: } DIF_{GF} = 1 + m \left(\frac{\dot{\delta}}{\dot{\delta}_0} \right)^r = 1 + 0.30 \left(\frac{\dot{\delta}}{\dot{\delta}_0} \right)^{0.16} \quad \text{Eq. (3.15)}$$

Where coefficients m and r are parameter settings without units due to the fact that $\dot{\delta}_0$ is set as 1 mm/s as mentioned previously. In addition, the fracture behavior of the SFRC rate effect is slight in the low rate range.

Fig. 3.38 Load-displacement curves at different loading rates.

In our case, the rate effect on the fracture behavior of SFRC is slight in the low rate range as well. The presence of free water in gaps and porous structures in the samples as mentioned by (Ruiz et al., 2011), was considered to be caused by viscous effects mainly and the pull-out behavior between the fiber and the matrix. Under impact loading rates, the rate effect is obvious. However, the inertia of the whole specimen contributes significantly to the strength gain, and the micro-inertia around the crack tip makes fracture propagation more difficult (Bischoff & Perry, 1991; Brara & Klepaczko, 2007; Rossi & Toutlemonde, 1996; Vegt, Breugel, & Weerheijm, 2007; Weerheijm & Van Doormaal, 2007). On the other hand, the rate effect is more pronounced because of the presence of hooked-end steel (long) fibers embedded in the concrete sample which can withstand a higher load under impact and the pull-out energy is also greater, (Nemkumar Banthia & Trottier, 1991).

Fig. 3.39 Compare between DIF with P_{max} and G_F

Moreover, the high performance HPFRCs (PC and PB) is less sensitive to loading rate than the conventional SFRC (PA) as well, but not like the rate effect on the flexural strength, that shows a large difference, DIF of fracture energy is just from 1.52 to 2.80 under impact loading conditions Fig. 3.39.

3.4.2 Comparing between different concretes in load-deflection and load-CMOD curves

The mean values and standard deviation of the equivalent tensile strength ($f_{eq,1}, f_{eq,2}$) and the residual flexural tensile strength shown in Table 3.12.

Table 3.12 Experimental results of three-point bending test on prismatic specimens.

Concrete type	f_{R1}	f_{R2}	f_{R3}	f_{R4}	$f_{ct,l}$	f_{eq2}	f_{eq3}
PA	8.02 (0.4)	7.25 (0.7)	5.95 (0.6)	4.92 (0.5)	7.75 (0.2)	8.01 (0.5)	5.52 (0.7)
PB	13.32 (4.4)	12.79 (2.6)	8.82 (3.0)	8.24 (2.5)	9.12 (1.0)	14.16 (3.2)	9.23 (2.6)
PC	23.82 (2.2)	24.85 (3.0)	23.17 (3.4)	20.90 (3.6)	13.28 (0.8)	24.02 (2.6)	21.99 (5.0)

The mean values and standard deviations of the post-peak parameters evaluated according to RILEM TC 162-TDF (Rilem, 2002a) and EN 14651 standard (EN, 2005a) from the load-deflection and load-CMOD curves are presented in Table 3.12. The significant point of this table is that there is an increase of every parameter when you increase the quantity of fibers. The table also reveals that $f_{eq,3}$ is slightly lower than $f_{eq,2}$ in concrete type (C) whereas the difference is bigger in type (A). That mean the energy absorption capacity of the designed SFRC was maintained (about 8% the decreasing percentage) in type (C) up to a deflection δ_3 and less maintained in the other one (more than 30% the decreasing percentage).

Table 3.13 Residual loads in every type with the rate different between them.

CMOD Type	0.5	Increase rate (%)	1.5	Increase rate (%)	2.5	Increase rate (%)	3.5	Increase rate (%)
PA	8.02	-	7.25	-	5.95	-	4.92	-
PB	14.11	76	13.63	74	9.96	67	8.13	65
PC	23.82	197	24.85	242	23.17	222	20.90	324

The increase rate of the residual load with long fibers quantity increased together with the expansion of the cracking width (see Table 3.13).

Comparing the load-CMOD curves with three different fiber contents in Fig. 3.40 (a), (b) and (c) as well as in Table 3.13, it can be seen that at the same CMOD values after cracking, the SFRC beam with larger fiber contents can resist much higher load. Also, the highest load value and equivalent tensile strength deflection increases with the increase in the fiber content.

Fig. 3.40 Residual strengths in every type of concretes under $2.2 \times 10^{-3} \text{ mm/s}$.

Fig. 3.41 and 3.42 show the influence of the dosage of steel fibers on the post-peak parameters. From Fig. 3.41, it can clearly be noted that a pronounced effect on post peak is related to the type of steel fiber. Also, the addition of hooked end (long) fibers to the concrete mix produce a significant change in post peak parameters.

Fig. 3.41 Equivalent flexural tensile strength with respect to the fiber volume ratio: (a) and (b)

Fig. 3.42 Residual flexural tensile strength with respect to the fiber volume ratio: (a) f_{R1} , (b) f_{R2} , (c) f_{R3} and f_{R4} .

This behavior of the material is due to the bond strength of the matrix and steel fiber type (hooked end (long) fibers have more bond than straight ones) and the fiber length. More contribution of long fiber on macro crack than the short fiber which means the effect of the long fiber (hooked end) will become obvious in the peak load (A. Naaman & Reinhardt, 2006; A. E. Naaman & Reinhardt, 1996).

As shown, in Fig. 3.41 (a) and 3.42 (a) and (b) the relation between the increase in fiber contents and f_{eq2} , f_{R1} and f_{R2} respectively. We can note that the relation was linear in all of them. On the other hand when you look at Fig. 3.41 (b) and 3.42 (c) and (d) which show the relation between the increase of fiber contents and f_{eq3} , and f_{R3} and f_{R4} were not linear, and the increase between concrete type A and type B was slight whereas the increase between type B and C was dramatic in the three parameters. The reason for this behavior is caused by the increase of hooked end (long) fiber content which increases of bond between the mixture and the fibers.

According to RILEM TC 162-TDF (Rilem, 2002a), the investigation by (J. Barros & Antunes, 2003; J. A. Barros et al., 2005) identified a correlation for the post-peak parameters. His investigation suggests a linear relationship between equivalent tensile strengths of concrete ($f_{eq,2}$, $f_{eq,3}$). Fig. 3.43 shows the relationship between $f_{eq,2}$ and $f_{eq,3}$ was not mimicking his results because there is a dramatic increasing from (B) type to (C) type whereas the relation between (A) type and (B) type was not pronounced.

Fig. 3.43 Relationship between $f_{eq,2}$ and $f_{eq,3}$.

Fig. 3.44 shows the scatter relationship between f_{R1} and f_{R4} which have same trend as the relationship between $f_{eq,2}$ and $f_{eq,3}$, but larger than its relation. That indicates to f_R parameters are more susceptible to local irregularities of the $F-\delta$ curve.

Fig. 3.44 Relationship between f_{R1} and f_{R4} .

The relationship between $f_{eq,2}$ and f_{R1} in Fig. 3.45 is matched by the relationships between $f_{eq,3}$ and f_{R4} in Fig. 3.46. The model proposed by Barros has a strong correlation between equivalent and residual flexural tensile strength parameters and is registered in series reinforced with the two types of fibers.

Fig. 3.45 Relationship between $f_{eq,2}$ and f_{R1} .

Fig. 3.46 Relationship between $f_{eq,3}$ and f_{R4} .

3.4.3 Effect of number of fibers on the area section

(Barr et al., 2003) suggests the way to count fibers in the section area of the broken surfaces. In this research, Table 3.14 and Fig. 3.47 show the quantity of fibers in the quasi-static and in impact loading rates for all types of SFRC.

Fig. 3.47 The quantity of fibers VS Concrete type in all loading rates.

Table 3.14 The quantity of fibers in all loading rates.

Concrete type	Quasi- static load		Impact load height		
	2.20×10^{-3} mm/s	2.20×10^1 mm/s	40 mm	160 mm	360 mm
A	373	385	338	394	353
B	519	537	553	525	513
C	850	758	902	711	863

Fig. 3.47 and table 3.14 prove that with increase in the quantity fibers in material, the number of fibers in the fracture surface area is increased regardless the loading rate. Furthermore, failure modes of beams related to concrete type (A) under different loading rates are shown in Fig. 3.48, Fig. 3.49 relates to concrete type (B) and in Fig. 3.50 related to concrete type (C). From the figures, it can be observed that there is more deformation in the crack surfaces when fibers are added to the concrete. On other hand, the increase in loading rate did not affect the surface deformation.

Fig. 3.48 Morphology of fracture surface of concrete type A at different loading rates.

Fig. 3.49 Morphology of fracture surface of concrete type B at different loading rates.

Fig. 3.50 Morphology of fracture surface of concrete type C at different loading rates.

3.4.4 Relation between the loading rates and the distribution of fibers

Using (Barr et al., 2003)'s work on the distribution of the fibers in the beams with the peak load, it can be noted that there is a relation between the type of beam and the rating rate.

Fig. 3.51 A broken half of a beam divided into 9 areas (Barr et al., 2003) (a) and one of our samples (b).

3.4.4.1 Type (A) concrete

At the loading rate 2.20×10^{-3} mm/s, we noted that the effect of the peak load affected only the number of fibers in the section No.6 which is shown in Fig. 3.51. Also, at the loading rate of 2.20×10^1 mm/s, the control section of the loading rate is the same as under loading 2.20×10^{-3} mm/s for PA6, which didn't follow this pattern that due to (D), the depth of the whole beam, was higher than the others (100.74 mm).

With the impact load from a height of 160 mm, the area is includes sections 3,6, and 9 which means the crack increases starting at the center of the area. In this loading rate PA14.

With the impact load from a height 360 mm, the increase in the load rate affected the surface area still with 3 sections 3,6, and 9. On other hand, the only beam which did not follow this

trajectory path was PA19 because of the depth (D) of the whole beam which is very high 101.04 mm. All the information related to these observations is summarized in Table 3.15.

Table 3.15 The quantity of fibers in all loading rates for beam Type (A).

Loading rate	The beam #	Flexural strength (MPa)	No. of fibers in sections	Section #
2.20×10^{-3} mm/s	PA1	8.62	36	1
	PA2	8.82	37	
	PA3	8.38	33	
	PA4	8.02	26	
2.20×10^1 mm/s	PA6	10.97	29	1
	PA7	11.28	39	
	PA8	11.59	42	
	PA9	10.69	35	
160 mm	PA12	35.21	121	3
	PA13	42.73	132	
	PA14	36.65	146	
	PA15	38.77	125	
360 mm	PA16	41.80	115	3
	PA17	41.49	101	
	PA18	54.94	134	
	PA19	46.26	104	

3.4.4.2 Type (B) concrete

As in Type (A), we investigated the distribution of the fibers in the beams with the peak load on Type (B) beams. Under the loading rate 2.20×10^{-3} mm/s, it was noted that the effect of the peak load affected only the number of fibers in the section No. 3, 6, 9 and 5 that are shown in Fig. 3.51.

Also, at loading rate of 2.20×10^1 mm/s, the control section of the loading rate that affected the fibers was enlarged to be sections 3, 6, 9 and 5, for PB6, which was very low (400 fibers) compared to others beams because of the quantity of fibers in the first part of the crack initiating surface (3, 6 and 9); PB6 has only 37 fibers compared to other beams which approximately 60% of them only.

With the impact load from a height of 160 mm, the area included sections 3, 6, 9, 2, 5 and 8 in the beam.

Under a height of 360 mm, the increase in the loading rate affected the surface area that also increased to be seven sections. On other hand, the only beam which did not follow this trajectory way was PB19. All the information related to these observations is summarized in Table 3.16.

Table 3.16 The quantity of fibers in all loading rates for beam Type (B).

Loading rate	The beam #	Flexural strength (MPa)	No. of effect fibers in sections	Section #
2.20×10^{-3} mm/s	PB1	17.53	30	4
	PB2	17.34	25	
	PB3	12.32	23	
	PB4	11.77	12	
2.20×10^1 mm/s	PB6	15.24	18	4
	PB7	20.99	18	
	PB8	20.27	15	
	PB9	19.21	14	
160 mm	PB12	33.76	12	6
	PB13	34.62	18	
	PB14	40.39	18	
	PB15	52.66	24	
360 mm	PB16	50.44	34	7
	PB17	66.07	50	
	PB18	59.79	50	
	PA19	56.94	57	

3.4.4.3 Type (C) concrete

As we did in Types (A) and (B), we investigated the distribution of the fibers in the beams with the peak load on Type (C) beams. Under the loading rate of 2.20×10^1 mm/s, it was noted that the effect of the peak load affected only the number of the fibers in the section No. 3, 6, 9, and 5 which are shown in Fig. 3.51. Also, in loading rate 2.20×10^1 mm/s, the control section that affected the fibers were sections 3, 6, 9 and 5.

With the impact load from a height of 160 mm, the area was included all sections in the beam. On other hand, the only beam which did not follow this trajectory was PC15.

At a height of 360 mm, the section affected was the quantity of section which had an effect on this . All the information related to these observations are summarized in Table 3.17.

Table 3.17 The quantity of fibers in all loading rates for beam Type (C).

Loading rate	The beam #	Flexural strength (MPa)	No. of effect fibers in sections	Section #
2.20×10^{-3} mm/s	PC1	23.24	30	4
	PC2	27.37	68	
	PC3	23.31	44	
	PC4	27.79	70	
2.20×10^1 mm/s	PC6	35.05	72	4
	PC7	25.07	24	
	PC8	32.97	54	
	PC9	32.04	37	
160 mm	PC12	66.89	54	7
	PC13	79.10	96	
	PC14	72.67	87	
	PC15	57.86	92	
360 mm	PC16	96.44	102	7
	PC17	110.53	154	
	PC18	118.70	180	
	PC19	87.59	88	

From these results, we note that the affected area of the surface crack that produced the peak flexural strength increases when the loading rate incremented, especially from quasi-static to impact load. In addition, the area also increased when more fibers were added. On the other hand, the rate effect being remarkable because of the presence of hooked-end steel fibers embedded in concrete matrix which support a higher load under impact and the pull-out energy is also greater, (Nemkumar Banthia & Trottier, 1991).

Fig. 3.52 Compare between DIF with P_{max} and G_F

Moreover, the high performance HPFRCs (PC and PB) is less sensitive to loading rate than the conventional SFRC (PA) as well, but not like the rate effect on the flexural strength, showing a large difference, DIF of fracture energy is just from 1.89 to 2.82 under impact loading conditions Fig. 3.52.

In the next chapter we will continue with another experiment related to same materials used in this chapter, but with a higher loading rate which is a blast test.

Blast tests

4.1 Introduction

One of the main reasons for studying the mechanical behaviour of concrete under high strain rates is to assess its performance when it is subjected to blast loading. As already mentioned in Chapter 1, the increase of terrorist attacks in sensitive buildings and structures has raised the concern of researchers and structural engineers for improving structural security against this kind of loading. However, to reach such security improvement first it is necessary to improve the knowledge about the behaviour of concrete and concrete structures against blast loading. Besides this, many researchers points out to fiber reinforced concrete as an effective way to retrofit structural members for improving their resilience against blast attacks. This fact makes even more appealing to conduct blast tests on this kind of material.

To this aim, we have conducted a series of blast tests on square slabs of the three fiber reinforced concretes A, B and C, characterized in the previous chapter. The blast tests have been made in the same experimental test set-up presented by (Morales Alonso, 2013) due to the benefits it offers for this type of tests, since four slabs can be subjected to the same blast loading with one single detonation.

The results of the tests provide valuable information about the behaviour fiber reinforced concretes subjected to blast loading. Moreover, since the experimental set-up ensures that all slabs tested with the same detonation are subjected to the same explosive load, these tests are especially well suited for comparing the behaviour of the three concretes considered in the thesis. Beside this, these tests also constitute an experimental benchmark for calibrating and validating numerical models, as will be done in Chapter 6 of this thesis.

On the other hand, the experience gained from intended and accidental explosions occurred in buildings along the last years shows that the main cause of human injuries is not the direct effect of the explosion (blast wave), but the subsequent progressive collapse of the structure. For this

reason, the mechanical characterization of the residual strength of structural members subjected to blast loading is of paramount importance.

In this thesis an experimental campaign aimed at characterizing the residual strength of the three concretes A, B and C analyzed in the thesis is presented. This experimental campaign consists on retesting under quasi-static conditions and three point bending the slabs already tested under blast loading. In order to have a reference to compare with, intact slabs of each of the three concretes have been also tested under the same quasi-static conditions. The results from these residual strength tests show the influence of the amount and the shape of the fiber reinforcement in both the residual peak load and the residual stiffness.

4.2 Material and specimens

The slab dimensions used for the blast tests were $500 \text{ mm} \times 500 \text{ mm} \times 50 \text{ mm}$. These square slabs have been cast in steel molds in all samples (see Fig. 4.1). For the sake of test simplicity, these testing concrete samples are light weight with each slab being below 30 kg, allowing them to be lifted without using cranes or other mechanical devices.

Fig. 4.1 Test specimen geometry in (mm)

The materials used were the same already mentioned in Chapter 3.

Fig. 4.2 Steel molds for casting slab samples

The total number of slabs cast (Fig. 4.2) was 6 for each concrete type. From these 6 specimens, 4 were tested under blast loading and 2 were reserved for being tested as reference in the residual strength tests, as will be explained later.

4.3 Blast tests

Slabs of the three concrete types have been subjected to a campaign of blast tests consisting in submitting them, to the same explosive load. After these blast tests, the slabs were retested under static conditions in order to measure residual load of them.

4.3.1 Blast tests experimental set-up

The experimental procedure is the same already presented in (Morales Alonso et al., 2013). The main benefit from using this experimental set-up is that unlike in most previous works that can be found in the literature (Ellis & Tsui, 1997; Luccioni & Luege, 2006; J Magnusson & Hallgren, 2000; Mays, Hetherington, & Rose, 1999; Silva & Lu, 2007; Zhou, Kuznetsov, Hao, & Waschl, 2008), it is possible to test up to 4 specimens simultaneously subjected to the same blast load. This characteristic of the experimental device may be used to control the experimental scatter of a given concrete, if the 4 specimens tested belong to the same concrete, or as in the case of this thesis, to efficiently compare the behavior of different concretes under blast loading if the slabs tested with the same detonation belong to different concretes.

In the following section, the main characteristics of the experimental set up are presented.

4.3.1.1 Steel frame

The experimental test bench used in these blast tests consists of a square steel frame, as shown in Fig. 4.3. It allows to test up to 4 specimens with one single detonation, ensuring that the 4 samples are subjected to the same blast load. The set-up is a symmetric device in which the explosive is detonated from its geometrical center. Both control of the experimental scatter and cost efficiency can be met by the use of a symmetrical experimental set-up, since several targets can be tested with each detonation.

Fig. 4.3 General view of the steel frame

Due to the steel frame design and dimensions, it can be assumed that the slabs are simply supported at their four corners and that the explosive pressure load is uniformly applied on the slab side facing the explosive load (see(Morales Alonso, 2013)). In recent years, blast incidents showed that most of the terrorist attacks were explosions within short standoff distance less than 10 m (Bao & Li, 2010). So, the dimension of the steel frame between columns is only 3 m, which implies a 1.50m stand-off from explosive to the slabs. The distance from the centre of the

explosive to the ground is 1.70m, avoiding thus undesirable interferences between the direct blast wave acting on the slabs and the wave reflected from the ground. Fig. 4.4 sketches the test bench including its dimensions. From these dimensions setup we guarantee that there will be no difference greater than 6 - 8% in the arrival time of the blast shock between the center and the corner of the slab.

Fig. 4.4 Location of targets and explosive

4.3.1.2 Explosive material

The explosive type used in all detonations was Goma-2 ECO, which is the commercial name of a dynamite class explosive manufactured by Maxam Company. This explosive is provided in cylindrical cartridges containing 151.5 g with 200 mm high and 22 mm diameter, and tied together to form a bunch with the required amount of explosive Fig. 4.5. The TNT equivalent of this explosive is approximately equal to 0.9535.

Fig. 4.5 Goma-2 ECO cartridges

4.3.1.3 Bunker

A bunker placed near the experimental set-up was used for the safe location of the data acquisition systems and filming camera as shown in Fig. 4.6.

Fig. 4.6 View of bunker

4.3.1.4 Piezoelectric pressure sensor

A piezoelectric pressure sensor was placed under two of the concrete slabs to measure every explosion pressure wave (one of these piezoelectric pressure sensors didn't work), since it is an important issue in the numerical simulation. The signal of this pressure has been recorded by a Tektronics DPO2024 oscilloscope. The PCB Piezotronic 102B piezoelectric pressure gauges (Fig. 4.7) was used to measure the pressure wave in two locations as shown in Fig. 4.8.

Fig. 4.7 Piezoelectric pressure sensor (Morales Alonso, 2013)

Fig. 4.8 Location of Piezoelectric pressure sensor

4.3.1.5 High speed recording camera

To film one of the samples during the tests a high speed recording Phantom V12 camera was placed inside the bunker, facing the rear of the sample and connected to the computer, as shown in Fig. 4.9.

Fig. 4.9 Equipment located inside the bunker with speed recording camera(Morales Alonso, 2013)

4.3.1.6 Testing samples supports

Each slab was subjected to the blast load at the front face and supported on each of its four corners on the rear face. The supports were built by fitting four steel plates to the slab corners and putting them together with the slab through the threaded rods, as shown in Fig. 4.10.

Fig. 4.10 Details of bolt end, steel support plate and nuts in four corners

4.3.2 Testing details

A total of four tests were performed on October 10th 2014 at the testing range of Fundación Santa Bárbara in La Ribera de Folgoso, Spain. The first detonation took place with the aim of checking all the data acquisition devices, such as the triggering system, the pressure sensor, the data acquisition system, etc.

4.3.2.1 Test procedure

Before each test the following procedure took place:

1. The data acquisition system and the camera were retest.
2. Concrete slabs were put on their testing position and were fixed with wooden pegs.
3. Explosive load was placed in the geometrical center of the testing set-up.

4. A cable powered by DC battery tied to the explosive and connected to the data acquisition systems to be used as trigger signal.
5. Detonation of explosive (Fig. 4.11).

For a better comparison of the performance of the three concretes tested, it was decided to place one slab of each concrete per detonation leaving clear one of the four testing positions. Therefore only three slabs were tested with each detonation. The smoother face of slabs (due to being in contact with the molds during the concrete casting) was placed in contact with the supports making rough surface facing the explosion.

In order to check the data acquisition devices and to have a preliminary measure of the damage caused by the detonations in the concrete slabs, a first test with 17 cartridges of Goma-2 was conducted. Given that the level of damage registered in this preliminary test was limited, it was decided to increase the explosive load to 22 cartridges, making thus 3.322kg of Goma-2. Given the usual experimental scatter associated to concrete samples and explosive testing, this test arrangement was repeated three times in order to have three slabs of each concrete subjected to the same blast load. Table 4.1 summarizes the explosive loads used during the test campaign.

Table 4.1 Explosive loads and concrete types tested.

Test #	# of cartridges	"Goma 2"	TNT eqv. (kg)
1	17	2.58	2.46
2 to 4	22	3.33	3.18

Fig. 4.11 Explosion test

4.3.2.2 Blast pressure histories

For numerical analyses to be conducted later, the blast pressure histories acting on the plates were registered during the tests (see Fig. 4.12).

Fig. 4.12 Pressure with time in 4 explosions

4.4 Residual strength tests

In this thesis, there is an investigation to study how we can prevent or minimize the progressive collapse (Osteraas, 2006). As mentioned in Chapter 1, we follow the same way done by (C. P. Pantelides et al., 2014) to measure the remaining static force that a structural member can sustain after being subjected to the blast. The researchers in that work used concrete slabs that had been subjected to blast to determine their static post-blast load resistance by subjecting them to another test under monotonic static load. They noted that the cracking during the blast reduced the initial stiffness and when the crack was smaller the panel would be stiffer during post-blast tests.

In our thesis, the slabs which have tested under blast loading have been retested under static loading in order to measure the flexural residual load of SFRC. Beside this, intact slabs have been also tested under the same conditions for reference. For these tests we used a servo-hydraulic machine (Servosis frame machine) 1000 kN capacity as shown in Fig. 4.13. The static tests were carried at quasi-static loading rate (7×10^{-3} mm/s). Under these loading conditions, we measured the peak load and the energy required to break the slabs up to 3.5 mm displacement. All load-displacement curves were drawn. Curves stopped at 3.5 mm and the other continued more than 3.5 mm, so we applied our calculation up to 3.5 mm to make a fair comparison.

4.4.1 Residual strength tests set-up

In order to study the fracture behavior of these slabs, three-point bending tests were conducted on slabs by using a servo-hydraulic machine coupled to a robust frame in the Laboratory of Materials and Structures of the Civil Engineering School of Castilla-La Mancha University of Ciudad Real, Spain. Three-point bending tests were performed on the slabs, as shown in Fig. 4.13, where the schematic diagram of the servo-hydraulic testing machine is shown in Fig. 4.14.

Fig. 4.13 the servo-hydraulic machine

As Fig. 4.13 shows, in order to materialize this test set-up a rigid steel beam was attached to the floor of the laboratory. Over this beam, two supports were installed to sustain the slab. Here, the distance between these two supports was fixed at 333 mm. Over the slab, a loading bar was set joined to the actuator through a rigid profiled beam and some platens. Upon installation of each slab, we checked the alignment of the supports and the loading line. One of the supports was fixed, whereas the other was free to rotate around the axis of the frame. To measure the loading point displacement, we used two vertical LVDTs, as shown in Fig. 3.13. To measure the load, a load cell (± 1000 kN, model PCI 200, HBM Company, resolution 0.1 kN, maximum non-linearity of transducer: $\pm 0.32\%$ of full scale) was adopted.

Fig. 4.14 Schematic diagram of the servo-hydraulic machine

4.5 Results of blast tests

4.5.1 Overall behavior of the slabs

Fig. 4.15 shows a picture of the slabs just after one test, with increasing fiber content from bottom to top. The preliminary results show a clear influence of the fiber content and length on the fracture patterns as well as on the structural response of the slabs.

From bottom to top, concretes A, B and C are shown. While in the case of the concrete with the lowest content of fiber, A, the energy released through the crack patterns was not able to preserve the structural integrity and some concrete fragments were pulled off the slabs, in the

case of the concrete C, with the use of longer fibers and an overall highest content of fiber, crack patterns were hard to see by naked eye in the tensioned face of the slab whereas the other one was apparently undamaged. The slabs made of the concrete with the intermediate content of fiber (B) exhibited also intermediate failure behavior, with crack patterns clearly visible but still preserving the slab integrity.

Fig. 4.15 General view of the slabs after one test.

4.5.2 Failure patterns

After the tests, the failure patterns were carefully tracked and registered for all slabs (Fig 4.16 and Fig. 4.17). Generally speaking, a trend towards shear failure (curved cracks surrounding the supports) is dominant in these crack patterns.

Fig. 4.16 Crack trajectories registered after the test with TNT equivalent 2.46 kg : a) for the A concrete slabs; b) for the B concrete slabs; c) for the C concrete slabs.

Apart from the obvious differences in the strengths, the increase in fibers content induces a strain hardening behavior that redistributes efforts in the slabs, making thus possible the appearance of cracks in wider areas of the slabs. Actually, by comparing the crack patterns between concretes A and B, it is possible to affirm that the density of cracks in the latter is higher. In the author's opinion this result is probably due to the more marked hardening behavior of the B concrete in comparison with the A one.

In the case of the C concrete, it exhibits a higher resistance to crack appearance, what justifies the lower density of cracks registered in the slabs after the blast tests. This result demonstrates the ability of the concretes studied in this thesis to dissipate large amounts of energy by crack propagation since, besides the higher levels of specific fracture energies exhibited by these kinds of concretes, the redistribution of efforts creates new crack surfaces, increasing thus the overall amount of energy dissipated.

Fig. 4.17 Crack trajectories registered after the tests with TNT equivalent 3.18 kg: a) for the A concrete slabs; b) for the B concrete slabs; c) for the C concrete slabs.

4.6 Results of residual strength tests

In this part of thesis we will retest the slabs subjected to the blast to determine their static post-blast load resistance and compare the results with intact slabs. For the lowest quantity of fibers concrete type (A), there are two of slabs that are destroyed completely and we could not make a static test for them (A3 and A6) as shown in Fig. 4.18 and 4.19. So, the result curves have not been include.

Fig. 4.18 Slab A3 after explosion Test.

Fig. 4.19 Slab A6 after explosion Test.

The slabs condition after the tests made extremely difficult to get a homogeneous support when retesting some slabs. This fact introduced a characteristic hardening behaviour in the initial part of the load-displacement curves until the specimens were fully settled that was remarkable different for each specimen. For this reason, with the aim of comparing the behaviour of intact slabs and slabs previously subjected to blast, it was decided to condition the load-displacement curves according to the following treatment:

First of all, the peak load was identified for each curve (Fig. 4.20).

Fig. 4.20 The point corresponding to the peak load

The point corresponding to 40% of the maximum load was identified in the initial part of the curve (Fig. 4.21).

Fig. 4.21 The point corresponding to 40% of peak load

The initial hardening was removed in all curves (Fig. 4.22).

Fig. 4.22 The neglecting initial hardening

A linear curve was fitted in the points between the remaining minimum load and the 40% of the maximum load. This linear curve substituted the beginning of the load-displacement curve (Fig. 4.23).

Fig. 4.23 The linear fitting

Fig. 4.24, Fig. 4.25 and Fig. 4.26 show the load displacement curves for the concretes A, B and C respectively. In these figures, green colour has been used for the intact slabs, red colour has been used for those slabs subjected to 3.18 kg of TNT equivalent and orange colour for the slabs subjected to 2.46 kg of TNT equivalent.

In the case of the A type concrete slabs (Fig. 4.24) the effect of the blast load is remarkable in both the peak load and the stiffness of the slabs. Moreover, the effect of the amount of explosive charge is also noticeable, since the slab previously tested with 2.46 kg of TNT equivalent reached values of peak load and stiffness between the ones achieved by the intact specimens and those achieved by the slab tested with 3.18 kg of TNT equivalent.

Fig. 4.24 Load VS displacement for A type.

About the B type concrete results, a quick view (Fig. 4.25) could suggest that blast loading has no effect on the behaviour of the slabs, given that intact and retested slabs exhibit similar values of peak load. However, if we focus on the stiffness of the specimens it is possible to observe that retested specimens show lower values of the stiffness than the intact ones. As Fig. 4.26 shows, the peak load in the retested specimens is reached at displacement values greater than their counterparts in the intact specimens. It might be argued that this shift in the displacement of the peak load could be also the result of an uneven alignment of the supports at the beginning of the test, but this would have led to load-displacement curve tails also shifted in the X axis, something that does not happen in these results. For this reason, our hypotheses is that the previous blast test caused significant damage in the slabs, but this was not large enough to have an effect on the peak load. It must be reminded that the large amount of fibre reinforcement of these concretes is responsible for their high level of tensile strength and therefore the peak in strength is achieved when the concrete matrix has been already damaged and the fibres are carrying most stress. For this reason, the peak load remains unaffected, but the initial stiffness is significantly different.

Fig. 4.25 Load VS displacement for B type.

In order to quantify this change in the stiffness of the slabs previously tested under blast skipping the influence of the initial hardening due to the inhomogeneous support of the specimens, we have calculated an Equivalent Stiffness defined as:

$$K_{eq} = \frac{P_{peak} - P_{40\%peak}}{\delta_{peak} - \delta_{40\%peak}} \quad \text{Eq. (4.1)}$$

Being K_{eq} the equivalent stiffness, P_{peak} the peak load, $P_{40\%peak}$ the 40% of the peak load in the ascending branch of the curve, as shown in Fig. 4.25 (obviously $P_{40\%peak} = 0.4P_{peak}$) and δ_{peak} and $\delta_{40\%peak}$ the displacements corresponding to the peak load and to the 40% of it respectively. The values of this equivalent stiffness and the peak load for all tested specimens are shown in Table 4.2. It can be seen how in the case of B type concrete the specimens previously tested under blast loading exhibited an equivalent stiffness significantly lower than that from the intact specimens.

Finally, the load-displacement curves for the C type specimens are shown in Fig. 4.26. In this case intact and retested specimens exhibit a similar behaviour, being all curves within the same experimental envelope. The results in terms of peak load and stiffness are given also in Table 4.2, in which it is possible to observe that intact and retested specimens show similar values for both parameters.

Fig. 4.26 Load VS displacement for C type.

Table 4.2 all results for slabs obtained (Servosis frame machine).

Concrete type	Specimen number	Intact/retested?	Explosive load (kg of TNT equivalent)	Peak load (kN)	Equivalent stiffness (kN/mm)
A	A1	Intact	-	28.4	60.29
	A2	Intact	-	28.8	87.49
	A3	Retested	2.46	24.7	17.47
	A4	Retested	3.18	9.0	2.60
B	B4	Intact	-	45.2	148.71
	B5	Intact	-	55.8	263.55
	B2	Retested	2.46	43.2	75.42
	B1	Retested	3.18	51.2	40.65
	B3	Retested	3.18	45.9	42.77
	B6	Retested	3.18	56.1	28.28
C	C1	Intact	-	61.5	167.28
	C2	Intact	-	60.9	82.55
	C4	Retested	2.46	55.8	83.14
	C3	Retested	3.18	70.7	163.29
	C5	Retested	3.18	60.7	109.86
	C6	Retested	3.18	65.7	45.41

To summarize the results previously discussed, Fig. 4.27 and Fig. 4.28 gathers the peak-load and the equivalent stiffness of all concrete tested comparing between intact and retested specimens.

Fig. 4.27 Peak load in every type concrete

Fig. 4.28 Stiffness of 40% peak load in every type concrete

Material model

5.1 Introduction

The cohesive zone model (CZM) is an alternative method to study the material failure. It must be considered a phenomenological model instead of an exact physical representation of the behavior inside the material (Cornec, Scheider, & Schwalbe, 2003). The CZM has an important hypothesis summarized in that anywhere in the material at the continuum is ready to be a starting point for a crack, even if it is not in a pre-existing crack or stress concentration. This feature has been exploited to use it for the critical load assessment of brittle materials with different sizes and under different loading modes.

One of the key ingredients of the CZM is the softening curve, which relates the stress transferred across the cohesive crack with the crack opening. The softening curve can be considered as a material property. So, to get accurate results with this model, the shape of the curve has to be carefully set. This task is somehow difficult especially with small characteristic length materials (Bazant ZP, 1998).

The purpose of this chapter is to explore the applicability of the CZM to our material. In order to set the softening function, a bilinear softening curve with a kink point that must be determined has been used. Also, to account for the effect of high strain rates, a novel procedure is proposed which is based on the modification of the softening curve under static conditions by a Dynamic Increase Factor (DIF) function linearly decreasing with the crack opening.

The chapter is structured in two main sections. The first one provides the main details about the implementation of the CZM through the embedded crack approach. The second section represents the application of the cohesive crack model to our HPFRC under different strain rates, by setting firstly the softening curve under static regime and to apply this novel DIF function thereafter to the specimens tested under dynamic conditions.

5.2 The Cohesive Zone Model

The CZM which is called also fictitious crack model ([Hillerborg et al., 1976](#)) is considered the most widely spread application in the analysis of concrete fracture. This model is based on seminal works from Barenbalatt, Dugdale and Rashid (Hillerborg et al., 1976; Planas et al., 2003). The subroutine which has been used in our model was developed by (Morales Alonso, 2013), is a combination of the Cohesive Zone Model with Strong Discontinuity Approach. Failure occurs under tensile stress and no failure appears under compressive stress as stated by (Hillerborg et al., 1976).

5.2.1 Basic of the Cohesive Zone Model for (plain concrete)

A simplification of the cracking process is represented by the cohesive model. The usual assumption for quasi-brittle materials is that the material located outside the crack behaves in a linear elastic manner. The crack initiates when the maximum principal stress at a point equals the tensile strength. According to the classical definition of the CZM, the softening curve relates the stress transferred across the crack with the crack opening under mode I crack opening. To account for multiple combination of crack opening modes (mixed mode = mode I + mode II + mode III), a central forces model (Sancho, Planas, Cendón, Reyes, & Gálvez, 2007; Sancho, Planas, Fathy, Galvez, & Cendon, 2007) has been used. According to this work, the traction vector $\mathbf{t}$ between crack tips works in the same direction as the crack displacement vector $\mathbf{w}$ (Fig. 5.1), according to:

$$\mathbf{t} = \left(\frac{f(\tilde{w})}{\tilde{w}} \right) \cdot \mathbf{w} \quad \text{Eq. (5.1)}$$

Where $\tilde{w} = \max(|\mathbf{w}|)$ is an equivalent crack opening value defined as the maximum registered euclidean norm of the crack opening vector, $\mathbf{w}$ and $f(\tilde{w})$ is the softening function that relates the stress carried across the crack with crack opening according to the classical CZM definition.

Fig 5.1. Central forces model.

5.2.2 Embedded Crack Approach

For quasi-brittle materials, the discrete crack approach and the smeared crack were the fracture modeling approaches used in the 1980s. These approaches have been successfully complemented by the strong discontinuity approach (SDA) (Simo et al. 1993). In the smeared crack model the fracture zone was represented as a continuous displacement surface whereas in the SDA, it is represented as discontinuous. On other hand, the crack geometry in the discrete crack approach is restricted to inter-element lines whereas in the SDA, it is not. Besides the

SDA, some other techniques like the extended finite element method have emerged during the last years for the analysis of fracture without the need for remeshing avoiding also mesh dependency. These methods however need more implementation effort compared to the SDA.

For SFRC's materials used in this thesis, we follow the material model used in (Morales-Alonso, Cendón, Gálvez, & Sánchez-Gálvez, 2013) with adaptations to make it suitable for our materials.

As a finite element, the kinematic description of a strong discontinuity embedded divides the displacement field into two parts (a continuous and a discontinuous). Additional degrees of freedom are required for the displacement and integrated in the displacement jump vector, $\mathbf{w}$. This kinematics are described in Fig. 5.2. It shows a two dimension arbitrary element that contains various nodes and a crack embedded on it. The crack orientation is defined as the normal to the direction defined by the unity vector $\mathbf{n}$.

Fig 5.2. Arbitrary finite element with (a) discontinuity line and (b) displacement jump through the discontinuity line. (Sancho, Planas, Cendón, et al., 2007)

We assume that the crack embedded in the element will split it into two regions A^- and A^+ , one of them being the reference (A^-). The displacement jump $\mathbf{w}$ can be calculated as the subtraction between the displacement fields at both sides of the crack, A^+ and A^- . By following the strong discontinuity approach (Oliver, 1996), the displacement field within the element can be expressed as:

For A^- :

$$\mathbf{u}^-(\mathbf{x}) = \sum_{\alpha \in A^-} \mathbf{N}_\alpha(\mathbf{x}) \cdot \mathbf{u}_\alpha + \sum_{\alpha \in A^+} \mathbf{N}_\alpha(\mathbf{x}) \cdot (\mathbf{u}_\alpha - \mathbf{w}) \quad \text{Eq. (5.2)}$$

For A^+ :

$$\mathbf{u}^+(\mathbf{x}) = \sum_{\alpha \in A^+} \mathbf{N}_\alpha(\mathbf{x}) \cdot \mathbf{u}_\alpha + \sum_{\alpha \in A^-} \mathbf{N}_\alpha(\mathbf{x}) \cdot (\mathbf{u}_\alpha + \mathbf{w}) \quad \text{Eq. (5.3)}$$

Where $\mathbf{u}^-$ and $\mathbf{u}^+$ correspond to the displacement fields for A^- and A^+ regions respectively, $\mathbf{N}_\alpha(\mathbf{x})$ is the shape function for node α , the displacement of this node is $\mathbf{u}_\alpha$ and $\mathbf{w}$ is the displacement jump vector. This displacement jump vector can be obtained by subtracting the displacements between the regions A^+ and A^- :

$$\begin{aligned} \mathbf{w} = \|\mathbf{u}(\mathbf{x}_s)\| &= \mathbf{u}^+(\mathbf{x}) - \mathbf{u}^-(\mathbf{x}) = \left[\sum_{\alpha \in A^-} \mathbf{N}_\alpha(\mathbf{x}_s) + \sum_{\alpha \in A^+} \mathbf{N}_\alpha(\mathbf{x}_s) \right] \cdot \mathbf{w} \\ &= [\sum_{\alpha \in A} \mathbf{N}_\alpha(\mathbf{x}_s)] \cdot \mathbf{w} = \mathbf{w} \end{aligned} \quad \text{Eq. (5.4)}$$

Where $\mathbf{x}_s$ is a point on the crack, and in expression (5.4) it has been used:

$$\sum_{\alpha \in A} N_{\alpha}(x_s) = 1 \quad \text{Eq. (5.5)}$$

Equations (5.2) and (5.3) can be rewritten in a single equation as used in (Oliver, 1996) to show the displacement field of a strong discontinuity:

$$\mathbf{u}(\mathbf{x}) = \sum_{\alpha \in A} N_{\alpha}(\mathbf{x}) \cdot \mathbf{u}_{\alpha} + [\mathbf{H}(\mathbf{x}) - \mathbf{N}^+(\mathbf{x})] \cdot \mathbf{w} \quad \text{Eq. (5.6)}$$

Where:

$$\mathbf{N}^+(\mathbf{x}) = \sum_{\alpha \in A^+} N_{\alpha}(\mathbf{x}) \quad \text{Eq. (5.7)}$$

$\mathbf{H}(\mathbf{x})$ is the Heaviside function ($H(\mathbf{x}) = 0$ if $\mathbf{x} \in A^-$; $H(\mathbf{x}) = 1$ if $\mathbf{x} \in A^+$)

The strain tensor can be calculated using the symmetric gradient expression (5.7). Assuming small strains:

$$\boldsymbol{\varepsilon}^c(\mathbf{x}) = \boldsymbol{\varepsilon}^a(\mathbf{x}) - [\mathbf{b}^+(\mathbf{x}) \otimes \mathbf{w}]^s \quad \text{Eq. (5.8)}$$

Where $\boldsymbol{\varepsilon}^a(\mathbf{x})$ and $\mathbf{b}^+(\mathbf{x})$ are obtained by:

$$\boldsymbol{\varepsilon}^a(\mathbf{x}) = \sum_{\alpha \in A} [\mathbf{b}_{\alpha}(\mathbf{x}) \otimes \mathbf{u}_{\alpha}]^s \quad \text{Eq. (5.9)}$$

$$\mathbf{b}^+(\mathbf{x}) = \sum_{\alpha \in A^+} \mathbf{b}_{\alpha}(\mathbf{x}) \quad \text{Eq. (5.10)}$$

$$\text{With: } \mathbf{b}_{\alpha}(\mathbf{x}) = \text{grad} N_{\alpha}(\mathbf{x}) \quad \text{Eq. (5.11)}$$

Where superscript **S** in $\boldsymbol{\varepsilon}^a(\mathbf{x})$ is indicating the symmetric part of a tensor.

The total nodal displacements are used to calculate the apparent strain tensor of the element ($\boldsymbol{\varepsilon}^a(\mathbf{x})$). The gradient of the shape functions for the nodes in the A^+ region (solitary nodes) defines the $\mathbf{b}^+$ vector. So, it is crucial in the formulation of the embedded crack model to determine the solitary nodes and hence, the A^+ region.

Vectors $\mathbf{n}$ and $\mathbf{b}^+$ are undefined in the un-fractured element and the displacement jump vector $\mathbf{w}$ equals $\mathbf{0}$. Assuming elastic stresses on such an element we find:

$$\boldsymbol{\sigma} = \mathbf{D} \cdot \boldsymbol{\varepsilon}^a = \mathbf{D} \cdot \boldsymbol{\varepsilon}^c \quad \text{Eq. (5.12)}$$

Where $\mathbf{D}$ is the elastic moduli fourth order tensor. The concrete behavior is linear elastic if there is no crack in it. This is why the stress tensor on the material can be controlled with the expression shown in equation (5.12). As mentioned before, when the maximum tensile stress reaches the value of the tensile strength, the crack initiates in the material and the equation (5.12) is no longer valid at this stress state. This means that if $\sigma_I > f_{ct}$ a crack appears and the strain is calculated by two terms: the apparent strain and the continuum strain according to Eq. (5.8). In this case, the stress tensor is calculated by:

$$\boldsymbol{\sigma} = \mathbf{D} \cdot \boldsymbol{\varepsilon}^c = \mathbf{D} \cdot [\boldsymbol{\varepsilon}^a - [\mathbf{b}^+ \otimes \mathbf{w}]^s] \quad \text{Eq. (5.13)}$$

To calculate the stress in equation (5.13) which shows the role plays by the vector $\mathbf{w}$, the displacement vector needs to be solved. This step requires local equilibrium between the cohesive crack embedded and the continuum in the element outside the crack. Such equilibrium is prescribed through the traction vector in the continuum, which can be obtained by applying linear elastic material behavior to the strain field given by Eq. (5.8):

$$\mathbf{t}_{continuum} = \boldsymbol{\sigma} \cdot \mathbf{n} = (\mathbf{D} \cdot \boldsymbol{\varepsilon}^c) \cdot \mathbf{n} = [\mathbf{D} \cdot \boldsymbol{\varepsilon}^a - \mathbf{D} \cdot [\mathbf{b}^+ \otimes \mathbf{w}]^s] \cdot \mathbf{n} \quad \text{Eq. (5.14)}$$

And the traction vector in the cohesive crack, given by the central forces model:

$$\mathbf{t}_{crack} = \left(\frac{f(\tilde{w})}{\tilde{w}} \right) \cdot \mathbf{w} \quad \text{Eq. (5.15)}$$

$$\mathbf{t}_{crack} = \mathbf{t}_{continuum} \quad \text{Eq. (5.16)}$$

$$\frac{f(\tilde{w})}{\tilde{w}} \cdot \mathbf{w} = \mathbf{D} \cdot [\boldsymbol{\varepsilon}^a - [\mathbf{b}^+ \otimes \mathbf{w}]^s] \cdot \mathbf{n} \quad \text{Eq. (5.17)}$$

Equation (5.17) is the basic equation that sets the cohesive-embedded crack formulation. The only unknown of this equation is the displacement vector across the crack, $\mathbf{w}$, which cannot be easily isolated and can be solved numerically, (Morales Alonso, 2013) for further details.

5.3 Application of the cohesive crack model to HPFRC under different strain rates

5.3.1 Softening behavior

Because the model (CZM) is suitable for plain concrete, but there is limited research for fiber reinforced concretes, there is no specific procedure for setting the softening curve. For this reason, we will use a trial and error procedure in order to determine the shape of the softening curve. The only fixed data is the initial point of the softening curve (which is the tensile strength) and the area under it, which is the fracture energy, both parameters analyzed in Chapter 3 of this document. For the sake of simplicity, we will use a bilinear softening curve as in (Petersson, 1981), whose kink point must be determined. In order to determine the kink point using a trial-error procedure, we have taken into account (Planas et al., 2003) results. That is to say, the maximum load of the three point bending specimens depends basically on the initial part of the softening curve as shown in Fig. 5.3. Following these instructions, the softening curves for the three concrete types have been modified until getting good agreement with the experiments under static conditions.

Fig. 5.3. Softening curve

5.3.2 Strain rates effect

Previous researches have shown that strain rates modify mechanical properties of plain concrete (Malvar, 1998). Generally speaking, the higher the strain rate, the higher the compressive and the tensile strengths. This mechanical properties enhancement is usually accounted for by means of the Dynamic Increase Factor (DIF) which can be defined as the ratio between the mechanical property considered under high strain rate and the same mechanical property under static conditions (see (M. L. Hughes, Tedesco, J.W., Ross, C.A., 1993; Malvar, 1998; C. A. Ross, Jerome, D.M., Tedesco, J.W., Hughes, M.L., July 1990 May-June 1996; C. A. Ross, Kuennen, S.T., Tedesco, J.W., March 1992; C. A. Ross, Tedesco, J.W., Kuennen, S.T., January-February 1995; C. A. Ross, Thompson, P.Y., Tedesco, J.W., September-October 1989; J. W. Tedesco, Ross, C.A., Brunair, R.M., 1989; J. W. Tedesco, Ross, C.A., Kuennen, S.T., March-April 1993 ; J. W. Tedesco, Ross, C.A., McGill, P.B., O'Neil, B.P., 1991)). Normally this DIF is a function of the strain rate.

Since the main ingredients of the CZM, that is to say the tensile strength, the fracture energy and the shape of the softening, are also mechanical properties it seems reasonable to assume that all these three ingredients should be also affect by the strain rate. However, although the research on the strain rate effect on the tensile strength of plain concrete has been extensive (Béton, FIB 2012), the research on the effect of the strain rate in the fracture energy is scarce and almost null in the case of the shape of the softening curve. In the case of fiber reinforced cementitious materials, as the ones considered in this thesis, this knowledge is even scarcer.

For the aforementioned reasons, we have made a hypothesis about the strain rate effect on the CZM parameters in fibre reinforced cementitious materials based on the behavior exhibited by the three point bending specimens tested in Chapter 3. For the three concrete types, the behavior of the three-point bending specimens has been found approximately as depicted in Fig. 5.4.

Fig. 5.4. Behavior of Load VS Displacement for the three-point bending specimens under dynamic and static conditions

From the previous figure, it is clear that the effect of the strain rate is higher at maximum load, and tends to vanish as load approaches to zero (at the end of the tests). In terms of the CZM, the initial part of the softening curve plays a major role in the peak load in a three-point bending test, while the tail of the softening curve has a higher influence on the tail of the load-displacement curve. For this reason and in view of the experimental behavior depicted in Fig. 5.4, we proposed a DIF which is function, not only of the strain rate, but also of the crack

opening, in such a way that is maximum for zero crack opening and vanishes ($DIF=1$) at critical crack opening (cohesive crack fully opened). This DIF is applied to the static softening curve in a multiplicative manner, according to:

$$f_{dyn}(\tilde{w}, \dot{\epsilon}) = DIF_{Gf_{dynamic}}(\tilde{w}, \dot{\epsilon}) \cdot f(\tilde{w}) \quad \text{Eq. (5.18)}$$

Being $f(\tilde{w})$ the softening curve under static conditions, $f_{dyn}(\tilde{w}, \dot{\epsilon})$ softening curves under dynamic conditions and $DIF_{Gf_{dynamic}}(\tilde{w}, \dot{\epsilon})$ we propose. For the sake of simplicity, we propose a linear function for the DIF with the crack opening according to:

$$DIF_{Gf_{dynamic}}(\tilde{w}, \dot{\epsilon}) = \left[1 - (DIF_{ft}(\dot{\epsilon}) - 1) \left(1 - \frac{\tilde{w}}{\tilde{w}_c} \right) \right] \quad \text{Eq. (5.19)}$$

In this expression $\tilde{w}_c$ is the critical crack opening (crack opening at which cohesive strength vanishes) under static condition and $DIF_{ft}(\dot{\epsilon})$ is the DIF for the tensile strength, which is function of the strain rate, $\dot{\epsilon}$. It must be noted that the introduction of this DIF function of the crack opening and the strain rate introduces two major features:

- The softening curve shape changes from static to dynamic conditions (see Fig. 5.5).
- The DIF for the specific fracture energy, $DIF_{Gf_{dynamic}} = \text{dynamic specific fracture energy} / \text{static specific fracture energy}$, is not equal to the DIF for tensile strength: $DIF_{ft}(\dot{\epsilon}) \neq DIF_{Gf_{dynamic}}(\dot{\epsilon})$

This latter differs from the hypothesis made in (Morales Alonso, 2013), in which the whole softening was multiplied by a constant $DIF_{ft}(\dot{\epsilon})$, resulting in the same DIF for the fracture energy and the tensile strength: $DIF_{ft}(\dot{\epsilon}) = DIF_{Gf_{dynamic}}(\dot{\epsilon})$

In the following chapter the model presented here is applied to the experimental tests carried out in this thesis.

Fig. 5.5. Researcher's observation of influence of the DIF on the softening curves for FRCs according to second criterion.

Numerical results

6.1 Introduction

In this part we apply the material model presented in the previous chapter to the experimental data generated in this thesis. The material model has been implemented in the Finite Element Code LS-DYNA v.971 by modeling the material model developed by (Morales Alonso, 2013). For this reason it shares with such implementation most characteristics related with the element type (single integration point solid elements) and programming language (Fortran 90). The reader is addressed to (Morales Alonso et al., 2013) for further details.

6.2 Three point bending tests under low loading rates

6.2.1 Modeling details

A total of 2316 elements of size $1 \times 4.3 \times 4.4$ mm have been used to simulate the concrete prismatic beam with constant stress solid elements (one point integration solid). We did our mesh with only 1 mm in thickness (shown in Fig. 6.1) and plain strain conditions were prescribed by preventing nodal displacements in the direction normal to the plane of the model. This model set-up allowed to save significant computational effort. The result of the simulations was multiplied by 100 (The true beam thickness) to give us the true result of the load.

6.2.1.1 Boundary conditions

Boundary conditions are depicted in Fig. 6.1. The supports have all their translational degrees of freedom fixed except in y direction. All nodes of the beam were not moved in x direction to prescribe plain strain conditions. Finally, the load was applied by prescribing a vertical displacement in the two upper nodes in the center of the model as shown in Fig. 6.1.

Fig. 6.1 Boundary conditions in the model: supports and load.

6.2.2 Setting the softening curves for the three concretes under static conditions ($2.2 \times 10^{-3} \text{ mm/s}$)

As already mentioned in the previous chapter, we assumed a bilinear softening curve for the CZM under static conditions. This kind of softening curve requires to determine the kink point (Fig. 6.2). In this work we have conducted a trial and error procedure in order to set the values of α and β according to the nomenclature shown in Fig. 6.2. As already pointed out in the previous chapter, there is no specific procedure to set the shape of the softening curve (the kink point in our case), however and generally speaking, the peak load of a three point bending test is related with the initial part of the softening curve, while the tail of the load-displacement in the three point bending tests is related with the final part of the softening curve.

Fig. 6.2. Bi-linear function using (α) and (β) variables

Apart from the shape of the curve, to properly set the softening curve we need:

Tensile strength: taken from the three-point bending quasi-static tests (see Chapter 3).

Fracture energy: taken also from the three-point bending quasi-static tests. However, the tests were stopped before dividing the specimens into two halves. For this reason, the fracture energy values measured in the tests are only a minimum limit of the actual fracture energy. The actual fracture energy has been estimated as higher than the measured value based on approaching P- δ curves as much as possible shown in Fig. 6.3.

Fig. 6.3. The fracture energy two divisions (Test region and estimating region)

The material parameters obtained from the mechanical tests are summarized in table 6.1. Regarding the kink point for the softening curve of each concrete α and β have been determined after several simulations changing α and β until we got good agreement between simulations and experiments. The parameters are related to the beam shown in table 6.1 except α and β shown in Table 6.2 whereas the parameters input in the simulations are related to support as shown in table 6.3.

Table 6.1 Mechanical properties set in the constitutive model of concrete for all concrete types

Variable	Value		
	A	B	C
Density [kg/m ³]	2328		2395
Bulk Modulus [GPa]	24.24	22.83	23.181801
Shear Modulus [GPa]	19.639999	19.32	19.615
Tensile strength [MPa]	8.46	14.74	25.43
Fracture energy [N/m]	5100	8136	25000

Table 6.2 α and β values set in the constitutive model of concrete for all concrete types

Variable	Value		
	A	B	C
A	0.385		0.4
B	0.002		0.0025
Type of softening curve	Bi-Linear		

Table 6.3 Mechanical properties set in the constitutive model of support

Variable	Value
Density [kg/m ³]	7850
Elastic modulus [GPa]	200
Poisson ratio [-]	0.20

6.2.3 Results for the three point bending tests under quasi-static conditions

For the lowest strength concrete (A) we can observe from Fig. 6.4 that the simulation has is in a good agreement with the experimental results.

Fig. 6.4 Comparison between the numerical results and the experimental results for concrete A under quasi-static conditions

Also, for the medium strength type of concrete (B), the numerical behavior related to the experimental results is shown in Fig. 6.5.

Fig. 6.5 Comparison between the numerical results and the experimental results for concrete B under quasi-static conditions

With C type of concrete, the behavior of the simulation is somewhat similar to the experiment results as shown in Fig. 6.6.

Fig. 6.6 Comparison between the numerical results and the experimental results for concrete C under quasi-static conditions

We note that there is a good level of agreement between the simulation and experimental results in all three types of concrete. This validates, not only the cohesive zone model for this steel fiber reinforced, but also the shape of the softening curve for these concretes under quasi-static conditions, which is paramount for the upcoming simulations under dynamic regime.

6.2.4 Results for the three-point bending tests under loading rate ($2.2 \times 10^1 \text{ mm/s}$)

With respect to this loading rate, it must be reminded that, as discussed in Chapter 3, the total specific fracture energy $G_{f_{dynamic}}$ was available until total failure of the specimen. In the case the tensile strength is not available. By dividing $G_{f_{dynamic}}$ over $G_{f_{stat}}$, which has been estimated for the simulations under quasi-static conditions (see table 6.1), it is possible to obtain the Dynamic Increase Factor for the fracture energy, DIF_{Gf} . However, according to Eq. (5.18) in order to define the expression for the softening curve under dynamic conditions, we need the Dynamic Increase Factor for the tensile strength, DIF_{ft} . As already discussed in the previous Chapter (see section 5.3) the approach proposed in this thesis for the softening curve under high strain rates lead to different Dynamic Increase Factor for the tensile strength and the specific fracture energy although according to Eq. (5.19) they are related. Such relation can be obtained by integrating Eq. (5.18) by parts: such integration is provided in Appendix B and results in:

$$DIF_{ft}(\dot{\epsilon}) = 1 + \frac{3 \cdot [(\alpha + \beta) \cdot DIF_{Gf}(\dot{\epsilon}) \cdot (1 - \beta) - \alpha - (\beta - \alpha\beta - \beta^2)]}{[(\beta - \alpha\beta - \beta^2)(3 - \beta) + 2\alpha]} \quad \text{Eq. (6.1)}$$

With this expression, the Dynamic Increase Factor for the tensile strength, $DIF_{ft}(\dot{\epsilon})$ can be obtained as a function of the Dynamic Increase Factor for the specific fracture energy, $DIF_{Gf}(\dot{\epsilon})$, which is obtained from the experimental results from Chapter 3 and the kink point of the softening curve under static conditions (α and β factors) from the softening curve under static conditions adjusted for each concrete, A, B and C as discussed in the previous section.

For this loading rate of 2.2×10^1 mm/s, the parameters input in the simulations are shown in table 6.4.

Table 6.4 Mechanical properties set in the constitutive model of concrete for all concrete types.

Variable	Value		
	A	B	C
$G_{fDynamic}$	6592	10120	29987
G_{fstat}	5100	8136	25000
DIF_{Gf}	1.293	1.244	1.199
α	0.385	0.385	0.385
β	0.002	0.002	0.002
DIF_{ft}	2.675	2.590	1.300

For the lowest type in strength (A), we note that the simulation result has good agreement with the experimental tests, as shown in Fig. 6.7.

Fig. 6.7 Comparison between the numerical results and the experimental results in PA

For the medium strength concrete (B), the numerical results related to the experimental results is shown in Fig. 6.8.

Fig. 6.8 Comparison between the numerical results and the experimental results in PB

With C concrete type, the behavior of the simulation is somewhat similar to the experimental results as shown in Fig. 6.9.

Fig. 6.9 Comparison between the numerical results and the experimental results in PC

Generally speaking, the results of the simulations show good agreement with the experimental results for the three types of concrete, A, B and C. This result gives support to our approach for the Dynamic Increase Factor variable with crack opening.

6.3 Drop tower tests

In order to feed the model proposed in this thesis, the whole fracture energy up to complete fracture of the beams is required. The procedure used to measure $G_{f,dynamic}$ from the drop tower tests was based on Bantia's method as already discussed in Chapter 3. This method however used only allows to measure the dynamic fracture energy up to 3 mm of beam deflection. To

this end, a Charpy like analysis based on energy balance has been conducted, as explained in the following lines.

6.3.1 Charpy like analysis

Many of the first studies of the strain rate effect on the fracture energy of concrete and mortars were conducted by using a Charpy testing device (Corp., Feb. 1969; Krenchel, 1974; Radomski, February 1981). The Charpy impact test is widely used for the characterization of fracture toughness of metallic materials under impact (E23-16b) and is based in obtaining the decrease of energy consumed by the specimen, which can be calculated by measuring the rise angle of the pendulum after impacting with the specimen. Although the Charpy test is credited to be an adequate procedure for measuring the specific fracture energy of materials, in the case of concrete the interest for measuring some other fracture-related parameters, such as the flexural strength, led to more complex testing setups, such as the instrumented Charpy test (Hibbert A.P, 1977), or the instrumented drop weight test (N Banthia et al., 1989; Zhang et al., 2008) which is the procedure used Chapter 3, which allowed for measuring the dynamic fracture energy only up to 3mm of beam deflection.

In order to allow for measuring the total dynamic fracture energy up to total failure of the specimen, the drop weight impact tests have been analyzed according to a Charpy-like perspective. To this aim the energy balance during the fracture process has been analyzed. At the end of the fracture of the specimen, the energy balance can be written as:

$$E_{\text{kinetic load}_0}^{\text{impactor}} = E_{\text{kinetic load}_f}^{\text{impactor}} + E_{\text{kinetic}_f}^{\text{specimen}} + E_{\text{fracture}} \quad \text{Eq. (6.2)}$$

Where $E_{\text{kinetic load}_0}^{\text{impactor}}$ and $E_{\text{kinetic load}_f}^{\text{impactor}}$ are the kinetic energies of the impactor at the beginning and at the end of the fracture process respectively, $E_{\text{kinetic}_f}^{\text{specimen}}$ is the kinetic energy of the specimen at the end of the fracture process and E_{fracture} is the total energy consumed in the specimen due to crack propagation. Other sources of energy dissipation, such as the residual elastic energy stored in the specimen remains or the local energy dissipated by concrete crushing in the loading points and the supports have been neglected (in agreement with the basic hypotheses of the Cohesive Zone Model).

By isolating the fracture energy:

$$E_{\text{fracture}} = E_{\text{kinetic load}}^{\text{impactor}} - E_{\text{kinetic}_f}^{\text{specimen}} \quad \text{Eq. (6.3)}$$

Where $E_{\text{kinetic load}}^{\text{impactor}}$ represents the drop of kinetic energy of the impactor (initial-final).

The drop of energy suffered by the impactor during the tests can be obtained from its records of displacements history. By taking the first derivative with respect to time is possible to obtain the velocity history. Fig. (6.10) shows an example of such history measured in one of the tests. An analysis of the velocity history shows an initial linear trend (a) that reveals a uniformly accelerated motion of the weight due to the free fall in the machine, followed by a sudden drop (b) caused by the impact with the specimens and then a second linear trend (c) with a similar slope than the first one that reveals a new free fall until the weight strikes the stopper at the bottom of the machine (d).

Fig. 6.10 Comparison between the numerical results and the experimental results in PC

From the drop of velocity between both linear trends (a) and (c), it is possible to obtain the impact velocity, v_{imp} , and the residual velocity, v_{res} , of the weight. Therefore, the drop of energy consumed by the specimen can be obtained as:

$$E_{\text{kinetic load}_f}^{\text{impactor}} = \frac{1}{2} m_{imp} (v_{imp}^2 - v_{res}^2) \quad \text{Eq. (6.4)}$$

Being m_{imp} the mass of the dropped weight and $(\Delta E_k)_{\text{impactor}}$ its drop of kinetic energy.

The classical Charpy impact methodology equals the drop of kinetic energy given by Eq. (6.4) to the energy required to propagate a crack in the material. In the case of concrete, due to the large size of specimens, the kinetic energy of the two halves after being broken may be appreciable. Actually the firsts experimental campaigns conducted using the Charpy technique led to artificially high fracture energy measurements because of ignoring this kinetic energy (Gopalaratnam & Shah, 1986)[Gopalaratnam&Shah]. In this study, as pointed out in Eq.(6.7), with the aim of estimating the kinetic energy of the specimen after the impact, a simple two rigid blocks kinematics model (Fig. 6.11) has been used, assuming perfect contact between the drop weight and the upper hinge connecting the blocks and also between the lower supports and the specimen, at least at the instant when the specimen is broken in two halves. The validity of this assumption has been confirmed by the high speed video recordings taken during the tests

Fig. 6.11 Comparison between the numerical results and the experimental results in PC

From the kinematics of this model, the kinetic energy of the two halves of the specimen can be related to the residual velocity of the weight. First, the rotation angle, θ , and the loading point displacement, δ , can be related through:

$$\frac{s}{2} \sin \theta + (H - \delta) \cos \theta = H \quad \text{Eq. (6.5)}$$

The meaning of each variable is represented in Fig. 6.11. By taking the first derivative of this expression with respect to time, the relation between the impactor velocity and the rotational velocity of the specimen can be obtained:

$$\dot{\theta} \frac{s}{2} \cos \theta - \dot{\delta} \cos \theta - \dot{\theta} (H - \delta) \sin \theta = 0 \Rightarrow \dot{\theta} = \frac{\dot{\delta}}{\frac{s}{2} - (H - \delta) \tan \theta} \quad \text{Eq. (6.6)}$$

According to Eq. (6.6), the rotational velocity of each half depends, not only on the velocity of the weight, $\dot{\delta}$, but also on the position of the loading point, δ and the rotation angle of the specimen, θ . The position of the loading point δ can be obtained by integrating the weight velocity $\dot{\delta}$ from the impact initial time to the end of the process when the weight reaches the residual velocity v_{res} , and the rotation angle can be calculated from Eq. (6.5). Once the rotational and the loading point velocities are known, the residual kinetic energy of the specimen can be obtained as:

$$E_{\text{kinetic f}}^{\text{specimen}} = 2(E_{k, \text{rotation}})_{\text{halfspecimen}} + 2(E_{k, \text{translational}})_{\text{halfspecimen}} = I_g \dot{\theta}^2 + m v_G^2 \quad \text{Eq. (6.7)}$$

$$v_G^2 = (\dot{\delta} - l \sin(\theta_0 + \theta))^2 + (l \cos(\theta_0 + \theta))^2 \quad \text{Eq. (6.8)}$$

Being m the mass of one half of the specimen and I_g the momentum of inertia of one half specimen with respect to its center of gravity, G . Therefore, the dynamic fracture energy can be obtained as:

$$G_{f, \text{dynamic}} = \frac{E_{\text{kinetic load}}^{\text{impactor}} + E_{\text{kinetic f}}^{\text{specimen}}}{B(H - a_0)} \quad \text{Eq. (6.9)}$$

Being B the thickness of the specimen, H the height (see Fig. 6.11) and a_0 the initial notch depth.

The procedure described in the previous lines provides the whole specific fracture energy of the material considered, but it can be applied only if the specimen is completely broken in to pieces after the test, something that did not occur for the specimens tested with a height of the impactor equal to 40mm, and for the C type concrete tested at 160mm height. Those cases in which the specimens were not fully broken after the tests could be clearly identified in the velocity history of the impactor, as shown in Fig. 6.12. The figure shows the velocity histories of the impactor for the specimens PC11 and PC13: after hitting the specimen the velocity drops to negative values due to the impactor rebound.

Table 6.5 summarizes the results obtained with this Charpy-like analyses together with the results obtained with the Reaction force-displacement procedure (up to 3mm displacement).

Table 6.5 The Charpy-like and Reaction force-displacement procedure results.

Concrete type	Drop height (mm)	Charpy-like specific fracture energy (J/m^2)	Reaction-displacement specific fracture energy (J/m^2)
PA	160	8869	7920
	360	10971	8600
PB	160	13350	11025
	360	17162	13810
PC	160	-	16217
	360	30153	24946

Fig .6.12 Change of velocity chart for impactor in (a) PC11 and (b) PC13concretes

6.3.2 Modeling details

A total of 2635 elements of size $1 \times 4.3 \times 4.5$ mm have been used to simulate the concrete prismatic beam with constant stress solid elements (one point integration solid) while for the contact with the supports fully integrated elements with eight integration point were used. The steel supports and the impactor were explicitly meshed, as shown in Fig. 6.13. Just like in the

previous simulations, we did our design with only 1 mm thick to make the simulations faster. The result of this simulation must be multiplied by 100 (The depth of the beam) to give us the true result of the load. The parameters input in the simulations are related to the supports and impactor for all loading rates as shown in table 6.6.

Fig. 6.13 Parts comprised in the model: concrete support, steel concrete, impactor and concrete beam.

Table 6.6 Mechanical properties set in the constitutive model of supports and impactor.

	Steel of impactor	Concrete of support	Steel of support
Density [kg/m ³]	5889000	2400	7800
Elastic modulus [GPa]	200	40	200
Poisson ratio [-]	0.20		

The density of the impactor cylinder was adjusted in order to have a total mass of it equal to 1/100 of the actual mass of the drop weight (120 kg).

6.3.2.1 Boundary conditions

In the Impact test, the supports have all their translational degrees of freedom fixed. All nodes of the beam were not moved in x direction.

6.3.3 Numerical simulation of 160 mm height drop weight tests

As pointed out previously, in the tests conducted at this height (160 mm) in drop tower instrument, all beams related to concrete types A and B were broken whereas only one specimen of type C was broken. For this reason, only the types A and B were modeled although this result suggests that the initial energy of the impactor was close to the fracture energy of C type concrete at this loading rate.

6.3.3.1 Material parameters

Although load-displacement curves were available for the tests conducted with the drop weight tower, their experimental scatter is considerably higher than that from the static tests (see Fig 3.30, Fig. 3.31 and Fig. 3.32). While in same case the peak loads can exhibit differences close to 50% (see Fig. 3.31 for concrete B), in other cases the time shift for secondary peaks may also be displaced by a 50% factor (see Fig. 3.32 for concrete C). This behavior is usual in dynamic tests like the ones modeled in this section and is due to small misalignments and gaps that have no effect on static tests, but lead to these differences in these events characterized by a total duration of fractions of a second. The ideally-perfect alignment and boundary conditions present in numerical simulations may lead to even higher differences when comparing numerical

simulations with experimental tests. For this reason, for the validation of the numerical model applied to the drop weight tests we have opted for comparing the energies (initial and residual) of the impactor in the tests and in the simulations.

The material parameters for both concretes types (A and B) were kept exactly the same as for the quasi-static simulations (see table 6.1) except for the ones related to the Cohesive Zone Model. Looking at the dynamic fracture energy values determined by the Charpy-like analysis (Table 6.5) and applying Eq. 6.1, the following parameters were used (Table 6.7).

Table 6.7 Mechanical properties set in the constitutive model of concrete for A and B concrete types.

Variable	A	B
$G_{fDynamic @ 160 \text{ mm hight}}$	8869	13350
G_{fstat}	5100	8136
DIF_{Gf}	1.74	1.64
A	0.385	0.385
B	0.002	0.0025
DIF_{ft}	2.074	1.910

6.3.3.2 Results

Fig. 6.14 shows the kinetic energy of the impactor for the A type under 160 mm height. The initial kinetic energy of the impactor is 1.783 J and its kinetic residual is 0.967 J. The value of subtraction between the two magnitudes needs to be multiplied by 100 because the reduce thickness of the model.

Fig. 6.14 Kinetic energy of the impactor with PA type in 160mm height

On the other hand, Fig. 6.15 shows the kinetic energy of the specimen in the same test. As shown in the figure, the residual kinetic velocity is near to 0.03 J. In the same way as the kinetic energy of the impactor, this residual kinetic energy must be multiplied by 100.

Fig. 6.15 Kinetic energy of the type A concrete specimen in 160mm height.

From the previous values it is possible to calculate the whole fracture energy for the specimen by subtracting the kinetic energy of the specimen from the kinetic energy of the impactor and divide the result by the area of the fracture surface (0.1×0.083 mm) repeating the same reasoning we have used in the Charpy-like analysis of the experiments.

The same procedure was followed for the B type concrete. In this case, Fig. 6.16 shows the kinetic energy history of the impactor while Fig. 6.17 shows the kinetic energy history of the sample.

Fig. 6.16 Kinetic energy of the impactor with PB type in 160mm height.

Fig. 6.17 Kinetic energy of the concrete specimen with PB type in 160mm height.

Table 6.8 shows the differences between the energies calculated from result of the simulation and the Charpy-like method applied to the experimental results. As shown in the table, the numerical results are in good agreement with the experiments.

Table 6.8 Differences between Energies calculated from experimental and simulation

	Impactor height (mm)	Impactor (J)			Specimen (J)		Kinatic Energy (J)	Fracture energy [N/m]	
		Impactor kinetic	Residual kinetic	Difference $\times 100$	Residual kinetic	$\times 100$		Simulation result	Charpy like method
PA	160	1.783	0.967	81.630	0.025	2.500	97.100	9530.120	8869.860
PB			0.541	124.200	0.017	1.700	122.500	14759.036	13350.500

6.3.4 Numerical simulation of 360 mm height drop weight tests

With respect to this height (360 mm) in drop tower instrument, all concrete beams related to types A and B were broken whereas only half of specimens of type C were broken. This result reveals that the initial energy of the impactor dropped from a height of 360 mm must be almost equal to the dynamic fracture energy of the C concrete at this loading rate. For this reason, in this case all concrete types A, B and C have been modeled. Following the same procedure already explained in the 160 mm height simulations, the material parameters for the three concretes were taken from the quasi-static simulations, except for those concerning to Cohesive Zone Model. In this case, according to the measurements from the Charpy-like analysis (Table 6.5), the parameters used for the CZM are the ones provided in Table 6.9.

Table 6.9 Mechanical properties set in the constitutive model of concrete for A and B concrete types.

Variable	A	B	C
$G_{fDynamic} @ 360 \text{ mm hight}$	10971	17162	30153
G_{fstat}	5100	8136	25000
DIF_{Gf}	2.15	2.11	1.21
A	0.385	0.385	0.385
B	0.002	0.002	0.002
DIF_{ft}	2.675	2.590	1.300

6.3.4.1 Results

Fig. 6.18 shows the kinetic energy history of the impactor in the case is 4.120 Joule (obviously this value is the same for all tests in 360 m height) and the residual kinetic energy is 3.032 Joule.

Fig. 6.18 Kinetic energy of the impactor with PA type in 360mm height.

Fig. 6.19 Kinetic energy of the concrete with PA type in 360mm height.

As shown in Fig. 6.19, residual kinetic energy in the specimen is 0.09 Joule. These values result in a fracture energy of 12025.30 N/m. As in the procedure done with concrete A, the residual kinetic energy in the case of concrete B simulations is 2.475 Joule (Fig. 6.20).

Fig. 6.20 Kinetic energy of the impactor with PB type in 360mm height.

The residual kinetic energy of the sample is 0.06 Joule (Fig. 6.21), resulting in a fracture energy simulated of 19107.23 N/m.

Fig. 6.21 Kinetic energy of the concrete with PB type in 360mm height.

In the case of the C type concrete, the residual kinetic energy is 0.776 Joule (shown in Fig. 6.22). After subtracting the residual kinetic energy of the sample, which is approximately 0.014 Joule (Fig. 6.23), it results in a fracture energy of 40121.69 N/m.

Fig. 6.22 Kinetic energy of the impactor with PC type in 360mm height.

Fig. 6.23 Kinetic energy of the concrete with PC type in 360mm height.

All this information is summarized in table 6.10 which also shows its corresponding experimental values. The good agreement between experiments and simulations supports the use of the CZM for these specimens and specially the Charpy-like analysis of the tests.

Table 6.10 Differences between Energies calculated from experimental and simulation

	Impactor height (mm)	Impactor (J)			Specimen (J)		Kinatic Energy (J)	Fracture energy [N/m]	
		Impactor kinetic	Residual kinetic	Difference $\times 100$	Residual kinetic	$\times 100$		Simulation result	Charpy like method
PA	360	4.120	3.032	108.810	0.090	9.000	99.81	12025.30	10971.88
PB			2.475	164.590	0.060	6.000	158.59	19107.23	17162.17
PC			0.776	334.410	0.014	1.400	333.01	40121.69	30153.83

6.4 Numerical simulation of the blast tests

6.4.1 Evaluation of the cohesive model for high strain rate

In order to feed the Cohesive Zone Model with high strain rate effects presented in this thesis it is necessary to provide the dynamic fracture energy or the dynamic tensile strength, in order to calculate DIF_{G_f} or DIF_{f_t} for the strain rates induced in the problem modeled. In the three-point bending tests modeled in previous sections it was possible to derive $G_{f_{dynamic}}$, by direct integration of the load-displacement curves in the case of the tests carried out in the hydraulic machine, or by applying the Charpy-like analysis in the case of the tests conducted in the drop weight machine.

In the case of the blast tests, however, such measurement is not possible. Moreover, unlike the three-point bending tests in which fracture propagates along a single plane, in these tests several cracks appear simultaneously making extremely difficult to obtain even an approximated value

of the strain rate at which the cohesive cracks are propagated. For this reason it was decided to develop one step beyond the numerical model presented in Chapter 4.

In the numerical simulations discussed in the previous section, a constant DIF_{Gf} (and DIF_{ft}) is used for each simulation. The reason behind this procedure is that one single value of the dynamic fracture energy outcomes as a result of each experimental test. However, as the crack propagates along the ligament of a three-point bending specimen, the strain rate varies.

In fact, the strain rate at which the cohesive crack generates at the notch tip, is higher than the strain rate at which the cohesive crack is formed in the mid-depth of the beam where the crack propagates through this point. In summary, the $G_{f,dynamic}$ resulting from each three-point bending test is actually the average of the energy required to propagate the crack at all different strain rates induced along the ligament during the crack propagation. Therefore, the cohesive model as used up to this section is fed by such averaged $G_{f,dynamic}$ and uses this constant value (constant DIF_{Gf} and subsequently constant DIF_{ft}) in spite of having actually different strain rates along the specimens ligament.

For the slabs subjected to blast, no average value of $G_{f,dynamic}$ was available and the range of strain rates induced by these tests is presumably much larger than the three-point bending tests one. For this reason, we decided to improve the model by introducing the DIF_{Gf} as a function of the strain rate, and not as a constant parameter, leading to an also constant DIF_{ft} . In order to define DIF_{Gf} as a function of the strain rate, we have taken benefit from expressions Eq. 3.13, Eq. 3.14 and Eq. 3.15 in which the $G_{f,dynamic}$ was expressed as a function of the loading rate:

$$\text{PA: } DIF_{GF} = 1 + m \left(\frac{\dot{\delta}}{\dot{\delta}_0} \right)^r = 1 + 0.28 \left(\frac{\dot{\delta}}{\dot{\delta}_0} \right)^{0.24}, \text{ for } \dot{\delta} \text{ in mm/s} \quad \text{Eq. (3.13)}$$

$$\text{PB: } DIF_{GF} = 1 + m \left(\frac{\dot{\delta}}{\dot{\delta}_0} \right)^r = 1 + 0.24 \left(\frac{\dot{\delta}}{\dot{\delta}_0} \right)^{0.25} \quad \text{Eq. (3.14)}$$

$$\text{PC: } DIF_{GF} = 1 + m \left(\frac{\dot{\delta}}{\dot{\delta}_0} \right)^r = 1 + 0.30 \left(\frac{\dot{\delta}}{\dot{\delta}_0} \right)^{0.16} \quad \text{Eq. (3.15)}$$

In these expressions, however, the Dynamic Increase Factor is obtained as a function of the loading rate and not as a function of the strain rate. To this aim, we have used the three-point bending test simulations to correlate the load point velocity with the average strain rate across the ligament. This correlation allows us to translate $\dot{\delta}_0 = 1 \text{ mm/s}$ to a certain $\dot{\epsilon}_0$, in order to convert Eq. 3.13, Eq. 3.14 and Eq. 3.15 from load point velocity to strain rate. From the numerical simulation of the three point bending test at 22 mm/s we calculated an average strain rate along the ligament of $3.3510 \times 10^{-1} \text{ 1/s}$. Therefore, for $\dot{\delta} = 1 \text{ mm/s}$ we have a value of $3.3510 \times 10^{-1} / 22 = 0.0152318 \text{ 1/s} = 1.52318 \times 10^{-5} \text{ 1/}\mu\text{s}$ turning our equations to:

$$DIF_{GF} = 1 + m \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_0} \right)^r = 1 + 0.28 \left(\frac{\dot{\epsilon}}{1.52318 \times 10^{-5}} \right)^{0.24}, \quad \text{for } \dot{\epsilon} \text{ in (/s)} \quad \text{Eq. (6.10)}$$

$$DIF_{GF} = 1 + m \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_0} \right)^r = 1 + 0.24 \left(\frac{\dot{\epsilon}}{1.52318 \times 10^{-5}} \right)^{0.25} \quad \text{Eq. (6.11)}$$

$$DIF_{GF} = 1 + m \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_0} \right)^r = 1 + 0.30 \left(\frac{\dot{\epsilon}}{1.52318 \times 10^{-5}} \right)^{0.16} \quad \text{Eq. (6.12)}$$

for our concretes A, B and C respectively. With these equations the subroutine calculates the DIF_{GF} , calculate then DIF_{ft} through equation Eq. (6.1) and compares the maximum principal stress in the element with $f_{t_{dynamic}} = f_{t_{stat}} \times DIF_{ft}$. Once the maximum principal stress becomes greater than $f_{t_{dynamic}} = f_{t_{stat}} \times DIF_{ft}$, the last calculated values for DIF_{ft} and DIF_{Gf} are frozen for the rest of the simulation.

6.4.2 Modeling details

A total of 187500 elements of size $4 \times 4 \times 4.17$ mm have been used to simulate the concrete slab with our constant stress solid element (one point integration solid) and the size of the slab is $500 \times 500 \times 50$ mm while the elements for modeling the steel supports are fully integrated eight point integration elements of $4 \times 4 \times 4$ mm size. Fig. 6.24 shows the model set-up.

With this improved version of the constitutive model, the only material properties required are these corresponding to the quasi-static behaviour of the three concretes which can be found in Table 6. The steel for the supports was modelled as a linear elastic material with the mechanical properties given in Table 6.9.

For the model in LS-Dyna, we also opened a gap (approximately 1 mm) between the two supports and the slab whereas the others are in contact. Our assumption is that the behavior of the load isn't distributed symmetrically.

Fig. 6.24 Parts comprised in the model: steel support and concrete slab.

6.4.2.1 Interaction between concrete and support

On the other hand, for these blast test simulations with LS-DYNA's contact algorithm *CONTACT ERODING SINGLE SURFACE with a friction coefficient can be set in order to account for the sliding constraint between both surfaces. The value given to the dynamic and static friction coefficient in the current simulations is 0.3.

6.4.2.2 Boundary conditions

In the Explosion impact, the central node of the steel supports has the z-direction translational degree of freedom fixed, because of the impact tower loading it. So, the steel supports are free to rotate around the central node, as shown in Fig. 6.25.

Fig.6.25. View of the available movements in the steel supports: rotation around the central nodes (Morales Alonso, 2013).

6.4.2.3 Blast load

For these simulations, we used Ls-Dyna's capability for defining a certain explosive load at a given distance. With this capability, the user only must define the equivalent mass of TNT and the X, Y and Z coordinates of the explosion point.

6.4.3 Results

With this last evolution of the constitutive model we run a set of simulations for the slabs tested under blast loading. As already described in Chapter 4, the only outputs from these tests were the failure crack patterns and the general condition of the specimens after the tests. Consequently these have been the criteria used to check the validity of the simulations.

In order to set a fixed criterion to compare numerical crack patterns with the experimental ones, we display in Fig. 6.26, Fig. 6.27 and Fig. 6.28 the crack patterns and crack opening contours for the instant of maximum deflection (maximum crack opening).

Fig. 6.26 compares experimental and numerical crack patterns for concrete A. The figure caption reflects maximum crack opening ($\tilde{w}_{max}=5.091$), although it must be multiplies by $G_{f_{dynamic}}/f_{t_{dynamic}}$ given that the subroutine is non-dimensional. Since DIF_{ft} and DIF_{Gf} have

the same order of magnitude, we have opted to use $G_{f_{stat}}/f_{t_{stat}}$ to recover the crack opening

dimensions. In this case, this leads to $\tilde{w}_{max} = 5.091 \times 5.1 / 8.460 = 3.07$ mm, which is a clearly visible crack.

When comparing the simulation with the experimental results, it can be observed that crack trajectories are similar but, on top of this, the most important result is that in both, simulation and experiments, multiple cracks have nucleated into a well defined crack pattern which is the one represented in the figure.

Fig. 6.26 Comparison between the behavior shape of the experiment and its simulation for type A

About B concrete, the comparison between simulation and experiments is shown in Fig. 6.27. In this case, maximum crack opening is $\tilde{w}_{max}=2.68$ which is equal to $\tilde{w}_{max}=2.68 \times 8.136 / 14.74 = 1.48$ mm.

In this case, the crack pattern predicted is not so similar to the experimental one. However the numerical simulation exhibit a more distributed crack pattern when compared to A concrete, in line with experimental results.

Fig. 6.27 Comparison between the behavior shape of the experiment and its simulation for type B

C type concrete results are shown in Fig. 6.28. In this case maximum crack opening was $\tilde{w}_{max}=0.498 \times 25 / 25.43 = 0.49$ mm, which is a crack opening hardly visible by naked eye, in accordance with the experimental results. Moreover, the highly distributed crack patterns, with curved shear cracks surrounding the supports approaches adequately the experimental results.

Fig. 6.28 Comparison between the behavior shape of the experiment and its simulation for type C

Conclusion and future works

7.1 Conclusion

7.1.1 Experimental

This thesis has analyzed three types of fiber reinforced concretes containing the same compositions for the cementitious matrix, with the only difference being the type and dosage volume contents in fiber (straight fiber and long hooked end fiber). The first concrete type was Type A which had only straight short fibers with 40 kg/m^3 . The second one, Type (B), contained the same quantity of straight short fiber with long hooked end fibers with 20 kg/m^3 . The third concrete Type (C) contained the same amount of straight short fibers as the previous two types of in addition to long hooked end fibers with 60 kg/m^3 . The fracture behavior of the three concretes at a wide range of loading rates was investigated.

The main achievements of the thesis from the experimental point of view are summarized in the following lines.

7.1.1.1 Three point bending test

In the three point bending tests, the RILEM TC 162-TDF (Rilem, 2002a) and EN 14651 standard (EN, 2005a) recommendations were followed with a scale factor according to (Giaccio et al., 2008). The loading rate used in three point bending tests varied considerably from a quasi-static level to a dynamic level, and the order of magnitude changed from 10^{-3} to 10^3 mm/s. For quasi-static loading rates the thesis used a Servo-hydraulic machine, and for high loading rates a drop tower instrument was used. We conclude from these tests that:

To follow from the recommendation of the RILEM TC 89-FMT committee for estimating the elastic modulus from a three-point bending test is still valid for fiber-reinforced concrete, though it is set up for plain concrete.

There is an increase of every parameter related to RILEM TC 162-TDF when the quantity of fibers is increased.

The relationship between $f_{eq,2}$ and f_{R1} is similar to the relationships between $f_{eq,3}$ and f_{R4} . This means there is a strong correlation between equivalent and residual flexural tensile strength parameters with the two types of fibers.

In addition this thesis emphasizes that the LVDT -CMOD curves relationship is almost linear for all types of SFRC, and the relationship slope is increased when the volume ratio is increased.

The higher the fiber content, the more involved the crack surface regardless of the loading rate. The affected surface crack area is directly related to the peak flexural strength increasing thus also with the loading rate from quasi-static to impact load.

For both the conventional and the high performance SFRCs, the flexural strength and the fracture energy are rate sensitive. Under low loading rates, the rate effect is minor, but is remarkable under high loading rates. At a low loading rate (2.20×10^{-1} mm/s) the three SFRCs exhibit a flexural strength enhancement around 30%. However at high loading rates, the dynamic increase factor for the conventional SFRC is approximately 6 (600 % increase) while for the two high performance SFRCs it was near 4 (400 % increase). With regards to the fracture energy, the gain is less than 40% for the three different SFRCs at a low loading rate (2.20×10^{-1} mm/s), while it is less than 3 at high loading rates.

The increase in fiber content reduces the rate sensitivity.

Two empirical equations for the rate sensitivity of the flexural strength and the fracture energy are proposed for each type of SFRC. These are helpful in numerical simulations that evaluate the rate effect of the fracture behavior.

7.1.1.2 Blast tests

Slabs of the three concretes were subjected to blast loading under two explosive charges (2.46 kg and 3.18 kg of TNT equivalent) placed at a distance of 1.5m. A clear effect of the fiber shape and content was noticed in these tests. As the fiber content increase, the capability of preventing fragments projection was increased. Higher contents of fibers also lead to more distributed crack patterns. In summary induced, the post-cracking behavior of these concretes seems to have a crucial role in the crack patterns during the blast tests.

The tests carried out to characterize the residual strength of these slabs under the static loading exhibit also similar trends. In this part of the thesis we conclude that the increase in fiber content and strength lead to a hike in residual strength performance of the concrete after the explosion load. Moreover, for the B type concrete although the blast load did not affect the residual strength, the residual stiffness was remarkable decreased.

7.1.2 Simulation work

A constitutive model has been proposed using the Cohesive Crack Model in combination with the Strong Discontinuity Approach. The model is based on the one prepared by (Morales Alonso, 2013). Unlike in such model in which a constant Dynamic Increase Factor (DIF) was applied to the whole softening curve, in the approach presented here, especially intended for fiber reinforced concretes, the DIF is defined as a linear function of the crack opening. This leads to two slightly different DIFs for the tensile strength DIF_{ft} and for the specific fracture energy DIF_{Gf} . Besides being function of the crack opening it is obviously function of the strain rate.

The numerical simulations carried out with the model show good agreement with all experimental tests conducted in the thesis. Moreover, the numerical simulation has allowed providing novel information of the drop-weight impact tests based on the hypotheses of the Charpy impact test.

7.1.3 Future works

7.1.3.1 EXPERIMENTAL

The drop-weight impact instrument has proven its reliability for measuring the specific fracture energy of fiber reinforced concretes at high strain rates. However the information obtained from this type of test could be enriched even more by using Digital Image Correlation techniques. Such technique could be helpful in studying additional parameters such as crack propagation velocity, crack initiation time, possible crack branching during the test...etc.

Although the drop-weight impact instrument provides valuable information about the specific fracture energy under high strain rates, to fully define the Cohesive Zone Model the tensile strength and the shape of the softening curve must be measured under high strain rates. To this aim, some other experimental techniques such as the Split Hopkinson Bar could be also used.

The blast tests conducted in this thesis allow testing structural members at high strain rates using the same type of load targeted in this research (explosive loading). However, the information gathered from this test is scarce given that the only parameters that can be directly analyzed are the failure patterns and the overall condition of the slabs. To improve the instrumentation of these tests by strain gauges and Digital Image Correlation would help to obtain richer information from them.

An attempt for measuring the residual strength of structural elements previously subjected to blast loading has been conducted in the thesis. Although the experimental results obtained are considered valuable, to design specific residual strength tests for each structural member, replying the actual working conditions of each (concrete columns under compression, beams under flexion...) would be desirable.

7.1.3.2 NUMERICAL

The simulations carried out in this thesis using the Cohesive Zone Model have provided satisfactory results for modeling steel fiber reinforced concrete elements. However, as the amount of fibers increases, some effects such as crack bridging are not adequately reproduced by this model. This may lead to inaccurate results when modeling small concrete pieces and structural elements under tension with lower strain gradients than the ones analyzed in the thesis. For this reason, numerical models separating the effect of the concrete matrix and the fiber reinforcement (explicit representation of fibers) could help to solve this issue, providing more reliable results. They could be also helpful for the process of designing the fiber content of these kinds of concrete.

Although the Embedded Crack Approach is capable for modeling crack trajectories without significant mesh sensitivity, some spurious mesh geometry effect is always unavoidable. In cases of single crack propagation, such as three point bending tests, this mesh effect is noticeable only by minor differences in the load-displacement curves. However, when multiple cracks propagate within the same structural element, as in the case of the slabs subjected to blast load, this may lead to crack patterns predictions significantly different from the experimental ones. To avoid this effect, the use of mesh-free simulation techniques should be explored.

Appendix A

Experimental Recording Sheets

A.1 Compressive tests recording sheets

Compressive Test

Specimen Name: A1

Control: Load

Date of batch: 10/4/2014

Date of Test: 3/11/2014

Test Velocity:

(0.53 ton/Sec)

D (mm)	L (mm)	ρ (kg/m ³)	$P_{\max}$ (kN)	f_c (MPa)	E (GPa)	ν
149.42	296.64	2,366.19	1959.51	111.75	-	-

(a) Load – Displacement Curve

(b) specimen A1 Photo

Fig. A.1: Recording sheet of specimen A1

Compressive Test

Specimen Name: A2

Control: Load

Date of batch: 10/4/2014

Date of Test: 4/11/2014

Test Velocity:

(0.53 ton/Sec)

D (mm)	L (mm)	ρ (kg/m ³)	$P_{\max}$ (kN)	f_c (MPa)	E (GPa)	ν
150.16	297.73	2,361.66	1993.81	112.59	43.96	0.169

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(a) Load – Displacement Curve

(b) specimen A2 Photo

Fig. A.2: Recording sheet of specimen A2

Compressive Test

Specimen Name: A3

Control: Load

Date of batch: 10/4/2014

Date of Test: 4/11/2014

Test Velocity:

(0.53 ton/Sec)

D (mm)	L (mm)	ρ (kg/m ³)	$P_{\max}$ (kN)	f_c (MPa)	E (GPa)	ν
150.12	297.47	2,359.80	1965.88	112.59	46.14	0.178

(a) Load – Displacement Curve

(b) specimen A3 Photo

Fig. A.3: Recording sheet of specimen A3

Compressive Test

Specimen Name: A4

Control: Load

Date of batch: 10/4/2014

Date of Test: 4/11/2014

Test Velocity:

(0.53 ton/Sec)

D (mm)	L (mm)	ρ (kg/m ³)	$P_{\max}$ (kN)	f_c (MPa)	E (GPa)	ν
150.08	297.94	2,361.73	1973.62	111.57	49.07	0.196

(a) Load – Displacement Curve

(b) specimen A4 Photo

Fig. A.4: Recording sheet of specimen A4

Compressive Test

Specimen Name: B1

Control: Load

Date of batch: 10/4/2014

Date of Test: 4/11/2014

Test Velocity:

(0.53 ton/Sec)

D (mm)	L (mm)	ρ (kg/m ³)	$P_{\max}$ (kN)	f_c (MPa)	E (GPa)	ν
150.21	298.03	2,369.83	2010.67	113.46	-	-

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(a) Load – Displacement Curve

(b) specimen B1 Photo

Fig. B.1: Recording sheet of specimen B1

Compressive Test

Specimen Name: B1

Control: Load

Date of batch: 10/4/2014

Date of Test: 4/11/2014

Test Velocity:

(0.53 ton/Sec)

D (mm)	L (mm)	ρ (kg/m ³)	$P_{\text{máx}}$ (kN)	f_c (MPa)	E (GPa)	ν
150.71	298.19	2,346.45	1858.276	104.17	42.85	0.166

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(a) Load – Displacement Curve

(b) specimen B1 Photo

Fig. B.2: Recording sheet of specimen B2

Compressive Test

Specimen Name: B3

Control: Load

Date of batch: 10/4/2014

Date of Test: 5/11/2014

Test Velocity:

(0.53 ton/Sec)

D (mm)	L (mm)	ρ (kg/m ³)	$P_{\max}$ (kN)	f_c (MPa)	E (GPa)	ν
150.56	298.11	2,364.25	2060.65	104.17	46.55	0.175

(a) Load – Displacement Curve

(b) specimen B3 Photo

Fig. B.3: Recording sheet of specimen B3

Compressive Test

Specimen Name: B4

Control: Load

Date of batch: 10/4/2014

Date of Test: 5/11/2014

Test Velocity:

(0.53 ton/Sec)

D (mm)	L (mm)	ρ (kg/m ³)	$P_{\max}$ (kN)	f_c (MPa)	E (GPa)	ν
150.62	296.92	2,421.91	2042.03	116.22	46.22	0.160

(a) Load – Displacement Curve

(b) specimen B4 Photo

Fig. B.4: Recording sheet of specimen B4

Compressive Test

Specimen Name: C2

Control: Load

Date of batch: 10/4/2014

Date of Test: 5/11/2014

Test Velocity:

(0.53 ton/Sec)

D (mm)	L (mm)	ρ (kg/m ³)	$P_{\text{máx}}$ (kN)	f_c (MPa)	E (GPa)	ν
150.15	296.82	2,405.48	2023.60	114.29	48.92	0.180

(a) Load – Displacement Curve

(b) specimen C2 Photo

Fig. C.2: Recording sheet of specimen C2

Compressive Test

Specimen Name: C3

Control: Load

Date of batch: 10/4/2014

Date of Test: 5/11/2014

Test Velocity:

(0.53 ton/Sec)

D (mm)	L (mm)	ρ (kg/m ³)	$P_{\max}$ (kN)	f_c (MPa)	E (GPa)	ν
150.90	297.59	2,394.63	2077.80	116.19	44.71	0.171

(a) Load – Displacement Curve

(b) specimen C3 Photo

Fig. C.3: Recording sheet of specimen C3

Compressive Test

Specimen Name: C4

Control: Load

Date of batch: 10/4/2014

Date of Test: 6/11/2014

Test Velocity:

(0.53 ton/Sec)

D (mm)	L (mm)	ρ (kg/m ³)	$P_{\max}$ (kN)	f_c (MPa)	E (GPa)	ν
149.55	297.75	2,409.60	1948.83	110.97	44.05	0.161

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(a) Load – Displacement Curve

(b) specimen C4 Photo

Fig. C.4: Recording sheet of specimen C4

Appendix A

Experimental Recording Sheets

A.2 Three-point bending tests recording sheets

Test Velocity: 22 $\mu\text{m/s}$

Three-point bending test

Specimen Name: PA1

Control: Load

Date of batch: 10/4/2014

Date of Test: 18/11/2014

Test Velocity:

0.13 $\mu\text{m/s}$

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
99.9	100.6	451	333.3	17.6	11.9	2979.03	337

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(a) Side view of crack

(b) Morphology of crack surface

Fig. PA.1: Recording sheet of specimen PA1

Three-point bending test

Specimen Name: PA2

Control: Load

Date of batch: 10/4/2014

Date of Test: 24/11/2014

Test Velocity:

22 $\mu\text{m/s}$

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	$P_{\max}$ (kN)	G_f N/m	N^o Fibers
101.9	100.7	451	333.3	17.1	12.6	3452.06	399

(a) Side view of crack

(b) Morphology of crack surface

Fig. PA.2: Recording sheet of specimen PA2

Three-point bending test

Specimen Name: PA3

Control: Load

Date of batch: 10/4/2014

Date of Test: 28/11/2014

Test Velocity:

0.13 $\mu\text{m/s}$

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
102.8	99.9	450.0	333.3	16.5	12.0	2846.24	349

(a) Side view of crack

(b) Morphology of crack surface

Fig. PA.3: Recording sheet of specimen PA3

Three-point bending test

Specimen Name: PA4

Control: Load

Date of batch: 10/4/2014

Date of Test: 02/12/2014

Test Velocity:

22 $\mu\text{m/s}$

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	$P_{\max}$ (kN)	G_f N/m	N^o Fibers
101.4	100.8	450	333.3	17.2	11.4	2994.35	406

(a) Side view of crack

(b) Morphology of crack surface

Fig. PA.4: Recording sheet of specimen PA4

Three-point bending test

Specimen Name: PB1

Control: Load

Date of batch: 10/4/2014

Date of Test: 18/11/2014

Test Velocity:

22 $\mu\text{m/s}$

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
101.4	102.2	449.0	333.3	18.0	25.3	5995.06	534

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(a) Side view of crack

(b) Morphology of crack surface

Fig. PB.1: Recording sheet of specimen PB1

Three-point bending test

Specimen Name: PB2

Control: Load

Date of batch: 10/4/2014

Date of Test: 24/11/2014

Test Velocity:

22 $\mu\text{m/s}$

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
102.4	100.1	451.0	333.3	18.4	23.8	6142.19	605

(a) Side view of crack

(b) Morphology of crack surface

Fig. PB.2: Recording sheet of specimen PB2

Three-point bending test

Specimen Name: PB3

Control: Load

Date of batch: 10/4/2014

Date of Test: 28/11/2014

Test Velocity:

22 $\mu\text{m/s}$

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	$P_{\max}$ (kN)	G_f N/m	N^o Fibers
101.4	101.6	452.0	333.3	17.6	17.6	3990.70	486

(a) Side view of crack

(b) Morphology of crack surface

Fig. PB.3: Recording sheet of specimen PB3

Three-point bending test

Specimen Name: PB4

Control: Load

Date of batch: 10/4/2014

Date of Test: 02/12/2014

Test Velocity:

22 $\mu\text{m/s}$

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
100.5	101.7	451.0	333.3	17.6	16.8	4145.78	451

(a) Side view of crack

(b) Morphology of crack surface

Fig. PB.4: Recording sheet of specimen PB4

Three-point bending test

Specimen Name: PC1

Control: Load

Date of batch: 10/4/2014

Date of Test: 18/11/2014

Test Velocity:

22 $\mu\text{m/s}$

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
100.8	101.2	451.0	333.3	16.5	33.6	9159.14	720

(a) Side view of crack

(b) Morphology of crack surface

Fig. PC.1: Recording sheet of specimen PC1

Three-point bending test

Specimen Name: PC2

Control: Load

Date of batch: 10/4/2014

Date of Test: 26/11/2014

Test Velocity:

22 $\mu\text{m/s}$

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	$P_{\max}$ (kN)	G_f N/m	N^o Fibers
99.9	101.7	450.0	333.3	16.7	39.6	11852.39	935

(a) Side view of crack

(b) Morphology of crack surface

Fig. PC.2: Recording sheet of specimen PC2

Three-point bending test

Specimen Name: PC3

Control: Load

Date of batch: 10/4/2014

Date of Test: 28/11/2014

Test Velocity:

22 $\mu\text{m/s}$

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
99.5	100.9	450.0	333.3	17.3	32.5	9676.92	784

(a) Side view of crack

(b) Morphology of crack surface

Fig. PC.3: Recording sheet of specimen PC3

Three-point bending test

Specimen Name: PC4

Control: Load

Date of batch: 10/4/2014

Date of Test: 03/12/2014

Test Velocity:

22 $\mu\text{m/s}$

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
100.7	100.5	451.0	333.3	16.5	39.5	11936.58	962

(a) Side view of crack

(b) Morphology of crack surface

Fig. PC.4: Recording sheet of specimen PC4

Appendix A

Experimental Recording Sheets

A.3 Three-point bending tests recording sheets

Test Velocity: 22 mm/s

Control: Load

Date of Test: 08/12/2014

Test Velocity:

22 mm/s

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P_{max} (kN)	G_f N/m	N^o Fibers
101.6	100.7	451.0	333.3	17.3	15.5	3075.90	326

(a) Side view of crack

(b) Morphology of crack surface

Fig. PA.6: Recording sheet of specimen PA6

Three-point bending test

Specimen Name: PA7

Control: Load

Date of batch: 10/4/2014

Date of Test: 09/12/2014

Test Velocity:

22 mm/s

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
102.5	100.0	450.0	333.3	16.7	16.0	3966.15	435

:

(a) Side view of crack

(b) Morphology of crack surface

Fig. PA.7: Recording sheet of specimen PA7

Three-point bending test

Specimen Name: PA8

Control: Load

Date of batch: 10/4/2014

Date of Test: 09/12/2014

Test Velocity:

22 mm/s

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
99.7	99.4	451.0	333.3	16.2	16.0	4141.08	381

:

(a) Side view of crack

(b) Morphology of crack surface

Fig. PA.8: Recording sheet of specimen PA8

Three-point bending test

Specimen Name: PA9

Control: Load

Date of batch: 10/4/2014

Date of Test: 09/12/2014

Test Velocity:

22 mm/s

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
101.9	99.5	452.0	333.3	16.0	15.2	3930.38	397

(a) Side view of crack

(b) Morphology of crack surface

Fig. PA.9: Recording sheet of specimen PA9

Three-point bending test

Specimen Name: PB6

Control: Load

Date of batch: 10/4/2014

Date of Test: 08/12/2014

Test Velocity:

22 mm/s

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
101.1	100.1	450.0	333.3	16.4	21.6	4985.61	400

(a) Side view of crack

(b) Morphology of crack surface

Fig. PB.6: Recording sheet of specimen PB6

Three-point bending test

Specimen Name: PB7

Control: Load

Date of batch: 10/4/2014

Date of Test: 16/12/2014

Test Velocity:

22 mm/s

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
100.1	101.9	450.0	333.3	18.2	29.4	6556.03	549

:

:

(a) Side view of crack

(b) Morphology of crack surface

Fig. PB.7: Recording sheet of specimen PB7

Three-point bending test

Specimen Name: PB8

Control: Load

Date of batch: 10/4/2014

Date of Test: 16/12/2014

Test Velocity:

22 mm/s

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
101.9	102.6	449.0	333.3	18.7	29.1	6459.03	588

:

(a) Side view of crack

(b) Morphology of crack surface

Fig. PB.8: Recording sheet of specimen PB8

Three-point bending test

Specimen Name: PB9

Control: Load

Date of batch: 10/4/2014

Date of Test: 16/12/2014

Test Velocity:

22 mm/s

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
99.2	101.8	451.0	333.3	17.3	27.2	6624.44	611

:

:

(a) Side view of crack

(b) Morphology of crack surface

Fig. PB.9: Recording sheet of specimen PB9

Three-point bending test

Specimen Name: PC6

Control: Load

Date of batch: 10/4/2014

Date of Test: 08/12/2014

Test Velocity:

22 mm/s

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P_{max} (kN)	G_f N/m	N^o Fibers
100.9	100.5	451.0	333.3	17.0	49.5	14236.81	827

(a) Side view of crack

(b) Morphology of crack surface

Fig. PC.6: Recording sheet of specimen PC6

Three-point bending test

Specimen Name: PC7

Control: Load

Date of batch: 10/4/2014

Date of Test: 12/12/2014

Test Velocity:

22 mm/s

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
100.5	100.3	451.0	333.3	16.8	35.2	8860.71	637

:

(a) Side view of crack

(b) Morphology of crack surface

Fig. PC.7: Recording sheet of specimen PC7

Three-point bending test

Specimen Name: PC8

Control: Load

Date of batch: 10/4/2014

Date of Test: 12/12/2014

Test Velocity:

22 mm/s

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
100.5	101.0	451.0	333.3	17.2	46.5	13173.93	808

:

(a) Side view of crack

(b) Morphology of crack surface

Fig. PC.8: Recording sheet of specimen PC8

Three-point bending test

Specimen Name: PC9

Control: Load

Date of batch: 10/4/2014

Date of Test: 12/12/2014

Test Velocity:

22 mm/s

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
100.0	100.8	451.0	333.3	17.2	44.8	12350.70	761

(a) Side view of crack

(b) Morphology of crack surface

Fig. PC.9: Recording sheet of specimen PC9

Appendix A

Experimental Recording Sheets

A.4 Three-point bending tests recording sheets

Hammer Height: 40 mm (8.85×10^2 mm/s)

Three-point bending test

Specimen Name: PA11

Control: Load

Date of batch: 10/4/2014

Date of Test: 27/2/2015

Test Velocity:

Hammer height 40 mm (8.85×10^2 mm/s)

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
101.75	101.08	451.00	333.3	16.57	29.30	4022.88	373

:

(a) Side view of crack

(b) Morphology of crack surface

Fig. A.11: Recording sheet of specimen PA11

Three-point bending test

Specimen Name: PB11

Control: Load

Date of batch: 10/4/2014

Date of Test: 27/2/2015

Test Velocity:

Hammer height 40 mm (8.85×10^2 mm/s)

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
101.60	102.19	452.00	333.3	17.41	46.89	1923.15	543

:

(a) Side view of crack

(b) Morphology of crack surface

Fig. B.11: Recording sheet of specimen PB11

Three-point bending test

Specimen Name: PC11

Control: Load

Date of batch: 10/4/2014

Date of Test: 27/2/2015

Test Velocity:

Hammer height 40 mm (8.85×10^2 mm/s)

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
101.08	100.63	454.00	333.3	16.41	68.13	4173.69	886

(a) Side view of crack

(b) Morphology of crack surface

Fig. C.11: Recording sheet of specimen PC11

Appendix A

Experimental Recording Sheets

A.5 Three-point bending tests recording sheets

Hammer Height: 160 mm (1.75×10^3 mm/s)

Three-point bending test

Specimen Name: PA12

Control: Load

Date of batch: 10/4/2014

Date of Test: 9/3/2015

Test Velocity:

Hammer height 160 mm (1.75×10^3 mm/s)

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
101.83	100.27	451.00	333.3	16.95	49.30	8359.85	413

(a) Side view of crack

(b) Morphology of crack surface

Fig. A.12: Recording sheet of specimen PA12

Three-point bending test

Specimen Name: PA13

Control: Load

Date of batch: 10/4/2014

Date of Test: 9/3/2015

Test Velocity:

Hammer height 160 mm (1.75×10^3 mm/s)

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
102.07	100.54	451.00	333.3	16.09	62.27	8954.29	212

(a) Side view of crack

(b) Morphology of crack surface

Fig. A.13: Recording sheet of specimen PA13

Three-point bending test

Specimen Name: PA14

Control: Load

Date of batch: 10/4/2014

Date of Test: 9/3/2015

Test Velocity:

Hammer height 160 mm (1.75×10^3 mm/s)

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
101.45	100.09	452.00	333.3	15.91	52.75	7496.58	363

(a) Side view of crack

(b) Morphology of crack surface

Fig. A.14: Recording sheet of specimen PA14

Three-point bending test

Specimen Name: PA15

Control: Load

Date of batch: 10/4/2014

Date of Test: 9/3/2015

Test Velocity:

Hammer height 160 mm (1.75×10^3 mm/s)

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
101.34	100.68	450.00	333.3	16.55	55.68	6867.59	356

(a) Side view of crack

(b) Morphology of crack surface

Fig. A.15: Recording sheet of specimen PA15

Three-point bending test

Specimen Name: PB12

Control: Load

Date of batch: 10/4/2014

Date of Test: 9/3/2015

Test Velocity:

Hammer height 160 mm (1.75×10^3 mm/s)

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
100.95	102.98	450.00	333.3	18.79	58.61	8942.54	475

(a) Side view of crack

(b) Morphology of crack surface

Fig. B.12: Recording sheet of specimen PB12

Three-point bending test

Specimen Name: PB13

Control: Load

Date of batch: 10/4/2014

Date of Test: 9/3/2015

Test Velocity:

Hammer height 160 mm (1.75×10^3 mm/s)

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
101.24	102.53	450.00	333.3	18.27	49.82	12283.94	495

(a) Side view of crack

(b) Morphology of crack surface

Fig. B.13: Recording sheet of specimen PB13

Three-point bending test

Specimen Name: PB14

Control: Load

Date of batch: 10/4/2014

Date of Test: 9/3/2015

Test Velocity:

Hammer height 160 mm (1.75×10^3 mm/s)

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
101.45	101.10	449.00	333.3	17.63	57.14	10572.36	548

(a) Side view of crack

(b) Morphology of crack surface

Fig. B.14: Recording sheet of specimen PB14

Three-point bending test

Specimen Name: PB15

Control: Load

Date of batch: 10/4/2014

Date of Test: 9/3/2015

Test Velocity:

Hammer height 160 mm (1.75×10^3 mm/s)

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
99.74	101.42	451.00	333.3	15.08	78.39	12304.24	551

(a) Side view of crack

(b) Morphology of crack surface

Fig. B.15: Recording sheet of specimen PB15

Three-point bending test

Specimen Name: PC12

Control: Load

Date of batch: 10/4/2014

Date of Test: 6/3/2015

Test Velocity:

Hammer height 160 mm (1.75×10^3 mm/s)

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
99.14	100.43	451.00	333.3	16.71	93.04	13980.51	599

(a) Side view of crack

(b) Morphology of crack surface

Fig. C.12: Recording sheet of specimen PC12

Three-point bending test

Specimen Name: PC13

Control: Load

Date of batch: 10/4/2014

Date of Test: 6/3/2015

Test Velocity:

Hammer height 160 mm (1.75×10^3 mm/s)

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
99.42	100.53	452.00	333.3	16.43	112.09	16546.48	695

(a) Side view of crack

(b) Morphology of crack surface

Fig. C.13: Recording sheet of specimen PC13

Three-point bending test

Specimen Name: PC14

Control: Load

Date of batch: 10/4/2014

Date of Test: 9/3/2015

Test Velocity:

Hammer height 160 mm (1.75×10^3 mm/s)

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
100.83	100.96	452.00	333.3	17.94	101.83	17288.50	725

(a) Side view of crack

(b) Morphology of crack surface

Fig. C.14: Recording sheet of specimen PC14

Three-point bending test

Specimen Name: PC15

Control: Load

Date of batch: 10/4/2014

Date of Test: 9/3/2015

Test Velocity:

Hammer height 160 mm (1.75×10^3 mm/s)

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
101.22	102.74	450.00	333.3	17.96	84.98	17053.15	782

(a) Side view of crack

(b) Morphology of crack surface

Fig. C.15: Recording sheet of specimen PC15

Appendix A

Experimental Recording Sheets

A.6 Three-point bending tests recording sheets

Hammer Height: 360 mm (2.65×10^3 mm/s)

Three-point bending test

Specimen Name: PA16

Control: Load

Date of batch: 10/4/2014

Date of Test: 2/3/2015

Test Velocity:

Hammer height 360 mm (2.65×10^3 mm/s)

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
101.89	101.70	451.00	333.3	17.24	60.81	7732.74	327

(a) Side view of crack

(b) Morphology of crack surface

Fig. A.16: Recording sheet of specimen PA16

Three-point bending test

Specimen Name: PA17

Control: Load

Date of batch: 10/4/2014

Date of Test: 4/3/2015

Test Velocity:

Hammer height 360 mm (2.65×10^3 mm/s)

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
101.24	100.82	451.00	333.3	17.33	58.61	8129.76	339

(a) Side view of crack

(b) Morphology of crack surface

Fig. A.17: Recording sheet of specimen PA17

Three-point bending test

Specimen Name: PA18

Control: Load

Date of batch: 10/4/2014

Date of Test: 4/3/2015

Test Velocity:

Hammer height 360 mm (2.65×10^3 mm/s)

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
102.13	100.60	451.00	333.3	16.67	79.12	9585.55	409

(a) Side view of crack

(b) Morphology of crack surface

Fig. A.18: Recording sheet of specimen PA18

Three-point bending test

Specimen Name: PA19

Control: Load

Date of batch: 10/4/2014

Date of Test: 4/3/2015

Test Velocity:

Hammer height 360 mm (2.65×10^3 mm/s)

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
101.42	101.04	450.00	333.3	16.79	66.67	8953.79	304

(a) Side view of crack

(b) Morphology of crack surface

Fig. A.19: Recording sheet of specimen PA19

Three-point bending test

Specimen Name: PB16

Control: Load

Date of batch: 10/4/2014

Date of Test: 2/3/2015

Test Velocity:

Hammer height 360 mm (2.65×10^3 mm/s)

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
99.78	102.16	450.00	333.3	17.75	71.79	12159.20	473

(a) Side view of crack

(b) Morphology of crack surface

Fig. B.16: Recording sheet of specimen PB16

Three-point bending test

Specimen Name: PB17

Control: Load

Date of batch: 10/4/2014

Date of Test: 6/3/2015

Test Velocity:

Hammer height 360 mm (2.65×10^3 mm/s)

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
99.69	100.98	450.00	333.3	17.64	91.58	14229.67	556

(a) Side view of crack

(b) Morphology of crack surface

Fig. B.17: Recording sheet of specimen PB17

Three-point bending test

Specimen Name: PB18

Control: Load

Date of batch: 10/4/2014

Date of Test: 6/3/2015

Test Velocity:

Hammer height 360 mm (2.65×10^3 mm/s)

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
100.00	101.62	450.00	333.3	17.36	83.52	13993.43	458

(a) Side view of crack

(b) Morphology of crack surface

Fig. B.18: Recording sheet of specimen PB18

Three-point bending test

Specimen Name: PB19

Control: Load

Date of batch: 10/4/2014

Date of Test: 6/3/2015

Test Velocity:

Hammer height 360 mm (2.65×10^3 mm/s)

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
101.88	101.72	449.00	333.3	16.91	83.52	14858.61	528

(a) Side view of crack

(b) Morphology of crack surface

Fig. B.19: Recording sheet of specimen PB19

Three-point bending test

Specimen Name: PC16

Control: Load

Date of batch: 10/4/2014

Date of Test: 2/3/2015

Test Velocity:

Hammer height 360 mm (2.65×10^3 mm/s)

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
100.64	100.28	451.00	333.3	15.87	138.46	20929.75	762

:

:

(a) Side view of crack

(b) Morphology of crack surface

Fig. C.16: Recording sheet of specimen PC16

Three-point bending test

Specimen Name: PC17

Control: Load

Date of batch: 10/4/2014

Date of Test: 6/3/2015

Test Velocity:

Hammer height 360 mm (2.65×10^3 mm/s)

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	$P_{\max}$ (kN)	G_f N/m	N^o Fibers
100.05	99.97	452.00	333.3	15.62	157.51	28879.50	939

(a) Side view of crack

(b) Morphology of crack surface

Fig. C.17: Recording sheet of specimen PC17

Three-point bending test

Specimen Name: PC18

Control: Load

Date of batch: 10/4/2014

Date of Test: 6/3/2015

Test Velocity:

Hammer height 360 mm (2.65×10^3 mm/s)

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
99.72	100.69	451.00	333.3	15.81	170.70	29936.94	910

(a) Side view of crack

(b) Morphology of crack surface

Fig. C.18: Recording sheet of specimen PC18

Three-point bending test

Specimen Name: PC19

Control: Load

Date of batch: 10/4/2014

Date of Test: 6/3/2015

Test Velocity:

Hammer height 360 mm (2.65×10^3 mm/s)

GF Fracture energy up to 3 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	a(mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
100.60	99.54	452.00	333.3	16.26	122.34	20037.36	791

(a) Side view of crack

(b) Morphology of crack surface

Fig. C.19: Recording sheet of specimen PC19

Appendix A

Experimental Recording Sheets

A.7 Blast Test with Three-point bending tests (Residual Strength) (0.007 mm/s)

Three-point bending test

Specimen Name: A1

Control1: Blast Load

Date of batch: 10/4/2014

Date of Test: 10/10/2014

Explosive Quantity:

NOT EXPLODED (REFERENCE)

Control2: Load Test Velocity: 0.007 mm/s

Date of Test: 19/05/2015

GF Fracture energy up to 3.5 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
501	51.21	499	333.3	28.4	2335.49	1127

(a) Side view of crack

(b) Morphology of crack surface

Fig. PD.1: Recording sheet of specimen A1

Three-point bending test

Specimen Name: A2

Control1: Blast Load

Date of batch: 10/4/2014

Date of Test: 10/10/2014

Explosive Quantity:

NOT EXPLODED (REFERENCE)

Control2: Load

Date of Test: 12/05/2015 **Test Velocity: 0.007 mm/s**

GF Fracture energy up to 3.5 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	P_{max} (kN)	G_f N/m	N^o Fibers
503	50.25	499.5	333.3	28.8	2356.39	1237

(a) Side view of crack

(b) Morphology of crack surface

Fig. PD.2: Recording sheet of specimen A2

Three-point bending test

Specimen Name: B4

Control: Blast Load

Date of batch: 10/4/2014

Date of Test: 10/10/2014

Explosive Quantity:

NOT EXPLODED (REFERENCE)

Control2: Load

Date of Test: 8/05/2015

Test Velocity: 0.007 mm/s

GF Fracture energy up to 3.5 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
504	51.57	500	333.3	51.2	5198.73	1794

(a) Side view of crack

(b) Morphology of crack surface

Fig. PD.10: Recording sheet of specimen B4

Three-point bending test

Specimen Name: B5

Control: Blast Load

Date of batch: 10/4/2014

Date of Test: 10/10/2014

Explosive Quantity:

NOT EXPLODED (REFERENCE)

Control2: Load

Date of Test: 8/05/2015

Test Velocity: 0.007 mm/s

GF Fracture energy up to 3.5 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
499.5	55.33	503.5	333.3	43.2	5733.73	1770

(a) Side view of crack

(b) Morphology of crack surface

Fig. PD.11: Recording sheet of specimen B5

Three-point bending test

Specimen Name: C1

Control1: Blast Load

Date of batch: 10/4/2014

Date of Test: 10/10/2014

Explosive Quantity:

NOT EXPLODED (REFERENCE)

Control2: Load

Date of Test: 11/05/2015

Test Velocity: 0.007 mm/s

GF Fracture energy up to 3.5 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
500.5	51.24	504.5	333.3	61.5	7037.49	2008

(a) Side view of crack

(b) Morphology of crack surface

Fig. PD.13: Recording sheet of specimen C1

Three-point bending test

Specimen Name: C2

Control1: Blast Load

Date of batch: 10/4/2014

Date of Test: 10/10/2014

Explosive Quantity:

NOT EXPLODED (REFERENCE)

Control2: Load

Date of Test: 8/05/2015 **Test Velocity: 0.007 mm/s**

GF Fracture energy up to 3.5 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
500.5	49.95	504	333.3	60.9	7040.97	2216

(a) Side view of crack

(b) Morphology of crack surface

Fig. PD.14: Recording sheet of specimen C2

Appendix A

Experimental Recording Sheets

A.7 Blast Test with Three-point bending tests (Residual Strength) Blast Quantity: (22 Cartridges) = Goma 2: 3.33 kg = TNT: 3.18 kg

Test Velocity: 0.007 mm/s

Three-point bending test

Specimen Name: A4

Control: Blast Load

Date of batch: 10/4/2014

Date of Test: 10/10/2014

Explosive Quantity:

(17 Cartridges) = Goma 2: 2.58 kg = TNT: 2.46 kg

Control2: Load

Date of Test: 20/05/2015

Test Velocity: 0.007 mm/s

GF Fracture energy up to 3.5 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
501	52.15	501.5	333.3	24.7	2350.49	994

(a) Side view of crack

(b) Morphology of crack surface

(c) Slab after the explosion

Fig. PD.4: Recording sheet of specimen A4

Three-point bending test

Specimen Name: B2

Control1: Blast Load

Date of batch: 10/4/2014

Date of Test: 10/10/2014

Explosive Quantity:

(17 Cartridges) = Goma 2: 2.58 kg = TNT: 2.46 kg

Control2: Load

Date of Test: 15/05/2015 **Test Velocity: 0.007 mm/s**

GF Fracture energy up to 3.5 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	P_{max} (kN)	G_f N/m	N^o Fibers
502.5	55.07	500.5	333.3	45.2	4523.07	1586

(a) Side view of crack

(b) Morphology of crack surface

(c) Slab after the explosion

Fig. PD.8: Recording sheet of specimen B2

Three-point bending test

Specimen Name: C4

Control: Blast Load

Date of batch: 10/4/2014

Date of Test: 10/10/2014

Explosive Quantity:

(17 Cartridges) = Goma 2: 2.58 kg = TNT: 2.46 kg

Control2: Load

Date of Test: 13/05/2015

Test Velocity: 0.007 mm/s

GF Fracture energy up to 3.5 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
503	50.99	500.5	333.3	55.8	6397.44	2419

(a) Side view of crack

(b) Morphology of crack surface

(c) Slab after the explosion

Fig. PD.16: Recording sheet of specimen C4

Appendix A

Experimental Recording Sheets

A.7 Blast Test with Three-point bending tests (Residual Strength) Blast Quantity: (22 Cartridges) = Goma 2: 3.33 kg = TNT: 3.18 kg

Test Velocity: 0.007 mm/s

Three-point bending test

Specimen Name: A3

Control: Blast Load

Date of batch: 10/4/2014

Date of Test: 10/10/2014

Explosive Quantity:

BROKEN COMPLETELY IN THE TEST

GF Fracture energy up to 3.5 mm displacement

B (mm)	D (mm)	L (mm)
501	52.15	501.5

:

(a) Side view of Slab

(b) Slab before the test

Fig. PD.3: Fragment of the slab

Three-point bending test

Specimen Name: A5

Control: Blast Load

Date of batch: 10/4/2014

Date of Test: 10/10/2014

Explosive Quantity:

(22 Cartridges) = Goma 2: 3.33 kg = TNT: 3.18 kg

Control2: Load

Date of Test: 20/05/2015

Test Velocity: 0.007 mm/s

GF Fracture energy up to 3.5 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
503.5	51.76	452.0	333.3	9	618.82	977

(a) Side view of crack

(b) Morphology of crack surface

(c) Slab after the explosion

Fig. PD.5: Recording sheet of specimen A5

Three-point bending test

Specimen Name: A6

Control: Blast Load

Date of batch: 10/4/2014

Date of Test: 10/10/2014

Explosive Quantity:

(22 Cartridges) = Goma 2: 3.33 kg = TNT: 3.18 kg

BROKEN COMPLETELY IN THE TEST

GF Fracture energy up to 3.5 mm displacement

B (mm)	D (mm)	L (mm)
503.5	51.76	452.0

(a) Slab after the test

(b) Fragments slab after the blast

Three-point bending test

Specimen Name: B1

Control1: Blast Load

Date of batch: 10/4/2014

Date of Test: 10/10/2014

Explosive Quantity:

(22 Cartridges) = Goma 2: 3.33 kg = TNT: 3.18 kg

Control2: Load

Date of Test: 5/05/2015

Test Velocity: 0.007 mm/s

GF Fracture energy up to 3.5 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
502.5	56.42	501.5	333.3	48.9	4718.32	1465

(a) Side view of crack

(b) Morphology of crack surface

(c) Slab after the explosion

Fig. PD.7: Recording sheet of specimen B1

Three-point bending test

Specimen Name: B3

Control1: Blast Load

Date of batch: 10/4/2014

Date of Test: 10/10/2014

Explosive Quantity:

(22 Cartridges) = Goma 2: 3.33 kg = TNT: 3.18 kg

Control2: Load

Date of Test: 7/05/2015

Test Velocity: 0.007 mm/s

GF Fracture energy up to 3.5 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
502.5	52.3	499.5	333.3	55.8	4427.19	1511

(a) Side view of crack

(b) Morphology of crack surface

(c) Slab after the explosion

Fig. PD.9: Recording sheet of specimen B3

Three-point bending test

Specimen Name: B6

Control: Blast Load

Date of batch: 10/4/2014

Date of Test: 10/10/2014

Explosive Quantity:

(22 Cartridges) = Goma 2: 3.33 kg = TNT: 3.18 kg

Control2: Load

Date of Test: 7/05/2015

Test Velocity: 0.007 mm/s

GF Fracture energy up to 3.5 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
503	55.21	500	333.3	56.1	4908.97	1723

(a) Side view of crack

(b) Morphology of crack surface

(c) Slab after the explosion

Fig. PD.12: Recording sheet of specimen B6

Three-point bending test

Specimen Name: C3

Control1: Blast Load

Date of batch: 10/4/2014

Date of Test: 10/10/2014

Explosive Quantity:

(22 Cartridges) = Goma 2: 3.33 kg = TNT: 3.18 kg

Control2: Load

Date of Test: 7/05/2015

Test Velocity: 0.007 mm/s

GF Fracture energy up to 3.5 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
504	52.05	499.5	333.3	70.7	7788.08	2027

(a) Side view of crack

(b) Morphology of crack surface

(c) Slab after the explosion

Fig. PD.15: Recording sheet of specimen C3

Three-point bending test

Specimen Name: C5

Control: Blast Load

Date of batch: 10/4/2014

Date of Test: 10/10/2014

Explosive Quantity:

(22 Cartridges) = Goma 2: 3.33 kg = TNT: 3.18 kg

Control2: Load

Date of Test: 6/05/2015

Test Velocity: 0.007 mm/s

GF Fracture energy up to 3.5 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
502.5	51.38	500.5	333.3	60.7	6517.37	2007

(a) Side view of crack

(b) Morphology of crack surface

(c) Slab after the explosion

Fig. PD.17: Recording sheet of specimen C5

Three-point bending test

Specimen Name: C6

Control: Blast Load

Date of batch: 10/4/2014

Date of Test: 10/10/2014

Explosive Quantity:

(22 Cartridges) = Goma 2: 3.33 kg = TNT: 3.18 kg

Control2: Load

Date of Test: 6/05/2015

Test Velocity: 0.007 mm/s

GF Fracture energy up to 3.5 mm displacement

B (mm)	D (mm)	L (mm)	S (mm)	P _{max} (kN)	G _f N/m	N ^o Fibers
501	52.14	504	333.3	65.7	6874.58	2143

(a) Side view of crack

(b) Morphology of crack surface

(c) Slab after the explosion

Fig. PD.18: Recording sheet of specimen C6

Appendix B

Integration to calculate DIF_{ft}

$$G_{F_{Dynamic}} = \int d(\tilde{w}) \cdot f(\tilde{w}) d\tilde{w}$$

$$= \text{Dynamic Increase Factor} \cdot G_{F_{stat}}$$

$$d(\tilde{w}) = DIF_f(\tilde{w}, \dot{\epsilon}) = \left[1 + (DIF_t(\dot{\epsilon}) - 1) \left(1 - \frac{\tilde{w}}{\tilde{w}_c} \right) \right] \quad \text{Eq. (5.19)}$$

$$\rho = DIF_t(\dot{\epsilon}) - 1$$

Dynamic Increase Factor (DIF):

$$d(\tilde{w}) = d = 1 + \rho \left(1 - \frac{\tilde{w}}{w_c} \right) = 1 + \rho - \rho \frac{\tilde{w}}{w_c}$$

$$i_d = (1 + \rho)\tilde{w} - \frac{\rho}{2w_c} \tilde{w}^2$$

$$I_d = \frac{(1 + \rho)\tilde{w}^2}{2} - \frac{\rho}{6w_c} \tilde{w}^3$$

$$\mu = f$$

$$dv = d$$

$$v = i_d$$

$$G_{F_{Dynamic}} = \int d(\tilde{w}) f(\tilde{w}) d\tilde{w}$$

$$f i_d \Big|_0^{w_c} - \int_0^{w_c} i_d f' dw$$

$$= f(w_c) \cdot i_d(w_c) - f(0) \cdot i_d(0) - \int_0^{w_k} i_d f' dw - \int_{w_k}^{w_c} i_d f' dw$$

$$= -f(0) \cdot i_d(0) - f'_1 I_d \Big|_0^{w_k} - f'_2 I_d \Big|_{w_k}^{w_c}$$

$$\begin{aligned}
 &= -f_1' \left[\frac{(1+\rho)}{2} w_k^2 - \frac{\rho}{6w_c} w_k^3 \right] \\
 &\quad - f_2' \left[\frac{(1+\rho)}{2} w_c^2 - \frac{\rho}{6w_c} w_c^3 - \frac{(1+\rho)}{2} w_k^2 + \frac{\rho}{6w_c} w_k^3 \right] \\
 &= (-f_1' + f_2') \left[\frac{1+\rho}{2} w_k^2 - \frac{\rho}{6w_c} w_k^3 \right] - f_2' \left[\frac{1+\rho}{2} w_c^2 - \frac{\rho}{6w_c} w_c^3 \right] \\
 &= (-f_1' + f_2') \beta^2 \cdot w_c^2 \left[\frac{1+\rho}{2} - \frac{\beta \cdot \rho}{6} \right] - f_2' \cdot w_c^2 \left[\frac{1+\rho}{2} - \frac{\rho}{6} \right] \\
 G_{F_{Dynamic}} &= \left(\frac{f_t(1-\alpha)}{\beta w_c} - \frac{f_t \alpha}{w_c(1-\beta)} \right) \frac{\beta^2 \cdot w_c^2}{6} [3 + 3\rho - \beta\rho] + \frac{f_t \alpha}{w_c(1-\beta)} \cdot \\
 &\quad \frac{w_c^2}{6} [3 + 3\rho - \rho] \quad \text{Eq. 1s}
 \end{aligned}$$

From the static fracture energy ($G_{f_{stat}}$) which is:

$$\begin{aligned}
 G_{f_{stat}} &= \frac{1}{2} \beta \tilde{w}_c \cdot f_t \cdot (1 - \alpha) + \beta \tilde{w}_c \cdot \alpha f_t + \frac{1}{2} \tilde{w}_c (1 - \beta) \cdot \alpha f_t \\
 &= \tilde{w}_c \cdot f_t \cdot \left[\frac{\beta}{2} - \frac{\alpha\beta}{2} + \beta\alpha + \frac{\alpha}{2} - \frac{\alpha\beta}{2} \right] = \tilde{w}_c \cdot f_t \cdot \frac{\alpha + \beta}{2}
 \end{aligned}$$

Now, we have to multiply ($G_{f_{stat}}$) by Dynamic Increase Factor (DIF) to get:

$$\begin{aligned}
 G_{F_{Dynamic}} &= G_{f_{stat}} \cdot DIF_{GF} = \left(f_t \cdot w_c \cdot \frac{\alpha + \beta}{2} \right) \cdot DIF_{GF} \\
 &\quad \text{Eq. 2s}
 \end{aligned}$$

From this Eq. 1s and Eq. 2s we can get:

$$\begin{aligned}
 &\left[\frac{1-\alpha}{\beta} - \frac{\alpha}{1-\beta} \right] \frac{\beta^2}{3} [3 + 3\rho - \beta\rho] + \frac{\alpha}{1-\beta} \frac{1}{3} [3 + 2\rho] = (\alpha + \beta) \cdot DIF_{GF} \\
 &\quad \frac{(1-\alpha)(1-\beta) - \alpha\beta}{(1-\beta)\beta} \cdot \frac{\beta^2}{3} [3 + 3\rho - \beta\rho] + \frac{\beta\alpha}{(1-\beta)\beta} \frac{1}{3} [3 + 2\rho] \\
 &\quad = (\alpha + \beta) \cdot DIF_{GF}
 \end{aligned}$$

$$\begin{aligned} & \beta^2(1 - \alpha - \beta + \alpha\beta - \alpha\beta) \cdot (3 + 3\rho - \beta\rho) + \alpha\beta(3 + 2\rho) \\ & = 3(\alpha + \beta)\beta(1 - \beta) \cdot DIF_{GF} \end{aligned}$$

$$(\beta - \alpha\beta - \beta^2) \cdot (3 + 3\rho - \beta\rho) + 3\alpha + 2\alpha\rho = 3(\alpha + \beta)(1 - \beta) \cdot DIF_{GF}$$

$$\begin{aligned} & (\beta - \alpha\beta - \beta^2) \cdot (3 - \beta)\rho + 2\alpha\rho \\ & = 3(\alpha + \beta) \cdot DIF_{GF}(1 - \beta) - 3\alpha - 3(\beta - \alpha\beta - \beta^2) \end{aligned}$$

$$\rho = \frac{3[(\alpha + \beta) \cdot DIF_{GF}(1 - \beta) - \alpha - (\beta - \alpha\beta - \beta^2)]}{(\beta - \alpha\beta - \beta^2) \cdot (3 - \beta)\rho + 2\alpha}$$

$$DIF_t(\dot{\epsilon}) = 1 + \rho = 1 + \frac{3[(\alpha + \beta) \cdot DIF_{GF}(1 - \beta) - \alpha - (\beta - \alpha\beta - \beta^2)]}{(\beta - \alpha\beta - \beta^2) \cdot (3 - \beta)\rho + 2\alpha}$$

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