Universidad Politécnica de Madrid

**Escuela Técnica Superior de Ingenieros de Caminos, Canales
y Puertos**

**Mechanical Characterization of Tensile
Behaviour of Concrete Under High Strain Rates**

Tesis Doctoral

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Ingeniero de Caminos, Canales y Puertos

Ingeniero de Materiales

2019

Departamento de Ciencia de Materiales

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2019

A mis padres

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Agradecimientos

Estas primeras líneas están dedicadas a las personas gracias a las cuales hoy puedo presentar mi tesis doctoral. A mis dos directores y amigos, Francisco Gálvez y David Cendón, porque gracias a su confianza he podido disfrutar estos cuatro años de una de las experiencias más enriquecedoras y bonitas de mi vida. Su apoyo y dedicación en el plano laboral y personal es algo que siempre agradeceré.

De igual manera, quiero agradecer el cariño y amistad de todos mis compañeros de Departamento en donde, después de cuatro años, he encontrado un grupo de personas con las que me gustaría seguir compartiendo muchas cosas en el futuro. De manera especial tengo que citar a aquellos con los que he tenido la suerte de compartir momentos más estrechos. A Adel, por ayudarme en la fabricación de los hormigones; a Josemi, por su apoyo, ánimo y cariño desde que entré en el Departamento; a Chus, por ayudarme en mis primeros días; a Gustavo y Álvaro, que me sugirieron durante mi etapa de estudiante de Materiales la remota idea de hacer un Doctorado; a la gente del taller, quien siempre está dispuesta a echarte una mano y aportar ideas para tu trabajo; a todos los profesores y catedráticos del Departamento, porque de cada uno he podido sacar valiosas lecciones. Como no, a mi compañero de grupo y de despacho, Rafa, porque me ha hecho reír y disfrutar estos años más allá del plano laboral, porque me ha ayudado si lo he necesitado y porque juntos hemos compartido muy buenos momentos.

A toda la gente que da forma y sentido al departamento, Beatriz, Elena, Tere, Toñi, Rafa, Monica, Mariceli, Dani, Sandra. A todos los que han pasado y ya no están. A Vicente, por haber hecho tan buena gestión de nuestro Departamento estos años. A Ana y a Rosa, por su cariño y amabilidad a la hora

de ayudarme con todo tipo de papeleos. Gracias a todos.

Quiero aprovechar estas lineas para agradecer a M^a Luisa Ruiz Ripoll asi como a Christoph Roller la posibilidad que me brindaron de poder vivir una experiencia inmejorable en Alemania. A M^o Luisa, por abrirme las puertas y allanarme el camino y a Christoph, por acogerme y buscar tiempo para mí durante mi estancia. Igualmente, gracias a los revisores internacionales de esta tesis, Dr. Sidney Chocron y Dr. Andrea Spagnoli, por aceptar tamaño encargo con tanta predisposición y tiempo tan ajustado.

Al Ministerio de Economía y Competitividad por hacer económicamente posible esta tesis doctoral a través de la beca FPI BES-2015-072269.

Y por último, a lo más importante, mi familia. A mis padres, a los que debo todo lo que soy. A Ana, porque en lo bueno y en lo malo, siempre está a mi lado. Y como no, a Bilma y a Beti, por alegrar mis vueltas a casa cada día.

Abstract

The use and development of concrete has seen a continuous evolution since its first use 2000 years ago. Manufacturing processes, new mix designs, innovative ingredients and combinations with other materials have made concrete a key building material for all type of modern structures. Even though the knowledge around concrete properties has reached the maximum standards, the use of this material for the most challenging infrastructures has introduced new loads and variables in the projection of structures. Bridges, military structures, dams, off-shore platforms, train structures, all of them subjected not only to the classical quasi-static loads but also to different dynamic actions.

As opposite to quasi-static loads, the characterization of concrete under dynamic actions is still a field of scarce development, with few experimental data and without a clear experimental proposed methodology. The importance of having a deeper knowledge in the dynamic behavior of concrete is justified over the well known fact that, several mechanical properties of concrete show a notorious increase under dynamic actions.

In the recent years some authors, noting the relevance of this change in the properties, have proposed different experimental techniques to estimate mechanical parameters defining the behavior of concrete. Thanks to the development of the experimental device designed by Hopkinson, a significant increase in the study of concrete subjected to impact loading has taken place during the last decades under compressive and tensile configurations. However, the nature of the dynamic actions in concrete, has focused the problem and interest on the tensile properties, due to the responsibility of these actions in the final collapse of structures subjected to spalling failure.

In an effort to increase the existing data, experimental methodologies using

the Hopkinson Bar and based on the spalling phenomenon have tried to recreate the transmission of pulses inside concrete material, in an effort to estimate dynamic mechanical properties governing the fracture of concrete such as the tensile strength and the fracture energy. Despite the work carried during the last decades, there is still some dispersion among published results with not a clear or reliable proposed methodology to predict the dynamic behavior of concrete.

In the present work an important effort has been carried to design an easy experimental methodology to estimate dynamic fracture properties of concrete. Experimental configurations based on the modified Hopkinson Bar are proposed both for estimating the tensile strength and fracture energy. Moreover, a validation experimental campaign has been carried out, comparing results with an alternative approach.

Finally, the developed methodology has been used to characterize and study two different concrete mixes under tensile spalling tests.

CHAPTER 1

Introduction and Objectives

1.1 Introduction

Technology, development and study of concrete began 2000 years ago. With the rise of the Roman Empire, technology of construction suffered its most relevant transformation. A new material called "opus caementicium" was developed and used in the forge of the greatest empire in history. The basic components of this revolutionary materials were two: mortar and stones. The use of this new material allowed the development of new structures, more complex in shapes and designs, overcoming the simplicity given by the previous technologies. From the first version of the "opus caementicium", some evolution during the roman period appeared, leading to a final product which has been hardly improved during these last decades, and leaving, as a proof of its great durability and reliability, many architectural examples.

From that initial mixture developed by the romans, the main difference with the modern concrete is the use of a Portland Cement as a mortar, instead of lime. The development of this new mixture began during the last part of the 18th century [1], where the use of concrete was recovered and deeply studied [2], arising up to our days and becoming the most popular material for building a huge variety of infrastructures. This popularity is well justified looking at its magnificent properties as the high compressive strength, low density, ease of molding and workability, low price, chemical and corrosion resistance and

resistance to fire and high temperatures. However, the good reputation cannot only be explained by these intrinsic properties but also due to its good behavior when it is combined with other materials. This ability of combination with different materials leads to mixed structural elements which can notably reduce one of the main drawbacks of concrete, which is its low or residual tensile strength. For this reason, design of concrete structures is always done from a compressive point of view being the study of concrete, during most of its life, centered around compressive properties.

As the technology and time evolves, requirements and limits also change. From the first roman use of concrete, where the importance laid in the construction of structures which could withstand gravity and natural actions, the requirements for the moderns structures are much higher and different in nature. The industrial human development has led to bigger and more sophisticated structures like dams, nuclear power plants, bridges, offshore platforms and different kind of military facilities, all of them subjected not only to well-known quasi-static loads but also to less frequent, but probable, dynamic unpredictable events like impacts, accidental explosions, vehicle accidents, terrorist attacks, earthquakes and many more. The unknown behavior of these structures under such type of loadings makes the study of concrete a permanent task.

Nowadays concrete remains as the key material for building civil and military constructions and the desire of having safer and long-lasting structures has increased the bandwidth of the considered design loads. Even though it may seem an old and well known material, its analysis remains an important duty for many researchers. From the classical quasi-static loads, usually consisting on compressive efforts, dynamic loads are now being considered into the design of new facilities.

Dynamic loads are introduced in structures in the form of pulses of short duration and differs from quasi-static events in three main facts; first, the acting time can be reduced to less than milliseconds, compared to the stable, long-lasting static loads; second, these dynamic loads travel through the material introducing inertia effects which have to be considered in the equations; third, once inside the material, these loads can change in sign, going from initial compression to tension after reflecting at the free faces of the structure [3].

1.1 Introduction

These tensile actions will cause the failure of brittle materials like concrete, leading to projection of fragments which can cause great damage [4] or even the progressive collapse of the structure, considered the main cause of deaths and injuries in structures subjected to impulsive loads [5].

The experimental study and analysis of concrete subjected to dynamic loading is a field of scarce development. A critical milestone regarding the dynamic testing of materials came with the studies of Prof. Bertram Hopkinson, whose famous Hopkinson Bar inspired later researchers to develop new techniques able to introduce impulse loadings in a wide range of materials. From that moment, loads introduced in the tested specimen through explosions or projectile impacts, could be recorded and studied in controlled experiments, which increased notably the experimental work carried in the dynamic range. However, new testing methodologies were still needed to obtain experimental results. In this sense, different authors began to establish some criteria and procedures around the common Hopkinson Bar device, in order to obtain particular mechanical parameters of the materials tested.

Regarding concrete study, the Hopkinson Bar was adapted to focus the analysis in the tensile fracture, considered the main reason for the failure of structures under dynamic regime. Starting with the first works by Reinhardt and Zielinski [6] and Ross [7] in the 80's decade, and continuing with an extensive investigation during the 90's with important experimental research carried by Weerheijm [8]. A big effort has been made since this initial works to increase the experimental background around the dynamic properties of concrete under high strain regime, deeply updated in the 2000 decade by Schuler [9], Brara [10], Erzar [11] and Forquin [12] among others.

All the experimental work carried by the previously commented authors has demonstrated that properties of concrete vary depending on whether a quasi-static or a dynamic analysis is performed. In the case of the tensile strength, experimental tests performed have shown an important increase in this value, meaning that more complex mechanisms take place under dynamic events. This leads to a different behavior and an important change in its properties, raising a new type of problem whose solution cannot be located in the classical codes and demand an intense effort to determine dynamic mechanical properties which

define the real behavior of materials under high strain rates. Achieving that is also important to improve material models so that they can be capable of predicting structure's life, not only under static loads but also subjected to sudden dynamic impacts.

High-strain-rate characterization parameters, as the dynamic tensile strength and fracture energy, are now being widely studied and as introduced, one of the most promising experimental techniques for dynamic analysis of concrete and other quasi-brittle materials is the Hopkinson Bar technique. Up to the date, this experimental device has been used by many researchers to study critical dynamic parameters. Even more, the modified configuration of the bar, which has been used in this work, has allowed the development of tensile pulses in concrete which lead to fracture by spallation.

In the case of tensile strength, many authors have proposed different ways for measuring it. An important amount of experimental data has been collected, showing in most of the cases a significant exponential growth of the corresponding values with increasing strain rate. On addition, there is not too much experimental data for fracture energy or at least a clear methodology proposed for measuring this parameter in an accurate way. Even though energy experiments have already been conducted by some authors, a deeper study and more experimental data is needed in order to have a clear understanding of the energetic response of concrete material under dynamic loads.

Despite all the experimental work carried up to date, there still exists great dispersion among published results, mainly due to a lack of a suitable or standardized experimental procedure. Implementing an easy, economical and reliable methodology is essential in order to be able to determine the main dynamic properties of concrete which permits a proper description of its dynamic behavior.

In this work a new experimental methodology, based on the Hopkinson Bar Technique, is proposed for measuring dynamic tensile strength and fracture energy of concrete subjected to high strain rates. A description on the experimental set-up as well as the data analysis procedure is fully carried in the present document. Experimental testing over unnotched and notched specimens has been carried, obtaining dynamic fracture parameters for different types of

1.2 Objectives

concrete mixes. An experimental validation campaign has been also conducted in order to check the suitability and versatility of both the proposed set-up and the defined methodology.

1.2 Objectives

The development of the present PhD Thesis pursues the achievement of the following objectives:

- Design and optimization of an experimental technique, based on the Hopkinson Bar device, for determining dynamic tensile strength and fracture energy of concrete.
- Perform initial experimental tests over cylindrical concrete specimens to calibrate the experimental device.
- Validate the proposed methodology and set-up by comparing the estimated results by two different approaches and an alternative set-up.
- Obtain and compare the dynamic fracture properties of two different concrete mixes. A Conventional and a High Performance Concrete will be tested.

CHAPTER 2

Literature Review

The interest in tensile properties of concrete has seen a notorious increase during the last decades, mainly caused by the rising interest in the high strain-rate analysis. However, having a controlled tensile test has always been a very complicated task, even under quasi-static regime. In this section, a review of the evolution of the tensile testing in concrete is carried, beginning from the first configurations used in quasi-static tests, in which the complexity of achieving such state will be outlined, up to the introduction of the spalling methods in dynamic experiments based on Hopkinson Bar schemes. These opened the possibility of having tensile efforts in concrete and ceramic materials, overcoming the main drawback introduced by quasi-static methods, which is the need for intermediate elements to transfer the tensile load into the specimen. Once the principles of tensile testing are set, a deep overview of the contribution and progress made by the different authors to the dynamic testing procedures and methodology will be carried.

Classically, due to its nature and main applications, the study of concrete has been focused in determining its properties and behavior when subjected to quasi-static compressive loads. Having a pure compressive state to characterize a material is relatively easy, because the load is directly transmitted to the specimen through the machine plates by simple contact. As opposite, when working with hard and brittle materials, performing a valid tensile test is not often possible. The reason is that other additional or intermediate materials are

required to achieve the tensile state. If a direct tensile test wants to be done, pulling the sides of the specimen to generate a tensile effort requires the use of special supports or supplementary materials, like glues, to be able to pass the load to the tested specimen [13]. All these can interfere the behavior of the complete specimen by creating critical load-concentration points which come from the extreme difference between the brittle material and the glue used to fix it.

One of the first authors to note the difficulty, as well as the relevance, of testing concrete materials under tensile efforts, was Joseph Derwent Todd. In his paper from 1955 [14], Todd presented the possibility of approaching the ideal stress-strain curve under a direct tensile test by what he defined as a beam test, in which a momentum was responsible for introducing tensile efforts in the specimen. This can be considered the first approach to the very well known three-point-bending test, currently used to define the quasi-static behavior of concrete under tension.

Continuing with the work developed by J. D. Todd, during the 60's, early studies first by Kupfer, Rusch and Hilsdorf [15] and later by Hughes [16] and Evans [17] focused on the characterization of concrete under tensile loadings. Hilsdorf showed the impossibility of the direct tensile test to identify any decrease in the stress after the maximum value was reached. Using the standard universal testing machine, Hughes' and Chapman's work presented a new technique based on the use of a stiffening member and special glues in order to minimize the problems generated by contact and differences in Young Modulus. The existence of a maximum peak and a subsequent falling branch in the tensile stress-strain curve were observed. Following this line, a deeper analysis on the micro-cracking and the failure branch was carried by Evans and Marathe [17]. Using a modified testing machine, complete stress-strain curves in direct tension were obtained.

At the same time, the study of materials subjected to high strain rates began to acquire more interest. The revolutionary Hopkinson Configuration, proposed in 1914 by Prof. Betram Hopkinson [18], opened a wide horizon of possibilities for testing materials under dynamic conditions. At that moment, measuring the effects of such dynamic loadings was not feasible, and the only

reference of the impact load that could be recorded was the momentum transfer into the impacted element, being the measurement of its components, that is, pressure and acting time, a very complicated task. Prof. Bertram Hopkinson overcame this problem with an original device able to transmit pressure waves into different metallic pieces and record the momentum and acting time on them. Looking at the basic working principle, it can be even stated that the original design of Prof. Hopkinson (see Figure 2.1) is the basis of the most recent spalling configurations to develop tensile waves into brittle materials. The device shown in the previous figure is composed of four different parts. A main steel shaft hanged by wires (B), free to move in horizontal and vertical directions and connected to a pencil and paper recording all its displacements. In one end of this shaft, the gun-cotton cylinder is fixed by short splints of wood (A), producing the compressive pulse after detonation. At the end of this shaft, a smaller steel piece is hold in contact by using a magnetic field (C). At a certain distance, a wood box is suspended again by wires (D), being free to move in horizontal and vertical directions and, as in the case of the shaft, it is also connected to pencil and paper to record the distances moved.

Figure 2.1: Original Hopkinson Bar

In this initial device, a projectile is fired against one end of the shaft or an explosive charge is detonated at that end. A compressive pulse is then

introduced into the shaft in contact with the steel piece tested. Once reflected at the end of the piece, the arising tensile pulse will separate the piece from the shaft and fly with a trapped momentum until is captured by the wood box. This momentum can be measured with the help of the ballistic pendulum defined by the wood box. In order to reconstruct the approximate pressure-time curve, Hopkinson used different lengths of projectiles so, apart from registering the momentum increase between them, the different intervals of the pressure-time curve could be obtained thanks to the increase of the length of the tested end, which led to an increment of the time interval.

Many researches worked over the basis of the theory exposed by Hopkinson, improving the configuration and adapting it to different type of tests. The first known application, using the Hopkinson Bar to determine dynamic mechanical properties of materials, was presented by Taylor [19] in 1946. Some years before, Taylor and Davies had used a Hopkinson Bar to test soft materials following the configuration showed in Figure 2.2.

Figure 2.2: Scheme of the Hopkinson Bar configuration used by Taylor and Davies to test soft materials in 1942

Volterra, who had worked with Taylor at Cambridge between 1938 and 1946, was responsible for the previous design of the Hopkinson apparatus. He later updated this design and presented it in his own paper in 1948 [20], with the first

description of a Split Hopkinson Bar (SHB) formed by incident and transmitted bars hanged by strings (see Figure 2.3). Volterra used this Split configuration to test polymer materials under dynamic compressive loads.

Figure 2.3: Scheme of the Split Hopkinson Bar configuration used by E. Volterra to test polymeric materials in 1948

A critical development in the use of Hopkinson Bars was achieved with Kolsky in 1949 [21]. Based on the split design by Volterra, Kolsky introduced the use of explosive charges to reach higher pulses and thus increase the strain rate reached up to that moment. Similarly to the original idea by Hopkinson, Volterra's design used a pendulum system to impact the sample, with the consequent limit in the stress reached during the test, but Kolsky changed the triggering system to explosive charges as described in his scheme shown in Figure 2.4. From that moment, the Kolsky Bar became the most used configuration for testing under compressive and tensile dynamic efforts. This use has continued up to date, being this system adopted by many researchers to implement own methodologies to obtain different dynamical properties of a huge variety of materials.

Regarding tensile characterization, first applications of the Kolsky Bar allowed to perform direct tensile testing up to strain rates of $1\text{E}+01\text{ s}^{-1}$. Results

Figure 2.4: Scheme of the SHB configuration designed by Kolsky in 1949

of dynamic tensile strength were obtained in this range first by Birkimer in the early 70's [22], showing a notorious dependence of the tensile strength with the strain rate reached. During the 80's, Zielinski and Reinhardt [23] focused their efforts in the study of the influence of the strain rate on concrete fracture under tensile loading. The SHB technique, with the specimens glued to incident and transmitted bars, was then used to create uniaxial tensile loading with maximum stress rates of $30 \text{ N/mm}^2\text{ms}$. From the obtained stress-strain diagrams, as for the case of Birkimer, an increase in the maximum strength reached by tested concrete was observed. In this work, some ideas about the influence of multi-fracturing in that increase were outlined. More recently in the 90's, similar configurations were used by Weerheijm [8] and Zheng [24], to contribute with new valuable data in the previous given strain rate range, confirming the dependence of the tensile strength on the strain rate tested.

The SHB configuration has also been adopted by some authors introducing particular modifications in the cross-section of incident and transmitted bars, different from the usual cylindrical geometry. Albertini and Montagnani [25] designed a rectangular cross-section aluminum bar system to test alternative geometries for the concrete specimens (see Figure 2.5). Also, a new triggering procedure was adopted in this configuration by using a prestressed steel bar

Figure 2.5: Scheme of the Split Hopkinson Bar configuration used by Albertini to test rectangular concrete specimens under tensile loading

to introduce a rectangular pulse, avoiding the need for projectiles or explosive charges.

Despite all the benefits exposed, the use of a SHB configuration to promote a tensile-stress state was heavily discussed, mainly due to the problems derived from the use of glued contacts which can lead to a lack of repeatability and cause bad transmission of the incident and transmitted pulses, to and from the concrete specimen respectively. Moreover, the use of the methodology, based on the analysis of incident and transmitted pulses, which relies in the existence of equilibrium between initial and end faces of the tested specimen, can lead to inexact estimations of the measured stresses and strains due to testing of long concrete specimens in which the equilibrium cannot be reached before fracture. In the aim of going deeper into the tensile characterization and trying to overcome the commented intrinsic problems of the SHB configuration, some authors proposed an alternative specimen configuration to indirectly obtain the tensile strength. As in the case of indirect quasi-static tensile test, or commonly known as Brazilian test, the SHB technique was also used to test concrete discs under compressive loads [26]. The advantages of this technique can be derived from the quasi-static tests, where compression is used to quantify the tensile mechanical properties thanks to some approach and hypothesis. In dynamic Brazilian tests, a compression pulse is introduced into the system and

transferred to the concrete specimen. With this technique, equilibrium can be reached in both contact ends of the concrete disc, and so a good estimation of the indirect tensile strength can be obtained. The problem of the Brazilian test comes from the limitation in the size of the disk that can be tested with some geometries of SHB. The diameter of incident and transmitted bars determines the maximum length of the tested disc, which can either limit the maximum aggregate size of the tested specimen or difficult the use of a representative sample regarding the ratio of the length with respect to the aggregate size.

To overcome this limitation, a new idea for obtaining tensile pulses in brittle materials comes from the so called “Spalling phenomenon”. The spalling process can be understood as the natural failure process of structural elements subjected to impulse loads. As described in the introduction, dynamic actions introduce compression waves in concrete structures which travel through the material and are reflected at the free surfaces. This reflection leads to a change in the sign of the introduced efforts in concrete, from compression to tension. These reflected tensile stresses are responsible for the failure of many structural elements, due to the weak resistance to these type of efforts. This failure phenomena, caused by reflected tensile pulses, is known as spallation or spalling, and is the base for many different experimental approaches regarding the study of mechanical properties of concrete under dynamic tensile loading.

The interest in using this phenomena to study dynamic behavior of concrete comes from the particularity of brittle materials, which have a very high compressive strength compared to the tensile resistance. This difference allows introducing compressive waves into the material, high enough to promote fracture after reflection but, at the same time, limited below the compressive strength and thus ensuring no previous damage to the tensile failure.

One of the pioneers using this spallation technique was Mark k. McVay who conducted spall experiments over concrete slabs by directly applying explosive charges [27]. To reach this spallation state, other more controlled option came with the development of the modified configuration of the Hopkinson Bar. The modified configuration is basically reached by removing the transmitted bar from the original SHB configuration, and so leaving a free-end specimen in contact with the incident bar. The modified configuration is defined by three

elements: a projectile or explosive charge, an incident bar and the specimen located at the end in contact with the previous bar. The spallation of the concrete specimen can be achieved by the developed tensile stresses after reflection of the compressive pulse at the free end of the sample. The development of this experimental technique, combined with a more detailed analysis of the fracture evolution and micro-cracking, led to an increase in the precision in the fracture identification which allowed the estimation of the dynamic fracture properties of concrete at increasing strain rates. This modified configuration was seen in the works by Klepaczko [28] and Brara [29] where, using a projectile to generate the pulse, strain rates in the range of 10 to 120 s⁻¹ were reached. The results showed, not only the expected increase in the tensile strength, but also the definition of an exponential trend in the dynamic increase factor reached for strain rates over 100 s⁻¹. This increase was later confirmed by experimental data provided by other authors and very well summarized in the plot by Schuler (see Figure 2.6). A first interest for the influence of the content of water in the dynamic behavior was also outlined. Dry and wet specimens were tested in the previous works by Klepaczko and Brara, and a detailed analysis of the waves was carried in order to ensure the validity of the estimation of the tensile strength, as strain gauges were only used to record at the incident bar. This analysis was required to be able to shift the transmitted pulses into the concrete specimen, allowing an analytical reconstruction of the reflection of the pulse. This was the base for later methods using the analytical reconstruction of waves to derive the tensile strength. These methods have to fulfill many different criteria to make a good prediction of the tensile strength. Possibly, among all the requirements, the most important fact is to be able to identify the exact position of the first fracture plane, and is there where the geometry of the projectile plays a critical role.

The precision of the analytical methodologies to measure the dynamic tensile strength relies in two main facts; first, the validity of the shifting of the pulse at the specimen free end requires no superposition of waves. This can be ensured by using long concrete specimens, at least in the order of the wavelength. Once the nonexistence of superposition of waves is ensured, the precise identification of the first fracture is critical. Apart from using the latest technology, other

Figure 2.6: DIF as a function of the strain rate obtained by different authors [9]

considerations, as the previously commented projectile geometry, can be taken into account. Regarding the length of the tested specimen, configurations in which the specimen is linked to the incident bar by gluing are very limited in the length of the sample that can be tested. If the specimen is just supported by the glue, the weight introduces a momentum which can interfere the contact or even separate the specimen from the incident bar. Testing longer specimens, with a sufficiently high length-to-diameter ratio, requires an alternative system to support the sample, ensuring a good contact to pass the compression pulse into the concrete tested.

In the case of ceramic materials, the size of the specimen is not so critical as no aggregate size has to be taken into account. Gálvez [30] used a similar spalling configuration to test ceramic materials under tension. In his work, the importance of the initial fracture in the specimen was outlined. The use of a modified projectile in order to shift the shape of the pulse was proposed and a deep analysis of the influence of the projectile geometry in the fracture process was carried. As exposed, the use of cylindrical projectiles can lead to the simultaneous fracture in the specimen, complicating the identification of the first or initial fracture. For that reason, Gálvez proposed the modification of the tail of the projectile in order to obtain non symmetrical pulses which

results in a clear identification of the maximum tensile peak at every instant of the reflection process.

Other interesting solution to overcome the problem of multi-fracture is the use of pulse shapers. Pulse shapers appeared as a way of having more controlled and defined pulses, specially when using explosive charges, instead of projectiles, to generate de compressive pulse. This form of creating the incident pulse leads to many uncertainties, as for example controlling the exact amount of explosive charge required to produce a certain pulse. Trying to have more control over the detonation charges, pulse shapers appeared as a good solution [31]. However, the use of these pulse shapers does not offer the resolution and precision given by the use of projectiles to generate controlled pulses. Regarding the shaping of the introduced pulses, one interesting technique was proposed by H. Wu [32] to test long concrete specimens. In this work, despite cylindrical projectiles are used, a non-rectangular shape of the compressive pulse can be achieved thanks to the use of an intermediate conical transmission element between projectile and incident bar. Even though it shows very good results when shifting the pulse, the use of intermediate elements, apart from complicating the geometry of the bar, complicates the prediction of the real shape of the final compressive pulse, as shown by the difference in the length of the pulse predicted by the numerical simulations with respect to the experimental recorded value obtained in the work. Shifting the pulse through the projectile geometry has shown better performance as the length of the resulting pulse can be accurately approached by modifying the projectile length.

In the recent years, finding the dynamic tensile strength has been only a part of the total problem of concrete subjected to tensile efforts. To fully describe the fracture process, also the dynamic fracture energy needs to be known. The fracture energy can be described as the consumed energy during the fracture process, and can be expressed in terms of crack opening distance and tensile stress required. The quasi-static case can be easily solved with the well known Three Point Bending test (TPB) over notched specimens in which, at every instant, the applied tensile stress and the crack opening can be measured. The dynamic case does not offer such simplicity for the energy computation, as no

equilibrium is reached during the tests performed in a Hopkinson Bar under spalling configuration.

Knowing the intrinsic difficulty of the energy problem, some authors tried to estimate the fracture energy by alternative ways as the exposed for the quasi-static case. As opposite to the tensile strength, there is not so many experimental data regarding the fracture energy and there is even not a clear proposed methodology to compute it in an accurate way. The common fact among all the experimental techniques suggested by the authors is the use of indirect ways for estimating the energy consumed during the fracture process. The reason is that numerous are the problems when trying to compute the value. First, the difficulty of focusing the fracture in one fixed section; second the impossibility of avoiding the formation of multiple cracks or micro-cracking, which makes the real calculation of the fracture surface an impossible task. Third, there is no way for measuring the crack opening together with the applied stress, which prevents from having the well known load-displacement curves used in quasi-static test to compute the consumed energy during the fracture process

H. Schuler tried to overcome the previous difficulties by proposing a methodology for computing the energy based on an indirect kinetic approach. In his work [9], Schuler performed a combined research of the two exposed mechanical properties, the tensile strength and fracture energy. His estimations of the tensile strength were based on the Novikov's approximation for fracture of metals under explosive loadings [33]. The advantage of the Novikov's approximation, extensively used by many researchers, is that it offers a very closed estimation of the tensile strength by using only the measurement of the free-end velocity. It was proposed first for the estimation of the tensile strength for metals and lately extended to brittle and concrete materials. Schuler performed spalling tests over unnotched and notched specimens at different loading stages which resulted in a range of strain rates from 30 to 80 s^{-1} . The Hopkinson configuration used by Schuler included a short cylindrical steel projectile and a long aluminum incident bar. The tested specimens were glued and hanged to the end of the incident bar, limiting the maximum tested length to 250 mm due to the effect of momentum produced by the weight of the specimen. From the Novikov's approach, an increase in the tensile strength can be observed, leading

to an exponential law. In the case of the fracture energy, increase factors up to 3 for the tested strain rates were found, following a similar trend as the one found for the tensile strength. As commented, the measurement of the fracture energy is done in an indirect process, by computing the change in the momentum before and after spallation occurs. The main difficulty of this technique is to be able to determine with enough accuracy the moments before and after the crack initiates, so that the velocities are measured at the exact moment to obtain accurate results of the change in momentum. This approach used by Schuler can be considered as a partial estimator of the fracture energy as it does not take into account the internal energy.

Another indirect approach for the estimation of the fracture energy, was proposed by Weerheijm and Van Doormaal [34]. The main objective of their study was focused in developing a material numerical model able to predict the behavior of concrete in a wide variety of strain rates. The main limitation for these models is the lack of experimental data required to validate them. Up to date, it is true that experimental tests are very limited, mainly in the case of fracture energy, in number and most important, in the range of strain rates tested. Overcoming this issue is not a simple task. Reaching higher impact velocities is not the only aspect to consider. As opposite to the tensile case, an optimal estimation of the fracture energy is limited by the damage introduced in concrete when testing over a certain regime. In the search of higher strain rates, Weerheijm and Van Doormaal proposed the use of detonators instead of metallic projectiles and the indirect approach used to estimate the fracture energy was based on a balance of energies during the spalling process. Being this a good approach, some aspects should be considered to avoid wrong estimations of all the parameters. First, the location of the notch in an appropriate section is fundamental. An analysis on the reflection of the pulse at the free end has to be carried in order to identify the section in which the probability of reaching the tensile strength is higher. A combination of experiments on unnotched specimens with an analytical study of the reflection should be considered to locate the notch at the most suitable position. Second, the correct measurement of every energy participating in the fracture process requires clear records at the strain gauges. Any superposition at the gauges of the incident and reflected

pulses has to be avoided. Superposition of the pulses is mainly related to the relation between the pulse length and the length of the tested specimen. The pulse length is almost fully dependent on the projectile length, when using projectiles to produce the incident pulse. Controlling the projectile geometry is a very good way to adequate the wavelength to the specimen length. Third, the damage during the energy tests have to be limited to the notched section. In order to have a close estimation of the consumed energy during the fracture process, any other source of energy consumption has to be avoided or at least, reduced to the minimum.

During the last decade, an extensive effort has been carried by P. Forquin and B. Erzar who have been working deeply over the study of tensile strength of concrete, developing a particular set-up, analyzing and comparing different methodologies. In [35] a comparison of the approaches by Novikov, Klepaczko and Gálvez is carried, showing a very good agreement among all of them. A special care has been taken in many of their works to control the pulse and reach a uniform stress in a wide length of the tested specimen. Reaching this uniform distribution is done in order to achieve a fracture under constant strain rate in several sections however, this way of testing can lead to the formation of multiple fractures at the same time. Further studies by Forquin and Erzar, focused in the importance of water content in the dynamic behavior of tensile fracture of concrete [12]. In these studies, a certain influence of water presence in the pores was remarked, showing an increase of the tensile strength in wet specimens over dry specimens. This sets the problem of the dynamic behavior of porous materials like concrete into the microscale, with a need of understanding those interactions between water and concrete matrix. Some progress has been made in this field with the help of mesoscale modeling and analysis [36], trying to define the macroscopic behavior based on the interaction of the heterogeneous mix which conforms concrete. However, validating those models requires much more experimental data in the macroscale, adding new and reliable data, at least for the two main fracture dynamic properties which define the tensile fracture. The application of these models to a wide range of dynamic events demands also the development of experimental procedures able to reach higher strain-rate testing ranges.

CHAPTER 3

Experimental and numerical basis

3.1 Introduction

In this chapter a brief description of the Hopkinson Bar system is given. From the innovative technique presented by Bertram Hopkinson, the evolution and changes made over the basic scheme are described. For the study of the dynamic mechanical properties of concrete, the spalling configuration has been chosen as the basis for a new experimental technique. This spalling test is fully summarized here and, for a complete understanding of the proposed methodology, the basis of the unidimensional wave propagation theory needs to be briefly explained. For the analysis and validation of the experiments, a numerical model has been also developed in LS-DYNA. This numerical model is briefly introduced in this section and will be widely described in [Chapter 4](#).

3.2 The Hopkinson Bar

The initial idea of what we call today Hopkinson Bars, comes from the test performed and published by Prof. Bertram Hopkinson in the year 1913 [18], whose original device supposed a revolution for measuring the pulse shapes

produced by an impact of a projectile or a detonation caused by explosives (Figure 2.1).

That initial device was later adopted and modified by Kolsky [21] to add a second transmitted bar to the system, with the idea of using it to test materials subjected to compression. Kolsky was also responsible for the introduction of explosive charges to trigger the projectile, and thus rose the range of strain rates reached. The design by Kolsky is the basis for the today's known Split Hopkinson Bar (SHB) whose experimental configuration is given in Figure 3.1. In the case of metals, polymers and composite materials, this SHB configuration can be used to perform compression (see Figure 3.1[a]) or tensile tests (see Figure 3.1[b]).

Figure 3.1: Compressive and tensile configurations for the Split-Hopkinson-Bar

To test brittle materials under tension, two bar configurations are possible. First, the use of the previous split configuration to perform Brazilian tests (Figure 3.2), where the tested specimens are disks limited to a certain length and diameter, and second, in the majority of cases the spalling phenomenon is frequently used, leading to a different experimental configuration known as the modified or spalling configuration. The main difference with respect to the SHB is that the transmitted bar is removed and so the specimen is placed next to the incident bar. The basic scheme is shown in Figure 3.3.

Three main elements can be found in this configuration; first, the projectile,

3.2 The Hopkinson Bar

Figure 3.2: SHB scheme under dynamic Indirect Tensile Tests

Figure 3.3: Modified Hopkinson Bar

which is fired against the incident bar with a certain air pressure to generate a initial compression pulse. As exposed in Chapter 2, the geometry and length of the projectile are responsible for defining the final shape and wavelength of the initial pulse. Depending on the author, the material chosen for the projectile can also vary, being the most usual steel or aluminum.

Second, the incident bar which is responsible for collecting the initial pulse generated during the projectile impact and transferring it to the tested specimen. The ideal incident bar should have a large length-to-diameter ratio in order to allow for perfect unidimensional wave propagation, avoiding wave dispersion and 3-D effects. Incident bars ranging from 1000 mm to a couple of meters can be seen. Regarding the material, in order to be able to test hard ceramics, incident carbon steel bars as well as high-performing aluminum alloys are used.

Third, the tested specimen is placed at the end of the incident bar. This configuration, with the specimen having a free-end face, is responsible for the reflection of the initial compressive pulses into tensile efforts, which lead to fracture of brittle materials under tension.

3.3 Unidimensional wave propagation

To understand the spalling phenomenon it is necessary to review the unidimensional wave propagation theory. A theoretical development is carried in this section in order to clarify the evolution of the introduced pulses into the different elements of the Hopkinson scheme as well as the reflection process responsible for the spalling fracture of concrete. For a more detailed view on this topic, the texts by Zukas [3], Kolsky [37] and Rinehart [38] offer a wider theoretical development on the matter of the elastic stress wave propagation.

The unidimensional wave propagation can be studied taking a close look at a finite section of an infinite bar (Figure 3.4), through which a stress pulse travels in its longitudinal direction x .

Figure 3.4: Finite section of an infinite bar subjected to a stress wave

Applying the movement equation (3.1) to the finite section

$$dF = dm \cdot da \quad (3.1)$$

and knowing that applied force can be written as the stress times the area, the movement expression yields

$$\frac{\delta \sigma}{\delta x} = \rho \frac{\delta v}{\delta t} \quad (3.2)$$

with ρ being the density of the material and v the particle velocity. Both

3.3 Unidimensional wave propagation

the stress and the particle velocity can be expressed in terms of the particle displacement u

$$\sigma = \varepsilon E = \frac{\delta u}{\delta x} E \quad (3.3)$$

$$v = \frac{\delta u}{\delta t} \quad (3.4)$$

and so equation (3.2) can be rewritten as

$$\frac{\delta^2 u}{\delta x^2} = \frac{\rho}{E} \frac{\delta^2 u}{\delta t^2} \quad (3.5)$$

From the unidimensional wave propagation, it is known that the propagation velocity (c) of the waves depends on the density and the Young modulus of the material

$$c = \sqrt{\frac{\rho}{E}} \quad (3.6)$$

entering in (3.5), the equation for the unidimensional wave motion is finally derived

$$\frac{\delta^2 u}{\delta x^2} - c^2 \frac{\delta^2 u}{\delta t^2} = 0 \quad (3.7)$$

The previous motion equation is valid for uniaxial stress or strain states and it is defined by the particle movement u , the time t , the velocity of the waves c and x being the position of a certain point from the bar origin and the positive growing marking the propagation direction of the pulse.

The solution to equation (3.7) can be obtained by integration, leading to

$$u(x, t) = f(x - ct) + g(x + ct) \quad (3.8)$$

The movement $u(x, t)$ can be expressed as the sum of two waves; $f(x - ct)$ which represents a pulse traveling in the positive direction of the x-axis, and $g(x + ct)$ representing a pulse moving in the opposite direction, both waves with the same velocity c .

The relation between σ and particle velocity v can be obtained if we derive equation (3.8) with respect to position and time to obtain the strain ε and velocity v respectively

$$\varepsilon(x, t) = \frac{\delta u(x, t)}{\delta x} = f'(x - ct) + g'(x + ct) \quad (3.9)$$

$$\sigma(x, t) = E\varepsilon(x, t) = E[f'(x - ct) + g'(x + ct)] \quad (3.10)$$

$$v(x, t) = \frac{\delta u(x, t)}{\delta t} = -cf'(x - ct) + cg'(x + ct) \quad (3.11)$$

Relating the previous expressions, the stress and velocity of a particle at a certain section are linked by

$$\sigma = \pm \rho cv \quad (3.12)$$

Depending on the adopted sign criteria for compression and tension pulses and the direction of propagation of them, the positive or negative form of the previous expression will be chosen. For the case of the present work, compressions have been chosen to be negative, while tension is taken as positive. In such a case, a wave traveling to the positive direction of the x-axis will be represented in negative form ($g(x + ct)$), and in positive when traveling in the negative direction ($f(x - ct)$).

The way of representing different magnitudes of the pulse as the sum of two waves, acquires its maximum relevance when describing the reflection process at the free end of a bar, which can be considered the basis for the spalling phenomenon. As exposed in the previous paragraphs, a pulse traveling to the positive direction of the x-axis can be directly represented by the corresponding term $f(x - ct)$, and for the same reason, any pulse traveling back through the bar to the negative direction will be expressed with one term $g(x + ct)$. Just when the reflection process begins, the evolving pulse is represented by the superposition or addition of the two terms (see Figure 3.5). This theoretical basis will be later used to illustrate the result of that process.

3.4 The Spalling test

Figure 3.5: Process of pulse reflection at the free end

3.4 The Spalling test

In a Hopkinson test, when a projectile hits the incident bar, a compressive pulse is created at the contact face. The shape and length of this initial pulse depends entirely on the geometry of the projectile used. For the case of a cylindrical projectile hitting a long cylindrical bar, both made of the same material, the stress peak (σ_{peak}) developed during the impact time (t_c) is directly related to the projectile impact velocity (v_{proj}) through the expressions of the elastic waves

$$\sigma_{peak} = \rho c \frac{v_{proj}}{2} \quad (3.13)$$

The maximum stress introduced into the system can be controlled by just adjusting the shooting pressure, since it changes the velocity of the projectile. The pulse duration (t_p), and thus the wavelength, can be determined by the projectile length (L_{proj}) through the next relation

$$t_p = 2 \frac{L_{proj}}{c} \quad (3.14)$$

The pulse duration is directly proportional to the contact time. This contact time can be deduced from the time that the waves need to travel from the contact face to the opposite end of the projectile and back again to the contact face. As it will be later explained, due to the reflection at the free end of the projectile, a tensile pulse is developed, which separates the projectile and the

incident bar when arriving at the interface. In the particular case of identical materials for the projectile and incident bar, and assuming an ideal cylindrical projectile, a good estimation of the expected wavelength (L_p) can be directly derived from the previous expression as

$$L_p \approx 2L_{proj} \quad (3.15)$$

After hitting the incident bar, a compressive pulse with the magnitude σ_{peak} and length t_p defined previously, is created traveling through the incident bar at a wave speed c defined by the material. Let us assume that, at the end of the incident bar, we place a concrete specimen. When the compressive pulse reaches the end of the bar, it is partially transmitted to the concrete specimen and partially reflected back to the incident bar. The amount of pulse that can be passed into the specimen from the incident bar, and the corresponding that is reflected, is given by the coefficients determined by the materials in contact and also by the transversal sections at the interface. These, called transmission and reflection coefficients, can be derived by imposing equilibrium of forces and compatibility of velocities at the contact face

$$A_1(\sigma_I - \sigma_R) = A_2(\sigma_T) \quad (3.16)$$

$$V_I - V_R = V_T \quad (3.17)$$

(3.17) can be expressed in terms of stresses using (3.12)

$$\frac{\sigma_I}{\rho_1 c_1} - \frac{\sigma_R}{\rho_1 c_1} = \frac{\sigma_T}{\rho_2 c_2} \quad (3.18)$$

From these last two expressions, the reflected stress pulse (σ_R) and the corresponding transmitted one (σ_T) can be defined in terms of the incident pulse (σ_I), the areas of the sections in contact (A_1, A_2) and the material constants (ρ_1, c_1, ρ_2, c_2)

$$\sigma_T = \frac{2A_1\rho_2c_2}{A_1\rho_1c_1 + A_2\rho_2c_2}\sigma_I \quad (3.19)$$

3.4 The Spalling test

$$\sigma_R = \frac{2A_2\rho_2c_2 - A_1\rho_1c_1}{A_1\rho_1c_1 + A_2\rho_2c_2}\sigma_I \quad (3.20)$$

Depending on the previous transmission coefficient between the incident bar and the concrete, a certain amount of compressive pulse enters the specimen. For the success of the spalling test, this compression has to be limited below the compressive strength of concrete, so no damage is introduced in the specimen before the tensile efforts can act. Due to the high compressive resistance in brittle materials, which can be up to 10 times bigger with respect to the tensile strength, pulses with enough tension to produce the fracture of concrete can be introduced without damaging it by compressions. The pulse inside concrete travels through the specimen until it reaches the free end and is reflected back. Different frames of the reflection process are shown in Figure 3.6. As explained in the previous section, this process can be understood as the superposition of two waves; one compressive wave traveling in the positive direction while leaving the specimen and the other, a tensile wave which travels in the negative direction while entering the specimen. This composition of two identical waves with opposite sign is necessary to achieve null stress at the free end of the specimen, which defines the boundary condition. During this reflection, the stress state at every instant can be obtained from the sum of the two pulses, with the compressive stresses tending to be reduced while the tensile efforts arise. In an ideal situation without any fracture, at the end of the reflection, the result should be a tensile pulse, identical to the initial compressive one, traveling back through the specimen.

More things happen during the reflection process apart from this variation on the sign of the efforts. Regarding the energy for example, an interesting transference between kinetic and elastic energies takes place. As illustrated in Figure 3.7 four phases can be distinguished regarding the energy evolution during a spalling tensile test. Phase 1 is defined by a sudden increase in the total energy introduced into the system, caused by progressive introduction of the compression pulse into the concrete specimen. This total energy is the result of the sum of two energy components, kinetic and elastic or internal. Kinetic energy comes from the movement transmitted by the pulse to the particles

Figure 3.6: Evolution of tensile stresses after the pulse reflection

forming the specimen. The velocity of each particle under the effect of the pulse will lead to a kinetic equivalent energy of

$$E_k = \frac{1}{2} \int v^2 dm \quad (3.21)$$

Internal or elastic energy depends on the deformation of each of the sections conforming the specimen, caused by compressive or tensile pulses. Then, the resulting internal energy of the specimen under the effect of the pulse can be computed by

$$E_i = \frac{1}{2} \int E \varepsilon^2 dV \quad (3.22)$$

Once the compressive pulse has completely entered the specimen, the maximum energy into the system is reached, entering Phase 2, where all the energies keep a constant value and suffer no change as the pulse travels through the specimen. When the compressive pulse reaches the free end, Phase 3 begins, with a rise in the kinetic energy and an equivalent fall of the internal part. The superposition of a compressive pulse exiting the specimen and a tensile pulse entering, both moving the particles in the positive direction of the x-axis, results in the increase of the kinetic energy as a result of the duplicated velocity.

3.5 Numerical model

Figure 3.7: Energy balance along the spalling test

By contrast, the same superposition of pulses, with opposite signs, results in the removal of the initial compressive strains, resulting in the fall of the internal energy. Despite this balance of energies, the total energy into the system remains constant as no loss is recorded during this reflection process.

As the compressive pulse disappears from the specimen, leading to a pure tensile state, velocities and strains recover the initial state, caused this time by a tensile pulse instead of the initial compressive one, leading to Phase 4 where again, kinetic and internal energies remain constant.

3.5 Numerical model

As part of this work, a numerical study on the technique and methodology proposed has been carried. To do so, the software LS-DYNA has been used to perform the simulations regarding the tensile and energy tests in a modified Hopkinson Bar. Specially in the case of energy tests, a particular material model subroutine, based on the cohesive crack theory, has been implemented

and used both to design a suitable position for the notch and to validate the proposed experimental procedure.

The user-programmed subroutine for the concrete material was implemented in the explicit finite element solver LS-DYNA. This constitutive model is based on the Cohesive Crack Model with Strong Discontinuity Approach developed by J. Planas and J.M. Sancho [39] and has been largely used for the analysis of concrete under quasi-static regime [40]. The constitutive model was later modified [41] to account for high strain rate effects. The model can be described as linear elastic with failure just under tensile efforts once the maximum principal exceeds the value of the tensile strength. To do so, different shapes of softening curves can be selected in order to fully represent the cohesive behavior of each particular concrete.

CHAPTER 4

Experimental concept and design

4.1 Introduction

To obtain the dynamical properties of concrete, an experimental methodology has been developed in this work. This new technique is focused in the estimation of the tensile strength and fracture energy from concrete specimens subjected to high strain rate under spallation fracture.

In this section, the design of the set-up and methodology are described. Based on the spalling configuration, a new experimental device is manufactured at the UPM Materials Department to test concrete specimens. Continuing with the developments proposed by previous authors, the actual design intends to overcome the problems and difficulties exposed in Chapter 2. The design is combined with a new methodology, using the latest sensor technology and a high-speed video camera to extract precise information and obtain the mechanical properties of concrete with an increased reliability.

The present section begins with the description of the Hopkinson Bar system configuration. Details on the different parts of the adopted modified configuration are given. Complementing the experimental scheme, a focused study over the projectile geometry is carried to improve the accuracy in the experimental testing.

The second part of this section is devoted to the description of the proposed methodology. Regarding the tensile strength configuration, new critical aspects influencing the final result of the test have been reviewed. Among these, the previously mentioned projectile design and the proper fracture plane identification have been carefully treated. In the case of the projectile and its influence on the fracture pattern, a mixed experimental-numerical study has been carried out. For the fracture identification, the combined technique of high-speed camera with Digital Image Correlation (DIC), has been paramount to determine the fracture initiation time.

In the case of the estimation of the dynamic fracture energy, an alternative experimental configuration is needed. To this end, some variations in the set-up with respect to the tensile configuration are required, being the introduction of a circumferential notch in the specimens the main difference. All the process followed to have a proper design and position of the notch is described. Moreover, a new methodology is also proposed to estimate accurately the fracture energy.

In the final part, the initial calibration tests are presented. Results for both the tensile strength and the dynamic fracture energy are given for a conventional concrete in order to check the viability of the technique to measure both mechanical parameters.

4.2 Experimental set-up design

The experimental set-up for the spalling tests consists on a modified Hopkinson Bar shown in Figure 4.1. This modified configuration is composed of three main parts: a conic projectile, an incident bar and the specimen.

Along this work, conic steel projectiles with different lengths and constant impact diameter of 22 mm will be used. The projectile is placed inside the cannon connected to a pressure air tank. The alignment of the projectile is done with the help of polymeric rings around it, to guide it and keep it straight. Thanks to these rings, pressure can be effectively transmitted to the projectile without any loss of air between projectile and inner wall of the cannon. The adjustment of the projectile velocity can be achieved by fitting the tank pressure.

4.2 Experimental set-up design

Figure 4.1: Scheme of the experimental set-up used in this work

However, a perfect shift of the velocity is not possible, due to external factors which interfere this velocity, like the friction coefficient between supports of the projectile and inner cannon part, the pressure release control and the effect of temperature on the polymeric supports.

The incident bar, of 1000 mm in length and 22 mm in diameter, is made up of a high-yield-strength steel. This bar is instrumented in its center with one strain gauge connected through a Wheatstone Bridge. At the end of the impact bar, the specimen is located, placed over a steel frame of 1500 mm in length, consisting on vertical stair rods to which two horizontal squared rods are screwed (see Figure 4.2). Fixed to the rods, multiple steel bearings are connected over which the sample is laid, acting as rails for the concrete specimen to move freely in the longitudinal direction. These small bearings allow the cylindrical specimen to move in the horizontal plane without any restrictions, thus enabling the fracture process by spalling. The whole support system is fixed to the main bench preventing other displacements.

Inside the specimen, located at the impact end and centered with the longitudinal axis of the specimen, a metallic rounded piece is embedded in the concrete (Figure 4.3). The interaction between the concrete specimen and the

Figure 4.2: Manufactured steel frame to support the concrete specimen

Figure 4.3: Metallic piece embedded in the concrete specimen

incident bar is accomplished thanks to this piece. This steel piece of 30 mm in length and 25 mm in diameter was previously mechanized at the surface in contact with the concrete in order to have a better link between both parts reaching a perfect engagement, needing no glues or cements to have a good interaction between incident bar and specimen and standing both together by simple contact using a special oil. The wave is then passed from the incident bar into the specimen, minimizing the effects of the change in sections and materials in contact.

Depending on the test performed, the concrete specimens are instrumented with different strain gauges. Also, velocities of the ejected free end are recorded

4.3 Projectile design

by using a PCB Piezotronics accelerometer, model 350B04, with a measurement range of 50000 g placed at that point.

The data acquisition system is composed of a Vishay signal conditioner, model 2210, with four channels to shift the signal from the gauges, a digital Tektronix oscilloscope, model TDS 714 L, of 500 MHz response to register the data from gauges and a computer. The testing device is completed with a Vision Research high-speed camera, model Phantom V12.1. The images from the high-speed camera are post-processed using a digital image correlation software [42] in order to identify the chronological appearance of the different cracks. The oscilloscope as well as the camera are triggered by the signal registered in the strain gauge placed in the incident bar. The acquisition period of the gauges was set to 10000 points ms^{-1} with a total record of 5 ms. That makes a total record of 50000 points.

4.3 Projectile design

A critical point of the proposed experimental set-up is the definition of the projectile geometry. It has been shown in the previous sections how the projectile length influences the resulting compressive incident pulse, determining the pulse duration. Moreover, the shape of the generated pulse can be determined by the shape of the projectile. To illustrate this influence, the case of an ideal cylindrical projectile is taken as an example. Ideal projectiles are those in which the length is much higher than the diameter so that dispersion or 3-D effects are minimized and can be neglected. When an ideal cylindrical projectile impacts the incident bar, a rectangular pulse is introduced (Figure 4.4 (a)), defined by two ascending and descending branches and an intermediate horizontal plate. The height of the horizontal region is defined by equation 3.13, while the slope of the ascending and descending branches, ideally perfectly vertical, is actually determined by the interaction between projectile and incident bar faces. A good finish of both surfaces, improving the whole contact, will lead to a vertical slope while a bad contact will decrease this slope. The impact velocity can also affect the slopes, as the higher the velocity, the deeper the surface contact.

Some authors have remarked the importance of rectangular pulses due to

the need of having a uniform tensile stress in a wide amplitude of the tested specimen [35]. By having a sufficiently long projectile, a pulse in the length range of the specimen can be introduced in concrete, and thus, achieve a constant strain rate in practically all the sections. However, when testing brittle materials and depending on the methodology used to estimate the mechanical properties, this could be a non ideal situation. By introducing a rectangular pulse we are having the same strain level at every section and hence the same risk of producing a fracture in each of those sections. It is then easy to see that, in the majority of cases, fracture at concrete material will happen in several sections at practically the same time. This can be defined as multi-fracture. In the case of the tensile strength estimation, the methodology proposed in this thesis relies on the formation of one single initial fracture in concrete, clearly defined and differentiated from the rest. The influence of the fracture location in the study of the dynamic fracture energy is much more critical due to the need of computing accurately the fracture surface in order to estimate the specific fracture energy. To overcome this problem of multi-fracture, the use of triangular pulses with different ascending and descending slopes emerges as a good solution.

The rectangular shape can be modified in two ways; first, as it has been exposed in the previous text, by altering the impact velocity of the contact surface between projectile and incident bar; and second, by introducing some modifications in the shape or geometry of the projectile. The idea of using shaped-modified projectiles was first proposed by Prof. Francisco Gálvez [43] [30] in the analysis of ceramic materials subjected to spallation and this same idea has been adopted in the present work to develop triangular pulses inside concrete specimen by using conical projectiles.

Opposed to the ideal rectangular pulses, the triangular pulses are becoming of great interest in order to avoid multi-fracture and locate the fracture section in a predefined position of the specimen. Apart from the shape, the way the pulses are reflected is also different. The idea behind the conical shape, as opposed to the classical cylindrical one, is to have an asymmetric reflection of the pulse at the projectile end and thus, a difference in the ascending and descending slopes of the generated pulse. As the shape of the projectile is

4.3 Projectile design

Figure 4.4: Pulse shapes as a function of the projectile geometry

changed to having a conical end, the descending slope of the pulse is modified. Altering this rectangular shape while approaching to a triangular pulse will lead to two differences after reflection of the compressive pulse at the free end of the specimen; first, fewer sections would be subjected at the same time to equivalent tensile stresses, reducing the peak-stress influence to a tighter section of the tested specimen and second, the evolution of tensile stresses after reflection of the pulse at the free end will follow a progressive increase instead of a sudden rising (Figure 4.4 (b)).

In the present work several conical geometries have been tested with the help of numerical simulations using LS-DYNA software, with the aim of introducing triangular-shaped pulses in concrete. Parameters such as the total length as well as the lengths of the main cylindrical body and the conical tail of the projectile were evaluated. The main idea of this study was to obtain a pulse with clearly differentiated ascending and descending slopes, maximizing the pulse length such as no superposition of waves occurred during the reflection at the specimen free end. The predicted pulses from the numerical simulations

Figure 4.5: Predicted and recorded compression pulse at the incident bar

were compared to the experimental records at the incident bar (see Figure 4.5) to check the accuracy of the design.

Depending on the test performed and the size of the tested specimen, the final geometry and dimensions of the projectile will vary. In Figure 4.6 the basic scheme of the projectile is given. Two main parts can be identified; a front cylindrical part of varying length from 45 to 90 mm and 22 mm in diameter, and a conic tail ranging from 180 to 360 mm in length and minor diameter of 4 mm. The conic projectile is kept aligned inside the cannon thanks to two PE supporting discs which have a very low friction coefficient. The outer diameter of these discs is coincident with the inner diameter of the cannon.

Table 4.1: Projectile lengths used in this work

Test	Total Length (m)	Cylindrical part (m)	Conical part (m)
Tensile	450	90	360
Energy	225	45	180

4.4 Methodology

Figure 4.6: Projectile geometry

4.4 Methodology

In the present section, a description on the methodologies used to estimate the dynamic tensile strength and the dynamic fracture energy is given. Two type of tests have been performed using the modified Hopkinson Bar and, even though the basic experimental setup described in previous chapters is the same for both of them, the main difference relies in the use of notched specimens in the case of energy tests. The use of a notch is justified due to the need of promoting a single crack in the specimen to accurately measure the specific fracture energy. Having a notched specimen is also useful to avoid or reduce multi-fracture as, thanks to the area reduction at the notch, tensile stresses can be kept below the dynamic tensile strength of concrete in every section outside the notch.

In Figure 4.7 dimensions of the specimens, for the two types of test are given. Figure 4.7 (a) shows the geometry for unnotched specimens employed in the tensile tests. The length of the tested specimens in this work varies from 450 to 975 mm. One strain gauge is always used, positioned at half of the total length of the tested specimen. An extra gauge, positioned at $L/4$ from the impact end, is also used in some tests conducted to compute the wave speed.

In figure 4.7 (b) a sketch of the notched specimen is shown. The position of the notch, at 100 mm from the free end, is given according to the numerical simulations detailed in section 4.6. Apart from the use of notched specimens, some differences or requirements need to be taken into account in the experimental procedure which will be described in the present section. A methodology, based

on the computation and balance of the different energies taking place in the fracture process is illustrated and numerically validated using FEM.

Physical accelerometers will also be used in some of the tests, mainly to validate the velocities measured by the DIC technique. These accelerometers are screwed to a metallic cylindrical piece embedded at the free end of the specimen.

Figure 4.7: Unnotched and notched specimens used for tensile and energy tests respectively

4.4.1 Dynamic Tensile Strength

In previous chapters it has been explained how, initial compressive pulses are transformed into tensile efforts after reaching a free surface of a certain structure. These secondary developed stresses, can be considered as the main source of failure of structures under dynamic events. Apart from the functional requirement of understanding these events, a second interest for the measurement of dynamic tensile strength of concrete comes from the well known fact that a significant increase in the mechanical properties has been observed under dynamic ranges. Different authors have proposed alternative ways for measur-

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ing this dynamic tensile strength and concur more or less in confirming that a notable rise can be appreciated over the quasi-static values, commonly referred as Dynamic Increase Factor

$$DIF = \frac{\textit{Dynamic parameter}}{\textit{Quasi - static parameter}} \quad (4.1)$$

In a first attempt to measure directly the maximum tensile stress reached in a spalling test, direct methods rely on the use of strain gauges placed after the fracture position, as close as possible to it, to record the residual stress passing through the crack. As opposite to its simplicity, two main disadvantages arise when performing this technique; first, the uncertainty in the estimation of the most probable fracture position, which can be more critical depending on the possibility of having simultaneous multi-fracture related to the use of rectangular pulses. Second, as some studies have shown [30], after the fracture process occurs, there is a continuous decrease in the tensile peak as the pulse travels back the broken specimen.

A second group of techniques are those based on approximated expressions to estimate the tensile strength, making use of other relevant parameters measured during the test. A good example of this type of expressions is the well known Novikov's formula already introduced.

Finally, a third type of technique is based on the wave propagation theory and the analysis of the pulse reflection phenomenon to estimate the maximum stress reached according to the fracture position. This approach will be used in the present work and is based on the analytical reconstruction of the reflection process of the compressive pulse at the free end.

In a tensile-strength test, the conic projectile is fired against the incident bar at varying air pressures to generate an initial triangular-shaped pulse. There is no limit in the shooting pressure used more than the need to keep the initial stresses below the compressive strength of concrete in order to avoid damage by compression which could interfere the results estimated for tension. As the shooting pressure is increased, higher stress peaks can be reached in concrete and so higher strain rates, leading to a growing number of spallation-fracture planes. As opposed to energy tests, the number of fracture planes does not interfere with the results obtained for the tensile strength in any of the proposed

methodologies. The compressive pulse generated inside the incident bar travels along it, reaching the interface with the concrete specimen. Thanks to the embedded metallic piece in concrete, the compressive pulse can be transferred into the specimen minimizing the dispersion effects and avoiding the local failure of concrete due to the penetration of the incident bar. This pulse-transfer system enables the use of higher stress pulses giving the possibility of performing spalling tests under increasing strain rates.

In the next phase, the compressive pulse is recorded at the strain gauge located at the concrete specimen. The position of the gauge is determined by the relation between the wave length and the length of the concrete specimen. The main objective is to have a clear record of the initial compressive pulse, without superposition of the incident and reflected waves. For that reason, it was decided to place the gauge at the middle section of the specimen, far enough either from the interface, to avoid possible interference recording in the transmission of the pulse from the incident bar, or from the free end, avoiding superposition of incident and reflected pulses. As introduced some lines ago, this recorded compressive pulse is used to shift, analytically, the pulse reflection at the free end. This reflection process can be understood as the interaction of two pulses (see Figure 4.8); a compressive pulse moving in the positive direction of the specimen while leaving it through the free end, and a tensile pulse, identical in magnitude but opposite in sign, moving towards the negative direction of the specimen. Thus, the stress state of the free end, can be computed as the sum of both pulses at every instant. Taking this into account, during the first stage of the reflection, compressions are going to be predominant as no tensile pulse has yet entered the specimen (Time 1), but as time evolves, an increasing part of the tensile pulse compensates the compressions, leading to a reduction of these initial compressive efforts and the introduction of some tensile stresses in the concrete (Time 2). Due to the asymmetry of the ascending and descending branches of the pulses, a growing tensile peak arises progressively, usually moving to negative direction of the specimen (Time 3), until it reaches the tensile strength of concrete leading to the fracture by spalling (Time 4).

Depending on the level of stress introduced, one or more fractures can occur. Thanks to the triangular pulses, coming from the use of a conical projectile, the

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Figure 4.8: Pulse reflection at the free end

formation of these cracks takes place at different time instants, as opposite to rectangular pulses in which simultaneous fractures can occur. This time spacing of the crack formation is determinant to be able to identify the first or initial crack which determines the accuracy in the estimation of the maximum peak reached.

Once the test is finished, the shifting of the pulses at the free end and the estimation of the tensile strength is carried thanks to a Matlab script in which, at every instant, the stress state can be obtained and plotted. The tensile strength can be derived by just identifying the instant in which the peak of the pulse matches the fracture position.

This methodology is supported and completed by the DIC technique, which has two main objectives; first, the identification of the first crack combined with the use of the high-speed camera. Second, the analysis of the evolution with time of the strains at the fracture position, in order to obtain the strain rate at fracture.

Figure 4.9: Damage introduced in concrete due to tensile pulses

4.4.2 Dynamic Fracture Energy

The main objective for determining the fracture energy of concrete with enough accuracy is to carry out a test in which a unique crack is developed, which means locating the fracture such that no other damage is introduced in the concrete specimen due to the tensile pulse. Regarding the energy release, in concrete subjected to spallation, as the tensile pulse travels through the specimen after reflecting at the free end, energy relief is going to take place in two forms (see Figure 4.9). First, in the form of complete surface cracks leading to separation of pieces, and second, as micro-cracking along the length of the specimen and between surface cracks. If more than one fracture occurs during the process, energy is going to be dissipated not only in the fracture positions but also between these positions due to continuous micro-cracking. Accounting for all this surface micro-cracking is not possible and so estimation of the fracture area will not be accurate enough to compute the specific fracture energy.

Reducing the previous inaccuracies, which could lead to wrong estimations of the fracture energy, is one of the main objectives of this work. It is known that, completely avoiding micro-cracking when subjecting concrete bars to tensile pulses is not possible, but several actions have been adopted in order to minimize this last effect and reach a good approximation of the specific fracture energy.

First, the use of a conic projectile allows introducing triangular pulses in the specimen ensuring that, at a given time, there is only one single maximum

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peak with different ascending and descending slopes, as opposite to rectangular pulses, where the stress is constant along the whole time-pulse with a segment of the specimen subjected to the maximum stress. The existence of a single maximum peak enables crack nucleation and prevents other sections from damaging. Moreover, when the incident wave is reflected, tensile stresses are going to arise progressively while returning back from the free end. This is very important as it allows us to determine, before the test, the most suitable position of the notch, that is, where the tensile stress is going to reach the expected tensile strength, avoiding the fracture out of the notch.

Second, as already mentioned in the previous section, notched specimens are used. The idea behind using a notch is to be able to locate the fracture in just one section so that the fracture area is previously known with complete accuracy. Moreover, by introducing a notch with a reduced section, tensile stresses are locally increased at that point, overpassing in that particular section the dynamic tensile strength of concrete ($f_{t,dyn}$) while in the rest of the specimen sections stresses are kept below the tensile strength, and thus minimizing multi-fracture and the diffused relief of fracture energy.

Several steps are needed to achieve an optimal specimen design in order to accurately measure the dynamic fracture energy (Figure 4.10):

1. The key starting point for these tests is knowing the dynamic tensile strength of the tested concrete which is obtained during the tensile spalling tests. It is important to notice that spalling tests conducted for determining $f_{t,dyn}$ and the ones for $G_{F,dyn}$ are independent but have to be related. With the test set-up presented in this thesis, it is not possible to obtain fracture energy and tensile strength at the same time or under the same configuration but, as opposite, the information derived from the initial tensile spalling tests is required for a successful fracture energy test.
2. Once dynamic tensile strength is known, either a numerical simulation or an analytical process can be performed to estimate the position of the initial in-time fracture. This analysis should be performed using compression pulses in concrete with limited peak values in the order of $f_{t,dyn}$ with the idea of promoting just one fracture. This fracture position will cor-

Figure 4.10: Performed steps to design the notched specimens

respond to the instant in which the increasing peak of the reflected pulse reaches its maximum value.

3. After locating the fracture position, specimens were finally designed and manufactured, placing the notch at 100 mm from the free end.

Energies involved

The method proposed in this paper for the determination of the fracture energy is based on the analysis of the evolution of the energies inside the concrete specimen after the projectile impact in one end. When the incident bar hits the concrete specimen, a compression pulse is transmitted. This pulse travels across the specimen and, thanks to its high compressive strength, concrete is able to withstand this pulse without damage. The total energy introduced in the specimen has two components, kinetic and internal. Both of them are determined by that initial pulse as any compression in the specimen will entail

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first, a movement in the particles leading to a kinetic energy (4.2) and second, a deformation of the infinitesimal sections, which results in an internal energy (4.3).

$$E_k = \frac{1}{2} \int v^2 \rho dV \quad (4.2)$$

$$E_i = \frac{1}{2} \int \sigma \cdot \varepsilon dV \quad (4.3)$$

Being E_k the kinetic energy, E_i the internal energy, v the velocity of the particles, ρ the density, σ the stress, ε the strain and dV an infinitesimal volume.

As the whole process takes place theoretically under elastic regime, it can be demonstrated that, during the pulse-traveling through the specimen, kinetic and internal energies are equivalent and so both can be derived by just using the information of strains contained in the pulse. This last assumption can be justified by analyzing the kinetic energy and knowing that for a elastic wave propagation the following relations can be found:

$$\sigma = \rho c v \quad (4.4)$$

$$\sigma = E \varepsilon \quad (4.5)$$

$$E = \rho c^2 \quad (4.6)$$

$$\rho = \frac{dm}{dV} \quad (4.7)$$

Using these expressions, the kinetic energy can be reconfigured. Entering in expression (4.2) with (4.4)

$$E_k = \frac{1}{2} \int \frac{\sigma^2}{\rho^2 c^2} dm \quad (4.8)$$

Using expression (4.5)

$$E_k = \frac{1}{2} \int \frac{E^2 \varepsilon^2}{\rho^2 c^2} dm \quad (4.9)$$

Using expressions (4.6) and (4.7)

$$E_k = \frac{1}{2} \int c^2 \varepsilon^2 \rho dV \quad (4.10)$$

$$E_k = \frac{1}{2} \int E \varepsilon^2 dV \quad (4.11)$$

$$E_k = \frac{1}{2} \int \sigma \varepsilon dV \quad (4.12)$$

Which equals expression (4.3).

Given a constant section along the specimen

$$dV = A \cdot dL \quad (4.13)$$

and knowing that the length and time traveled by the pulse across the specimen are related by the wave speed

$$c = \frac{dL}{dt} \quad (4.14)$$

expression (4.3) can be rewritten as

$$E_i = \frac{1}{2} E \cdot A \cdot c \int \varepsilon(t)^2 dt \quad (4.15)$$

Numerical integration of the records of one strain pulse can be done leading to

$$E_i = \frac{1}{2} E \cdot A \cdot c \sum \varepsilon(t)^2 \Delta t \quad (4.16)$$

with A being the section of the cylindrical specimen, E the Young modulus, c the wave speed, $\varepsilon(t)$ the recorded strain with time at the gauge and Δt the time step between gauge records .

Thus, the total initial energy introduced into the specimen can be computed from the strain records of the compressive pulse at the specimen

$$E_T = E_k + E_i = 2 \cdot E_i = E \cdot A \cdot c \sum \varepsilon(t)^2 \Delta t \quad (4.17)$$

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Energy balance

In the experimental tests carried out in this work, the projectile was shot with controlled air pressure in order to limit the maximum stress peak (σ_{peak}) reached by the pulse below the dynamic tensile strength. After hitting the specimen, the initial compression pulse (ε_0) travels through it, reflects at the free end and returns back as a tensile pulse. Given that the tensile peak remains controlled below the dynamic tensile strength, no fracture is expected until reaching the notch. Due to the reduced area at that section, tensile stresses are increased and so a unique fracture takes place. The cohesive nature of the fracture process enables part of the returning tensile pulse (ε_r) passing through the cracking area to the next part of the specimen while part of the pulse remains trapped in the ejected free end (Figure 4.11). This means that part of the energy returns to the initial part of the specimen (E_r) and part is retained in the ejected end (E_{fe}). As it will be later demonstrated, the retained energy at the free end is predominately kinetic. The velocity of the ejected end just after the fracture can be measured by using DIC over the images of the high-speed camera and so the kinetic energy of the ejected free end (E_{fe}) can be easily computed by

$$E_{fe} = \frac{1}{2} m_{fe} \cdot v_{fe}^2 \quad (4.18)$$

The initial (E_0) energy and the residual (E_r) energy in the initial part of the specimen can be computed thanks to the records at the strain gauge, respectively ε_0 and ε_r (Figure 4.12). Both E_0 and E_r can be decomposed into kinetic and internal components and so they can be computed applying previous expression (4.17)

$$E_0 = E_{0k} + E_{0i} = 2 \cdot E_{0i} = E \cdot A \cdot c \sum \varepsilon_0(t)^2 \Delta t \quad (4.19)$$

$$E_r = E_{rk} + E_{ri} = 2 \cdot E_{ri} = E \cdot A \cdot c \sum \varepsilon_r(t)^2 \Delta t \quad (4.20)$$

Finally, a small portion of the energy (E_F) is consumed to produce the fracture surface.

Figure 4.11: Energy balance during the fracture process

Figure 4.12: Records of initial and residual pulses at the strain gauge

4.5 Calibration tests

Thus the energy balance for the whole fracture process corresponds to the next expression:

$$E_0 = E_r + E_{fe} + E_F \quad (4.21)$$

From all of them, E_0 , E_r and E_{fe} can be measured as explained, allowing E_F to be derived:

$$E_F = E_0 - E_{fe} - E_r \quad (4.22)$$

Finally, the dynamic specific fracture energy can be obtained dividing the total fracture energy by the fracture surface.

$$G_{F,dyn} = \frac{E_F}{A_F} \quad (4.23)$$

4.5 Calibration tests

Calibration tests over conventional concrete specimens were carried to check the whole system and methodologies proposed. The two types of procedures described were tested and the dynamic tensile strength and fracture energy obtained.

4.5.1 Concrete manufacturing

For these calibration tests, long concrete cylinders were manufactured in the Materials Department of the UPM. A specific mix was designed to obtain a conventional self-compacting concrete with a maximum aggregate size of 12 mm. The tested specimens were cylinders of 69 mm in diameter, with a length of 975 mm for the case of the tensile tests and 450 mm for the energy tests, keeping a length-to-diameter ratio of 14 in the first case and 7 in the second one, which is large enough to ensure that dispersion and 3-D effects can be neglected leading to an unidimensional wave state. Tensile tests were carried over 975 mm in length specimens to achieve a good unidimensional wave propagation state. After the primary tensile tests, for energy tests the length of the tested

Table 4.2: Concrete dosage

Component	Dosage (% in Volume)	Density (kg/m ³)
Cement	11	360
Crushed stone	32	799
Sand	36	940
Filler	4	116
Water	15	147
SP (ADVA)	1	6
Pores	2	0
Total	100	2367

specimens was reduced to 450 mm, enough to obtain a good unidimensional state.

The manufacture of these bars was done thanks to the use of long individual molds made of PVC pipes cut to the desired length. The use of these long molds, with a narrow diameter, complicates performing proper vibration, with the risk of having an heterogeneous mix after curing. For that reason, the use of the self-compacting concrete (SCC) was decided, thus improving the air exiting and the homogeneity of the mix. The employed dosage is given in Table 4.2

The proportions of water and cement were adjusted in order to obtain a compressive strength near 50 MPa. Moreover, to have a self-compacting mixture, filler and fluidifying agents (SP) were required.

The manufacturing process was carried at the Department concrete lab using a conventional mixer. A total amount of 24 specimens were initially manufactured with a total of 4 different batches using the same dosage on all of them. After mixing the ingredients, the fresh mix was then dropped into the PVC pipes placed in vertical position. In the lower end, a rounded PVC top was placed (Figure 4.13) preventing the concrete to flow away from the mold. Is in this top where a metallic cylindrical piece of 20 mm in length and 25 mm in diameter is previously screwed, thus ensuring a proper centered location in the place of contact with the incident bar. This allows the transmission of the compressive pulse from a metallic bar of a smaller diameter into the concrete specimen, reducing local damage by concentration of compressive stresses. The

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Figure 4.13: PVC top with the embedded metallic piece fixed

flat contact face in the concrete is also achieved thanks to this top. In the upper end, another metallic piece is embedded in order to be able to place an accelerometer.

4.5.2 Characterization tests

Prior to dynamic tests, characterization tests under static conditions were performed. To this end, compressive strength, tensile strength and fracture energy tests were carried out. All the tests were carried in an Instron Servo-hydraulic Testing Machine (Figure 4.14).

Figure 4.14: Quasi-static tests

Specimens devoted to these static tests were directly obtained from the previous concrete cylinders, cut to the required length depending on the test

carried. For compression and tensile tests, cylindrical specimens with a length-to-diameter ratio of 2:1 were used. In both cases, the tests were carried under a displacement control program set to 0.648 and 0.2 mm/min respectively. For the tensile strength measurement, splitting (Brazilian) tensile tests were performed. These tests are often used for rock characterization and, due to its simplicity and efficiency, they are also frequently used to characterize concrete [44]. The determination of the tensile strength from these indirect tensile tests can be done with the maximum compressive load reached, using the expression (4.24)

$$\sigma_t = \frac{2P}{\pi\phi L} \quad (4.24)$$

being P the maximum applied load, ϕ the diameter and L the thickness of the sample. The above equation uses the theory of elasticity for isotropic continuous media and gives the tensile stress perpendicular to the loaded diameter at the center of the cylinder at the time of failure.

Regarding the fracture energy tests, the three-point-bending configuration [45] was used. However, instead of prismatic specimens, cylindrical notched specimens were used. As explained, concrete specimens for dynamic tests were casted using cylindrical PVC molds of 1 meter in length and 69 mm in diameter. Due to the special geometry of the molds, effective vibration was not feasible and so a SCC was used to try to minimize possible segregation of the mix. However, some heterogeneities introduced by the concrete molds are always unavoidable. This may lead to differences in the material parameters measured, influenced by the specimens geometry. In this thesis we have attempted to reduce such differences as much as possible and hence, we have used the same molds for all specimens tested, regardless the test and strain rate.

The static configuration and procedure showed in Figure 4.14c followed the RILEM TC 162-TDF recommendations except for the specimen geometry. The length of the specimens was chosen ensuring a distance between supports $S > 3 \cdot D$, being $D=69$ mm the diameter of the specimen and 15 mm the depth of the central notch. The testing machine was configured under a displacement control program set to 5.5E-04 mm/s.

The total fracture energy (E_F) can be obtained from load-displacement curves (Figure 4.15) by numerical integration, calculating the area under them.

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Figure 4.15: Load-displacement curve for a three-point-bending test over cylindrical notched concrete specimen

This area represents the consumed energy during the fracture process and, dividing by the fracture area (A_F), the specific fracture energy is obtained (G_F). In Table 4.3 results for both total and specific energies are given for all the tests.

Table 4.3: Fracture Energy results for quasi-static TPB tests

Test	L(mm)	A(mm ²)	A _F (mm ²)	E _F (J)	G _F (kN/mm)
Test 1	345	3739.28	3139.60	0.638	0.000203
Test 2	375	3739.28	3139.60	0.491	0.000156
Test 3	390	3739.28	3139.60	0.587	0.000187
Mean					0.000182
Stdev					2.4E-05

Regarding the compressive strength and tensile strength tests, a total of 6 and 3 tests respectively were conducted. Results from these tests are shown in Tables 4.4 and 4.5 respectively.

Table 4.6 shows the properties of the manufactured SCC.

Table 4.4: Compression characterization tests

Test	Max. Load (kN)	Mean section (mm ²)	f_c (MPa)
Comp1	63	3731	17.0
Comp2	92	3657	25.1
Comp3	108	3718	29.1
Comp4	99	3696	26.9
Comp5	101	3711	27.3
Comp6	109	3710	29.4
Mean	95		25.8
Stdev	17.0		4.6

Table 4.5: Brazilian characterization tests

Test	Max. Load (kN)	Fracture section (mm ²)	f_c (MPa)
Tens1	42	8821	3.0
Tens2	36	8996	2.5
Tens3	40	8734	2.9
Mean	39		2.8
Stdev	3.1		0.3

Table 4.6: Concrete properties

Characteristics of the self-compacting concrete	
Constitutive equation	Linear Elastic
Maximum aggregate size (mm)	12
Density (kg/m ³)	2.202
Wave speed (m/s)	4081
Dynamic Young Modulus (GPa)	36.93
Poisson's ratio	0.3
Compressive strength (MPa)	25.8
Tensile strength (MPa)	2.8
Fracture energy (kJ/mm)	1.82E-04

4.5 Calibration tests

4.5.3 Dynamic Tensile tests

For these initial calibration tests, a total of 11 spalling tests were done using the configuration explained in section 4.2. Figure 4.16 shows details of the experimental set-up. The specimen is laid over the metallic frame, placing the incident bar in contact with the embedded metallic piece. No glues are used to keep the contact, only a special grease to ensure that both parts do not separate. The concrete specimen is instrumented with one or two strain gauges, depending on the test, to record the strains along the bar. The use of two gauges in some of the tests is also used to estimate the wave velocity in the material.

Along the tests, the conic projectile was fired with variable pressure within the range of 2 to 3 Bar, in order to induce different strain rates in the specimens. The initial idea was to vary the projectile velocity as a function of the introduced pressure however, other unexpected factors had a significant influence in the final velocity reached by the projectile. For example, the existing friction between PE supports and steel cannon led to a deformation of those by the effect of the temperature, difficult to foresee, with the subsequent decrease in the output projectile velocity. This caused that for identical shooting pressures, impact velocities could differ from one test to another.

Apart from influencing the strain rate and the maximum compressive peak introduced, the impact velocity determines the number of cracks developed. As it has been exposed, the existence of numerous cracks, for the case of the tensile strength determination, does not influence the final result but the first crack generated in time must be clearly identified. Table 4.7 shows the relation of tested specimens with the shooting pressure, the projectile velocity and the position of the different fractures detected in the test.

The identification of the crack initiation cannot be successfully achieved by only using the images of the high speed camera and requires a combined action with a Digital Image Correlation software. The resolution when detecting the formation of a crack that the DIC software can achieve is well over the capability of the human eye and, in some cases, an analysis of the images of the camera itself with the naked eye does not guarantee a correct identification of the first

(a) General view of the test setup

(b) Concrete specimen instrumented and in contact with the incident bar

Figure 4.16: Experimental configuration previous to the spalling test

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Table 4.7: Parameters of the dynamic tests

Specimen	Pressure (Bar)	v_{proj} (m/s)	Crack position wrt. Free End(mm)		
			1 st crack	2 nd crack	3 rd crack
CAL01	2.7	8.1	200	365	259
CAL02	3.0	10.2	182	327	218
CAL03	2.7	4.6	382		
CAL04	2.7	12.6	177	310	233
CAL05	2.0	11.0	193	300	
CAL06	2.0	9.0	193	305	
CAL07	2.0	6.7	230		
CAL08	2.0	7.6	185	270	410
CAL09	2.0	7.6	203	355	

crack as, for example, in the case of partially-opened cracks, which can only be distinguished by performing a DIC analysis. To illustrate this, the analysis of test CAL01 is shown in Figure 4.17. The image of the high-speed camera at the end of the test identifies an apparent first (in time) crack due to its opening. However, after running the corresponding DIC analysis, partially-opened cracks are identified at previous time instants.

Figure 4.17: DIC analysis of the images from the High-speed camera

Figure 4.18: Records from the strain gauges positioned at 150 mm and 450 mm from the impact end

To estimate the tensile strength, the wave speed in concrete has to be previously measured. This can be done by using the recorded information of two sensors located at different positions in the cylinder. These records can come from the strain gauge at the incident bar, at the concrete specimen or from the accelerometers. Knowing the distance between the two selected sensors, the time instants in which the signals arrive to the sensors are noted (Figure 4.18). Knowing the distance between the two sensors, the concrete wave speed ($c_{concrete}$) can be derived by

$$c_{concrete} = \frac{\Delta x}{\Delta t_{signals}} \quad (4.25)$$

being Δx the distance between sensors and $\Delta t_{signals}$ the time difference between the records of the selected sensors.

To improve the accuracy, different sensors are used to obtain a mean value. In these first tests, records from the strain gauges in concrete and accelerom-

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Table 4.8: Measured parameters of the specimens

Specimen	CAL01	CAL02	CAL03	CAL04	CAL05	CAL06	CAL07	CAL08	CAL09	Mean	St.Dev.
Wave speed (m/s)	3992	4288	4187	4373	4170	4294	3890	3387	4152	4081	301
Weight (kg)	7.612	8.186	8.145	8.161	8.410	8.141	8.159	7.757	7.659	8.026	
Density (kg/m ³)	2.090	2.248	2.236	2.243	2.283	2.240	2.247	2.128	2.101	2.202	73
E (GPa)	33.31	41.32	39.20	42.90	39.70	41.31	34.01	24.40	36.22	36.93	5.77

eter are used. Once the wave speed is known, the dynamic Young Modulus is obtained by

$$E = \rho c_{concrete}^2 \quad (4.26)$$

being ρ the specimen density. Table 4.8 shows the values for the wave speed, weight, density and Young modulus obtained for the tested specimens. The average value and standard deviation are also given for each magnitude.

The strain rate estimation can be done in different ways. Some authors have proposed the time integration of the analytic reconstruction of the recorded pulses at the moment of failure [30], while more recent approaches are base in the direct measurement of the evolution of stresses at the concrete specimen with the help of several strain gauges positioned close to the fracture position [35]. In this work a straight approach is suggested, using DIC analysis over the strain evolution with time at the fracture position.

Besides this, due to the triangular shape of the compressive pulses used, the strain rate is not constant over time so a criterion has to be adopted for its determination in each particular test. For the proposed methodology, the adopted value corresponds to the maximum strain rate reached at the fracture position. In Figure 4.19 a plot of the evolution during the test of the strain rate with time at the fracture position is shown. A first negative increase can be seen, as the compressive pulse passes through the point of interest, while immediately after, a sharp increase in the strain rate is recorded as the returning tensile pulse deforms the section. This strain rate grows up to the fracture of the specimen, which is represented in the plot by the maximum peak reached after the tensile slope. This maximum peak represents the strain rate at fracture for the considered test.

In order to obtain the dynamic tensile strength, it is necessary to get the

Figure 4.19: Record of the strain rate at the fracture position

stress history at the free end. The analytical process to shift the stress state at the free end has been widely described in previous sections. For that purpose, a Matlab script has been defined. The program is fed with the records of the strain gauge and, as the output, it plots the evolution of stresses with time at the free end (Figure 4.20). The rising of the tensile peak at the consecutive instants can be tracked in the plot as the compressive pulse is transformed into a tensile one. The instant in which this maximum peak matches the first fracture position determines the dynamic tensile strength reached by the section ($f_{t,dyn}$).

From these initial calibration tests, results for nine tests at varying strain rates ranging from 60 s^{-1} to 140 s^{-1} are obtained. The variation in strain rates can be considered a combination of the shooting pressure with the friction between the PE supports of the projectile and the cannon. To check the evolution of the pulses in concrete, four tests were conducted using two strain gauges located at $\frac{1}{4}$ and $\frac{1}{2}$ of the total length of the specimen. By comparing the records of both gauges, some slight differences in the pulse were observed due to dispersion effects and for that reason, for all the tests, the gauge positioned

4.5 Calibration tests

Figure 4.20: Stress evolution at the free end after the reflection of the compressive pulse

at $1/2$, closer to the fracture section, is used to compute the dynamic tensile strength.

Table 4.9 shows the relation of performed tests, the strain rate reached for each of them and the estimated dynamic tensile strength for the gauges used. These results show consistent values among the tests and a clear influence of the strain rate in the tensile strength reached. The Dynamic Increase Factor, which relates the dynamic and quasi-static values for the tensile strength, is also given.

During these calibration tests, a comparison of the results given by the proposed technique with the Novikov's formula is carried. This approach, as already mentioned in Chapter 2, is based on the pull-back velocity at the free end which can be defined as the difference in velocities recorded during the reflection process of the pulse before and after fracture occurs. It can also be described as the difference between the maximum velocity reached during the reflection process and the ejection velocity of the free end after fracture. The record of these velocities can be done with the help of an accelerometer placed

Table 4.9: Results for the dynamic tensile strength

Specimen	$\dot{\epsilon}$ (s ⁻¹)	Gauge 1 (MPa)	DIF G1	Gauge 2 (MPa)	DIF G2
CAL01	90	11.1	4.0	9.0	3.2
CAL02	100	20.4	7.3	13.2	4.7
CAL03	-	13.1	4.7	11.6	4.2
CAL04	140	16.4	5.8	14.9	5.3
CAL05	74	-	-	14.1	5.0
CAL06	80	-	-	10.6	3.8
CAL07	-	-	-	10.8	3.8
CAL08	60	-	-	11.1	4.0
CAL09	60	-	-	12.5	4.5

at the end-face of the specimen. By numerical integration of the previous signal, the velocities are obtained. Another way of measuring velocities is possible by using DIC technique. In this case, displacements fields at the point of interest are derived with respect to time to obtain the same result. The use of physical accelerometers offers a higher time resolution with respect to DIC but, as opposite, these sensors result expensive and very brittle, specially when performing dynamic tests. On the other hand, the economy and versatility of the DIC technique makes it an interesting substitute, as long as the time resolution is enough to obtain a good approach.

In Figure 4.21 an example of the recorded velocity integrated from a physical accelerometer is shown. Both the maximum velocity before fracture (V_1) and the ejection velocity after fracture (V_2), are identified. The pull-back velocity (V_{pb}) can then be computed as

$$V_{pb} = V_1 - V_2 = 4.3 - 1.8 = 2.5ms^{-1} \quad (4.27)$$

and the dynamic tensile strength can be estimated following expression 4.28

$$f_{t,dyn} = \frac{1}{2} \cdot 2202 \cdot 4081 \cdot 2.5 = 11232952Pa = 11.23MPa \quad (4.28)$$

The dynamic tensile strength obtained by the Novikov's formula for all the

4.5 Calibration tests

Figure 4.21: Free-end velocity recorded by the accelerometer

tests uses mean values for the density and wave speed computed and shown in previous Table 4.8, have been used.

Regarding the accuracy given by the accelerometer and DIC, a comparison can be seen in Figure 4.22. Despite the lower resolution of the DIC, both methods allow for a good enough record of the maximum and minimum velocity reached by the free end.

In Table 4.10 results for the dynamic tensile strength obtained by the Novikov's approach are given for 8 tests. In CAL07 the record of the free end velocity failed. For the rest of the tests, parameters required for the Novikov's formula are included.

In Figure 4.23 a comparison of the measured DIFs with both approaches is shown. It can be seen that homogeneous results are obtained, being the main difference the higher increases over the quasi-static values predicted by the proposed methodology as compared to the Novikov's approach.

All these results confirm the reliability of the developed methodology for the dynamic tensile strength measurement.

Figure 4.22: Free-end velocities derived from the physical accelerometer and DIC

Table 4.10: Dynamic tensile strength results with the Novikov's approach

Specimen	$\dot{\epsilon}$ (s ⁻¹)	v_1 (m/s)	v_2 (m/s)	Δu_{pb}	σ_{Nov} (MPa)	DIF_{Nov}
CAL01	90	4.3	1.8	2.5	11.2	4.0
CAL02	100	4.9	2.8	2.1	9.4	3.4
CAL03	-	3.0	0.2	2.8	12.6	4.5
CAL04	140	6.7	3.9	2.8	12.6	4.5
CAL05	74	6.1	3.0	3.1	13.9	5.0
CAL06	80	4.3	2.1	2.2	9.9	3.5
CAL07	-	-	-		-	-
CAL08	60	4.0	1.8	2.2	9.9	3.5
CAL09	60	4.0	1.9	2.1	9.4	3.4

4.5 Calibration tests

Figure 4.23: Compared DIF for the dynamic tensile strength tests

4.5.4 Dynamic Energy tests

A total of 9 fracture energy tests over notched specimens were performed at different strain rates under the scheme shown in Figure 4.24. In this case, 450 mm long specimens were used in contrast to the 975 mm long specimens used for the tensile tests. An optimization in the length was done after the dynamic tensile testing, reducing it to half of the initially proposed and thus economizing the experimental procedure. The projectile was also adapted to the new geometry of the specimen, being also resized to a total length of 225 mm.

Unfortunately, due to the complexity of the test, not all of them were carried successfully. Controlling the pulse intensity was one of the most challenging task for these tests as no exact relation exists between air pressure and projectile velocity. The shooting pressure was kept under 0.5 Bar in order to limit the

Figure 4.24: Experimental set-up for the energy tests

maximum stress peak introduced in concrete below the previously computed dynamic tensile strength (12 MPa). Unsuccessful tests showed either multiple or out-of-the-notch fractures (Figure 4.25a), not allowing for an accurate estimation of the energy. Successful tests are shown in Figure 4.25b. In those, a unique and clear fracture is located at the notch position without any other fractures.

From the performed tests, five were carried out correctly (FE02, FE03, FE05, FE08, FE09) and using the information recorded at the strain gauges, E_0 and E_r were integrated. The analysis of the images from the high-speed camera allows to obtain the velocity of the ejected free end just after the fracture process and, knowing the mass of the ejected piece, the kinetic energy is easily obtained. In Figure 4.26 results for all the energies involved in the process as a function of the strain rate are plotted. An exponential trend can be identified for every energy, increasing as the strain rate grows.

The specific fracture energy is then computed from the previous E_F dividing

4.5 Calibration tests

(a) Unsuccessful tests with fracture out-of-the-notch

(b) successful tests fractured at the notch

Figure 4.25: Fractured specimen after unsuccessful and successful fracture energy tests

Figure 4.26: Energies involved in the fracture process, as a function of the strain rate

Table 4.11: Energy results for the performed tests

Specimen	$\dot{\epsilon}$ (s ⁻¹)	E_0 (J)	E_r (J)	E_k (J)	E_F (J)	G_F (kN/mm)	DIF
FE01	150	1.246	0.160	0.866	0.220	0.78E-04	0.40
FE02	17	1.057	0.240	0.227	0.590	2.09E-04	1.05
FE03	50	1.079	0.159	0.288	0.631	2.23E-04	1.12
FE04	-	-	-	-	-	-	-
FE05	80	1.765	0.384	1.114	0.266	0.94E-04	0.46
FE06	-	-	-	-	-	-	-
FE07	-	-	-	-	-	-	-
FE08	75	1.575	0.492	0.717	0.365	1.29E-04	0.63
FE09	110	3.197	0.720	1.309	1.167	4.13E-04	2.10

it by the fracture surface, which in all the cases was given by the area of the notched section (3739 mm²).

In Table 4.11 the measured energies and the obtained dynamic fracture energy with the DIF are given. The measurement of the strain rate for every test was done using DIC, analysing the strains with time at the notched section. DIC allowed also to check that no fracture occurs in more than one section.

In Figure 4.27 the DIF results for the specific fracture energy (G_F) are shown for the successful tests. A critical strain rate can be identified for a value of 100 s⁻¹, where a sharp change in the dependency of the fracture energy with respect to the strain rate can be appreciated. In this sense, a initial regime, ranging from static tests to strain rates up to 80 s⁻¹, is characterized for keeping the fracture energy not much higher than the static one, while for strain rates over 100 s⁻¹, a significant increase in the estimated fracture energy is seen. Even though there are just some values of the fracture energy, these can be taken as an initial approximation for the behavior of concrete, helping in the identification of this critical strain rate value.

As it can be appreciated in Figure 4.27, values for the specific fracture energy corresponding to strain rates around 80 s⁻¹ suffer a decrease compared to the expected trend given by the rest of the values and, what is more significant, these values show an illogical estimation below the computed quasi-static value

4.5 Calibration tests

Figure 4.27: DIF for the Specific Fracture Energy of concrete

of the fracture energy. In the case of FE08 ($G_F = 1.29E-04$ kN/mm) it is probably due to some dispersion in the results similar to the one identified for the values of the quasi-static fracture energy. The result obtained for FE05 ($G_F = 0.94E-04$ kN/mm) cannot only be justified by the previous dispersion, due to the strong difference with respect to the static values. In the later case, a qualitative analysis on the surface fracture has been carried (Figure 4.28) in order to identify possible reasons for that deviation. In Figure 4.28a the fracture surface for specimen FE05 is showed. The inspection of this surface revealed a predominant intergranular fracture, with decohesion between stone and cement. Analyzing the corresponding fracture surface for specimen FE09 (Figure 4.28b), a different mechanism can be identified. In this last case, the path followed by the crack crosses through the stones, as opposite to the previous decohesion. In the case of specimen FE05, the exposed fracture process could lead to a decrease in the estimated fracture energy which, combined with certain dispersion, would explain the unexpected low energy value measured.

(a) Decohesion of stones in fracture surface of specimen FE05

(b) Fractured stones in fracture surface in specimen FE09

Figure 4.28: Fracture surface analysis

4.6 Numerical validation of the fracture energy testing methodology

Simulations with different material criteria have been performed in this work for several purposes (Figure 4.29). In the case of tensile test modeling, an elastic model without failure was initially used to design the projectile. The interest of this model was obtaining the introduced strain pulse by different projectile geometries and lengths. In this case, both projectile and incident bar had to be modeled in LS-DYNA without the need for modeling any specimen.

After the first part of the experimental work, a model with the material subroutine fed with the dynamic tensile strength measured in the tests, according to the procedure developed in this thesis, was done to compare numerical and experimental results. With this initial calibration, the model could be tested with varying incident compressive pulses. Decreasing pulses with a maximum peak in the order of the previously computed dynamic tensile strength, were then introduced in the model to try to produce one single fracture. The position of this fracture was used as a reference for the future location of the notch. The idea was to ensure that the notch is placed at the section of the specimen in which the probability of reaching the dynamic tensile strength is maximum.

The design of the notched specimen was done thanks to the information

4.6 Numerical validation of the fracture energy testing methodology

Figure 4.29: Evolution of the different numerical models used in this work

retrieved from these previous simulations. The position of the notch was determined by the fracture position given by the simulations. A second model was then prepared in LS-DYNA with a notched specimen. In a first round, the model was fed again with a elastic material without fracture. The aim in this case was to check and study the evolution of the different energies inside the concrete specimen. After the experimental energy tests, the model is finally defined with the material subroutine fed now with the obtained $f_{t,dyn}$ and $G_{F,dyn}$. This last model will be used to validate the experimental methodology to estimate the dynamic fracture energy.

Metallic parts, such as the incident bar and the embedded steel piece in concrete, were defined using an elastic material model with no failure. For the concrete specimen, the behavior is defined as elastic up to failure, determined by the dynamic tensile strength and once reached, the transmitted stress is determined by the softening curve, which controls the relation between crack opening and stress transferred through it. For all the simulations an exponential law [46] for the softening curve has been used (Figure 4.30) and the specific fracture energy set to 2.0E-4 kN/mm. The parameters for all the parts conforming the model are given in Table 4.12.

4.6.1 Model definition

As in the experimental set-up, three parts are defined in LS-DYNA: projectile, incident bar and concrete specimen. The mesh size was adapted for every

Figure 4.30: Exponential softening curve

Table 4.12: Parameters adopted for the numerical model

Part	Material	E (GPa)	ν	Failure	$f_{t,dyn}$ (MPa)	$G_{F,dyn}$ (kN/mm)	Length (mm)	Element size (mm)	#Elements
Metallic components									
Incident bar	Elastic	208	0.3	-	-	-	1000	4	4000
Embedded piece	Elastic	208	0.3	-	-	-	20	2	120
Concrete specimen									
Main body	Elastic	37	0.3	$f_{t,dyn}$	12	2.00E-04	378	2	13488
Notch	Elastic	37	0.3	$f_{t,dyn}$	12	2.00E-04	2	2	56
Free end	Elastic	37	0.3	$f_{t,dyn}$	12	2.00E-04	100	2	3600

part, formed by 8-node constant-stress solid elements ranging from 2 mm in the concrete specimen to 4 mm in the incident bar. To reduce the computational cost, two symmetries were defined, in both horizontal and vertical planes resulting in a model formed by 21000 elements.

The model was fed with the recorded experimental pulse at the strain gauge of the incident bar. This pulse was introduced in LS-DYNA as curve of velocities at the nodes in contact with the projectile. Thus, the model is simplified as no projectile needs to be modeled, improving the computation velocity due to the suppression of small elements used to define the conical tail of the projectile.

The geometry of the modeled concrete specimen is adapted depending on the performed test. In this sense, different lengths for the specimen have been defined, ranging from 450 to 975 mm. Another variation in the geometry can be seen for notched specimens used for energy tests. In the Finite Element Model, the notch is defined by creating three different parts inside the specimen component: one for the main body, a second one for the notch section and a

4.6 Numerical validation of the fracture energy testing methodology

third one for the free end. This way of modeling the whole specimen will be useful for the analysis of the energy balance of each part.

4.6.2 Energy balance

The energy balance described in previous sections is validated using numerical simulations. One of the hypothesis adopted in that balance was that the energy of the ejected free end after spallation corresponds mostly to kinetic energy, as the elastic energy retained in this free end is negligible. This effect could be confirmed analyzing the energies of the different parts of the numerical model. Figure 4.31 shows numerical results for the evolution of the different energies of the whole specimen, while Figure 4.32 and Figure 4.33 show the isolated energies corresponding both to the main body and the free end. It can be seen that, for the main body, the residual energies after the fracture correspond to both kinetic and internal or elastic components while, for the case of the ejected free end, the total energy corresponds entirely to kinetic energy.

Figure 4.31: Evolution of energies in the numerical model for the whole specimen

Figure 4.32: Evolution of energies in the numerical model for the main body of the specimen

Figure 4.33: Evolution of energies in the numerical model for the free end

4.6 Numerical validation of the fracture energy testing methodology

Numerical energy balance

The notched numerical model was fed with a stress pulse with a maximum peak limited below the dynamic tensile strength obtained from the experimental work. In all the simulations using this limited pulse, the fracture was located at the notch, with no fractures developed at other sections. The procedure described for experimentally obtaining the dynamic fracture energy of concrete was tested using the results from the strain histories computed in the numerical simulations as if we had virtual strain gauges.

With the records from a finite element at the middle part of the specimen, initial (ε_0) and residual (ε_r) strains were plotted and the corresponding energies E_0 and E_r integrated using expressions (4.19) and (4.20) respectively. E_{fe} was computed using the velocity of the ejected free end predicted by the numerical simulation. Applying the methodology described and performing the balance given by equation (4.22), E_F was derived. Finally, dividing by the fracture surface of the model, G_F was obtained (see Table 4.13), giving a value of 2.01E-04 kN/mm (Model 1), which coincides with the defined for the model, validating the proposed methodology.

Table 4.13: Energy estimations for different impact pulses

#Model	$v_{p,rel}$	E_0 (J)	E_r (J)	E_{fe} (J)	E_F (J)	A_F (mm ²)	G_F (kN/mm)
1	1.0V	0.474	0.202	0.130	0.142	707	2.01E-04
2	1.5V	1.073	0.259	0.610	0.204	707	2.89E-04
3	2.0V	1.896	0.343	1.190	0.363	707	5.14E-04
4	2.5V	2.965	0.418	2.050	0.497	707	7.03E-04

These simulations were also useful to check the influence of the pulse magnitude introduced in the test on the specimen's damage (Figure 4.34) and consequently, on the measured fracture energy. It is important to note that high-controlled pulses have to be used, always limiting the maximum compression peak below the dynamic tensile strength. Figure 4.34 shows that if non-controlled pulses are used and so the maximum tensile peak introduced in concrete is not kept below the dynamic tensile strength, several sections are going to be damaged without the formation of apparent fractures, impossible

Figure 4.34: Cracked elements in the model depending on the magnitude of the pulse used

to account during a real experimental test and so leading to incorrect measurement of the fracture energy (Table 4.13) due to the wrong estimation of the fracture surface (Models 2, 3 and 4).

CHAPTER 5

Technique validation

5.1 Introduction

The experimental methodology presented is defined by two parts, the test set-up and the techniques used to obtain the searched dynamic parameters from the data acquired during the tests. In the present thesis, the set-up used to conduct the experimental tests has been carefully designed, taking into account all the different pieces that conforms it. The projectile geometry, the sizes of the Bar and specimen, the supporting system and boundary conditions, the use of several sensors and DIC, etc., all of them focused in the extraction of certain records from the test which enables the proposed methodology to offer more accurate results.

Some experimental works regarding the study of the dynamic properties of concrete have been slightly commented on Chapter 2, not existing up to date a common accepted reference methodology. In the case of the dynamic tensile strength, the best approach seems to be the already commented Novikov's formula. However, in the case of the fracture energy, only a few experimental methodologies can be found in the literature, lacking enough amount of experimental data to establish any comparison to validate this newly proposed methodology. Among them, the experimental work carried by Schuler at Fraunhofer EMI [9] was the one that offered a deepest insight and a complete study of the two main dynamic properties of concrete at several strain rates. In the

case of dynamic tensile strength, Novikov's approach has been extensively used by Schuler at EMI with very coherent results. Regarding the dynamic fracture energy, an alternative momentum approach was proposed in the referenced work, testing notched specimens at a wide variety of strain rates and making this approach a good reference to compare the accuracy of the results obtained through the proposed methodology in this thesis.

In the present section, the experimental validation of the proposed methodology for measuring the dynamic properties of concrete is carried out. A complete experimental campaign involving tensile and fracture energy tests is conducted at the Hopkinson Bar located at Fraunhofer EMI's facilities with the aim of comparing the results obtained by both the proposed methodology and the EMI's procedure. Moreover, the versatility of the proposed technique is also checked as an alternative Hopkinson Bar configuration is used for these tests.

5.2 Experimental program

5.2.1 Specimen manufacturing

Specimens for spalling tests at EMI, for both tensile and fracture energy tests, consist of concrete cylinders of 300 mm in length and 75 mm in diameter. A total of 22 samples were directly obtained by drilling into a uniform concrete block of dimensions 1000 x 300 x 250 mm (Figure 5.1). Compared to other techniques in which every specimen is manufactured individually using a cylindrical mould, the extraction from a large block allows obtaining a very homogeneous mixture for every specimen, avoiding the wall-effect. Moreover, the use of the drilling technique avoids the problem of vibration of high length-to-diameter ratio moulds.

The mix was designed by a private company under the requirement of having a concrete with a maximum compressive strength of 50 MPa. Due to property reasons, the dosage was not released by the company.

5.2 Experimental program

Figure 5.1: Specimen drilling from concrete block

5.2.2 Quasi-static characterization

Previous to dynamic tests, compression, Brazilian and TPB tests are performed in order to have a complete quasi-static mechanical characterization of the concrete mixture.

Compressive strength was obtained from standard compression tests carried over normalized cubical specimens of 150 mm side length. Specimens for Brazilian tests were obtained from the drilled cylinders after cutting them to the required length of 150 mm, thus keeping a length-to-diameter ratio of 2:1 as defined by the testing norm. Both compression and Brazilian tests were carried at Fraunhofer Institute under displacement-control at an actuator velocity of 0.05 mm/s in the case of compression (Figure 5.2a), and to 0.01 mm/s for the Brazilian tests (Figure 5.2b).

Regarding the quasi-static fracture energy, the corresponding TPB tests were performed at the Materials Department of the UPM using an Instron machine with a load cell of 5 kN and a displacement-control program. Instead of prismatic specimens, the same cylindrical notched specimens used for dynamic

Figure 5.2: Quasi-static characterization tests at EMI

Table 5.1: Concrete properties

Concrete properties	M1	M2	Mean	Stdev
Density (kg/m^3)	2288	2324	2306	25
σ_c (MPa)	65	66.3	65.7	0.9
f_t (MPa)	5.2	4.3	4.8	0.6
G_F (kN/mm)				
$c_{concrete}$ (m/s)	3800	3802	3801	1
E_{dyn} (GPa)	33.3	33.3	33.3	0.0

tests and drilled from the original concrete block were characterized under this configuration. The total fracture energy (E_F) can be obtained from the force-displacement curves by numerical integration, computing the area under them. This area represents the consumed energy during the fracture process and, dividing by the fracture area (A_F), the specific fracture energy is calculated (G_F).

For each test typology, two specimens were tested. Results, average values and standard deviations for the quasi-static tests are shown in Table 5.1.

5.2.3 Dynamic Test setup

Dynamic spalling tests were carried out at the Fraunhofer Institute for High-Speed Dynamics, Ernst-Mach-Institute (EMI), employing a modified configuration of the Hopkinson Bar (Figure 5.3). This configuration shows some impor-

5.2 Experimental program

Figure 5.3: SHB scheme at Fraunhofer Institut

tant differences with respect to the one employed at the UPM facilities, both in geometry and materials. The experimental device is composed of a steel projectile of 60 mm length inside the air cannon and an aluminium incident bar of 5500 mm length and 75 mm in diameter. The incident bar is instrumented with four strain gauges at 250, 1750, 4000 and 4750 mm from the impact end. The use of several strain gauges allows checking the evolution of stresses along the incident bar and the influence of the 3D effects on the final introduced pulse inside concrete. At the end of this bar, the concrete specimen is glued directly to the aluminum. The concrete specimen is also instrumented with strain gauges at 30 and 70 mm from the contact end. The testing device is completed with a Photron high-speed camera, model Fastcam SA5. The images from the high-speed camera are post-processed using Ncorr, a Digital Image Correlation (DIC) software.

The setup at EMI facilities at Efringen-Kirchen can be seen in Figure 5.4

5.2.4 Projectile shape

As already pointed out, the shape of the projectile plays a paramount role in the spalling tests. Therefore, in order to increase the experimental cases used for validation of the results, two geometries for the projectile are considered.

Before manufacturing the projectile, several shapes were checked using FEM with the LS-Dyna code. Both cylindrical and conical projectiles were simulated and the duration, shape and maximum peak reached by the pulses, adjusted.

(a) View of the air chamber and aluminum incident bar

(b) View of the concrete specimen in contact with the incident bar

Figure 5.4: Modified Hopkinson at EMI lab at Efringen-Kirchen

In Figure 5.5, experimental and numerical pulses developed in the incident bar are shown, with both cylindrical and conical projectiles.

As it can be seen, the pulse from the conical projectile has a more regular and stable shape, both in the ascending and the descending slopes. In the case of the cylindrical projectile, the descending slope suffers some distortion, which could be caused by a projectile length-to-diameter ratio equal to 1. The reduction in the diameter of the reflecting face in the conical projectile could be the reason for this improvement in the shape of the developed pulse. Even though the length of both projectiles is the same, the conical one leads to a smaller time-pulse. This means that after reflection, the portion of the sample affected by superimposition of the waves is lower, which offers some benefits depending on the technique used to estimate the dynamic tensile strength. After checking the suitability of the conical design, the final projectile was manufactured at the EMI Workshop under the geometry shown in Figure 5.6.

5.2 Experimental program

Figure 5.5: Experimental and numerical comparison of the pulse shapes

Figure 5.6: Final design of the conic projectile. Distances in mm

5.3 EMI analysis techniques

In this section, a brief description of the used techniques for both measuring the tensile strength and fracture energy with EMI's technique is given. Only two configurations for the concrete specimen are required, one for the estimation of the tensile strength and one for the fracture energy (Figure 5.7). They basically differ in the introduction of a circumferential notch in the middle section of the specimens used in the fracture energy tests. As in the case of the methodology presented in this thesis, the use of a notch is justified due to the need of promoting a single fracture plane in the specimen to accurately measure the specific fracture energy. Having a notched specimen is also useful to avoid or reduce multi-fracture as, thanks to the area reduction at the notch, tensile stresses can be kept below the dynamic tensile strength of concrete in every section outside the notch

In the tensile strength configuration (Figures 5.7 (a) and 5.8 (a)), the specimen is directly glued to the incident bar, using an epoxy resin, with no other supports under it. The length of the possible tested specimen is then limited due to the self weight of concrete, and for that reason, a maximum length of 300 mm could be tested.

In the fracture energy configuration, the specimen is not glued to the bar, but linked to it using a mortar mix, enough to transmit the compressive pulse from the incident bar to the specimen. The EMI methodology requires that all the specimen parts are free to be ejected after the fracture occurs, so the momentum of each ejected piece of the concrete specimen can be accurately measured which results in a better approximation of the fracture energy. Extra supports are then needed under the specimen, as shown in Figures 5.7 (b) and 5.8 (b), to withstand the self weight of concrete.

Two strain gauges are used in both configurations however, the need for two different records for the strains has a special relevance in the case of the energy tests where initial and residual strains inside the concrete specimen have to be recorded at different points in order to avoid superposition of the pulses.

5.3 EMI analysis techniques

Figure 5.7: Specimen setup for a) Tensile strength test and b) Fracture energy test

(a) Unnotched specimen glued to the incident bar

(b) Notched specimen in contact with the incident bar and supports

Figure 5.8: Specimens configuration and instrumentation

5.3.1 Tensile strength

With the configuration detailed in the previous section, the dynamic tensile strength is determined at EMI using the Novikov's formula

$$f_{t,dyn} = \frac{1}{2} \rho c \Delta u_{pb} \quad (5.1)$$

This expression uses the density (ρ) and wave speed (c) of concrete and the so-called pullback velocity (Δu_{pb}) of the ejected end of the specimen to estimate the tensile strength. The pullback velocity, given by the integral of the accelerometer record, can be defined as the difference in velocities of the free end before and after the spallation occurs. In this case the wave speed (c) has been estimated using the records at the first strain gauge in concrete (SGC1) and the accelerometer (Figure 5.9).

Figure 5.9: Records from the strain gauge and the accelerometer placed at 30 mm from the incident bar and free end respectively

5.3 EMI analysis techniques

5.3.2 Dynamic Fracture Energy

The Momentum approach for computing the dynamic fracture energy of concrete is based on the momentum transfer proposed by Schuler [9]. The requirements regarding specimens geometry and test configuration have been given in the previous sections. As exposed, one of the critical factors of this technique is that in notched specimens, after fracture occurs, the formed pieces are ejected freely so that the velocities, and thus the momentum of both, can be accurately measured.

In this test, after the reflection of the pulse at the specimen's free end, the formed tensile wave produces the fracture of the specimen at the notched section. If the test is performed successfully, two pieces are ejected from the incident bar due to the tensile wave that goes back through the whole specimen. The velocity of the fragments can then be measured by using different sensors, such as digital extensometers, accelerometers or DIC. As the mass of the separated fragments can be measured after the test, the moment of inertia can be easily computed.

The estimation of the fracture energy is derived by Schuler starting from its definition as the integral of the force over the crack opening. The fracture energy can then be rewritten as

$$G_f = \int F d\delta = \int \frac{dI}{dt} \dot{\delta} dt \quad (5.2)$$

The previous integral can be expressed in terms of momentum transfer between fragments (ΔI) and crack opening velocity ($\dot{\delta}$). The variation in the momentum of inertia can be understood as the consumed energy during the fracture process and can be estimated by the difference in the solid rigid velocity of the ejected free end, before ($v_2(t_1)$) and after fracture occurs ($v_2(t_2)$) (Figure 5.10).

$$\Delta I = I_{12} = [v_2(t_1) - v_2(t_2)]m_2 \quad (5.3)$$

The crack opening velocity ($\dot{\delta}$) is estimated as the difference in the mean

Figure 5.10: Considered time instants during the estimation of the transferred momentum between fragments

values of the fragment velocities at both sides of the notch during the whole fracture process

$$\dot{\delta} = \dot{\delta}_{12} = \frac{[v_2(t_2) - v_2(t_1)]}{2} - \frac{[v_1(t_2) - v_1(t_1)]}{2} \quad (5.4)$$

Assuming constant crack opening velocity, the fracture energy can be estimated by:

$$G_f = \Delta I \cdot \dot{\delta} \quad (5.5)$$

Fragments velocities before and after spallation are measured and the energy dissipated between those instants can be computed from the reduction of these velocities. For this procedure, two instants are considered: t_1 , the time where crack initiation starts (Figure 5.10 (a)), and t_2 is the moment in which the fracture is completely opened (Figure 5.10 (b)). The accuracy in the estimation of the momentum transfer relies in the fact that the fragments are ejected freely after the fracture process.

5.4 Tensile strength analysis

A total amount of 6 tensile tests have been carried out with varying strain rates ranging from 60 s^{-1} to 130 s^{-1} (Figure 5.11). The variation in the strain rates is achieved by adjusting the air pressure in the cannon. Three different

5.4 Tensile strength analysis

Figure 5.11: Strain rates reached for different projectile velocities

pressures were used: 0.5 bar, 1.0 bar, 1.5 bar. For every pressure value, two specimens were tested, one of them using the cylindrical projectile and the other with the conical-shaped one.

After testing, qualitative and quantitative analysis of the fractured specimens were done. From the qualitative point of view, an analysis of the fracture pattern was conducted, comparing the influence of the projectile shape as well as the strain rate in the number and spacing of fractures. Moreover, particularly the influence of the projectile shape over the evolution of strains inside the specimen was analyzed.

From the quantitative point of view, after developing the experimental tests, results for dynamic tensile strength are obtained applying both the proposed methodology in this thesis and the described EMI's approach, with the idea of comparing the influence in the results.

5.4.1 Fracture pattern

In this section, the number and spacing of the fractures for every test are analyzed. Figure 5.12 shows pictures of the state of the specimens after the tests. For the three pressures used (1.5 bar, 1.0 bar and 0.5 bar), differences in the fracture patterns can be observed depending on whether a cylindrical (Figure 5.12a) or a conical projectile (Figure 5.12b) is used for the test. In the case of the cylindrical projectile, more damage can be appreciated in the specimens compared to the corresponding conical-projectile tests. In the first case, a higher density of fractures can be seen, even though the strain rate reached

Figure 5.12: Fractured specimens after spalling tensile tests

by both projectile geometries for the same pressure is similar. As testing pressure is decreased, the number of fracture positions, as well as diffused damage (partially opened cracks), remains more controlled in the case of the conical projectile. This result reveals that the conical projectile is more convenient if a limited number of cracks in the specimen is targeted. This fact becomes even more important when testing notched specimens, proposed in this thesis for estimating the fracture energy, in which the control of the damage has a critical role in determining precisely the specific fracture energy, due to the need of promoting just one located fracture, and thus avoiding the diffused damage out of the notch.

5.4.2 Evolution of strains

Together with the reduction of the fracture planes, shown in the previous section, the progressive evolution of the tensile peak after reflection is an additional benefit from the use of conical projectiles. For high-pressure tests leading to multi-fracture in the specimen, conical projectiles lead to a progressive growth of the tensile peak after reflection, allowing a better determination of the first-in-time crack as the maximum stress is reached in one particular section at a moment, well differentiated from the rest (Figure 5.13b). In the case of cylin-

5.4 Tensile strength analysis

drical projectiles (Figure 5.13a), strains arise with a less-defined tensile peak, leading to a wide portion of the specimen subjected to equivalent stress at a certain moment, causing simultaneous fractures and making the identification of the initial crack a complicated task. This aspect is especially critical for certain methodologies like the one presented in this thesis, as it depends on a clear identification of the first position of the fracture which must be linked to the stress profile in order to obtain the dynamic tensile strength.

Figure 5.13: Strains evolution with time inside the specimen

5.4.3 Tensile strength results

Table 5.2 shows the resulting values of the tensile strength obtained with the EMI's approximation together with the velocities used to obtain these values. Shooting pressure (P), specimen length (L), number of apparent cracks and strain rate ($\dot{\epsilon}$) are also given. Finally, the dynamic increase factor ($DIF_{f_t,EMI}$) with respect to the quasi-static tensile strength value is computed. The results predicted by this approach show reasonable and stable values for all the strain rates analyzed. An increasing trend of the DIF can be identified reaching a value close to 7 at a strain rate of 125 s^{-1} .

Table 5.3 shows the same values obtained using the analytical methodology presented in this thesis. In this case, the position of the first-in-time crack with respect to the free end is given. With this second approach, DIFs also show

Table 5.2: Results and measured dynamic tensile strength by EMI's methodology

#Test	Projectile	P (bar)	L (mm)	$\dot{\epsilon}$ (s ⁻¹)	v_{max} (m/s)	v_{min} (m/s)	u_{pb} (m/s)	σ (MPa)	DIF _{<i>f_t,EMI</i>}
1	Cylindrical	1.5	277	130	18.46	11.30	7.14	31.3	6.5
2	Conical	1.5	300	125	14.43	7.00	7.43	32.6	6.8
3	Cylindrical	1.0	297	88	18.40	12.50	5.86	25.7	5.3
4	Conical	1.0	300	83	12.97	7.20	5.77	25.3	5.3
5	Cylindrical	0.5	300	60	10.15	6.24	3.91	17.1	3.6
6	Conical	0.5	300	50	7.56	5.18	2.38	10.4	2.2

Table 5.3: Results and measured dynamic tensile strength obtained by the proposed analytical methodology

#Test	Projectile	P (bar)	L (mm)	$\dot{\epsilon}$ (s ⁻¹)	1 st crack (mm _{<i>free end</i>})	σ (MPa)	DIF _{<i>f_t,analytical</i>}
1	Cylindrical	1.5	277	130	105-125	38.0-24.0	8.0-5.0 (6.5)
2	Conical	1.5	300	125	100	37.3	7.8
3	Cylindrical	1.0	297	88	110-130	26.0-16.0	5.4-3.3 (4.4)
4	Conical	1.0	300	83	120	24.0	5.0
5	Cylindrical	0.5	300	60	118	14.4	3.0
6	Conical	0.5	300	50	100	14.3	3.0

good agreement as well as the growing trend with increasing strain rates shown for the EMI's procedure. On the other hand, the relevance of the projectile shape can be seen in tests 1 and 3. In these particular cases, the identification of the first-in-time fracture was not possible, as several cracks were formed simultaneously. Multiple apparent fracture planes arise simultaneously as a result of the planar strain evolution seen in previous Figure 5.13, and as a consequence, the only affirmation that can be done regarding the initial fracture position is that it is located between to observed spatial crack positions along the specimen. The tensile strength was obtained in these two cases as a range limited by the position of these cracks and the DIF as the mean value within the range.

The expected raise in the tensile strength with increasing strain rates can be seen in Figure 5.14, where the DIFs obtained with both approaches are shown. Small differences can be seen in the estimation of the stresses reached for higher values of the strain rates, where the proposed analytic methodology predicts increases in the order of 8 times over the corresponding quasi-static parameter.

5.5 Fracture energy

Figure 5.14: DIFs of the tensile strength predicted by both approaches at increasing strain rates

5.5 Fracture energy

A total of 6 notched specimens were tested under controlled shooting pressure below 0.25 Bar. As the impact velocity was then kept constant, the range of strain rates reached for these energy test was narrower, with values going in the majority of cases from 80-100 s⁻¹. Even though a lower pressure is used for these tests, the range of strain rates is similar to those reached during the tensile tests, where higher shooting pressures were used. This is due to the area reduction at the notched section.

To analyze the possible influence of the projectile shape, three tests were performed using the conical projectile and three using the cylindrical one.

Results of the dynamic fracture energy have been again obtained by two different methodologies, the energy balance approach proposed in this thesis and the EMI's momentum approach proposed by Schuler. Both approaches require measuring different parameters. In the case of the Momentum approach, what is measured are the velocities of the fragments generated during spallation

Table 5.4: Energy results for the momentum approach

#Test	$\dot{\epsilon}$ (s ⁻¹)	m_2 (kg)	$v_1(t_1)$ (m/s)	$v_2(t_1)$ (m/s)	$v_1(t_2)$ (m/s)	$v_2(t_2)$ (m/s)	ΔI (kg m/s)	$\dot{\delta}$ (m/s)	E_F (J)	G_F (kN/mm)
Con1	125	1.47	2.60	4.77	0.12	3.50	1.87	2.78	5.18	0.001832
Con2	85	1.46	2.14	3.65	0.91	2.32	1.94	1.46	2.84	0.001003
Con3	80	1.50	2.15	4.10	0.88	2.86	1.86	1.97	3.65	0.001293
Cyl1	86	1.47	2.36	4.10	1.51	2.80	1.91	1.52	2.90	0.001024
Cyl2	98	1.47	2.56	4.50	1.40	3.40	1.62	1.97	3.19	0.001127
Cyl3	99	1.48	2.65	4.70	1.26	3.55	1.70	2.17	3.69	0.001306
Mean value										0.001150
Standard deviation										0.000144

through the notch. As the specimen is designed to be fractured at that point, only two fragments are generated. For the measurement of the velocities, two time-instants are considered, t_1 corresponding to the moment right before the fracture occurs and t_2 corresponding to the moment after fracture. In the work by Schuler, velocities corresponding to different instants were measured by different ways. In the case of t_1 , an analytical procedure was used to derivate the velocities of both fragments, while for t_2 velocities were measured using a high-speed extensometer. In the present work, both instants have been measured using a DIC analysis over the images of the high-speed camera. The idea is to obtain a more direct approach to the instant previous to the fracture. Energy results as well as the computed values for all the required velocities are shown in Table 5.4.

The results with this first approach show consistency for all the tests performed except for test 1, which was done with an alternative configuration to check the influence of the boundary conditions. In this test, instead of leaving the whole projectile to be ejected freely, the part of the specimen in contact with the incident bar was glued to it. After the fracture of the specimen, the free end was ejected but this part remained glued to the bar. An increase in the measured fracture energy for this test can be noted, related either to a inaccurate measurement of the momentum or the increased strain rate for this configuration, but a deeper analysis should be done to clarify this point. From the rest of the tests (2-6), performed with the configuration detailed in section 5.3, a mean value of 0,001150 kN/mm was obtained. No significant differences in the G_F results can be appreciated between cylindrical and conical projec-

5.5 Fracture energy

Table 5.5: Results for the energy balance approach

#Test	$\dot{\epsilon}$ (s ⁻¹)	E_0 (J)	E_r (J)	m (kg)	v_{rel} (m/s)	E_k (J)	E_F (J)	G_F (kN/mm)
Con1	125	11.64	1.26	1.47	3.38	2.48	7.90	0.002793
Con2	85	4.06	0.50	1.46	1.41	1.03	2.53	0.000895
Con3	80	6.61	0.33	1.50	1.98	1.49	4.80	0.001696
Cyl1	86	5.17	0.46	1.47	1.29	0.95	3.76	0.001331
Cyl2	98	6.10	0.67	1.47	2.00	1.47	3.96	0.001401
Cyl3	99	5.40	0.40	1.48	2.29	1.69	3.31	0.001169
Mean value								0.001298
Standard deviation								0.000295

tiles, just a small difference in the higher strain rate reached by the cylindrical projectile.

In the case of the energy balance approach proposed in this thesis, the required parameters are different. As explained, the initial energy (E_0) introduced into the concrete specimen as well as the residual energy (E_r) after the pulse are both recorded at the strain gauges glued to the first part of the specimen. Due to the length of the introduced pulse, some superposition of the waves can occur when recording both initial and residual pulses. To minimize this effect, the two strain gauges positioned at different distances from the contact and the free end are helpful. Strain gauge 1 (SGC1) is located close enough to the contact end to avoid superposition from the reflected pulse at the free end. On the other hand, the strain gauge 2 (SGC2) is positioned far enough from the contact end, to have a clear record of the residual pulse without superposition due to secondary reflections at the contact end. The velocity of the ejected free end is computed with the help of DIC analysis. The computed values for this second approach are shown in Table 5.5 showing a mean value of 0.001298 kN/mm.

All the results obtained for the fracture energy are compared in Figure 5.15 and even though the strain rate is kept within a narrow range, a common increasing trend can be seen for both approaches, following a same dynamic response as for the tensile strength. In the case of the thesis methodology, higher values for the fracture energy are obtained for all the strain rates tested,

Figure 5.15: Fracture Energy results predicted by both approaches at increasing strain rates

with a significant increase at 125 s^{-1} as compared to the value predicted by the EMI's approach.

5.6 Summary

In this chapter, results obtained for the proposed methodology in this thesis have been compared to the ones obtained with the methodology used at EMI. Moreover, the versatility of the proposed methodology has been tested under EMI's alternative Hopkinson Bar configuration. The dynamic tensile strength and fracture energy have been studied to have a complete validation of the technique proposed. As an additional task, the influence of the projectile shape in the results has also been considered in this chapter.

Regarding the tensile strength, very similar results have been obtained by the two methodologies. From a quantitative point of view, the main difference between them seems to be in the higher DIF predicted by the analytical proposed methodology, with tensile strength increments up to 8 times over the

5.6 Summary

quasi-static value at 125 s^{-1} , as compared to the 7 times predicted by the Novikov's formula. In this sense, the DIFs obtained by both methodologies seem to follow different trends and so a wider study should be performed in the range of strain rates over 125 s^{-1} to check their evolution at higher strain rates. From a qualitative point of view, focusing in the influence of the projectile shape over the results, the critical difference is found in the identification of the first fracture position, which is paramount for an accurate measurement of the dynamic tensile strength in the case of the proposed methodology in this thesis. As shown across the present chapter, the use of conical-shaped projectiles helps with the identification of the first crack position. With this type of projectile, fractures in the concrete specimen arise progressively as opposite to cylindrical projectiles in which multiple apparent fractures can be developed at the same time.

In the case of the fracture energy, the results obtained by the proposed methodology show higher values with respect to the ones predicted by the momentum approach. Even though both compared methodologies derive the fracture energy through an indirect procedure, the main difference between them is centered in the considered energies involved in the fracture process. While the momentum approach is based on the kinetic evolution of the whole system, in the case of the balance of energies approach, a deeper insight into the involved energies is done, measuring both the kinetic and internal energies. The importance of this fact has been already justified in Section 4.6, where the numerical analysis of the methodology showed the importance of evaluating all the energies involved in the process. This difference in the way that the involved energies are considered in the derivation of the fracture energy could be the reason of the lower values predicted by the momentum approach, as an important elastic component is not taken into account. The energy balance technique also offers a clear tracking of the all the energies involved, allowing for a better interpretation of the process and results obtained.

CHAPTER 6

Dynamic fracture analysis of two concrete mixes

6.1 Introduction

In this chapter, the developed experimental technique is used to perform spalling tests over two types of concrete mixtures. Dynamic tensile tests are performed over conventional concrete (CC) and high-performance concrete (HPC) cylindrical specimens under the experimental configuration developed in the present thesis.

The introduction of a high-performance concrete in the present study is motivated by the development of newer and more sophisticated concrete mixtures, designed to resist higher compressive and tensile stresses as well as other chemical and corrosive actions. HPC emerge to overcome actual limits in the design of infrastructures, reducing the loads introduced due the own weight by employing concrete mixtures with and increased ratio strength/weight. These types of concretes are indicated for very particular buildings like skyscrapers, special bridges and infrastructures subjected to extreme actions like offshore platforms or residual treatment plants. The HPC is a type of concrete with an enhanced compressive resistance with respect to what is called a conventional concrete. A precise classification is not clearly done in the codes but generally, concretes are considered to be high-performance when their compressive strength exceeds

50 MPa, while a conventional concrete is usually designed to have between 25 to 35 MPa. However, as time and technology develops, this limit is permanently changing and nowadays conventional concrete used in the majority of infrastructures reach the previously specified limit with certain facility. For this reason, actual codes suggest to consider a HPC when the compressive strength reached is above 80 MPa.

As quasi-static mechanical parameters are notably increased, a certain influence in the dynamic properties is also expected, giving these concrete mixtures an additional interest for their use in certain civil and military facilities sensitive to possible dynamic solicitations.

In this chapter, results for the dynamic tensile strength and DIF for both types of concrete mixtures are given. A qualitative analysis of the fracture pattern is also carried out to bring some light over the fracture process determined by each type of concrete mix under dynamic events.

6.2 Concrete manufacturing and characterization

6.2.1 Mix design and manufacturing

For this work, two concrete dosages were developed. The initial idea was to have two different mixtures regarding their mechanical properties (Compressive and tensile strength) so that a sufficient comparison can be done. First, a conventional self-compacting concrete was designed limiting its compressive strength below 50 MPa. Two main actions were adopted to have a proper mix, which consisted in controlling the water/cement relation and using filler to give the self-compacting properties needed to have an homogeneous mix inside the moulds. Concretes not overpassing this value of resistance are considered as conventional concretes and are the most frequently used in several infrastructures.

To establish a comparison and check the influence on the mechanical properties, a second HPC dosage was designed and manufactured at the Department of Materials science from the UPM. There are no specific methodologies for

6.2 Concrete manufacturing and characterization

the design of HPC and the conventional dosage methods do not offer satisfactory results because of the need of use of discontinuous curves, employing high quantities of cement. In other words, the water/cement ratio is extremely low in these types of concrete mixtures. This makes the process of HPC design a very hard task, based on a trial and error process, adjusting the proportions of the different components and checking, via compressive tests, the suitability of the final recipe. For this reason, in the manufacture of both concrete mixtures, the help and advice of a concrete specialist at the Materials Department of the UPM was required. Based on previous works by Adel Fatty [47], both the CC and the HPC were designed and manufactured with his help.

The main difference of the HPC with respect to the CC was the use of a lower relation water/cement and the inclusion of silica fume inside the mix. This last addition is frequently used when higher compressive strengths are required, due to the chemical reactions which enhance the concrete resistance. A fluidizing agent is also required to increase the poor workability of the fresh mix, caused by the low ratio water/cement. The size and origin of the aggregate is also important to reach higher values of resistance and it is usual to employ siliceous material with a maximum size below 20 mm. In this work siliceous aggregates with a maximum size of 12 mm have been used.

Dynamic tested specimens, both CC and HPC, consist of concrete cylinders of 450 mm in length and 69 mm in diameter. PVC tubes with this inner diameter were used to manufacture the specimens. Due the high length-to-diameter ratio, difficulties during concrete vibration may appear, but the use of the small fractions of filler in the case of CC and silica fume for the HPC, helped to have a more fluid mix avoiding possible segregations. The dosage of both CC and HPC concretes is given in Table 6.1. The maximum aggregate size used was, in both cases, 12 mm.

Several batches were needed to obtain a whole set of specimens for both the CC and the HPC. In the case of CC, a total of 4 batches were done, while for the HPC, only 2 batches were manufactured. In every mix process, specimens for the different types of quasi-static and dynamic tests were obtained.

Table 6.1: Concrete dosage

Component	Dosage (% in volume)		Density (kg/m ³)	
	CC	HPC	CC	HPC
Cement	12	14	390	450
Crushed stone + Sand	68	65	1429	1375
Filler	3	-	59	-
Silica Fume	-	3	-	59
Water	14	14	144	143
SP (ADVA)	1	1	6	8
Pores	2	2	0	0
Total	100	100	2026	2045

6.2.2 Quasi-static characterisation

Compression and Brazilian tests were performed over CC and HPC specimens to characterize both concrete mixtures. Compressive strength was obtained from standard compression tests carried out over normalized cylindrical specimens of 150 mm length and 75 mm in diameter, using a displacement-control program set to 0.648 mm/min. Results of this parameter given in Table 6.2 show that there is a significant difference between CC and HPC, with a value of 55 MPa in the first case and 81 in the case second one.

Specimens for indirect tensile tests (Brazilian) were obtained from cylinders, cut to the required length. Again, as in the case of compression, the testing machine was configured under a displacement control program set to 0.5 mm/min. From the tensile characterization it can be seen that the HPC has a higher resistance to tension as compared to CC. Usually a tensile strength of around 10% of the compressive value is expected for concrete. In this case, HPC and CC show 7.5% and 5.6% respectively.

6.3 Dynamic tensile tests

Table 6.2: Quasi-static properties of the different mixtures (average values (standard deviations in parentheses))

	HPC	CC
Bulk density ρ (kg/m ³)	2290(62)	2293(25)
Maximum aggregate size (mm)	12	12
Dynamic Young Modulus (GPa)	47 (1)	37(2)
Compressive Strength (MPa)	81 (11)	55 (8)
Tensile Strength (MPa)	6.1 (0.8)	3.1 (0.3)
Tensile/Compressive ratio	0.075	0.056

6.3 Dynamic tensile tests

In this last tensile campaign, a total of 18 spalling tests were performed using the set-up and methodology developed and presented across this work. 12 tests corresponded to the CC while 6 were performed over HPC specimens. The configuration used in both of them is the same, as detailed in Chapter 4 for the tensile tests. The set-up at the laboratory is shown in Figure 6.1 for both types of concrete mixtures.

From all the performed test, 9 CC and 4 in the case of HPC, were successful. Non successful tests were caused by lack of records in the strain gauge and the camera due to failure of the trigger, even though a normal fracture occurred (3 CC and 2 HPC).

The aim of this campaign is to compare the mechanical behavior of the two different concrete mixtures in a certain dynamic range. In all the tests, the same shooting pressure was used in order to limit the strain rate within a certain range, however, as explained previously, other factors apart from the established pressure affected the final strain measured in the tests.

The analysis process started with the determination of the wave speed recorded in every specimen during the spalling test. This task was done with the help of DIC analysis, registering the arrival time of the pulse at two different positions in the specimen (Figure 6.2). The distance between the considered positions divided by the difference in the arrival time throws the estimated

(a) CC specimen placed in the experimental device

(b) HPC specimen placed in the experimental device

Figure 6.1: Experimental configuration previous to the spalling test

wave speed for every test. The dynamic Young modulus is then computed with expression

$$E = \rho c^2 \quad (6.1)$$

To reduce uncertainties caused by dispersion in the estimated parameters, measurements for every specimen are made over the density, wave speed and dynamic Young Modulus. Moreover, for every batch, a mean value is computed for each parameter. Table 6.3 shows the previous parameters for every specimen of each of the 4 CC batches. Very homogeneous values can be seen among specimens of the same batch, and also among average values from the different

6.3 Dynamic tensile tests

Figure 6.2: DIC signals at point x_1 and x_2

Table 6.3: Material properties for CC mixes

CC mix	CC-1			CC-2			CC-3			CC-4		
Specimen	ρ	c	E	ρ	c	E	ρ	c	E	ρ	c	E
1	2282	4200	40	2294	4175	40	2290	3713	32	-	-	-
2	2272	3950	35	2250	3900	34	2283	3865	34	2311	4100	39
3	-	-	-	2271	3923	35	2308	4160	40	-	-	-
Mean	2281	4075	38	2272	3999	36	2294	3913	35	2326	4100	39
Stdev.	9	177	3	22	153	3	13	227	4	21	-	-

batches resulting in an estimated dynamic Young Modulus between 35 and 39 MPa for the CC.

In Table 6.4 the same previous parameters for the corresponding 2 batches of HPC are obtained. In this case and compared to the CC, the measured values for the wave speed and the dynamic Young modulus suffer some more dispersion among specimens belonging to the same HPC batch, as shown by the computed standard deviation.

After the analysis of the previous parameters, no great difference can be

Table 6.4: Material properties for HPC mixes

HPC mix	HPC-1			HPC-2		
Specimen	ρ	c	E	ρ	c	E
1	2250	4548	47	2336	-	-
2	2236	4200	39	2347	4804	54
3	2255	4845	53	2319	4256	42
Mean	2247	4531	46	2334	4530	48
Stdev.	10	323	7	14	387	9

observed in the density between CC and HPC. However, the measured wave speed shows a notorious increase in the case of the HPC with respect to CC, which results in a higher Young modulus for the HPC as expected and exposed in other studies. This type of concrete mixes are characterized for presenting higher and more linear slopes in the stress-strain (σ - ϵ) diagrams up to fracture as compared to conventional concretes (Figure 6.3), which makes the HPC to have a more brittle behavior.

Figure 6.3: Stress-Strain Curves for Conventional and High Performance Concretes [48]

6.3 Dynamic tensile tests

The shooting pressure was set to 0.5 bar for all the tests, which resulted in the fracture of concrete specimens with the formation of 2-3 cracks. No difference can be stated between CC and HCP in the number of cracks however, the spacing, as well as the distance between the first crack and the last one, seems to have some dependence on the type of concrete tested. A deeper analysis on these facts will be done in the next section of the present chapter.

The strain rate reached at each test was estimated using DIC analysis, selecting the fracture point and placing a virtual extensometer at both sides of the formed crack. Strain rate results for all the successful tests of the different batches of CC are shown in Figure 6.4, where a range between 70 and 200 s^{-1} is achieved.

Figure 6.4: Computed strain rates for the CC spalling tests

The same strain-rate plots are obtained for the HPC mixes, shown in Figure 6.5. In this case, a narrower range is obtained for the tests, with values varying between 60 and 100 s^{-1} . Apart from the reduced variation of the range, the

smaller values of strain rate reached by the HPC in comparison to CC tests, are consistent with the previously obtained Young moduli for both mixes and the commented more brittle behavior of the HPC.

Figure 6.5: Computed strain rates for the HPC spalling tests

6.4 Results

6.4.1 Tensile strength

The first part of the testing process was devoted to the CC specimens, in which, from the 12 programmed tests, 9 were successfully broken and recorded. The estimation of the tensile strength was done, as explained in previous chapters, using a Matlab subroutine, plotting the stress state at every time instant. The first fracture position was identified from the DIC analysis and matched to the evolution of stresses to estimate the maximum tensile strength reached by the concrete sample (Figure 6.6).

Results for the 9 successful CC spalling tests are shown in Table 6.5. The specimen length, strain rate, the number of fractures, the strain at fracture, the tensile strength and the Dynamic Increase Factor are given. This DIF represents the relation between the dynamic tensile strength for each test with respect to the quasi-static measured value, which for the case of CC specimens was 3.1 MPa.

6.4 Results

Figure 6.6: Stress evolution in test CC-M1-2 and estimated tensile strength

Table 6.5: Results and estimated dynamic tensile strength for the CC specimens

Specimen	Length (mm)	# Fractures	$\dot{\epsilon}$ (s ⁻¹)	$\epsilon_{fracture}$	f_{ctd} (MPa)	DIF
CC-M1-1	471	1	110	2.45E-04	9.3	3.0
CC-M1-2	485	2	150	3.61E-04	13.7	4.4
CC-M2-1	471	3	125	2.72E-04	9.8	3.2
CC-M2-2	470	3	156	4.31E-04	15.4	5.0
CC-M2-3	474	2	70	2.64E-04	9.5	3.1
CC-M3-1	470	2	124	3.23E-04	11.3	3.6
CC-M3-2	470	2	157	5.37E-04	18.8	6.1
CC-M3-3	482	2	83	2.03E-04	7.1	2.3
CC-M4-2	411	1	200	5.52E-04	21.5	6.9

All these previous results are summarized within Figure 6.7, where the estimated tensile strength reached for the conventional specimens are plotted with respect to the strain rate, ranging from 70 to 200 s⁻¹. The four tested CC batches are referenced in the plot and very homogeneous results can be observed among them.

Figure 6.7: Dynamic tensile strength results for CC specimens

The analysis of the complete results for CC mixes show a common trend, with a clear increase in the tensile strength with increasing strain rates, fitting the classical exponential curve already predicted by previous authors. Two regimes can be distinguished in the plot. First, the initial branch, ranging from quasi-static values of the strain rate up to $100\text{--}120\text{ s}^{-1}$, defines a linear regime, in which the increase in the tensile strength follows a soft slope. Over the stated critical strain-rate value of 120 s^{-1} , an exponential regime can be defined, with the rise of the tensile strength following a sharper slope.

In Figure 6.8 the DIF's for every test are given. It can be seen that, for CC specimens, the minimum increase reached starts from a factor of two over the static value of 3.1 MPa , reaching a maximum value of 7 for the strain rate of 200 s^{-1} .

In a second stage, spalling tests corresponding to HPC specimens were carried out under the same previous procedure. From the 6 performed tests, 4 were recorded correctly. In this case, the analytical process performed using the Matlab subroutine (Figure 6.9) led, in almost all the cases, to higher values

6.4 Results

Figure 6.8: Dynamic Increase Factor for CC specimens

of the tensile strength reached. Results for the 4 successful tests are shown in Table 6.6.

The same plots for maximum tensile strength and DIF are given in Figures 6.10 and 6.11 respectively, where the estimated results are located in a strain rate range between 60 and 100 s^{-1} . Compared to CC results, two main differences can be stated in these second stage; first the smaller values of strain rate reached and second, HPC does not appear to follow an exponential law for the dynamic tensile strength as a function of the strain rate, instead, a single branch can be identified, which can be adjusted to a linear expression. Moreover, in spite of the higher values of tensile strength reached by HPC as compared to CC, a smaller increase of 6 over the quasi-static tensile value is found for the maximum strain rate of 100 s^{-1} .

To understand the dynamic response of both concrete mixes against impact loads, the analysis and comparison of the results for different parameters of the test is carried out. Regarding the dynamic tensile strength (Figure 6.12), results show higher stress ranges for the HPC, as expected, reaching values up to 36 MPa, which almost doubles the dynamic strength reached by CC. This

Figure 6.9: Stress evolution in test HPC-M1-1 and estimated tensile strength

Table 6.6: Results and estimated dynamic tensile strength for the HPC specimens

Specimen	Length (mm)	# Fractures	$\dot{\epsilon}$ (s ⁻¹)	$\epsilon_{fracture}$	f_{ctd}	DIF
HPC-M1-1	481	3	77	4.78E-04	22.0	3.6
HPC-M1-2	485	2	60	3.54E-04	16.3	2.7
HPC-M1-3	476	2	100	7.83E-04	36.0	6.0
HPC-M2-2	477	3	60	5.41E-04	26.0	4.3

ratio conserves the ratio found for the quasi-static values between both concrete mixes, however, the strain rate at which the maximum stress is reached differs between mixes, being the maximum strain rate reached at CC tests (200 s⁻¹), twice the corresponding at HPC (100 s⁻¹).

Continuing with the analysis, even though direct results of the tensile strength show the predominance of the HPC concrete to withstand high dynamic impacts, a more interesting conclusion can be obtained by analyzing and comparing the DIF of the two mixes. Figure 6.13 shows how, conventional concretes

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Figure 6.10: Dynamic tensile strength results for HPC specimens

are able to develop a higher increase in their tensile properties over improved HPC mixes, expressed by the higher DIF of the first one, mainly thanks to their ability of adapting the strain rate to higher values due to the lower Young Modulus coefficient. This increase could also be related with the strain response of both concrete mixes which will be deeply studied in the next section.

6.4.2 Fracture analysis

Apart from the usual quantitative analysis regarding mechanical concrete parameters, like the tensile strength, the analysis of the response of different types of concrete mixes against impact or dynamic loadings requires a further qualitative study on the fracture process. Understanding the way concrete reacts against sudden loads, and how properties are changed with respect to quasi-static situations, needs a deeper insight on the formation of the cracks.

For that reason, in this last section, a final study on the fracture pattern of the specimens is carried out. Number and spacing of the different cracks

Figure 6.11: Dynamic Increase Factor for HPC specimens

formed during the spalling process are compared for CC and HCP specimens, combined with analysis of the strain response of CC and HPC at fracture.

Figure 6.14 shows the images of the CC specimens after the spalling tensile tests. The number of developed cracks varies from 1 to 3, being the formation of 2 cracks the most usual fracture pattern in this type of concrete. After DIC analysis, it was confirmed that the temporal order of crack formation coincided with the proximity of the crack to the free end, being in all the cases the first crack formed the closest to the free end.

A similar trend in the rising of the cracks is found for the HPC specimens (Figure 6.15). However, the number of formed cracks ranges, in this case, from 2 to 3, depending on the strain rate reached.

Fracture patterns for the 9 CC specimens and 4 HPC specimens have been represented in Figures 6.16 and 6.17 respectively. The number of cracks and distance with respect to the free end are represented in the previous figure.

Analyzing the fracture patterns for both concrete mixes, the first important difference is found in the number and spacing of formed cracks. For the CC, usually fewer cracks are developed, as compared to HPC, where more cracks can

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Figure 6.12: Comparison of the dynamic tensile strength for CC and HPC as a function of the strain rate

Figure 6.13: Comparison of the Dynamic Increase Factor for CC and HPC as a function of strain rate

Figure 6.14: Fractured CC specimen after the spalling tensile test

Figure 6.15: Fractured HPC specimen after the spalling tensile test

Figure 6.16: Fracture patterns for the CC spalling tests. Impact face on the left and free end on the right. Distances given to the free end

6.4 Results

Figure 6.17: Fracture patterns for the HPC spalling tests. Same configuration as Figure 6.16

be found, being the spacing between them consequently reduced in the second case. This also determines the beginning and extension of the damage zone, which can be defined as the part of the specimen contained between the first and last fracture positions. This damage zone has been estimated in this work by extracting the average value of the first and last positions of the formed cracks (Table 6.7). The bigger spacing between fracture sections leads, in the case of the CC mixes, to longer damage zones with an average length value of 70 mm as compared to the corresponding 45 mm of the HPC. The initiation of this zone also differs from one type of concrete to the other, rising the first fracture at 127 mm from the free end in the CC, while the same first fracture position is found at 114 mm for the HPC.

All the previous qualitative results, combined with the tensile strength estimation show that, even though it has reached higher values of strength, the increased stiffness in the HPC makes these type of mixes to have very poor behavior against impact or dynamic loads. As the spacing between opened fractures has demonstrated, CC flexibility allows this type of concrete to redistribute the strain along a longer part of the specimen, without concentrating it in few critical sections. As opposite, HPC does not share this ability, instead, strain is very localized in some critical sections, which results in the formation of a higher number of opened fractures. The analysis of the strain at fracture for every specimen, which is represented in Figure 6.18, also confirms the idea of the high strain concentration at critical or fracture sections in HPC. It can

Table 6.7: Fracture positions in CC after spalling tests

Specimen	Free end distance (mm)		$\varepsilon_{fracture}$	$\dot{\varepsilon}$ (s ⁻¹)
	1 st crack	Last crack		
CC-M1-1	150		2.45E-04	110
CC-M1-2	125	185	3.61E-04	150
CC-M2-1	140	220	2.72E-04	125
CC-M2-2	125	220	4.31E-04	156
CC-M2-3	150	200	2.64E-04	70
CC-M3-1	123	210	3.23E-04	124
CC-M3-2	115	175	5.37E-04	157
CC-M3-3	107	165	2.03E-04	83
CC-M4-2	140		5.52E-04	200
Mean	127	196		

Table 6.8: Fracture positions in CC and HPC after spalling tests

Specimen	Free end distance (mm)		$\varepsilon_{fracture}$	$\dot{\varepsilon}$ (s ⁻¹)
	1 st crack	Last crack		
HPC-M1-1	110	160	4.78E-04	77
HPC-M1-2	125	165	3.54E-04	60
HPC-M1-3	120	140	7.83E-04	100
HPC-M2-2	100	170	5.41E-04	60
Mean	114	159		

be seen that, for the same strain rate, HPC suffers an increased strain at the fracture section, for similar pulses as the ones used for CC.

To conclude, both CC and HPC have demonstrated an interesting dynamic response under impulsive loads. Specially CC mixes have shown a very good behavior against impact loading, with a notorious increase in their mechanical properties with the strain rate as shown by the DIF analysis. More over, the redistribution of strain shown by these CC mixes, helps concrete to reduce the number of fractured sections at the expense of increasing the damage zone of the element. This, which can be asimilated to a more elastic behaviour, may

6.4 Results

Figure 6.18: Comparison of the maximum strain at fracture for CC and HPC as a function of strain rate

be responsible for the significant relative increase in the dynamic mechanical properties exhibited by CC as compared to HPC.

CHAPTER 7

Conclusions and Future Work

7.1 Conclusions

The design and development of a new experimental technique to measure the dynamic tensile strength and fracture energy of concrete has been carried in the present work. Based on the Hopkinson Bar system, a modified configuration was adopted in order to achieve tensile pulses in the tested specimens. By removing the transmission bar, initial compressive pulses in the concrete were successfully transformed into tension through the spalling phenomenon.

In a first stage, the design of the different parts of the experimental device was done, having a special care in the dimensions and geometry of all the components. A special attention was dedicated to the projectile, supporting system and specimen. In the case of the projectile, a modification in the shape was introduced in order to achieve triangular pulses inside the concrete. The benefits of these type of pulses have been fully justified across the document, and the conical geometry adopted for the projectile has shown good results to obtain suitable triangular pulses.

The supporting system was designed to test very long specimens, thus reaching an unidimensional wave propagation in concrete and reducing the 3-D dispersion effects. Ideal boundary conditions were also achieved through this steel frame, as no glues are required to fix the specimen to the incident bar. Moreover,

tested specimens are totally free to move inside the horizontal plane thanks to the steel bearings which support the specimen.

Finally, long concrete specimens were designed with an embedded metallic piece to achieve a perfect transmission of the pulse coming from the incident bar. This piece reduced the localized stress avoiding the local fracture by penetration.

Initial spalling tests were carried over conventional concrete specimens. A methodology based on the analytical reconstruction of the pulses at the free end was develop to estimate the dynamic tensile strength of the tested concrete. The geometry and length of the projectile and specimen allowed for a more precise location of the first initial fracture position, needed to have a reliable estimation of the tensile strength reached by the specimen.

Regarding the fracture energy and due to the impossibility in dynamic tests of directly measuring this parameter, an indirect approach was proposed and analyzed in this work. A balance of the energies taking place in the test was used to estimate the specific fracture energy of concrete. A special care was needed during the execution of the tests to have a valid estimation of the energy. Notched specimens were exposed as the only possibility of estimating a valid result, in order to promote a single fracture in concrete and thus having a well defined fracture surface. Combined with this, the limitation of the introduced compression pulses inside concrete was also a key factor to reach a successful energy test, preventing the damage out of the notch. This fact was fully demonstrated both through the non successful tests and also checked through numerical simulations, which showed the strong dependency of the estimated energy to the introduced damage in the specimen.

The previous numerical model was used with a material developed subroutine, based on the cohesive fracture theory, to validate both the tensile and energy procedures. In a tensile configuration, numerical results showed a good agreement on the initial fracture location. These first simulations were used to have a proper design of the notched specimens. Finally, the energy balance approach was also successfully validated by applying the same experimental methodology to the simulated procedure.

A experimental validation campaign was carried at Fraunhofer Institut with an alternative experimental scheme. The use of a different device allowed to test

7.1 Conclusions

the versatility of the proposed methodology to be applied to different configurations. The analytical proposed methodology for the estimation of the tensile strength was validated with the Novikov's approach, showing very homogeneous results. In the case of the fracture energy, the balance of energies approach and the momentum approach used at Fraunhofer showed some differences. Even though similar results for the fracture energy were achieved by both techniques, the higher values obtained by the balance of energies could be related to the inclusion of the internal and kinetic parts of the energy, as opposite to the momentum approach in which only kinetic parameters are taken into account.

This last validation campaign was also used to analyze the suitability and influence of the projectile geometry. Conic and cylindrical shapes were compared both for the estimation of the tensile strength and the fracture energy. Despite the similarity in the results, the conic projectile showed its better performance during the estimation of the tensile strength, justified by the clear formation of one single initial fracture as compared to the use of cylindrical projectiles, where the developed pulses led to the formation of simultaneous fractures at the same time.

A last tensile campaign was carried over two different concrete mixes in order to use the developed methodology to evaluate the influence of the concrete dosage in the dynamic properties. A High Performance Concrete (HPC) and a Conventional Concrete (CC) were designed and manufactured at the Department of Materials Science of the UPM. After the quasi-static characterization, dynamic tensile spalling tests were carried over the two concrete mixes. Both quantitative and qualitative analysis were performed to evaluate the response to impact loadings. As expected, in the quantitative level, higher values for the dynamic tensile strength were reached by the HPC over the CC. However, the referenced results to the quasi-static values revealed the capacity of CC mixes for showing significant increases in the dynamic properties, with a DIF of 7, higher than the one reached by the HPC. This capacity can be directly related to the lower Young Modulus of CC allowing for a more elastic behavior, increasing the capacity of strain distribution along the specimen without the formation of clear open fractures. This hypothesis is supported by the qualitative results, where the study of the number and spacing of fractures was performed. This

analysis showed the rising of more opened fractures in the case of HPC, with a smaller spacing between them. As opposite to CC, the stiffness of the HPC mix seems to have an important role in the poor distribution of strains across the length of the specimen, with a higher concentrated strain in certain critical locations. This effect is also consistent with the higher strain rates reached by the CC.

7.2 Future Work

In this work, an extensive experimental campaign has been carried, showing very coherent results when compared to other approaches. It is clear, continuing in the line hold by previous authors, that an increase in the mechanical properties can be measured and is dependent on the subjected strain rate. However, a physical explanation on the reason that underlays this behavior has not been already given. During dynamic events, complex mechanisms take place under pulse loadings, which combined with the duration of the performed tests make the exact measurement of the searched dynamic properties a very complicated task. For this reason, a deeper understanding of the fracture process under high strain rates is required, which can throw some light to the apparent increase shown by the experimental testing. Maybe, this study needs a combined work of the powerful Hopkinson device with other analysis performed in a smaller scale. This takes special value in the case of the fracture energy, where an exact definition of the softening curve can lead to a better understanding of the real fracture process.

Together with the experimental development, a parallel effort should be made in the improvement of the numerical models in order to accurately predict the behavior of structures undergoing impulsive loads. In this work, an initial material model has been defined and used to validate the experimental procedure. An extensive experimental campaign regarding tensile strength and fracture energy is needed to feed the numerical tools. This, combined with the previous mentioned understanding of the fracture process, are also critical to have an exact definition of the energy relief process, which can make the numerical simulation an exact tool for dynamic prediction.

7.2 Future Work

Finally, from the experience of the presented methodology, a more detailed analysis on the design of the notched specimens should be made in order to increase the success rate of the energy tests. A proper position of the notch with an adequate depth is determinant to reach a single fracture, minimizing the diffused damage along the specimen. Regarding this, it might be of great interest for the interpretation of the experimental results, a dedicated study on the influence of the depth of the notch in the obtained results.

References

- [1] A. J. Francis. *Cement Industry 1796-1914: A History*. David & Charles, Newton Abbot Eng. ; North Pomfret, Vt, May 1978.
- [2] L.-J. (Louis-Joseph) Vicat and b 1787 Thierry. *Recherches expérimentales sur les chaux de construction, les bétons et les mortiers ordinaires*. Paris, Goujon, 1818.
- [3] Jonas A Zukas. *High velocity impact dynamics*. Wiley, New York, 1990. OCLC: 21445603.
- [4] Vicente Sánchez Gálvez. *Materiales para la defensa*. Fundación Rogelio Segovia para el Desarrollo de las Telecomunicaciones, Madrid, edición: 1 edition, October 2012.
- [5] Donald O. Dusenberry. *Handbook for Blast Resistant Design of Buildings*. John Wiley & Sons, 2010. Google-Books-ID: 19Gr55RPcRQC.
- [6] Reinhardt, H.W. and Zielinski, A.J. *Fracture of concrete and mortar under uniaxial impact tensile loading*. PhD thesis, Delft University Press, November 1982.
- [7] C. A. Ross. *Split-Hopkinson Pressure Bar Tests*. PN, 1989.
- [8] J. Weerheijm. Concrete under impact tensile loading and lateral compression. 1992.
- [9] Harald Schuler, Christoph Mayrhofer, and Klaus Thoma. Spall experiments for the measurement of the tensile strength and fracture energy of

- concrete at high strain rates. *International Journal of Impact Engineering*, 32(10):1635–1650, October 2006.
- [10] Ahmed Brara and Janusz R. Klepaczko. Fracture energy of concrete at high loading rates in tension. *International Journal of Impact Engineering*, 34(3):424–435, March 2007.
 - [11] Benjamin Erzar, Eric Buzaud, Pascal Forquin, and C Pontiroli. Tensile strength of dried mortar over a wide range of strain rate. <http://dx.doi.org/10.1051/dymat/2009086>, 1, September 2009.
 - [12] P. Forquin and B. Erzar. Dynamic fragmentation process in concrete under impact and spalling tests. *International Journal of Fracture*, 163(1-2):193–215, May 2010.
 - [13] W. Zheng, A. K. H. Kwan, and P. K. K. Lee. Direct Tension Test of Concrete. *Materials Journal*, 98(1):63–71, January 2001.
 - [14] J D Todd. The determination of tensile stress/strain curves for concrete. *Proceedings of the Institution of Civil Engineers*, 4(2):201–211, March 1955.
 - [15] Helmut Kupfer, Hubert K. Hilsdorf, and Hubert Rusch. Behavior of Concrete Under Biaxial Stresses. *Journal Proceedings*, 66(8):656–666, August 1969.
 - [16] B P. Hughes and G P. Chapman. The deformation of concrete and micro-concrete in compression and tension with particular reference to aggregate size. *Magazine of Concrete Research*, 18:19–24, March 1966.
 - [17] R. H. Evans and M. S. Marathe. Microcracking and stress-strain curves for concrete in tension. *Matériaux et Construction*, 1(1):61–64, January 1968.
 - [18] Bertram Hopkinson. X. A method of measuring the pressure produced in the detonation of high explosives or by the impact of bullets. *Phil. Trans. R. Soc. Lond. A*, 213(497-508):437–456, January 1914.
 - [19] G I Taylor. James forrest lecture 1946. the testing of materials at high rates of loading. *Journal of the Institution of Civil Engineers*, 26(8):486–519, October 1946.

REFERENCES

- [20] E. Volterra. Alcuni risultati di prove dinamiche sui materiali. *Rivista Nuovo Cimento*, 4:1–28, 1948.
- [21] H. Kolsky. An Investigation of the Mechanical Properties of Materials at very High Rates of Loading. *Proceedings of the Physical Society. Section B*, 62(11):676, 1949.
- [22] D. L. Birkimer and R. Lindemann. DYNAMIC TENSILE STRENGTH OF CONCRETE MATERIAL. *Am Concrete Inst Journal & Proceedings*, January 1971.
- [23] A. J. Zielinski and H. W. Reinhardt. Stress-strain behaviour of concrete and mortar at high rates of tensile loading. *Cement and Concrete Research*, 12(3):309–319, May 1982.
- [24] Shishan Zheng. Beton bei variierender Dehngeschwindigkeit untersucht mit einer neuen modifizierten Split-Hopkinson-Bar-Technik. 1996.
- [25] C. Albertini and M. Montagnani. Study of the true tensile stress-strain diagram of plain concrete with real size aggregate ; need for and design of a large Hopkinson bar bundle. *Le Journal de Physique IV*, 04(C8):C8–113–C8–118, September 1994.
- [26] Joseph W. Tedesco and C. Allen Ross. Experimental and Numerical Analysis of High Strain Rate Splitting-Tensile Tests. *Materials Journal*, 90(2):162–169, March 1993.
- [27] Mark K. McVay. Spall Damage of Concrete Structures. page 431, June 1988.
- [28] J. R. Klepaczko and A. Brara. An experimental method for dynamic tensile testing of concrete by spalling. *International Journal of Impact Engineering*, 25(4):387–409, April 2001.
- [29] A. Brara and J. R. Klepaczko. Experimental characterization of concrete in dynamic tension. *Mechanics of Materials*, 38(3):253–267, March 2006.

- [30] F. Gálvez Díaz-Rubio, J. Rodríguez Pérez, and V. Sánchez Gálvez. The spalling of long bars as a reliable method of measuring the dynamic tensile strength of ceramics. *International Journal of Impact Engineering*, 27(2):161–177, February 2002.
- [31] D J Frew, Sandia National Laboratories, and M J Forrestal. Pulse Shaping Techniques for Testing Brittle Materials with a Split Hopkinson Pressure Bar. page 3.
- [32] Haijun Wu, Qingming Zhang, Fenglei Huang, and Qiankun Jin. Experimental and numerical investigation on the dynamic tensile strength of concrete. *International Journal of Impact Engineering*, 32(1):605–617, December 2005.
- [33] S.A. Novikov, I.I. Divnov, and A.G. Ivanov. Study of rupture of steel, aluminum and copper under explosive loading. *Fiz. Met. Metalloved.*, 21:23–32, January 1966.
- [34] J. Weerheijm and J. C. A. M. Van Doormaal. Tensile failure of concrete at high loading rates: New test data on strength and fracture energy from instrumented spalling tests. *International Journal of Impact Engineering*, 34(3):609–626, March 2007.
- [35] B. Erzar and P. Forquin. An Experimental Method to Determine the Tensile Strength of Concrete at High Rates of Strain. *Experimental Mechanics*, 50(7):941–955, September 2010.
- [36] B. Erzar and P. Forquin. Experiments and mesoscopic modelling of dynamic testing of concrete. *Mechanics of Materials*, 43(9):505–527, September 2011.
- [37] Herbert Kolsky. *Stress Waves in Solids*. Courier Corporation, 1963. Google-Books-ID: JRxUjBxcpyYC.
- [38] John Sargent Rinehart. *Stress transients in solids*. HyperDynamics, Santa Fe, N.M, 1975.

REFERENCES

- [39] José M. Sancho, Jaime Planas, Adel M. Fathy, Jaime C. Gálvez, and David A. Cendón. Three-dimensional simulation of concrete fracture using embedded crack elements without enforcing crack path continuity. *International Journal for Numerical and Analytical Methods in Geomechanics*, 31(2):173–187, February 2007.
- [40] J. M. Sancho, J. Planas, D. A. Cendón, E. Reyes, and J. C. Gálvez. An embedded crack model for finite element analysis of concrete fracture. *Engineering Fracture Mechanics*, 74(1):75–86, January 2007.
- [41] Gustavo Morales Alonso. *Experimental and numerical analysis of reinforced concrete elements subjected to blast loading*. phd, E.T.S.I. Caminos, Canales y Puertos (UPM), 2013.
- [42] J Blaber, B Adair, and Antonia Antoniou. Ncorr: Open-Source 2d Digital Image Correlation Matlab Software. *Experimental Mechanics*, 55, March 2015.
- [43] Francisco Galvez Diaz-Rubio. *Caracterización mecánica de materiales cerámicos avanzados a altas velocidades de deformación*. phd, E.T.S.I. Aeronáuticos (UPM), December 1999.
- [44] D18 Committee. Test Method for Splitting Tensile Strength of Intact Rock Core Specimens. Technical report, ASTM International.
- [45] Lucie Vandewalle, D Nemegeer, György Balázs, B Barr, Joaquim Barros, P Bartos, Nemkumar Banthia, M Criswell, E Denarie, Marco Prisco, H Falkner, Ravindra Gettu, Vellore Gopalaratnam, P Groth, V Hausler, A Kooiman, Konstantin Kovler, B Massicotte, S Mindess, and J Walraven. RILEM TC 162-TDF: 'Test and design methods for steel fibre reinforced concrete' - sigma-epsilon-design method - Final recommendation. *Materials and Structures*, 36:560–567, October 2003.
- [46] Zdenek P. Bazant, Z. P. Bazant, and Jaime Planas. *Fracture and Size Effect in Concrete and Other Quasibrittle Materials*. CRC Press, Boca Raton, edición: 1 edition, January 1998.

REFERENCES

- [47] A. M. Fathy, B. Sanz, J. M. Sancho, and J. Planas. Determination of the bilinear stress-crack opening curve for normal- and high-strength concrete. *Fatigue & Fracture of Engineering Materials & Structures*, 31(7):539–548, July 2008.
- [48] ETSICCP. Universidad de La Coruña. Hormigones de alta resistencia, 2011.

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