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A SHORT REPORT ON SOME ITALIAN PAPERS PRESENTED IN OCCASION OF THE 5th INTERNATIONAL SYMPOSIUM ON "UTILIZATION OF HIGH STRENGTH/HIGH PERFORMANCE CONCRETE" - SANDEFJORD (20th-24th JUNE 1999)

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SUMMARY

This paper reports a brief description of some selected proceedings of the 5th INTERNATIONAL SYMPOSIUM ON UTILIZATION OF HIGH STRENGTH/HIGH PERFORMANCE CONCRETE 1999, Sandefjord (Norway), 20th-24th June 1999. About 220 people participated to this symposium from 38 countries, mostly from Sweden, Norway, Japan, U.S.A. and Italy. This event has represented a good occasion for assessing the updated situation about the high performance concrete (HPC), which are almost currently used for significant applications in several Countries, such as U.S.A., Canada, France and Scandinavia (Norway, Denmark, Sweden).
The contribution to this symposium from Italy has been remarkable (11 papers). In particular, the research group arisen from the collaboration between the CTG – Italcementi Group and the Department of Structural Engineering – Polytechnic of Milan has presented 7 papers concerning the main topics (material developments, testing and structures design). In the present report these papers will be summarized.

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1. SYMPOSIUM DESCRIPTION

The 5th International Symposium on "Utilization of High Strength/High Performance Concrete" 1999 which was held in Sandefjord, has followed the previous symposiums held in Stavanger, Norway (1987), in Berkeley, California (1990) Lillehammer, Norway (1993) and Paris, France (1996), all dedicated to the recent developments and applications of HPC.

When the first symposium was held in Stavanger in 1987, the main utilization of HPC was mostly in offshore structures: at this moment, the applications of HPC are concerning all types of structures where the most required properties are strength, durability toughness fire resistance and fatigue resistance.

In this symposium particular attention was dedicated to the materials, the design criteria, the standards and the recent realizations, including case records.

Besides several key papers or invited notes, the symposium has included about 150 papers divided into five sections:

1. New Structural Concepts;
2. Design Method and Criteria, Recent and Current Research, Codes and Specifications;
3. Construction – Case Records, New Techniques and Applications;
4. Materials – Mix Design, Mechanical Properties, LWA-Concrete, Ultra HSC;
5. Durability.

In the first section "New Structural Concepts" some detailed information about the more recent evolution of innovative structural concepts, concerning bridges, submerged structures (tunnels) and other interesting HPC constructions were presented.

In the above-cited second section an updated state-of-the-art referred to the design and the testing of HPC structural elements and to the methods and criteria to be used for the design and the structural verification was given. In the third section several case records were presented: offshore structures (Scandinavia), bridges and High-rise buildings (above all in U.S.A., Japan, Scandinavia and Canada). Several American financed projects were also presented, concerning infrastructures and pavement applications (FHWA/SHRP Program). In the fourth wide section about materials and mix design, some proposals about new concepts for the definition of HPC mixtures (self-levelling/self-compacting concrete, fiber-reinforced materials, also using specific software, as a function of the final desired properties (e.g. tensile/bending strength, durability, controlled shrinkage, fire resistance, abrasion, creep) were presented.

Finally, the last section " Durability" included relevant considerations about the structures in marine environment.

This event has represented a good occasion for assessing the updated situation about the high performance concrete (HPC), which are currently used for significant applications in several Countries, such as U.S.A., Canada, France and Scandinavia (Norway, Denmark, Sweden).

The contribution to this symposium from Italy has been remarkable (11 papers). In particular, the research group arisen from the collaboration between the CTG – Italcementi Group and the Department of Structural Engineering – Polytechnic of Milan has presented 7 papers concerning the main topics from the new material development (HPC using white cement), to the testing and structures design) [1-7]. In the present report these papers will be summarized. Other two interesting papers were presented by researchers of the Polytechnic of Milan, concerning the thermal damage of HSC beams [8] and the advantages of using steel fibers for thin-web roof-elements [9].

2. PRESENTED PAPER

2.1 INTRODUCTION

The papers summarized in this section represent an updating of the R&D experimental activity about HPC/HSC conducted since 1995 by CTG Italcementi Group and the Department of Structural Engineering (Polytechnic of Milan). The research has concerned the mix design development, the physical-mechanical testing and the manufacturing development of high/very high performance concrete (HPC/VHPC) to be used for structural applications in Italy. Indeed, the use of HPC/VHPC is still limited in Italy, above all for the lack of adequate standards and recommendations: by this experimental work a wider and wider utilization of these special building materials would be encouraged, by demonstrating the technical and quality advantages.

After a complete physical, mechanical and structural characterization of HPC and relative structural elements, the first applications have been suggested and among them, the first case study is practically ended: it is a road bridge near Rezzato (Brescia), built with prestressed beams produced using fiber-reinforced very high strength concrete (FRVHSC), possessing a characteristic cubic strength of 100 MPa.

This project has been performed according to the recent recommendations edited by the Higher Council Ministry of Public Works (November 1996 [10]). According to this document, it is allowed to use VHPC having a characteristic cubic strength higher than 75 MPa, once the "product certification" has been obtained. The certification can be obtained after presenting to a specific committee (Central Technical Service) a complete documentation including the theoretical approach adopted, the design rules, an adequate experimental testing programme and an adequate quality assurance control (Fig. 1).

A part of this both theoretical and experimental work will be described in most of the following papers presented at the Symposium. Other new HPC applications are in progress in Italy: the outstanding example is the Church of the 2000 year (Rome), designed by the Architect Richard Meyer, whose shells will be built using white HPC precast elements (R_{ck} 75 MPa). The precast concrete will be produced using an innovative white cement, that offers a high degree of whiteness, which remains constant over time. This innovative concrete will be described in one of the following papers.

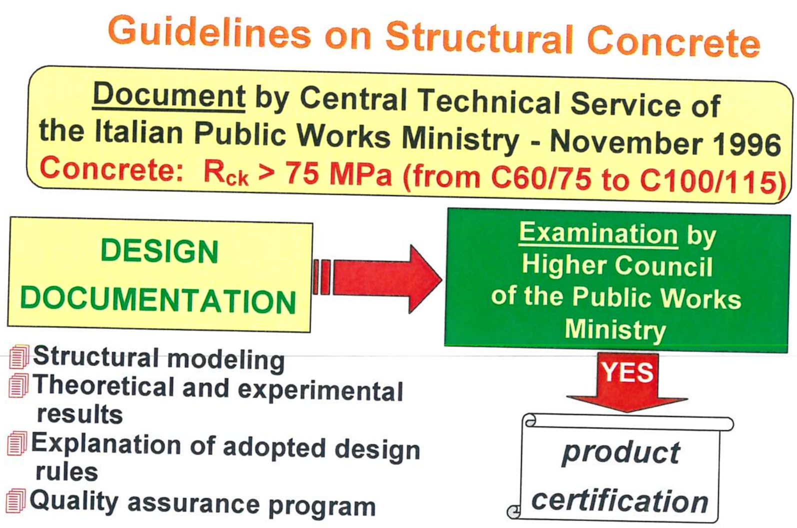

Figura 1 - Guidelines of Italian Public Works Ministry

2.2. PRESENTED PAPERS

2.2.1 "ON THE CORRELATION BETWEEN THE MODULUS OF ELASTICITY AND THE COMPRESSIVE STRENGTH IN VHSC", [1].

As it is well-known, the determination of the modulus of elasticity used for the production of structural members is of crucial importance in civil engineering, both for functional (serviceability limit states – SLS) and for safety reasons (ultimate limit states – ULS). The experimental results available in literature, together with the more recent standards and recommendations, both for normal and for high strength concrete point out that the modulus of elasticity E_c is correlated with the compressive strength of concrete. The suggested relationships, defined after experimental tests, can be well-referred only to the mix design utilized, so they are purely indicative if used with different compositions.

The modulus of elasticity for concrete depends on the modulus of elasticity both of the paste and of the aggregates used, other than their proportions in the mix design.

The modulus of elasticity for cement paste essentially depends on its porosity and consequently on the water/binder ratio, but also on mineral additions (micro silica, for example) and/or on chemical admixtures (water reducers and super plasticizers) which can be present in the mix, which allow to obtain a denser matrix. Thinking about the correlation between the water/binder ratio and the compressive strength as well as upon the correlation between the water/binder ratio and modulus of elasticity (for the same mix design), it can be understood why all the relationships proposed for the modulus of elasticity are referred to compression strength, which is much easier to determine and which any other parameter is usually referred to.

The influence of the modulus of elasticity of aggregates is very relevant on the determination of the modulus of elasticity of concrete, not necessarily on the compressive strength: some experimental tests seem to demonstrate that different concretes possessing the same values of compressive strength, prepared with different types of aggregates, have very different values of modulus of elasticity.

The need of determining proposals of relationships between the modulus of elasticity and the compressive strength for high/very high strength concrete is due to the fact that in most of standards and recommendations these materials are considered as ordinary concrete (up to 60 MPa about) and the suggested correlations are not valid beyond this limit. In fact, the mechanical behavior of these very high strength materials is very different in comparison with that of normal concrete.

The shape of the stress-strain curves is closely related to the nature of concrete components, especially of the aggregates: several studies have shown that high strength concrete of equal strength but prepared with different aggregates can significantly have different moduli of elasticity.

In this paper, the most recent correlation between strength and modulus of elasticity, included in national standards or proposed in literature, are mentioned.

The results obtained by compressive tests, made on cylindrical specimens (height/diameter=2) of very high strength concrete, also fiber-reinforced – which have been previously interested by a load-unload history (30% of the failure load) – which permitted to determine the beginning of the ascending branch of the stress-strain curve and having, from this, the elastic modulus, are also described.

Finally, the correlation between the measured compressive strength and the estimation of elastic modulus is discussed, and the comparison with expressions from the literature and from the national and international codes is analyzed (Fig. 1).

On the basis of tests executed on the experimental results (Cylindrical specimens of diameter=100 mm and height=200 mm prepared in steel moulds and water cured at 20°C until 28 days) obtained from the mix design series no. 1 and 2 (table 1) it can be derived that:

1. there are many correlations between modulus of elasticity and compressive strength, proposed in standards, recommendations and in literature: these differences are evident, even if the range of high/very high strength concrete is considered;
2. even the modulus for high/very high strength concrete is strongly dependent on the nature of (coarse) aggregates (values of modulus for series no. 2 (siliceous-calcareous aggregates) are lower than the value of modulus obtained from the series no. 1 (quartz aggregates), even if the strength values are higher (135 MPa instead of 110-120 MPa));
3. the best reproducibility was obtained not using a correlation proposed by norms, but using the correlation proposed by Gutierrez, which takes into account the aggregates quality (using the correlation proposed by Gutierrez, which takes into account the quality of the aggregates quality (through the α_β coefficient), it is possible to calculate from f_{cm} the corresponding E_c values with excellent accuracy, so that the differences between experimental and calculated values are almost negligible);
4. fibers do not influence the values of modulus even when relatively high contents by volume are used.

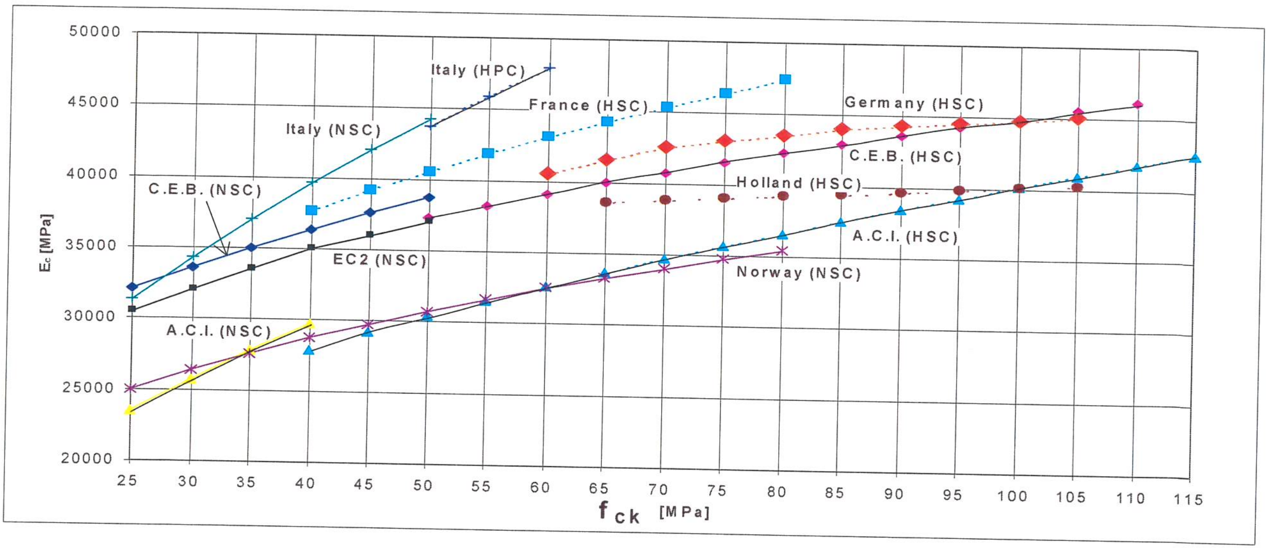

Figura 1

Table 1 - Mix designs, kg/m^3 (daN/m^3)

	MIX 1 - Series no. 1			MIX 2 - Series no. 2	
mix reference definition	2/0% Fiber	2/2% Fiber	2/4% Fiber	1/0% Fiber	1/1% Fiber
Portland cement CEM I 52.5 R	709	658	607	770	770
Micro silica, uncompacted	144	131	118	86	86
Natural sand (0-3 mm), silic.calcar.	-	-	-	543	512
Crushed aggregates (3-6 mm) silic.-calcar.	-	-	-	817	770
Quartz aggregates (0-3 mm)	1412	1316	1220	-	-
Acrylic superplasticizer (30% solid)	36	40	40	39	39
Steel micro fibers	0	160	320	0	78
Total water	191	177	163	215	213
Water/binder	0.225	0.225	0.225	0.25	0.25

2.2.2 "CHARACTERISTIC COMPRESSIVE STRENGTH OF VERY HIGH STRENGTH CEMENT-BASE MATERIALS", [2].

The use of a material for structural applications requires a complete knowledge of its mechanical performances. In particular, compressive and tensile strengths are fundamental properties, as well as other properties but only for particular applications (for ex., impact strength, wear and fatigue resistance). In these more recent years, it is also required that a structural material, including concrete, maintains its performances for a long time, in other words that it has a good chemical and physical durability. The concrete performances are generally correlated in a positive way, so that is recognized the improvement of a determined property as associated to the improvement of other properties/performances: for example, the increase of the concrete compressive strength is followed by an increase of the tensile strength.

In a such way, the global performance behavior can be estimated with reference to a single significant property.

In the case of normal concrete, all the norms and standards classify the material grades as a function of characteristic compressive strength (from the structural point of view). Other than the quality and properties of its composition, this strength (and the more well-known characteristic compressive strength, at 28 days) assumes the significance of fundamental parameter for the behavioral characterization of concrete. However, it is important to underline that for a good durability it is necessary to have a "good" concrete with reference to its physical characteristics (low porosity, low permeability, ...).

In this work the problem of the evaluation of very high strength concrete (VHSC) characteristic compressive strength is discussed, with particular reference to the possibility of incorporation of fibers to primarily increase the tensile/bending strength. This study is also aimed to put in evidence the effects of several process parameters (curing time, specimen dimensions, types of mold) on this fundamental property.

In this paper, the result of 228 compression tests performed on very high strength cement-based material specimens are discussed. The considered material is a very high performance mortar, having a maximum aggregate size of 3 mm, an aggregate/cement ratio of 2, silica fume, an acrylic superplasticizer and randomly dispersed micro-fibers whose contents are 0, 2 and 4% by volume.

A statistical description of the compressive strength is derived and the characteristic compressive strength values as a function of the micro-fibers volume content and of other parameters have been reported.

The results obtained put in evidence that the quality level of high performance concrete (and its components) used for structural applications influence deeply the design considerations and the final performances.

The cement-based materials used for this investigation are very high performance mortars, obtained with:

- Portland cement (CEM I 52.5 R, with a Blaine fineness of 4590 cm2/g;
- Grey microsilica in the form of a dry, not dense powder having a surface area of 20 m^2/g (B.E.T. method);
- natural crystalline quartz sand of high purity (99% SiO2), with dimensions 0-3 mm;
- acrylic copolymer superplasticizer, 30% solid content
- carbon steel micro-fibers, length 6 mm, and diameter 0.15 mm.

All the mixtures have:

- aggregate/binder ratio 2;
- superplasticizer/cement ratio 0.02;
- water/binder ratio 0.22;
- microsilica/binder ratio 0.2;
- fiber content: 0, 2 or 4% by volume.

The experimental results(*) enable us to notice what follows:

1. The presence of fibers, other than determining an increase of the ultimate strength due to the confinement effect, determines an improvement of material performances in the sense of a more granted quality, but only up to a certain volume content (about 2%);
2. The standard deviations of the fiber-reinforced materials (2% and 4%) are lower than the corresponding value for the plain material;
3. The standard deviation, referred to the specimens containing 2% by volume of fibers, is lower than 7% of the mean values (variation coefficient <7%);
4. Higher fiber contents (4%) can determine higher values of standard deviation (70 mm and 100 mm sided specimens), thus making the scarce increment of the mean strength negligible;
5. In order to obtain the best compaction of materials considered as well as the best mechanical performances, the use of steel moulds is required.

2.2.3 "A STATISTICAL EVALUATION OF SPECIMEN SIZE AND SHAPE EFFECTS ON COMPRESSIVE STRENGTH OF THE VHSC AND VHSFRC", [3].

In civil engineering the determination of the failure stress of the material used in the constructions is of primary importance. The experimental results that can be found in literature and referred to ordinary concrete indicate clearly that the failure stress (apparent) of the concrete decreases with the geometric dimensions of the test sample. This is one of the aspects of the size effect, characteristic of the heterogeneous and fragile materials as concrete and rocks, effect of primary importance when it comes to the «translation» of the strength of the material determined in laboratory to the material used in the construction of structures of real dimensions. Therefore the failure strength of the material with which a real structure is composed cannot be considered that obtained from usual laboratory tests on specimens of limited dimensions. This effects have been attributed in the past to several reasons; among these the influence of the confinement due to the plates of the test machines on the strength and the failure of the specimens, the difference in temperature and humidity between the surface and the core of a sample that generate tensions and differences in the degree of maturation of the specimens.

The codes, giving informations regarding the dimensions of the specimens to be tested to compression in order to evaluate the failure strength of concrete, try to guarantee significant dimensions for the determination of this mechanical characteristic.

The first rational approach to the problem is due to Weibull that, on a statistical basis, determined the strength of a structure in the basis of the probability of failure associated to the defect of critical dimensions. The decreasing strength with the dimensions is due to the increasing probability of the presence of these defects in the structures of big geometric dimensions. Later on, Neville suggested that the size effect in the concrete could depend on the probability of finding micro cracks having a critical orientation and dimension, probability that grows with geometric dimensions.

So, the first problem that is met, in the characterization of a material is the individuation of the optimal dimensions of the specimens for the compression tests. This is important specially in the case of very high strength concrete, because, as it can be intuited, the high strength of the material determines very high values of compressive failure loads, when the dimensions of the specimens are those used for ordinary concrete. Further on, the presses used in official laboratories for the tests on

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(*) Values calculated on the basis of statistic parameters (mean value, standard deviation, variation coefficient) for each set of 12 specimens, with reference to the statistic evaluation of experimental results of compression tests performed on cubic specimens, by varying several parameters: cube sides (40, 50, 70 and 100 mm), fiber content by volume (0, 2 and 4%), curing time (7 and 28 days) and, finally, mold material (steel, PVC, polystyrene).

constructions materials could result non compatible conserving for very high strength concrete the typical dimensions of those used for ordinary concrete.

The problem, on the basis of results obtained in a series of experimental compressive tests on very high strength plain and fiber reinforced concrete specimens, of cubical and cylindrical shapes of different dimensions is examined. The variation of compressive strength is examined, in relation to the dimension of the specimens and of the fiber content. The work has the purpose to suggest the optimal dimensions of the specimens for the compression tests and to individuate the correlation that exists between results obtained with cubical and cylindrical specimens.

In this paper the effects of specimen size and specimen shape on the compressive strength of the material are examined. Tests on cylindrical and cubic specimens of various sizes were performed. Two concrete mixes are examined, one having a maximum aggregate size of 3 mm, and the other having a maximum aggregate size of 6 mm. Both of them had in common: aggregate/binder ratio of 2, a silica fume/binder ratio of 0.2 and randomly dispersed steel micro fibers of 0, 2, 1 and 4% by volume. Eight different specimen types of moulds with four different high strength concrete compositions were considered. The compressive strengths of all specimen types and the conversion factors between them were evaluated and compared.

The materials used for this study present basically two different mixes. The first, commonly defined DSP (Densified Systems containing homogeneously arranged, ultra fine Particles), was proposed by the first time by H. Bache. It is about very high strength malts containing quartz sand, silica fumes and steel micro fibers.

The second differs only for the type and dimension (3-6 mm) of the calcareous crushed aggregate.
Mix designs for this study were defined as shown in Table 2.

Table 2 - Mix designs, kg/m^3

	MIX 1 - Series no. 1			MIX 2 - Series no. 2	
mix reference definition	2/0%Fiber	2/2% Fiber	2/4% Fiber	1/0% Fiber	1/1% Fiber
Portland cement CEM I 52.5 R	709	658	607	770	770
Micro silica, uncompacted	144	131	118	86	86
Natural sand (0-3 mm), silic.calcar.	-	-	-	543	512
Crushed aggregates (3-6 mm) silic.-calcar.	-	-	-	817	770
Quartz aggregates (0-3 mm)	1412	1316	1220	-	-
Acrylic superplasticizer (30% solid)	36	40	40	39	39
Steel micro fibers	0	160	320	0	78
Total water	191	177	163	215	213
Water/binder	0.225	0.225	0.225	0.25	0.25

The specimens, prepared using steel or plastic moulds and consolidated with a high frequency vibrating table, were unmoulded after 24 hours and cured in water at a temperature of 20 °C till the test (28 days).

The testing system consisted of universal testing machine Controls with a maximum capacity of 3000 kN.

The load has been applied following the standard procedures according the Italian UNI norms (UNI code 6132).

On the basis of the experimental results, it can be concluded that:

1. Regarding the considered geometry important variations of the compressive strength for the different dimensions, are not observed;
2. Regarding the fiber content, the compressive strength is lightly influenced;
3. The ratio between cylindrical and cubic strength, in the case of the 100 mm specimens results to be much higher (even greater than one) than that commonly accepted in the ordinary concrete.
4. For what concerns the safety factor, regarding the 100 mm specimens, variations of the shape coefficient k_c in relation to the strength fractile considered (50%, 5%, 0.5%) are not observed.

2.2.4 "WHITE CEMENT FOR HIGH PERFORMANCE CONCRETE", [4].

An accurate selection of the raw materials and a specifically devised production technology make white cement the most suitable binder in terms of aesthetic qualities and strength development for use in high technology concrete featuring high strength and durability.

This report describes the results of a laboratory research focused on the definition of mix designs and on the evaluation of the physical and mechanical properties of a white cement concrete with a cubic compressive strength greater than 80 MPa.

Verification tests performed on a large industrial production enabled us to set up a precasting technology for the production of large panels (placing, consolidation and curing) as well as to identify the best way to overcome or at least to minimise the occurrence of defects such as cracks, bugholes and mottling phenomena on large architectural concrete surfaces.

Without a doubt, it is its form that gives reinforced concrete greater architectural dignity and by concentrating on improvements to its surface, we can also achieve cost-cutting solutions at the same time.

By using white cement, in particular, the resulting concrete not only becomes an expressive material that, with an infinite range of colour tones, intensifies one of its aesthetic qualities, but also gains remarkable validity in terms of structural qualities due to its high mechanical strength.

Thanks to constant laboratory research into the chemical and physical structure of white industrial clinkers, white cement has been rendered increasingly and noticeably high-strength. Furthermore, it is now possible to get an excellent degree of whiteness and guarantee its constancy as a result of studies into the influence exerted by the minor components - the so-called dyes - and, in particular, the heat treatment received by the clinker.

White clinker is produced by taking the precaution to limit to not more than 0.15% the content of ferrous composites and other heavy metal composites, whose presence give common portland cement its distinctive grey colour. To achieve this, one starts by carefully selecting the raw materials: only mineralogically pure kaolins and white limestones are used.

This type of cement is, to all effects and purposes, a high-strength Portland cement; the essential characteristic that distinguishes it from the other cements is simply the fact that it is white.

MIX DESIGN

The mix design for a concrete made with white cement needs to be developed bearing in mind two properties that have an equal effect on this:

- the aesthetics or surface finish
- the strength or structural suitability

In other words, the right materials need to be chosen to create a delicate balance between the mix components and to guarantee the rheological behaviour of the resulting mix.

In the case of high-strength concrete (HSC), the raw materials are: water, cement and aggregates (the same as in the ordinary cement mix), to which mineral additions and superplasticiser admixtures may be added as required. As a result, HSC has a microstructure that differs significantly from that of ordinary concretes: more compact, with a system of much smaller capillary pores and far more intense interfacial binding of the stages, leading to different macroproperties in terms of strength and durability.

First of all, in HSC the water dosage is drastically limited by the action of the superplasticizer; besides, any variation is its dosage even if just a few % (4-5 litri/m^3) will have a substantial effect on the final properties (strength, compacity) of the concrete.

In general, white cements belonging to class 52.5 (i.e., featuring fast setting and high final strength, suitable also for high performance concretes with $Ø_{max}$ 15-25 mm) are dosed at 300-400 kg/m^3. When the $Ø_{max}$ drops - a practice currently adopted in this class of concrete, the cement content could reach high values (600-700 kg cement per m^3).

When deciding on the combination of the aggregates - sand and coarse aggregates -two basic requirements for our concrete must be borne in mind. In the first case - the aesthetics - the choice of coarse aggregate and thus its colour is of utmost importance for the worked open surfaces (e.g., bushhammered, sand-blasted and washed), while the choice of the fine aggregate is decisive for the colouring of the open surfaces without further treatment after the removal of the formworks. Very light sand is needed if we specifically want a perfectly white surface, while a coloured sand (often a common sand) is quite sufficient if a more particular tone is required.

In both cases, the use of white cement in the "open" concrete lets us have a far more brilliant mortar in contrast to the colour of the aggregate, thus enhancing this if the surfaces are "worked" and which makes casts brighter if the surfaces are left "as is".

In greater detail than the initial sand, the surface of a hardened mortar obtained with white cement:

- acquires more brilliance, the less brilliant the sand
- assumes an increasingly less pinkish and more bluish tone than that of the sands. In fact, the wavelength of the loose materials is longer than that of the hardened surfaces;
- loses colour percentage (the colour is weaker) and thus the use of coloured sands is not enough on its own to get a concrete with distinctly coloured surfaces (see Figure 3).

The results of the sand/mortar colour experiments show that it is not necessary to use particularly white sands, which are often hard to find, to create a white cement concrete: we can quite happily use normal sands.

The "colour loss" or rather the shades obtained by passing from sand to mortar can have a very pleasing chromatic effect, often preferable to pure white.

However, when the designer wants to achieve a brilliant and definite colour, he changes the colour of the surface by adding a mortar of the relevant colour capable of accentuating the colouring and giving the required dominant wave length: in this way, it is possible to get a virtually infinite spectrum of colour tones, as.

In HSC mixes, the generalised (and necessary) use of fine additional materials (pozzolanic additions) arise from the need to saturate the spaces between the particles in the cement matrix with solids rather than mixing water.

There are certain limitations involved in compacting the cement matrix, which is only possible by reducing the w/c ratio. Even when we cut the amount of water, the concrete continues to resemble ordinary concrete in most aspects, particularly in terms of the development of its strength and heat of hydration, but it especially maintains a microstructure that is rich in C-H and C-S-H with a high Ca/Si ratio with an obvious effect on the durability of this type of concrete. Furthermore, to guarantee that the mix is sufficiently workable, the amount of cement used needs to be increased, with further consequences on the heat and thermal shrinkage. Another important change in the microstructure of this concrete is the addition of fine mineral materials in the mix, thus improving both the chemical and physical characteristics of the concrete [4]. In fact, the very fine particles in the mix harden to block the interlocked pores, thus blocking the connection of the capillary network and increasing the nucleation sites with the precipitation of hydration products.

The effect of all this is to speed up the hydration process and reduce the size of the capillaries. Finally, the pozzolanic reaction produces more intense compacting in the mix/aggregate interfacial area with the build up of a smaller amount of C-H here than in the same area in normal concretes. Moreover, since the cement quickly picks up the limestone as it becomes hydrated, no macro crystals are allowed to form (if any, they are extremely small).

In the case of white cement concrete, it is clear that fine pozzolanic materials must be white and of all the materials currently available (such as silica fume, blastfurnace slag, fly ash, rice husk ash and metakaolin) only the latter can be considered. In fact, normal grades of silica fume - the most commonly used material in HSC - colour the mix, even if they be particularly light in colour or white. For this reason silica fume must be excluded, unless one can accept the appearance of their particular shades in the manufacts. Besides, use of "special" white silica fume seems to be disadvantageous from the economical point of view.

Metakaolin is by nature white and is highly reactive, thus making it a valid alternative to silica fume.In particular, it accelerates the hydration at greater intensity within the first 24 hours.

An interesting and somewhat adventurous new idea in the field of white cements, linked to wide-ranging moves to cut environmental pollution through the use of specific construction materials, is the development of a white binder called "Millennium TX" with a special titanium dioxide admixture mostly in the form of anatase. The cement product made with "White TX Millennium", thanks to the photocatalytic effect of the titanium dioxide, manages to keep its original look unaltered over time, with less mottling of exposed surfaces.

Trials on the formulation of this "self-cleaning" cement, plus in-depth research especially in Japan, confirm that the photocatalytic activity of the TiO_2 through oxidation in the presence of light and atmospheric oxygen on exposed plasters, mortars and concretes prepared in the laboratory manages to break down the various types of pollutant (organic substrata such as phenanthroquinone and poly-condensed aromatic compounds).

It has been shown that the aesthetic durability of the product (i.e., its ability to keep its colour unaltered over time) has no adverse effect on the strength of the material.

Since HSC demand water levels with a w/b ratio of 0.20-0.35, the technology for HSC turns to the use of new generation admixtures such as acrylics rather than melamine-formaldehyde or naphthalene-formaldehyde sulfonate condensates.

However there could be a problem caused by the incompatibility of these admixtures with many cements and with white cements in particular, causing quick workability losses.

Research into the interaction between the cement and the superplasticisers has yet to offer a good enough explanation for this phenomenon, although a few incompatibility factors have been identified. In the case of the cements: the C_3A content and its reactivity linked to morphology, the C_4AF content, the final form of the gypsum in the cement. In the case of the superplasticisers: the length of the molecular chain and the position of the sulfonate group in this, the type of cations and the presence of residual sulfates that influence the cement's deflocculating properties.

Neither is it a question of the cement's initial hydration kinetics as the fast thickening of the mix with the admixture occurs both when the w/c ratio is around 0.3 (when distances between the particles in the mix are short and the number of ions that can enter the solution is also low) and when the w/c ratio is greater than 0.5, as is the case with normal concretes.

The factors listed here on the basis of our experience are by no means exhaustive and it is quite possible that as research studies progress, we may find the answer by finding the specific admixture without having to modify the mineralogical make-up of the cement (a technologically difficult, though not impossible task).

Recently, the problem was solved (in CTG laboratories) by using a system of admixtures with two components: formula A (compatibility agent), free of water and that cannot be mixed with water, ready mixed when dry with the cement or aliquot (master) of the fine added material (metakaolin), and formula B, an acrylic added at the moment of mixing.

THE PANELS FOR THE "SHELLS" OF THE CHURCH OF THE YEAR 2000

To conclude and complete this study, we thought it would be interesting and useful to give a rough description of the prefabrication technology used for a construction in white HSC, which is of interest not only for the high level of aesthetic and strength qualities required of it. The manufacts produced are precast panel modules (3.0 x 2.0 x 0.8 m) of different size and double cambers, to be used to assembly the three shells in the Church of the Year 2000 in Rome designed by the American architect Richard Meier and engineered by Gennaro Guala at the CTG-Italcementi Group (Figure 1 and 2).

These panels had to be "open" with both curved faces vertically cast. Their enormous mass – 5 m^3 about of concrete – the low surface/volume ratio (3.5 m^2/m^3) and the presence of a strong and elaborate internal reinforcement frame are factors that required the development of a special technology covering all stages from the mix design to the storage and transportation of the panel. Not to mention the problems involved with the subsequent special assembly system.

The look of the two cambered surfaces measuring 6 m^2 each called, first of all, for the choice of a very narrow combination of rheological properties in the mix and the settlement system methods and times.

The huge increase in heat within such a large amount of cast concrete was controlled from the moment the formworks were removed to the moment when its temperature and humidity matched that of the surrounding atmosphere.

The strength of the concrete was linked not just to the mix design, but also to the aesthetic features of the structure mentioned above.

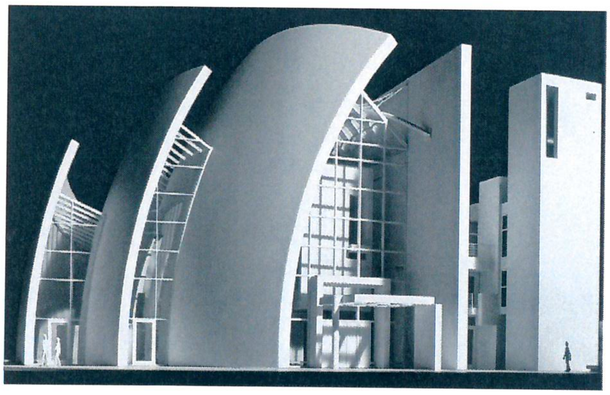


Figure 1. The Church of the Year 2000

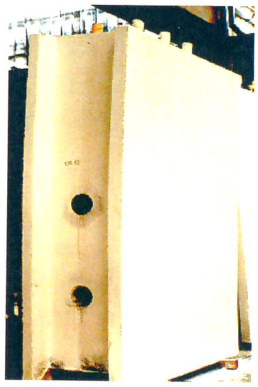


Figure 2. Type of panel

With the possibility of staying close to a w/b ratio of roughly 0.35 thanks to the double-admixture technique (compatibility agent + acrylic superplasticiser), the actual amount of White TX Millennium cement needed was quite low (350 kg/m^3) and the workability of the concrete was high.

As this structure had to be entirely white, a white Apuan marble aggregate was selected (from the Carrara area), whose particle size tends to produce a continuous curve (Füller).

For this reason, the maximum particle size was limited to 20 mm to ensure that the concrete would flow through the reinforcement and covers (40 mm) and at the same time to contain or at least greatly reduce surface defects (bugholes, gravel clusters, etc.) linked also to the elevated bulk of the aggregate due to the wall effect.

The appropriate content of fine aggregates < 0.2 mm makes it easier to obtain open faces with acceptable defect levels.

Metakaolin was used to increase durability and the strength over the medium/long-term.

The rheological behaviour of the mixes was such to ensure a slump value of almost 15 cm after 1 hour, starting from an initial slump value of 20 cm (minimum).

Mean values of water/binder ratio and density were respectively 0.33 and 2430 kg/m^3.

Table 1 shows the mechanical strengths of the concrete at different stages in the curing process. Table 2 shows the values of both modulus of elasticity (dynamic and static/secant) and Poisson's ratio.

The geometry of the product, that of the embedded reinforcement and the need to obtain aesthetically perfect surfaces virtually free of bugholes meant that the concrete had to be allowed to settle in specific formworks with the use of fixed vibrators with adjustable vibration frequency applied to the outside of these and internal vibrators with controlled embedding.

The hydraulic shrinkage, or drying shrinkage given the low amount of water in the mix, was relatively low and, in any case, the resulting tensions were counteracted by the high strengths of the concrete after just a short period of curing.

However, the wet curing that took place immediately after removing the forms obviously protected the structure from the insurgence of cracking.

Table 1. Mechanical strengths (MPa)

Compressive strength		Flexural strength	
24 hours	35	7 days	7
30 hours	41		
2 days	47.7	28 days	10.4
7 days	69.8	**Splitting strength**	
28 days	86.2	14 days	5.3
90 days	89.8	28 days	7

Table 2. Modulus of elasticity and Poisson's ratio (according to UNI 9771 and 6556)

	Dynamic elastic modulus GPa	Secant static modulus GPa
1 day 7 days 28 days	28,7 41,3 44,2	- 40 41
Poisson's Ratio (28 days)	0.24	0.25

When used for a somewhat massive product as was the case here, wet curing is required to avoid the formation of cracks due to the thermal expansion and shrinkage of the concrete.

Thermal expansion occurs through very slow dispersion of the heat that develops within the product due to the low diffusivity of the material. As a result, the temperature within the structure can reach temperatures of 60-70°C in a very short space of time,

On removal of the formworks, direct contact with the environment causes the outside of the product to cool rapidly. The new temperature condition causes the inner core to dilate in contrast to the shrinkage of the external shell and so give rise to possible cracking.

During the cooling process, if thermal contraction on connection with other structure elements at different temperatures were blocked, it would cause further cracks due to thermal shrinkage.

For this reason, immediately after the removal of the formworks, the panels were cured in a curing chamber with sprayed water at set temperatures to guarantee that the atmosphere was constantly saturated to suit the humidity and temperature of the product as these gradually changed.

The sprayed water in the curing chamber initially had a temperature just below that of the panels and then, at regular time intervals, its temperature was further reduced by steps of 10-15°C each time until the actual ambient temperature was reached. The panels stayed in this chamber for 3 days about, before being sent for storage.

2.2.5 "STRENGTH AND BRITTLENESS OF HIGH PERFORMANCE CONCRETE SLABS", [5].

A dependence of strength on size or scale is well known in concrete-like materials. It is also been widely reported in materials such as composites, rocks and ceramics, defined as quasi brittle, where a large span in scale between laboratory tests specimens and the structures typical of engineering practice exists. This aspect is only the first manifestation of the size effect that may appear with different features. Indeed, another demonstration of size effect is the different post-peak behavior (brittleness) that characterizes geometrically similar structures that fail under a given type loading.

These problems are particularly important when laboratory data must be employed in structural prediction, hence every time geometrically similar structures with different dimensions are to be compared.

The first rational approach to the problem is credited to Weibull who advanced a statistical theory of strength of materials by considering the probability of failure associated with defects present in the material. On the basis of fracture mechanics considerations, the problem has been extensively examined by Bazant.

The size effect is generally studied through a comparison of geometrically similar structures of different sizes and conveniently characterized by load parameters (such as the nominal strength) and overall structural response.

The objective of this paper is to present and to discuss experimental tests on plain and fiber-reinforced very high performance cement-based materials. It is shown that, when fibers are introduced into a brittle matrix, their influence on the control of crack bridging generates a reduction of the structural size effect. This is the reason why geometrically similar three-point bend beams were considered.

The tests were completed with interferometric measurements and location of acoustic emission of the damaged zone development in terms of shape and size. It is shown that in the fiber-reinforced materials the size effects appear reduced, in addition to an improvement in fracture toughness, strength and brittleness.

EXPERIMENTAL TECHNIQUES

Mix designs and test specimens

Mix-components of the cement-based materials used for this investigation are as follows: Portland cement CEM I 52.5 R, according to ENV 197/1 European Standard; an uncompacted grey microsilica; an acrylic copolymer superplasticizer, 30% solid content; a natural crystalline quartz of high purity (99% SiO_2), maximum aggregate size of 3 mm; carbon steel microfibers, diameter 0.15 mm, length 13 mm.

The concrete had an aggregate/binder ratio of 2 and a water/binder ratio of 0.22. The superplasticizer dosage, that is the ratio of the dry mass of superplasticizer solids to the mass of cement, was 0.02. Microsilica/binder ratio was 0.1, fiber contents were 0 and 2 (% by volume).
Four series of tests on prismatic specimens of high performance concrete, with and without steel reinforcing microfibers, were considered.

The specimens dimensions are as follows:

- type A - 160x40x40 mm, span 70 mm;
- type B -230x120x40 mm, span 210 mm;
- type C -460x240x40mm, span 420mm;
- type D - 900x480x40mm, span 840;

Testing apparatus

The testing system consisted of a closed-loop electromechanical Instron testing machine with a maximum capacity of 100 kN.

Strain measurement

A full field strain measurement technique is required to observe localization within the strain field: this is the reason why the ESPI (Electronic Speckle Pattern Interferometry) technique was used in these tests.

Acoustic emission

The acoustic emission signals generated in laboratory specimens are captured using piezoelectric (PZT-5A) transducers attached to the specimen surface, and pre-amplified before recording.

EXPERIMENTAL RESULTS

Two groups of beams were tested, one with and one without steel reinforcing microfibers randomly dispersed in the matrix (2% by volume). Specimens of four different sizes, three in each size, were cast and vibrated in steel molds.

They were left in the molds for 24 hours at 20°C, and then demolded and cured in water (20°C) until the tests (after almost 28 days). Details of the specimens geometry are presented thus resulting in a size range 1:12.

Typical load-displacement curves for different specimen sizes are shown in Fig. 1.

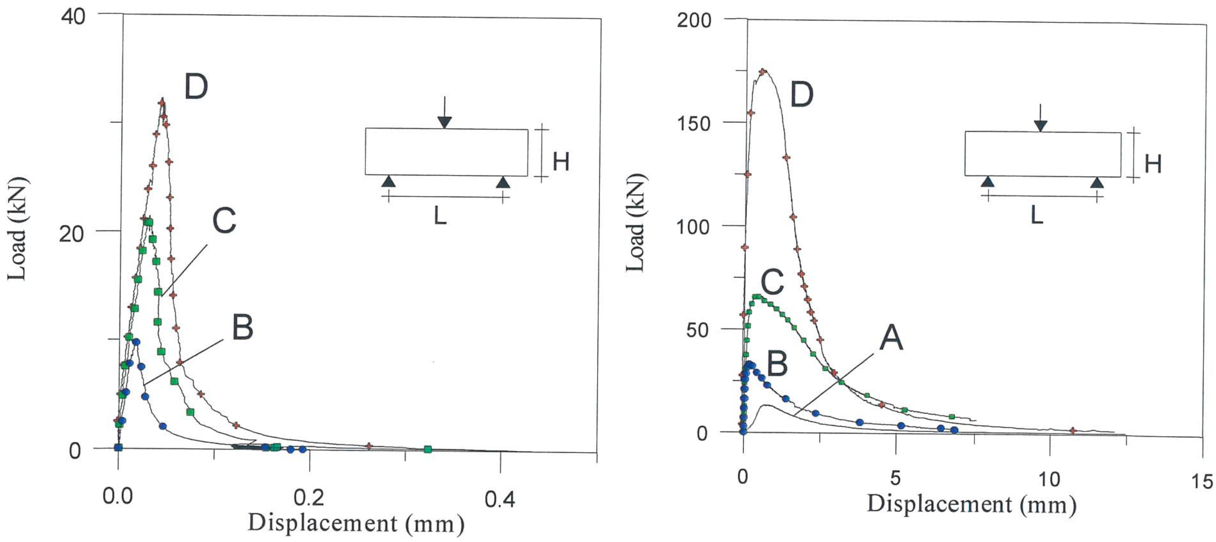


Figure 1. Load-displacement curves for plain beams (left) and fiber reinforced beams (right).

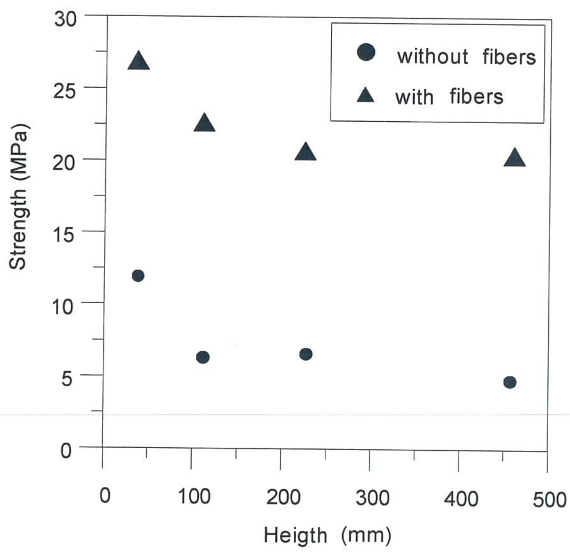


Figure 2. Nominal strength as a function of size.

Different is the response of the largest fiber-reinforced beams: a brittle behaviour was observed. This was probably due to fibers length, not long enough to bridge the large cracks (8-10 mm) observed in the first part of the post-peak branch. This was confirmed by AE activities not recorded after the peak load coming from a large volume in the lower part of the beams. This suggests that fibers length should be defined even as a function of the structural size to obtain an adequate ductile behaviour.

To discuss about strength, it is necessary to characterize the evolution of damage at the peak load.

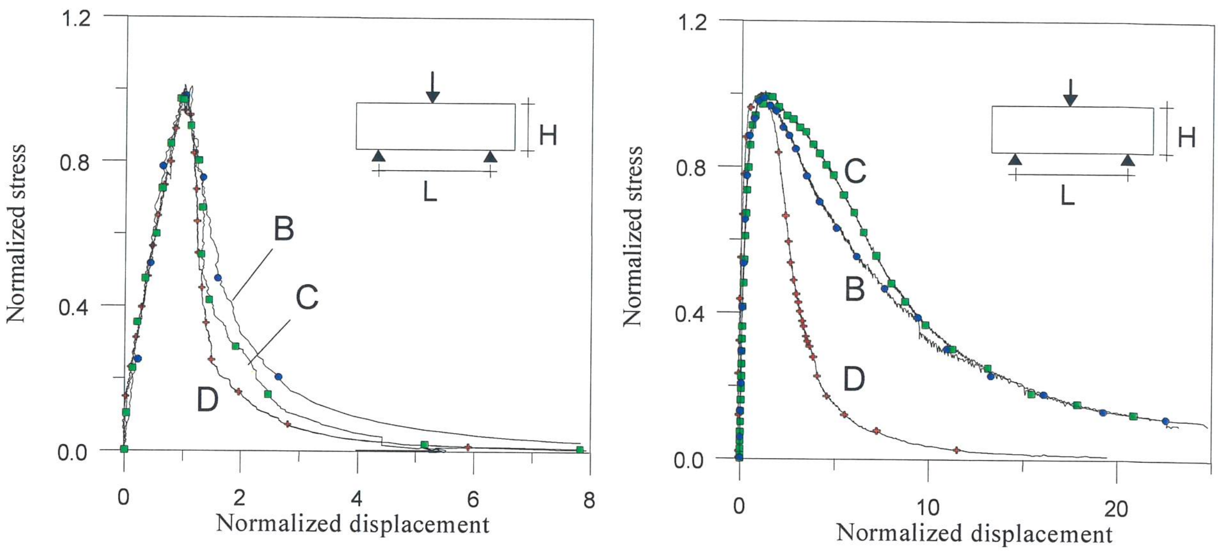


Figure 3. Normalized stress-displacement curves – plain (left) and fiber-reinforced (right) beams

From the ESPI fringe patterns, it is possible to identify the end of the localized damage zone as the point where the fringes appear sharply broken thus denoting a discontinuity in the displacement field, Fig. 4. The "crack tip" appears at the end of this region. The crack lengths are quite different in the two materials (fiber-reinforced and not). For the beams made with the plain material the crack length results to be almost constant (that is, independent of the beam size). In the second case, the fiber-reinforced beams were characterized at the peak load by a crack length related to the height of the beams. Table 1 summarizes the mean values of the peak loads and the relative crack lengths.

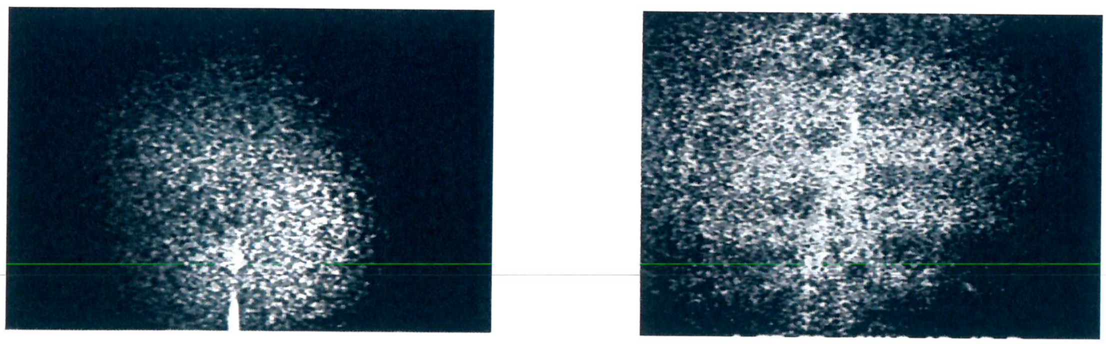


Figure 4a. Digitized images of the fringe pattern at the peak load for plain beams B (left) and fiber-reinforced beams B (right).

Table 1. Experimental results

Type	*spec. dim.*	*span (mm)*	*maximum load (KN)*		*(crack length)/(beam height) ratio*	
			with fibers	without fibers	with fibers	without fibers
A	160x40x40	70	11.90	6.74	0.75	0.40
B	230x120x40	210	38.14	10.05	0.77	0.13
C	460x240x40	420	66.50	21.48	0.83	0.07
D	900x480x40	840	175.44	32.36	0.69	0.03

A comparison of two identical slabs (type B), with and without fiber reinforcement, is shown in Fig. 5.

For the slab made of plain material, a discontinuity in the ESPI image was observed at around 80 percent of the peak load, where the non-linearity in the load displacement response of the structure became apparent.

Conversely, in the fiber-reinforced slabs the localization of the deformation appeared at about 50 percent of the maximum load capacity.

It is interesting to observe that for this size there was an order of magnitude of difference of the deflections recorded at the peak load.

Therefore, the experimental evidence shows that without fiber the peak load is reached with an inelastic trend almost independent from the beam size; on the contrary, when fibers are added to the matrix, at the peak load the inelastic region increases with the specimen size (Fig.6).

In this sense there is a behavioural analogy of fiber-reinforced materials with the ductile metallic materials.

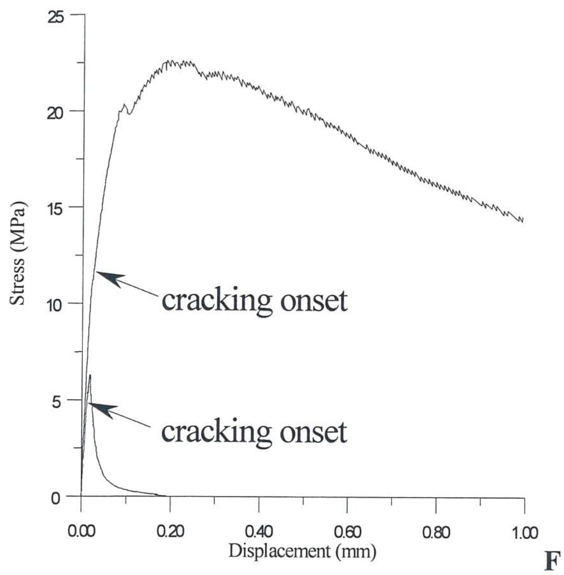


Figure 5a – Stress-displacement curves for plain and reinforced materials (type B geometry)

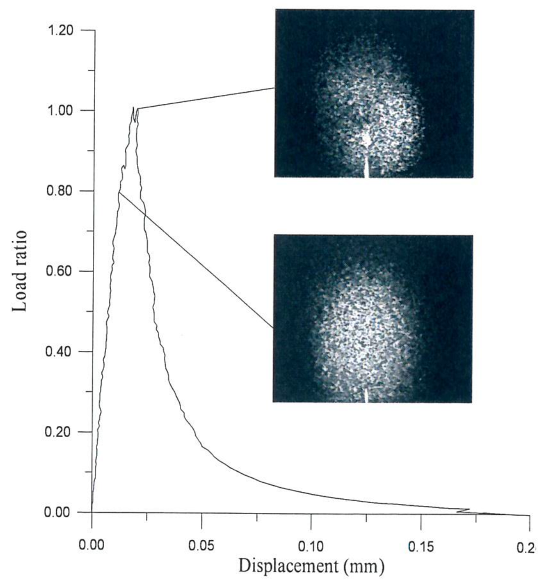
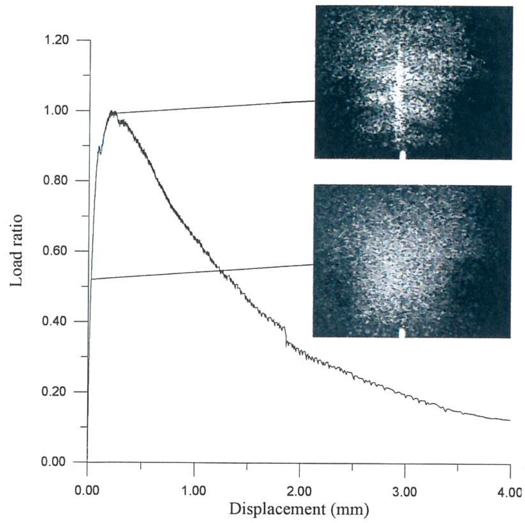


Figure 5b – Load ratio (referred to the peak load) - displacement curve for plain material (type B geometry)



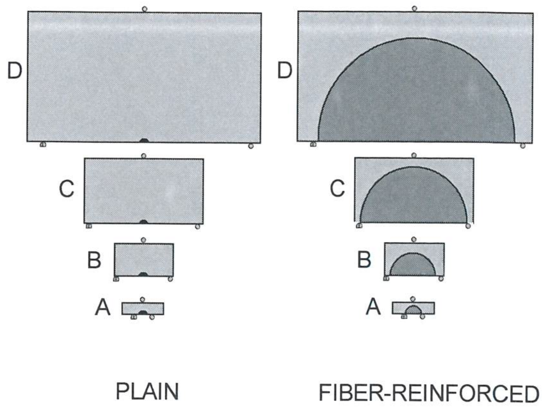


Figure 5c – Load ratio (referred to the peak load) - displacement curve for fiber-reinforced material (type B geometry)

Figure 6 – Qualitative evolution of inelastic region in geometrically similar beams

From a mechanical point of view, this behaviour justifies a reduced notch effect in the critical cross-section of the beam, that means a lower stress gradient in the undamaged volume. Consequently, the control of crack bridging, due to the fibers, provides increased strength and ductility in the damaged region and reduces the stress concentration in the elastic region. However, this behaviour is a function of the fibrous reinforcement (volume content, bulk distribution, fiber aspect ratio, fiber-matrix interface, etc.).

A fibrous reinforcement not only improves fracture toughness and bending strength, but at the same time reduces the structural size effect.

The multiple cracking that characterizes the fracture process in the fiber-reinforced materials leads to a decrease in notch sensitivity.

With reference to the unreinforced cement-based material evaluated, AE activity around the peak load has allowed a similar process zone length, for different beam sizes, to be observed.

As far as the fiber-reinforced material is concerned, AE activity and ESPI measurements around the peak load have shown that the process zone length increased with increasing size of specimens. This study confirms that a combination of these experimental techniques provides complementary pieces of information about the microcracking and cracks development.

2.2.6 "FLEXURAL BEHAVIOUR OF PRESTRESSED FIBER REINFORCED CONCRETE BRIDGE BEAMS", [6].

Composites are evolving toward new and improved materials and new applications for structures. High performance cement- based composites have been recently developed in the attempt to improve some poor properties of traditional materials such as strength-strain characteristics, toughness and durability. Cementitious materials are considered brittle, weak in tension and characterized by a substantially high porosity. The use of a correct amount of particular ultrafine mineral additions (e.g. microsilica), of a low water/cement ratio, the addition of discontinuous fibers and, sometimes, the use of special curing methods, lead to an increase in mechanical performance that may best fit the demands of the structural design.

Generally speaking, the definition "high performance" is meant with not only to differentiate structural materials from conventional ones, but also to optimize a combination of properties in terms of final applications in civil engineering. The most interesting properties are for example strength, ductility, toughness, durability, stiffness, an a thermal resistance, although the final cost of materials and, above all, of the manufactured products must be taken into account.

The specific "high performance fiber-reinforced cement-based composites" term refers to high performance cement-based materials, particularly developed for specific applications, for which toughness, ductility and energy absorption are fundamental properties. Furthermore, fibers allow to improve the quality of concrete, in order to obtain an increased its homogeneity and to better exploit the high strength matrix.

The objective of the present paper is to present the results of tests on very high strength fiber reinforced concrete beams in bending. The development and the optimization of mix design were performed for a special concrete composed of portland cement, microsilica, acrylic superplasticizer and steel fibers, having the following basic characteristics: nearly self-compacting concrete (slump more than 20 cm) for almost 1 hour from the batch preparation, mean compressive strength of almost 120 MPa.

Afterwards, six 3m-long reinforced beams (three of which lightened with polystyrene cores) and four 12.4 m-long prestressed concrete beams were produced and tested.
For the quality control, a complete mechanical characterization (compressive and bending tests) of the fiber-reinforced concrete was also made. The results confirmed the very good reproducibility passing from the laboratory scale to the industrial scale. A monitoring with optical interferometry, using a moiré grid with 40 lines/mm, was considered.

EXPERIMENTAL TECHNIQUES

Materials and mix design

To obtain a high performance concrete the choice of the materials is crucial. For this particular application, the mix components were chosen in terms of workability (time of almost 1 hour for a slump of about 20 cm), and of compressive strength (at least 130 MPa).
After a preliminary laboratory testing program, the material proportions were defined as shown in Table 1.

Table 1.Optimized mix design

Component	*Kg/m³*	*Observations*
Portland cement CEM I 52.5 R	770	
Microsilica, uncompacted	86	10% on binder basis
Natural sand (0-3 mm)	512	40% on total aggregates
Crushed aggregates (3-6 mm)	770	
Acrylic superplasticizer (30% solid)	39	Dry extract: 1.5% on cement basis
Steel microfibers d=0.16 mm L=13 mm	78	1% by volume
Total water	213	Including superplasticizer water
Water/binder	0.25	

Experimental program and testing equipment

To characterize the material and its structural behavior different tests were performed: compression tests on prismatic and cylindrical sample, tensile tests (direct and bending tests) and three point bending tests on the reinforced concrete beams and on the bridge beams. The specimens were cast in metallic molds.

EXPERIMENTAL RESULTS

Compression tests

Tests carried out under displacement control on cubic and cylindrical specimens were conducted after 3, 7 and 28 days from the production time of 3 m-long beams. The curing of the specimens was at the same environmental condition (temperature and humidity) (Fig.1a).

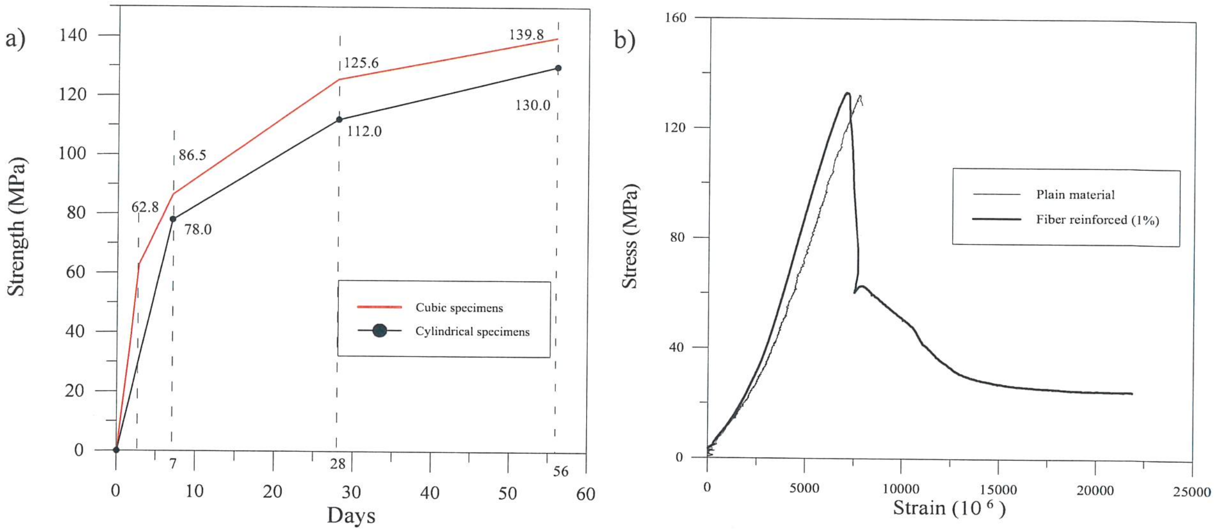


Figure 1.a- Strength as a function of aging time; b- Stress-strain curves.

Tensile tests (direct tension and three-point bending)

Direct tensile tests on specimens with different fibers percentage and fibers length were carried out in order to know the best performances/cost ratio (Fig. 2).

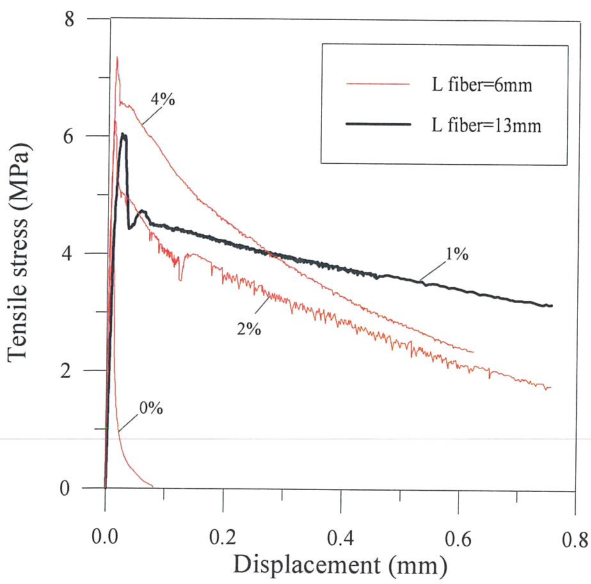


Figure 2. Stress-displacement curves.

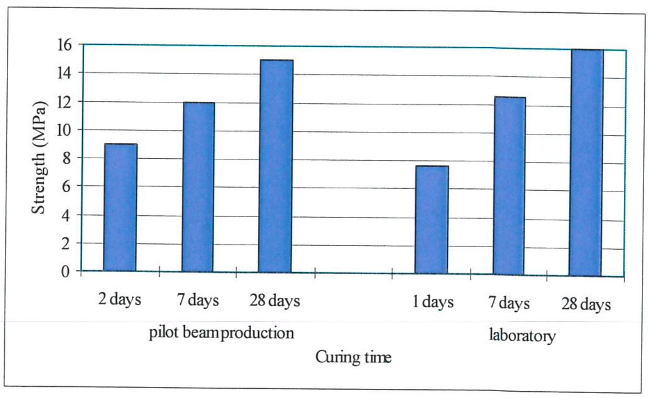


Figure 3. Bending strength

The results shown that already with 1% of long fibers (13 mm) the tensile strength and the ductility of the material increase heavily. In fact without steel fibers the material is very brittle, but by adding the steel fibers the ductility properties and the strength enhanced.

Three point bending tests were performed at different time of curing. The tests were performed in crack mouth displacement control at a constant rate of $2{\cdot}10^{-4}$ mm/sec.

In Fig. 3 are reported the results in terms of average value, obtained by testing almost three specimens at every time. The comparison of values confirms that the production procedures were correctly applied, passing from the laboratory scale to the industrial scale.

Beams tests

The 3 m-long beams were tested under a three point bending configuration. The cross-sections of both beam types are shown in Fig. 4.

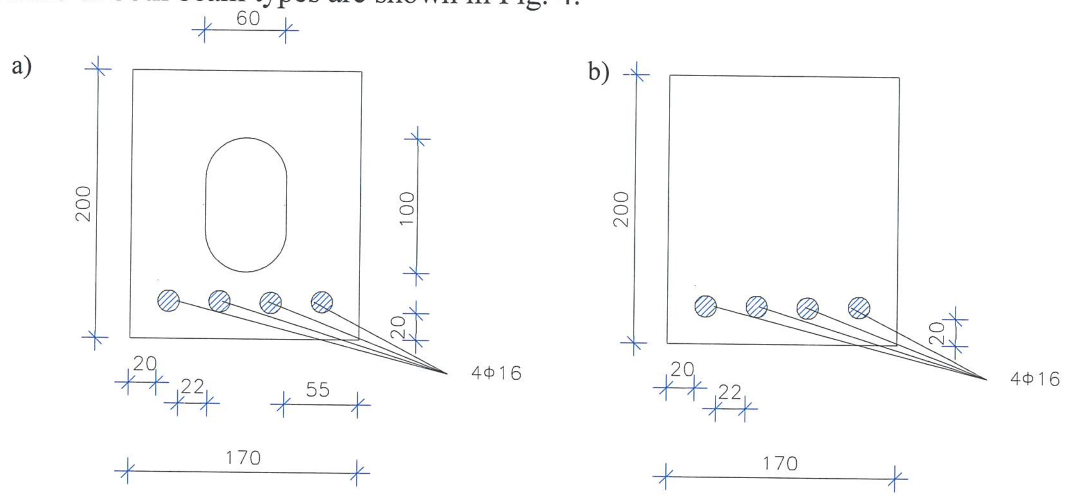


Figure 4. Cross section of the beams a- beam with polystyrene core; b- full beam.

The beams were reinforced with commercial steel ribbed wires with a nominal diameter ϕ=16mm. Four experimental load-deflection curves for the reinforced beams are reported in Fig. 5. There is not a significant difference between the load-displacement response of the two beams and all the curves have the same shape.

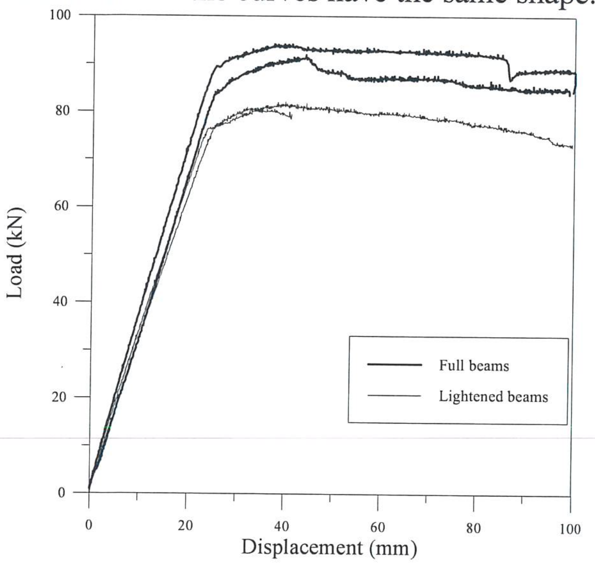


Figure 5. Load-deflection curves.

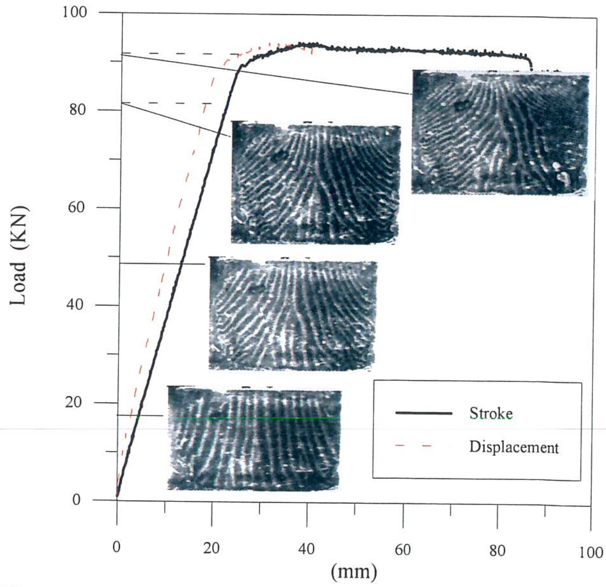


Figure 6. Moiré fringes at different load levels.

It may be observed that the crack penetration in the beams were comparable but the crack opening in the lightened beam was about twice (0.127 mm) of the crack opening (0.076 mm) for the full beam.
This difference is mainly due to the not negligible contribution of the tensile strength of the material that, in the tension zone of the beam improves the flexural behavior of the beam.

Bridge beam tests

The bridge beams, simply supported, were loaded as shown in Fig. 8.

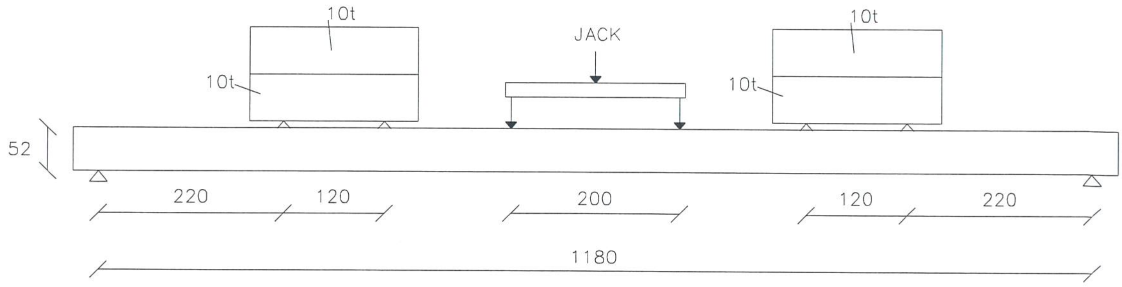


Figure 8. Loading device.

Four beams, two with and two without a completing slab were tested (Figs.9a, 9b). The specimens were prestressed by 21 straight seven-wire steel strand. Figs 9 show the cross section and the strand distribution of the beams.

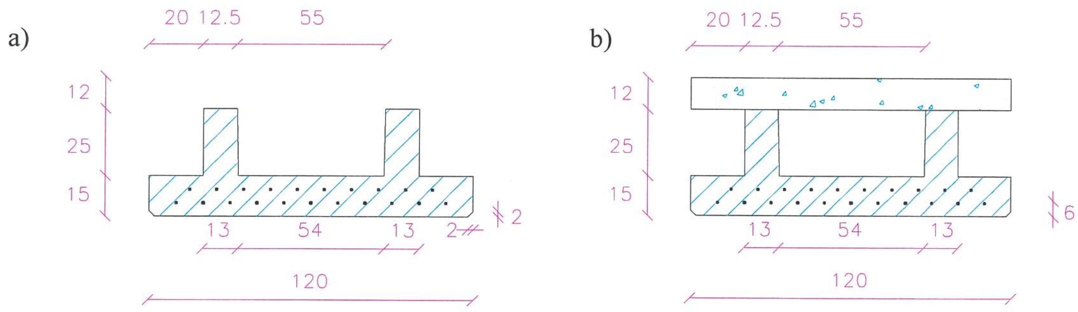


Figure 9. Cross section of the beams.

These cross sections has been designed for a possible application for a small road bridge that consider the production of prestressed fiber reinforced concrete beams (Fig.9a) and a subsequent casting of a normal concrete slab (Fig.9b).

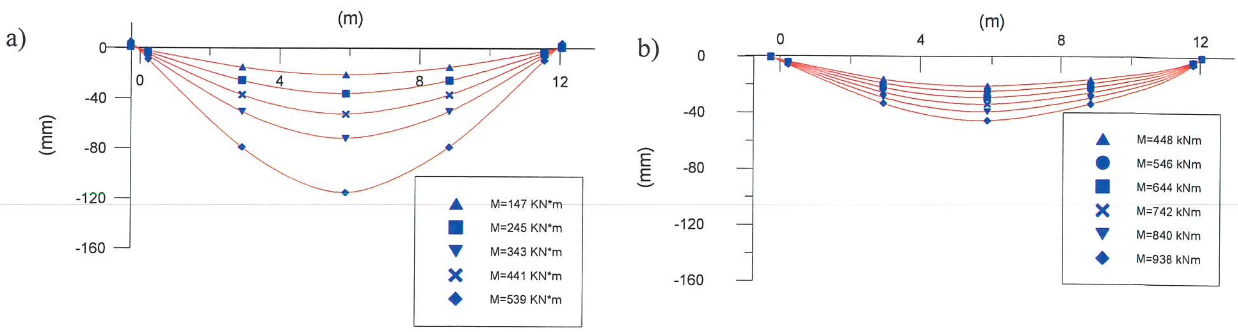


Figure 10. Deflection curves at different bending moment levels.

Fig. 10 shows the deflected shape of beams at different loading steps expressed as a function of the maximum moment for the beams without and with completing slab, respectively.

It may be observed the significant contribution of the completing slab to the beam stiffness. Fig. 11a shows the moment midspan displacement curves for both beams. The beams without the completing concrete slab failed suddenly due to crushing of the concrete matrix in the compression zone and subsequently instability of reinforcing wires.

Significant influence had the completing slab made with normal concrete (compressive strength of 35 MPa).

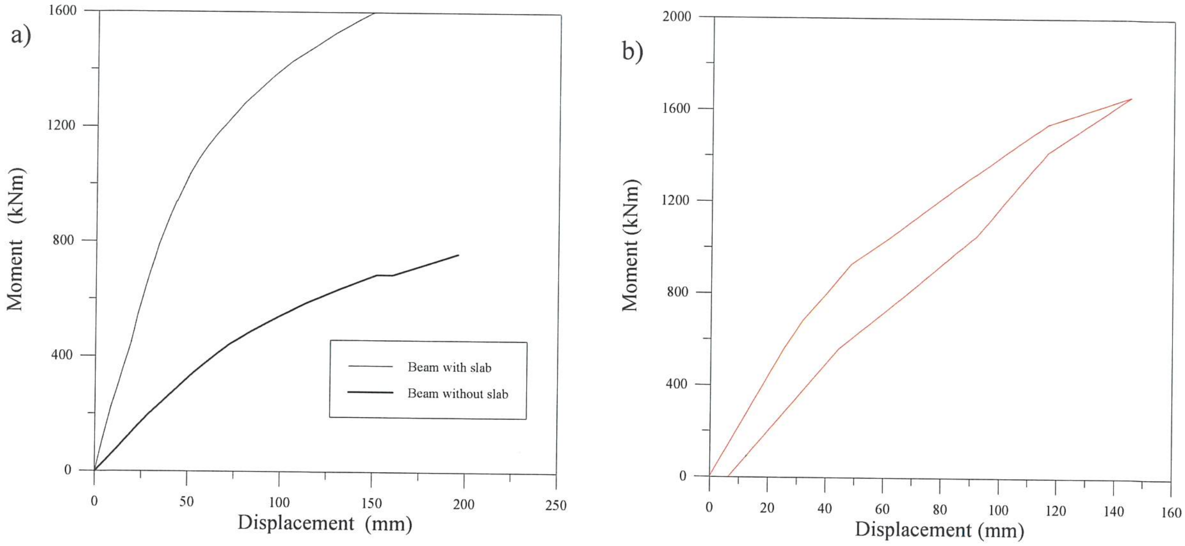


Figure 11. Bending moment - displacement curves

The residual displacements in the middle cross section recorded after the unloading were negligible (Fig.11b). All the cracks, barring one, were closed.

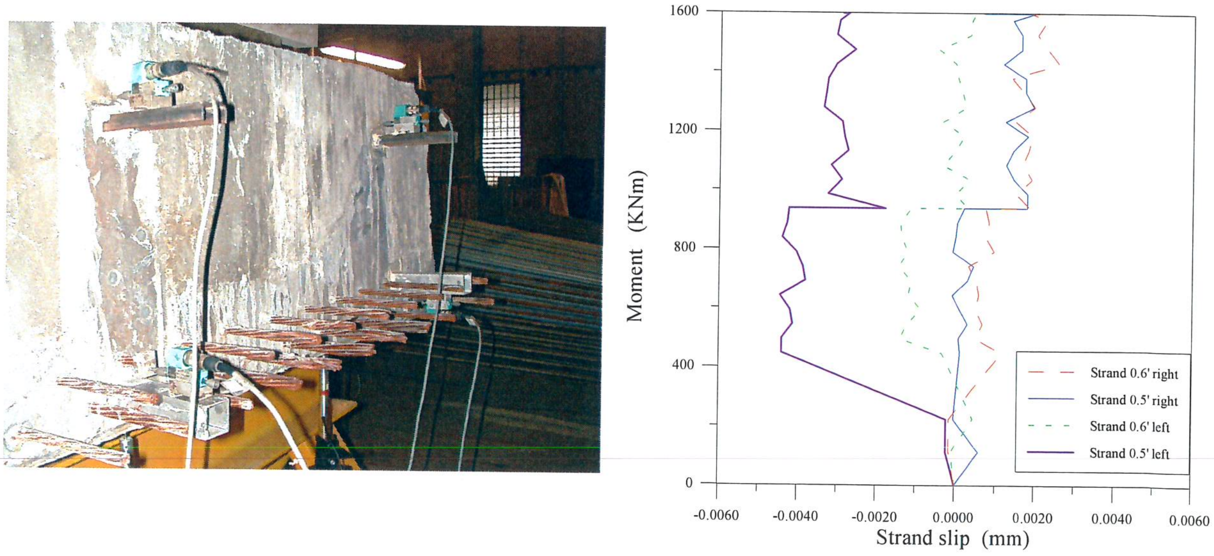


Figure 12. Potentiometers and strands slip as a function of the maximum moment.

The behavior of the specimens also denoted an excellent bond strength testified by negligible slip of the strands monitored with potentiometers (Figs 12). Problems were not observed in the region of application of the prestressing forces where significant tensile splitting stresses arise.

This paper presents tests on reinforced concrete beams and prestressed concrete beams made with a very high performance micro-concrete. The experimental evidence showed that:

Steel microfibers, added to a cement based matrix, improve the flexural behaviour of reinforced beams in terms of strength and stiffness.

The contribution of the tensile region is not negligible with a fiber reinforced material. An extension of the cracked zone was observed in the beams without polystyrene core. The full beams showed an improved cracking around the middle cross section of the tested beams.

The region of application of the prestressing forces where significant tensile splitting stresses arise did not showed localized evidence of inelastic phenomena.

2.2.7 "DIRECT TENSION TESTS ON HPC SPECIMENS", [7].

The mechanical characterization of a given material requires the knowledge of properties such as tensile strength, fracture energy and constitutive response. The *natural way* to determine these parameters is to perform an experiment, but this may produce several problems related to the test and the data interpretation.

For example, it is well recognized that the evaluation of the tensile strength of a quasi-brittle material is an intriguing problem. In addition, direct tensile tests are complicated to setup, with the specimen grips introducing a perturbation in the uniaxial stress field that usually cannot be ignored. Even in a perfect apparatus, slight imperfections in sample preparation and material inhomogeneity can produce nonuniform tensile stresses in the specimen. In any case, the experimental observations on a quasi-brittle structure show a size dependence whatever the geometry of the specimen and the condition of loading.

One objective of a direct tensile test is to obtain a complete stress-strain curve. In this context, two methods have been used to obtain a stable response in the post-peak region: one uses an elastic load-sharing system parallel to the specimen, the other uses a closed-loop control and introduces a notch in the specimen. Gopalaratnam and Shah used specimens with a double edge-notch, and Labuz et al. used specimens with and without a notch.

The influence of kinematic boundaries on the test results was discussed. Two different kinematic conditions have been considered, uniform displacement imposed (fixed platens) and tests where a spherical hinge was introduced between the head of the machine and the specimen (rotating platens). This work demonstrated the fundamental influence of the loading devices on the experimental results. Furthermore it was suggested to measure the roughness of the fracture surface in order to find the real constitutive behavior (softening branch) separated from the structural effects (due to the boundary condition).

A finite element modeling of direct tensile tests was considered by Rots and de Borst. They shown that the structural behavior, after the peak load, involves strongly non-symmetric deformations. As a consequence the descending branch measured in direct tensile tests cannot be considered as a pure «material» property, but as a combination of material and structural behavior.

The purpose of this paper is to show the influence of the boundary conditions (fixed platens and freely rotating platens) on the results of direct tensile tests, to identify the constitutive behaviour and the structural effects.

Some experimental results on cylindrical and prismatic specimens are presented. The tests on prismatic specimens were monitored with an additional interferometric strain measurement (Electronic Speckle Pattern Interferometry) that allows the full strain field to be shown. A model that explain the experimental behaviour is presented. At first the constitutive law for the elastic state and the cracked state are calculated. Then according to Neuber's theory the cracking propagation is modeled to investigate the effects of the boundary conditions on this «structural» phenomenon.

Finally considerations on the two test types are reported.

EXPERIMENTAL RESULTS AND THEIR INTERPRETATION

Direct tensile tests on cylindrical and prismatic notched specimens of high performance concrete were carried out to know the effects of different boundary conditions.

The testing system consisted of a closed-loop electromechanical Instron load frame with a maximum capacity of 100 kN. The feedback signal was the average value of displacements measured using two LVDTs, with a sensitivity of 0.2 μm, set across the notch at a gage length of 50 mm and arranged at 180° around the specimen axis. The loading system was provided with four adjustable dynamometric bars parallel to the two columns of the load frame, with the lower plate of the actuator connected with the upper cross beam. The bars allowed an almost continuous adjustment of the possible rotations of the specimen that can introduce undesired bending stresses. In this way, it was possible to impose a uniform crack opening with a tolerance of 1μm through the critical cross section of the specimen. The relative displacement of the machine heads was measured at four points, by means of four LVDTs.

Two different kinds of tests were considered: tests with fixed platens, where, through the four dynamometric bars an uniform displacement was imposed, and tests with rotating platens, without the bars control with freely platens.

Furthermore the strain field of the prismatic specimens was observed through the ESPI (Electronic speckle pattern interferometry) technique.

Figs 1 show an example of the comparison between the tensile stress-displacement curves for fixed and rotating platens, for cylindrical (a) and prismatic specimens (b).

The uniform displacement imposed (fixed platens) leads to higher loads both for cylindrical and prismatic specimens.

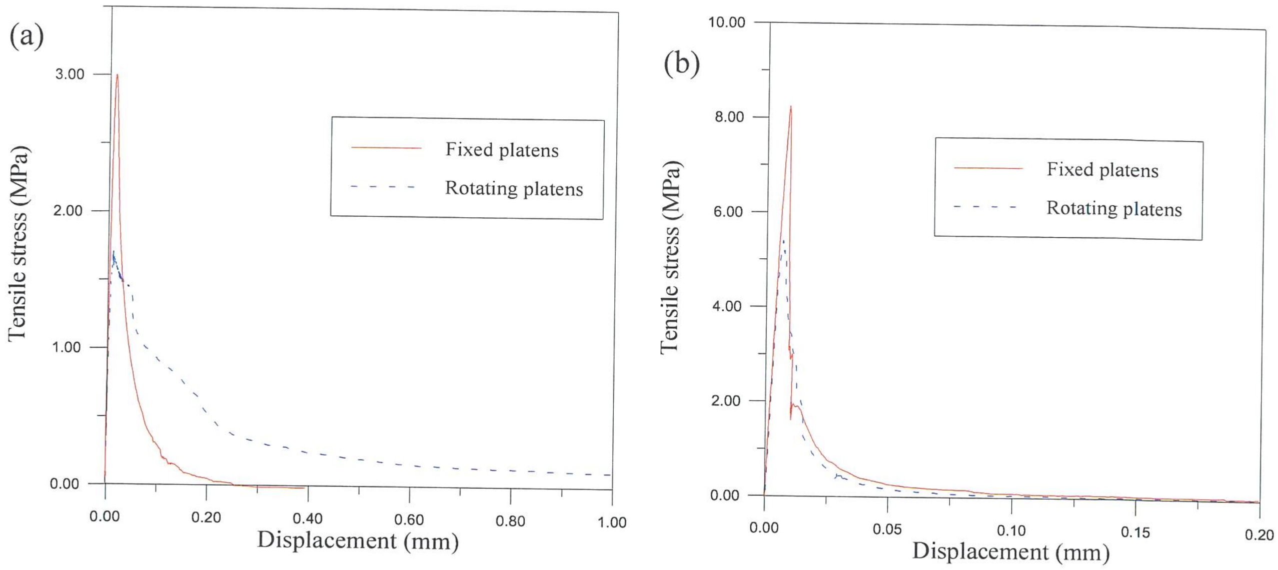


Figure 1 - Tensile stress-displacement curves: (a) cylindrical specimen (b) prismatic specimen.

In the test with fixed platens (Fig.2a) the four measurements of crack opening displacement (COD) were close together, while in the test with rotating platens (Fig.2b) the displacements were not uniform.

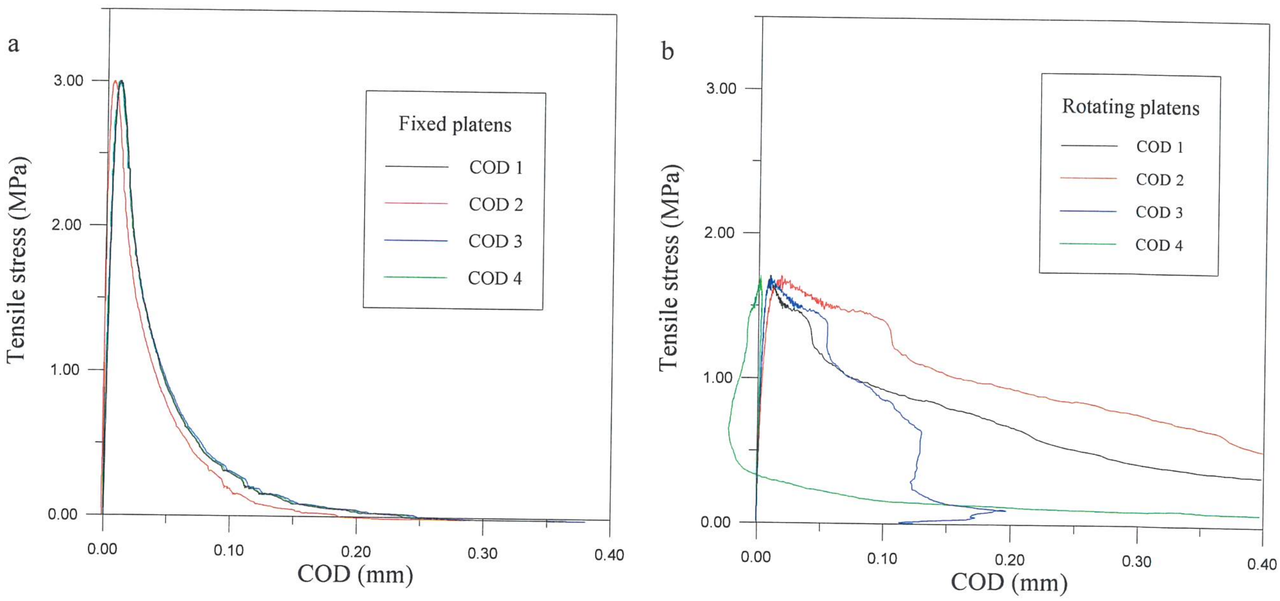


Figure 2 - Tensile stress-COD curves:(a) fixed and (b) rotating platens (cylindrical specimens).

The same behavior was observed by ESPI, where in a fixed platens test the strain field was uniform and there were two cracks that grew together (Fig.3a). Conversely in a rotating platens test the strain field was not uniform, so only one crack was present (Fig.3b).

a

b

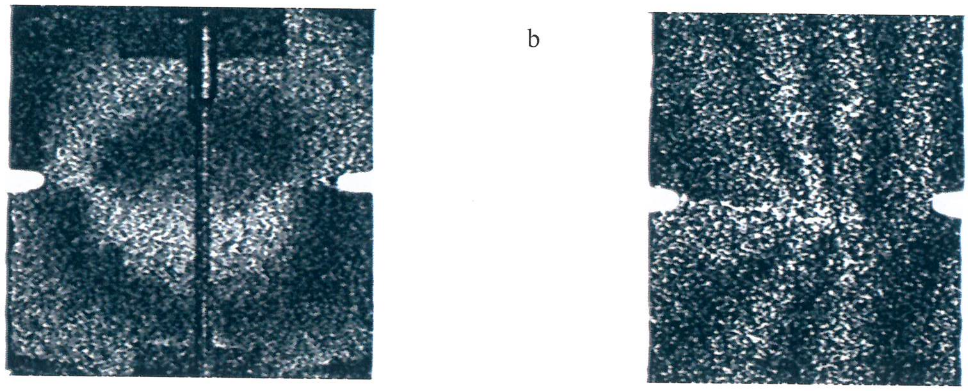


Figure 3 – ESPI image:(a) fixed and (b) rotating platens (prismatic specimens).

To explain the experimental results two important aspects must be considered: the concrete is not an homogeneous material, and undesirable eccentricity can be introduced, when a specimen is glued to the machine.

A simple numerical model can explain the influence of these aspects on the kinematic conditions.

A direct tensile test on a specimen made up by three trusses (1-2-3) with unitary section and unitary barycentric distance (Fig.4) is simulated.

a

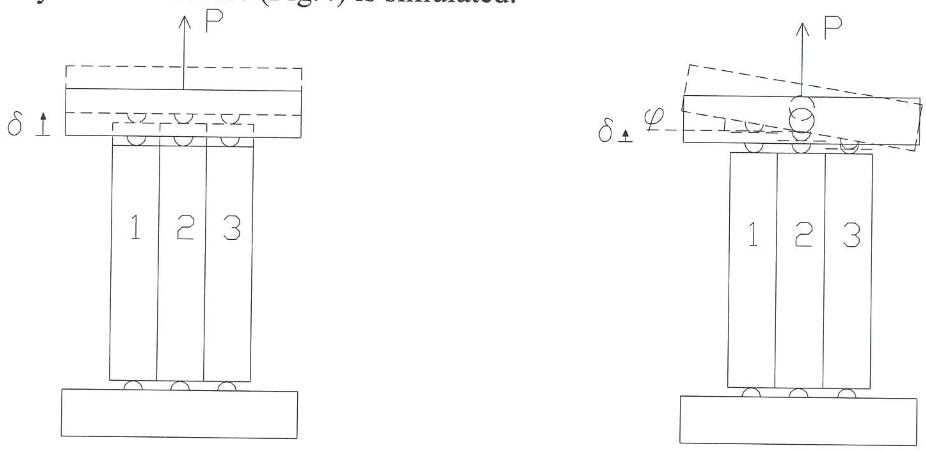


Figure 4 – Scheme of the specimen: (a) fixed platens (b) rotating platens.

The trusses are of elastic-cohesive material, with different constitutive laws (A-B-C), that simulate the non homogeneity of the concrete specimens, and also a geometric eccentricity can be introduced in the model.

There are three independent combinations of constitutive laws (ABC, ACB, BAC), but simulating a direct tensile test with fixed platens the response is always the same. Conversely in a rotating platens test the results depend on the combinations of the constitutive laws (Fig.5).

The uniform displacement imposed (fixed platens) leads to higher loads both for cylindrical and prismatic specimens.

All of this let us understand that the fixed platens tests are objective, not depending on the aleatory defects, disomogeneities and centering errors, realizing a direct tensile stress state while those with rotate platens bring to an unreliable result, dependent on several unpredictable factors and, above all, they do not give the results of a direct tensile test, but of a tensile flexural one.

Fixed/Rotating platens

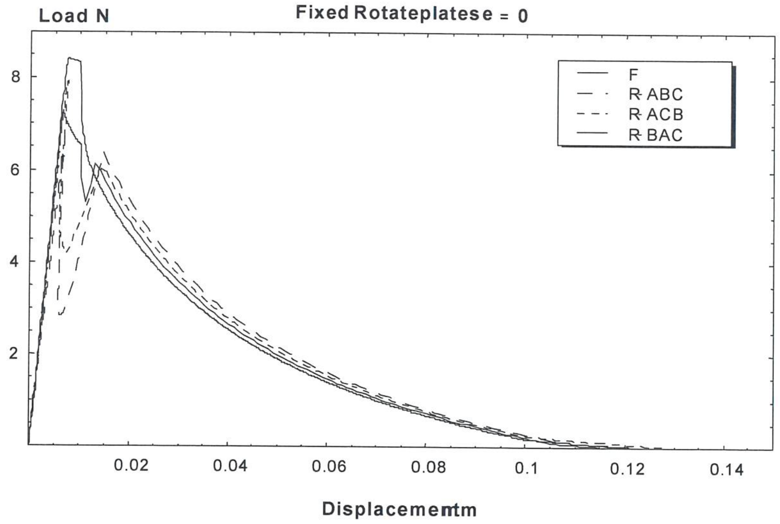


Figure 5 – Comparison of the simulation of a fixed and a rotating platens test

MODELLING

The previous consideration let us understand that to analyze the tensile behavior, in order to know the constitutive law, a test with fixed platens must be considered.

The proposed model is based on both a non-linear elastic analysis for the solid state and a cohesive analysis for the cracked state.

Solid concrete under tension

Referring to a prismatic specimen, with an elliptical notch under tension, a closed-form solution for the stress distribution in the notched section was formulated by Neuber.

In particular, the stress concentration factor of the longitudinal stress σ_{max} at the tip of a notch of arbitrary depth is:

$$\alpha_k' = \sigma_{max} / (P/ba) = 1 + (\alpha_{fk} - 1)(\alpha_{tk} - 1)\left((\alpha_{fk} - 1)^2 + (\alpha_{tk} - 1)^2\right)^{-1/2} \tag{1}$$

where b and a are the width and the ligament length respectively, P the applied load and α_{fk} and α_{tk} are the stress concentration factors for a shallow and a deep notch.

The expressions of these two parameters are as follows:

$$\alpha_{fk} = 3\sqrt{\frac{t}{2\rho}} - 1 + \frac{4}{2 + \sqrt{t/2\rho}} \qquad \alpha_{tk} = \frac{2(a/\rho + 1)\sqrt{a/\rho}}{(a/\rho + 1)\operatorname{arctg}\sqrt{a/\rho} + \sqrt{a/\rho}} \tag{2a,b}$$

where t is the notch depth and ρ is the curvature radius at the tip.

The non-linear elastic analysis of Neuber was based on the following constitutive relation:

$$\sigma = \frac{E_t \varepsilon}{\sqrt{1 + \left(\frac{E_t \varepsilon}{\sigma^*}\right)^2}} \tag{3}$$

where E_t is Young's modulus and σ^* is the asymptotic value of stress corresponding to infinite strain.

Rosati et al. extended these results to concrete-like materials by imposing a local energetic equivalence between the linear tensile stress-strain relation (based on perfectly elastic stress concentration factor α'_k (Eq.1).

As a result, the following expression for the asymptotic value of the stress σ^* was obtained:

$$\sigma^* = \frac{\left(\alpha'_k \sigma_{NM}\right)^2}{2\sqrt{\left(E_t \varepsilon_M\right)^2 - \left(\alpha'_k \sigma_{NM}\right)^2}} \qquad (4)$$

where σ_{NM} and ε_M are the stress and the strain at the notch tip corresponding to the onset of crack.

The value of the nominal strength f_{ct} can be obtained from eq.3 with ε equal to the maximum strain ε_M:

$$\sigma = \frac{E_t \varepsilon_M}{\sqrt{1 + \left(\frac{E_t \varepsilon_M}{\sigma^*}\right)^2}} \qquad (5)$$

The value of ε_M was obtained from ESPI, and at the corresponding time from the test data file the value of σ_{NM} was calculated.

Cracked concrete

The analysis of the descending branch can be based on the identification of a point, where the load-elongation curve changes its curvature. From this point on, a complete extension of a crack trough the whole cross-section can be assumed. As a consequence the crack opening distribution is linear over the fractured section.

In this purely cohesive phase, a hyperbolic stress σ_w-crack opening w relation is adopted:

$$\sigma_w / f_{ct} = \left(\phi\left(w_c - w\right)\right) / \left(w_c\left(\phi + kw\right)\right) \quad \text{for } 0 \le w \le w_c \qquad (6a)$$

$$\sigma_w / f_{ct} = 0 \quad \text{for } w \ge w_c \qquad (6b)$$

where σ_w are the stresses transferred through the crack surfaces, w_c is the critical width beyond which no cohesive stress σ_w is transferred, k is a coefficient related to the slope of the hyperbolic curve and ϕ is the maximum aggregate size.

Transition solid-cracked concrete

According to the previous considerations, the response of a double notched concrete specimen is governed by both an elastic and a cohesive contribution, produced by the stress transfer through the cracked surface (Fig.7). In this phase the kinematic conditions have an important role. In fact an uniform displacement leads to a tensile stress, while if the displacement is not uniform a bending moment is present.

At first, let us consider the case with uniform displacement imposed (fixed platens).

During crack propagation the cross section of the specimen can be considered subject both to a force due to the non-linear elastic behavior of the ligament, and to a force N_w which is the resultant of the cohesive stresses keeping the cracks closed. Thanks to this scheme the notch is no longer deep t_0, but t_0+t, where t is the depth of the crack at the analyzed loading step. In this way it is possible to consider the cracks as a pointed notch (ρ=0), with a technical

concentration factor α_{ST} according to Neuber's theory. Taking into account a material dependent radial dimension ρ' of the plastic region in front of the notch tip, the technical factor is related to the effective non-linear material behavior.
The expression of the technical concentration factor is:

$$\alpha_{ST} = 1 + 2\sqrt{a/\rho'}/\sqrt{a/t + \pi^2/4}\,. \tag{7}$$

The value of the dimension ρ' can be obtained from the equivalence between the tensile strength f_{ct}, coming from the non linear elastic analysis, and α_{ST} times σ_{NM}.

Assuming that the crack opening displacement w is linearly distributed along the crack and referring to the local coordinate x with origin in the middle of the cross section, its distribution through the crack length is:

$$w = w_m\,(x - h + t + t_0)/t \tag{8}$$

where w_m is the maximum crack width or c.o.d. at the distance t (crack depth) from the tip. Furthermore the relation between maximum crack width w_m and the crack depth t is:

$$t = (a - t_0)(1 + k_t)w_m/(w_c + k_t w_m) \tag{9}$$

where k_t is a parameter connected to the material.
The equilibrium condition leads to:

$$P = f_{ct}/\alpha_{ST}2(h-t-t_0)b + 2N_w \tag{10}$$

where:

$$N_w = \int_{h-t-t_0}^{h-t_0} b\sigma_w(x)dx\,. \tag{11}$$

According to this scheme let us consider the case of rotating platens.

The procedure is based on the hypothesis that there is a dominant defect on one side of the specimen. Furthermore the other defects do not influence the dominant defect. According to these hypothesis and to the experimental results only a crack is present. This condition leads to a presence of both a tensile and bending stress. The Neuber's theory suggests the technical concentration factor for a specimen with an external notch on one side both in tension α_{ST} and in bending α_{SM}.

To compare these results with those obtained with fixed platens, the same cross section areas for the two test manner were taken (Fig.8).

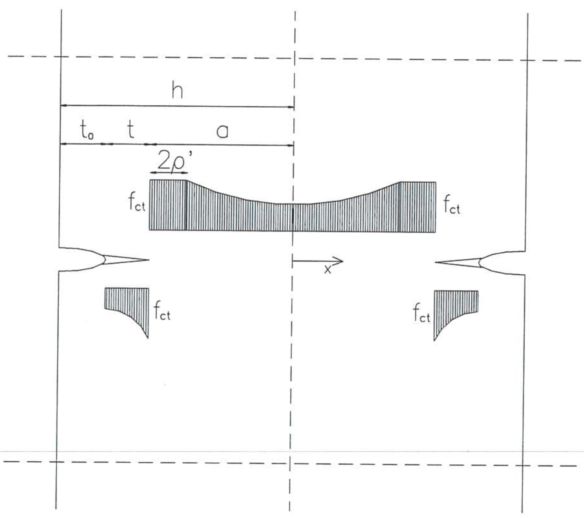


Figure 7 – Theoretical stress distribution for fixed platens.

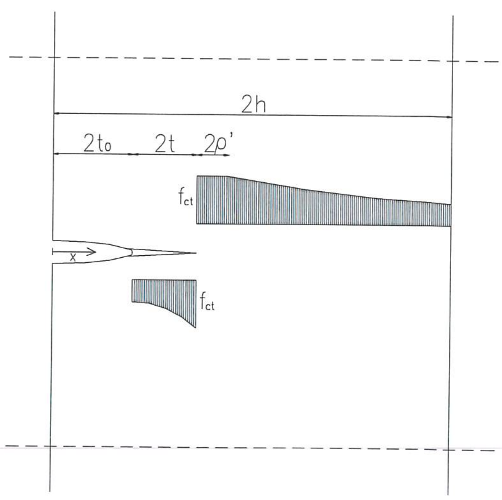


Figure 8 – Theoretical stress distribution for rotating platens.

Like in fixed platens, it is assumed that the crack opening displacement w is linearly distributed along the crack, and referring to the local coordinate x with origin at the boundary of the specimen, its distribution through the crack length is:

$$w = w_m\,(-x + 2(t + t_0))/2t. \tag{13}$$

Furthermore the same relation of fixed platens between maximum crack width w_m and the crack depth t is adopted.

The equilibrium condition leads to:

$$P = f_{ct}\,2b(-h+t+t_0)^2/(h\alpha_{ST} + (6\alpha_{SM}+\alpha_{ST})(t+t_0)+ N_w\,(h(6\alpha_{SM}+\alpha_{ST})- \alpha_{ST}\,(t+t_0)-3\,\alpha_{SM}\,(t+2t_0)/ /(h\alpha_{ST} + (6\alpha_{SM}+\alpha_{ST})(t+t_0) \tag{14}$$

where:

$$N_w = \int_{2t_0}^{2(t+t_0)} b\sigma_w(x)dx\,. \tag{15}$$

Comparison fixed – rotating platens

Fig. 9 shows a comparison for theoretical and experimental results for fixed and rotating platens, in the phase of crack propagation (from t = 0 to t = h).

Both results show that a fixed platens test leads to higher peak load value than a rotating platens test. The theoretical and experimental results of rotating platens are in good agreement, while the results of fixed platens are comparable till the peak stress, then there is a difference due to the loose of the bar control during the test.

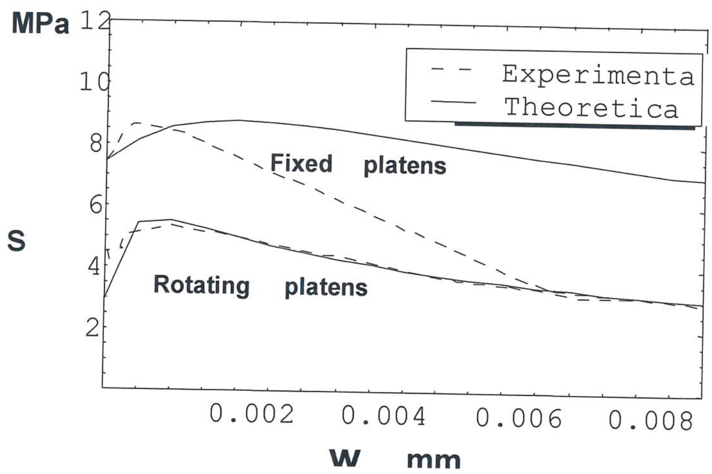


Figure 9 – Comparison experimental/theoretical results during the crack grow to the whole section for fixed and rotating platens.

The experiments have shown that the test procedure, imposing uniform displacement or leaving free rotation to the machine platens has a substantial influence on the results.

This is due to two main reasons: difficulties in centering the specimen and defects within the material, which produce a perturbation in the uniform stress field. The higher values in the peak stress (strength) were obtained with fixed platens.

In this kind of test a more regular development of cracking in the critical cross section has been observed: the strain measurements were more uniform, denoting a reduction in the stress concentration and the fracture energy dissipated was lower.

In particular the full field strain measurement with ESPI has shown that during the test there is strain localization on a side of the specimen, that clearly started before peak load. A more uniform strain field was possible with fixed platens. In this sense, this loading system appeared «more correct» for material characterization. Imposing symmetric displacement to the specimen, the parts of the section run their local constitutive laws. Their average provides the global response of the cross section, without structural effects.

The model proposed can describe the kinematic growth of crack and its influence on the stress distribution.

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ANNEX 2: SOMMARIO

Il presente lavoro costituisce un breve report dei lavori del 5th INTERNATIONAL SYMPOSIUM ON UTILIZATION OF HIGH STRENGTH/HIGH PERFORMANCE CONCRETE

1999, che si è svolto a Sandefjord (Norvegia) nelle giornate del 20-24 giugno 1999. Al simposio hanno partecipato circa 220 persone, in rappresentanza di 38 Paesi, in particolare, soprattutto provenienti da Norvegia, Giappone, Svezia, U.S.A. ed anche Italia. Tale avvenimento è stato una ottima occasione per fare il punto aggiornato sui calcestruzzi ad alte prestazioni divenuti ormai materiali quasi di corrente uso in certi Stati, quali ad esempio gli U.S.A., il Canada, la Francia ed i Paesi scandinavi (Norvegia, Danimarca). Il contributo dell'Italia è stato notevole, con un totale di 11 lavori. In particolare, il gruppo di ricerca nato dalla collaborazione del CTG - Italcementi Group e del Dipartimento di Ingegneria Strutturale (DIS) - Politecnico di Milano, con ben 6 lavori (di cui 5 lavori del CTG presentati oralmente) ha contribuito in modo attivo al simposio, presentando lavori riguardanti aspetti diversi dei calcestruzzi ad altissime prestazioni, dai materiali alla loro caratterizzazione meccanica al loro utilizzo strutturale. Nel presente lavoro, dopo un breve riassunto del Simposio, si fornisce la presentazione dei principali punti trattati nelle memorie del gruppo di lavoro DIS – CTG.