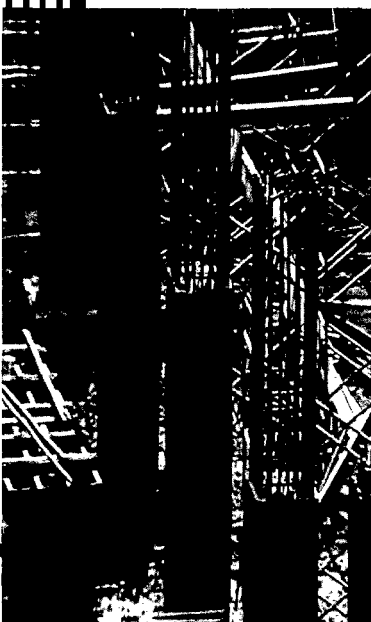


High-Performance Concrete

Performance and Quality of Concrete Structures



*Proceedings
Third International Conference
Recife, PE, Brazil, 2002*

Editors:

V. M. Malhotra

Paulo Helene

Enio P. Figueirdo

Armando Carneiro



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Front cover photo:

Name of Building: e-Tower (under construction)

Structural Designer: Ricardo França

Concrete Designer and Concrete Construction Consultant: Paulo Helene

Contractor: Tecnum



Documents

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High-Performance Concrete *Performance and Quality of Concrete Structures*

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PREFACE

The first conference on the subject of High-Performance Concrete (HPC), and Performance and Quality of Concrete Structures was held in Florianopolis, Brazil in 1996 as a result of the initiative of Prof. Luiz Roberto Prudêncio and Prof. Paulo Helene of Brazil, supported by Dr. Mohan Malhotra and Prof. Kumar Metha from Canada and the U.S.A., respectively. The second conference on this subject was held in Gramado, Brazil in 1999, organized by Prof. Denise dal Molin, with more than 350 participants worldwide. The success of these International Conferences is recognized by the ABCP-Brazilian Cement Industry Association (IBRACON), Brazilian Concrete Institute, and the academic and professional communities. A significant number of the participants in the HPC conferences were young researchers. Conferences are really contributing to the best engineering and holistic vision of Brazil's concrete professionals for tomorrow.

Brazil has a huge experience in concrete structures design and construction since the beginning of the 1900s. In spite of this experience of more than 90 years, there have been few opportunities to show and to share this experience with ACI and other important worldwide organizations.

The Guinle Building, a 7-floor concrete office tower (28-m high) was inaugurated in 1916 in São Paulo, and is still functional. In 1929, the Martinelli concrete tower (106-m high), reached the world record in high-rise buildings, and is still in good condition. Also, in Rio de Janeiro, the Night Building was one of the highest concrete buildings in the world for many years.

In addition, many other concrete structures, like Itaipu Dam (largest in the world) and 13-km large Rio-Niteroi Bridge, are important concrete structures. Since the 1980s, new concrete structures using high-performance concrete have been built. In 1997, the highest concrete building in Brazil, 180-m high United Nations Towers, used 30,000 m³ of high-strength concrete (HSC) with $f'_c = 50$ MPa. This year the e-Tower Building with HSC $f'_c = 125$ MPa in columns is a new record in the world.

To make sure of the safety of the above developments, major research projects were undertaken in the Brazilian universities, and important exchanges of information with well-known researchers and institutions in the world was necessary. The Third Conference demonstrates the significance of these exchanges. More than 90 papers were received and 30 were accepted by the ACI review panel for publication in the ACI Special Publication, SP-207, as the proceedings of this conference. The Organizing Committee decided to publish many of those submissions that could not be included in the Special Publication as supplementary papers. In addition to the papers that have been published in the proceedings and supplementary volume, a number of other papers were also presented in the Conference.



The refereed proceedings and the volume containing supplementary papers contain interesting papers about HPC and HSC. Also, there are papers describing experiences in the use of recycled materials to obtain HPC, HPC for pavements, HPC with fibers, comparisons of different methods for proportioning mixtures, durability properties of HPC, and others showing that there are still many questions that remain to be answered.

We appreciate the distinguished members of the ACI review panel who reviewed the draft manuscripts in 2001 in Iguassu Falls during the 2001 IBRACON Conference. Without their prompt review and constructive comments, it would not have been possible to publish the proceedings for distribution at the conference. The cooperation of the authors in accepting reviewers' suggestions, and revising their manuscripts accordingly, is greatly appreciated.

The help of the ACI staff for performing the administrative duties associated with this publication is greatly acknowledged.

Editors

V. Mohan Malhotra

Paulo Helene

Armando Cameiro

Enio P. Figueiredo

October 2002

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Behavior of High-Performance Concrete Subjected to Biaxial Tension-Compression Stresses

by J. M. Calixto

Synopsis: The results of an experimental investigation on the behavior of high-performance concrete subjected to biaxial tension-compression stresses are presented. Short-term static tests were performed on 125 mm square by 12.5 mm thick plates. Strain controlled tests were executed in a biaxial testing machine constructed at the University of Texas. The primary studied variables were the discontinuity and the ultimate stress levels at each stress ratio. Results indicated that even small amounts of tensile stress reduced the ultimate compressive strength of the specimens substantially. The failure mode of the plate specimens fell basically into one category: tensile splitting in a plane or planes perpendicular to the direction of the principal tensile strain. The failure surface contained both fractures through coarse aggregate and mortar. These results suggest that the failure criteria for high-performance concrete, under biaxial tension-compression, is a limiting value for the tensile strain. The magnitude of the failure tensile strains is not constant, but increases with the degree of compression.

Keywords: behavior; biaxial tension-compression stresses;
high-performance concrete; strength

José M. Calixto is an associate professor in the Structural Engineering Department at the Federal University of Minas Gerais, Belo Horizonte, Brazil. His research interests are in concrete materials and concrete structures.

INTRODUCTION

Concrete in normal conditions is a versatile, strong and durable construction material. However under several physical and chemical processes as well as certain environmental conditions it may deteriorate in a short period of time. This fact has led researchers in the last 30 years to develop the high-performance concrete. High-performance concrete (HPC) is a concrete which possess high workability, high strength and low permeability.

The production of a high-performance concrete can be achieved basically with a better selection of component materials, adequate mixture proportion, careful placement and proper curing. The required workability is normally attained with the use of superplasticizers. Since the manufacture of high-performance concrete does not involve the use of exotic materials and complicated procedures, its production is within the reach of most concrete producers.

In most structures, concrete is often subjected to biaxial states of stress, and the behavior of the material under these types of actions must be well understood. It is, therefore, not surprising that numerous investigations into the behavior and strength of conventional concrete under biaxial stress states have been conducted in the past 40 years (1,2,3). On the other hand, HPC differs from normal strength concrete (NSC) in several aspects and these differences are not yet totally understood. Thus more studies are necessary to better understand the behavior of this material and to facilitate the design and construction of more structures with HPC.

The aim of this paper is to gain further understanding on the behavior and failure mechanism of HPC when subjected to biaxial stresses. Short-term static tests were performed on 125 mm square by 12.5 mm thick plates subjected to biaxial tension-compression stresses at selected stress ratios (4). The strain controlled tests were executed in a biaxial testing machine constructed at the University of Texas. The primary variables studied were the discontinuity and the ultimate stress levels at each stress ratio. A comparative study with normal strength concrete is also presented.

MATERIALS AND EXPERIMENTAL PROCEDURES

Materials

The proportioning of the components for HPC mixtures is more critical than for normal concrete since for HPC optimum performance is required from each component used. This means that quality materials in the mixture proportion and good quality control in the field are necessary. For the current investigation, the materials used included ASTM Type I cement (Brazilian Type CP I), crushed limestone, with a maximum size of 1.0 cm, as coarse aggregate and a river sand as fine aggregate. For the necessary slump the superplasticizer employed was Pozzolith 400-N manufactured by Master Builders of Cleveland, Ohio. The mixture proportion consisted of 575 kg of cement per cubic meter of concrete, a W/C (by mass) of 0.28, an aggregate/cement ratio of 2.88 and a coarse-to-fine aggregate ratio of 1.72. The superplasticizer/cement ratio (by mass) was 0.013.

A total of nine 100 mm x 200 mm cylinders and six 150 mm x 150 mm x 500 mm beams were cast using steel molds. A pencil vibrator was used for compaction of the concrete into the molds. The newly cast molds were covered with a wet burlap for 24 hours. The cylinders and beams were then removed from the molds and placed in a curing room at a temperature of 22 ± 2 °C and 95 to 100 percent relative humidity.

Specimen Preparation

The preparation of the plate specimens included the cutting of the 125 mm square by 12.5 mm thick plates from the beams. Cutting of the beams was initiated after at least 56 days of casting and was performed using a slow-feeding diamond blade lapidary saw. For the 12.5 mm dimension, the plates were sliced slightly larger than needed and ground down employing a vibrating lapidary table. After grinding, the specimens were washed with soap and water, rinsed and again stored in the curing room.

At least three days prior to testing, each specimen was removed from the curing room for preparation of the loading edges. The edges were sand-blasted with fine silica sand to roughen the surfaces, facilitating a good bond between the edges of the specimen and the epoxy adhesive.

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For the transmission of the tensile forces, the edges of the specimen must be securely attached to the comb-platens and the binding material used must develop a sufficient strong bond with both the concrete and the aluminum to withstand the maximum tensile force expected. The exposed edges of the specimen were then coated with a thin layer of Adhesive Engineering Company 1411 Non Sag Paste Adhesive. The mating surfaces of the comb-platens and the specimen were then pressed together. The paste adhesive was allowed to cure at room temperature for at least 36 hours prior to testing.

Loading Apparatus and Testing Procedure

The loading apparatus used was a special biaxial testing machine built at The University of Texas. It consists of steel framing members, hydraulic rams and load cells as shown schematically in **Fig. 1**. The load was applied by hydraulic rams located in the vertical and horizontal directions. Each ram was controlled by an electronic hydraulic servo system. In order to prevent any significant confining stress, special steel brush-bearing platens were used for compressive loads (**Fig. 2**). They were of a design similar to those employed by Hilsdorf (5). For the tensile loads, a different set of platens (**Fig. 3**), made of a single aluminum plate and designed by Briggs (6), was used.

Instrumentation was necessary to measure and record loads and strains. Load cells were employed for measuring the loads, while strain gages were used for evaluation of the deformations. Two strain gages were glued on each face of the plate specimen parallel to the directions of the principal stresses. The rate of loading was conditioned to match the strain gage feedback signal by the electronic hydraulic servo system.

Three specimens were tested for each principal stress ratio. The principal stress-ratios used were $\sigma_2/\sigma_1 = 0$ (uniaxial compression); $\sigma_2/\sigma_1 = -0.10$, -0.5 and -1.0 for biaxial tension-compression; and $\sigma_2/\sigma_1 = \infty$ for uniaxial tension.

TEST RESULTS AND ANALYSIS

Concrete strength was evaluated using 100 mm x 200 mm cylinders tested at ages of 8, 56 and 172 days. The 56-day strength was of primary interest since the preparation of the plate specimens took place after the age of

56 days. The 172-day test was performed to compare the cylinder compressive strength with the uniaxial compressive strength of the plate specimens. The average cylinder compressive strengths were 52.7, 62.9 and 74.4 MPa at the ages of 8, 56 and 172 days respectively.

Typical stress-strain curves at the different principal stress ratio tested are shown in **Fig. 4**. It is interesting to note that only for the case of uniaxial compression, the stress-strain curve deviates significantly from linearity at high levels of straining. For all other cases tested, the stress-strain curves are almost linear up to failure.

The value of the modulus of elasticity for specimens tested in uniaxial tension ($E_t = 46755$ MPa) was only 3% greater than the average modulus of elasticity obtained in uniaxial compression ($E_c = 45279$ MPa). These values are larger than expected specially when compared to relation for determination of the modulus of elasticity proposed by ACI 363 (8). The value of Poisson's ratio was slightly higher in uniaxial compression ($\nu_{21} = 0,20$) than in uniaxial tension ($\nu_{12} = 0,19$). Similar differences were obtained by previous investigators (3, 7) for normal strength concrete.

The average stress and corresponding strain at failure for each principal stress ratio are presented in Table 1. The average uniaxial compressive strength determined using the plate specimens was 59.6 MPa, while the average cylinder compressive strength, tested at the same age, was 74.4 MPa. The difference between the two values can mainly be attributed to the effect of confinement, which practically does not exist in the case of the plate specimen.

The analysis of the results of Table 1 indicates also that the ultimate compressive strength of HPC under biaxial tension-compression is significantly less than the uniaxial compressive strength. Under uniaxial loading, the ultimate tensile strength of the HPC was 7 percent of the corresponding compressive strength. This relation is in good agreement with the equation proposed by ACI 363 (8) for tensile splitting strength if the plate compressive strength is employed in the equation.

With respect to the deformations, the results of Table 1 show that, in uniaxial compression, there is a significant increase in the tensile strains at failure in relation to the tensile strains at failure of all other ratios tested. As observed for normal strength concrete (3), high-performance concrete strength

can also sustain a larger amount of indirect tensile strain, due to Poisson's effect, than direct tensile strains.

The biaxial tension-compression strength envelope is shown in **Fig. 5**. The strength envelope for normal strength concrete, obtained by Tasuji et al. (3), is also shown. The data presented in the plot are normalized with respect to the plate uniaxial compressive strength. The shape of the strength envelope for biaxial tension-compression of HPC is much more linear than that obtained for normal strength concrete under similar loading conditions. The comparison also indicates that the biaxial tension-compression strength is different for different strength concretes. Under biaxial tension-compression, the decrease in the compressive strength is lower for concretes with a normal uniaxial compressive strength (NSC), and is higher for concretes having a higher uniaxial compressive strength (HPC).

This difference in behavior has important implications on the design of HPC structures. An example would be in the shear design of reinforced concrete flexural elements. Since biaxial tension-compression stresses exist in the shear region of these elements, the stresses corresponding to the formation of the diagonal cracks should be more carefully determined when HPC is used.

The failure mode of all plate specimen tested fell into one category, tensile splitting in a plane or planes perpendicular to the direction of the principal tensile strain. The failure surface contained fractures through both mortar and coarse aggregate. These results suggest that the failure criterion for high-performance concrete under biaxial stresses is a limiting value for the tensile strain.

In the present study, the behavior of HPC at discontinuity was considered in addition to that at failure. According to Newman (9), discontinuity in NSC corresponds to the stage at which major microcracking of concrete is initiated. Carino and Slate (7) have indicated that the use of discontinuity as the basis for failure of NSC is more appropriate than the ultimate strength since it represents the end of quasi-elastic behavior and the beginning of extensive mortar cracking. Moreover, it has been shown by Rüsçh (10) that the long term strength of NSC in compression corresponds to the stage at which extensive mortar cracking develops. Beyond discontinuity, NSC can no longer be considered a continuous material since any sustained load is expected to cause an eventual failure. This was the primary motivation for obtaining data at discontinuity.

Many procedures have been used to calculate the discontinuity point. In this study two procedures were employed. In uniaxial compression, discontinuity was defined as the point at which Poisson's ratio began to increase significantly. This method has been extensively employed by previous investigators (3,7) for NSC. In tests involving direct tension loading, discontinuity was defined as the point at which the stress versus the maximum tensile strain curve began to deviate significantly from a straight line. A 2% deviation was taken as a significant departure from linearity. Table 2 presents the average stresses and strains at discontinuity as well as the mean normal stresses for the various stress-ratios tested.

It can be seen from Table 2 that the principal tensile strain (ϵ_2) at discontinuity depends upon the state of stress, increasing with the degree of the compressive stress. A plot relating this principal tensile strain at discontinuity to the mean normal stress is presented in **Fig. 6**. A careful examination of the figure indicates that the data can be represented by a straight line whose corresponding equation is also shown. The correlation coefficient for the data used in obtaining the equation shown was 0.99. This finding suggests that there exists for HPC a linear relationship between the principal tensile strain and the mean normal stress at discontinuity for biaxial tension-compression stresses.

The principal tensile strain versus the mean normal stress for normal strength concrete (NSC), obtained by Tasuji et al. (3), is also presented in **Fig. 6**. A linear relationship also exists for NSC and the corresponding equation is indicated in the diagram. A comparison between the equations shows that the slope for NSC is 1.5 times the value corresponded to HPC. This fact implies that, under biaxial tension-compression, NSC can deform much more than HPC, confirming once more the more brittle behavior of HPC.

As for normal strength concrete, it is evident from the above results that a constant tensile strain failure criterion cannot fully describe the observed HPC behavior in biaxial tension-compression. A HPC failure criteria should take into account the fact that the limiting value of the tensile strain is dependent upon the applied state of stress.

CONCLUDING REMARKS

The results of an experimental investigation on the behavior of high-performance concrete under biaxial tension-compression stresses have been presented. Based on the analysis of the data, the following remarks can be drawn:

- ① The introduction of a principal tensile stress reduces HPC ultimate compressive strength under biaxial tension-compression stresses in relation to the uniaxial compressive strength.
- ② High-performance concrete, similar to normal strength concrete, can sustain higher indirect tensile strains, due to Poisson's effect, than direct tensile strains.
- ③ The failure mode of high-performance concrete plate specimen tested consisted of tensile splitting in a plane or planes perpendicular to the direction of the principal tensile strain.
- ④ The principal tensile strains at discontinuity were linear function of the mean normal stresses. This supports the idea that HPC failure criteria, under biaxial tension-compression, is a limiting value for the tensile strain. The magnitude of the failure tensile strains is not constant, but increases with the degree of compression.

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Table 1 – Average Stresses and Strains at Failure

Stress Ratio	σ_1 (MPa)	σ_2 (MPa)	ϵ_1 ($\times 10^{-6}$)	ϵ_2 ($\times 10^{-6}$)
Uniaxial Compression	59.58	0	1500	- 520
- 0,10	22.19	-2.22	480	- 152
- 0,50	6.85	- 3.43	135	- 95
- 1,0	3.81	- 3.81	63	- 95
Uniaxial Tension	0	- 4.20	18	- 98

Table 2 – Average Stresses and Strains at Discontinuity

Stress Ratio	σ_1 (MPa)	σ_2 (MPa)	ϵ_1 ($\times 10^{-6}$)	ϵ_2 ($\times 10^{-6}$)	σ_1 / f'_0	σ_m
Uniaxial Compr.	46.78	0	1050	- 315	0.79	15.59
- 0,10	14.74	- 1.47	308	- 108	0.25	4.42
- 0,50	4.81	- 2.41	113	- 68	0.08	0.80
- 1,0	2.17	- 2.17	50	- 50	0.04	0
Uniaxial Tension	0	- 2.06	9	- 45	0	-0.69

f'_0 – uniaxial compressive strength of the plate specimen = 59.58 MPa

σ_m – mean normal stress = $(\sigma_1 + \sigma_2) / 3$

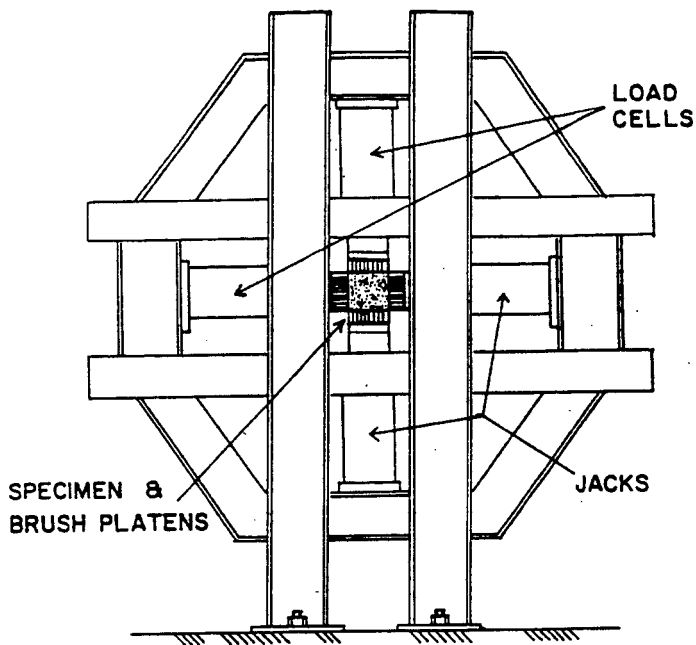


Figure 1 – Biaxial Loading Frame

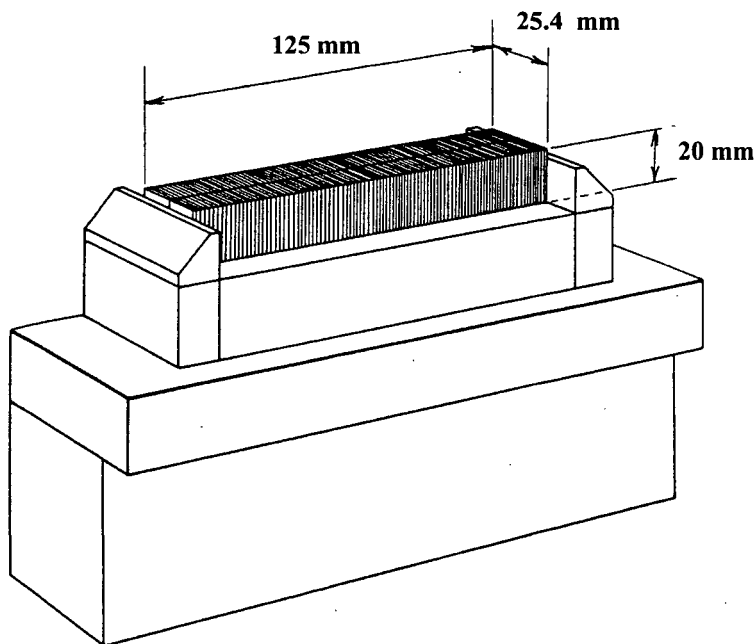


Figure 2 – Compression “Brush-Bearing” Platen

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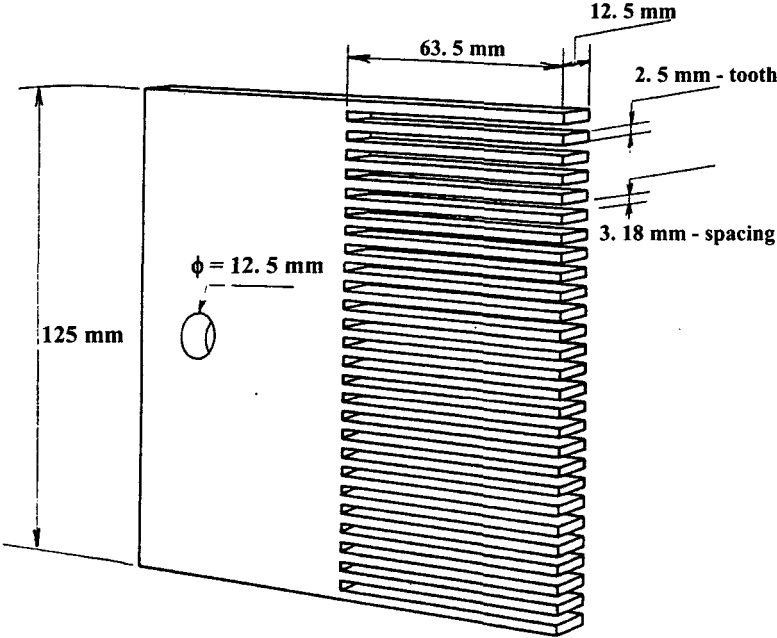


Figure 3 – Tension “Comb” Platen

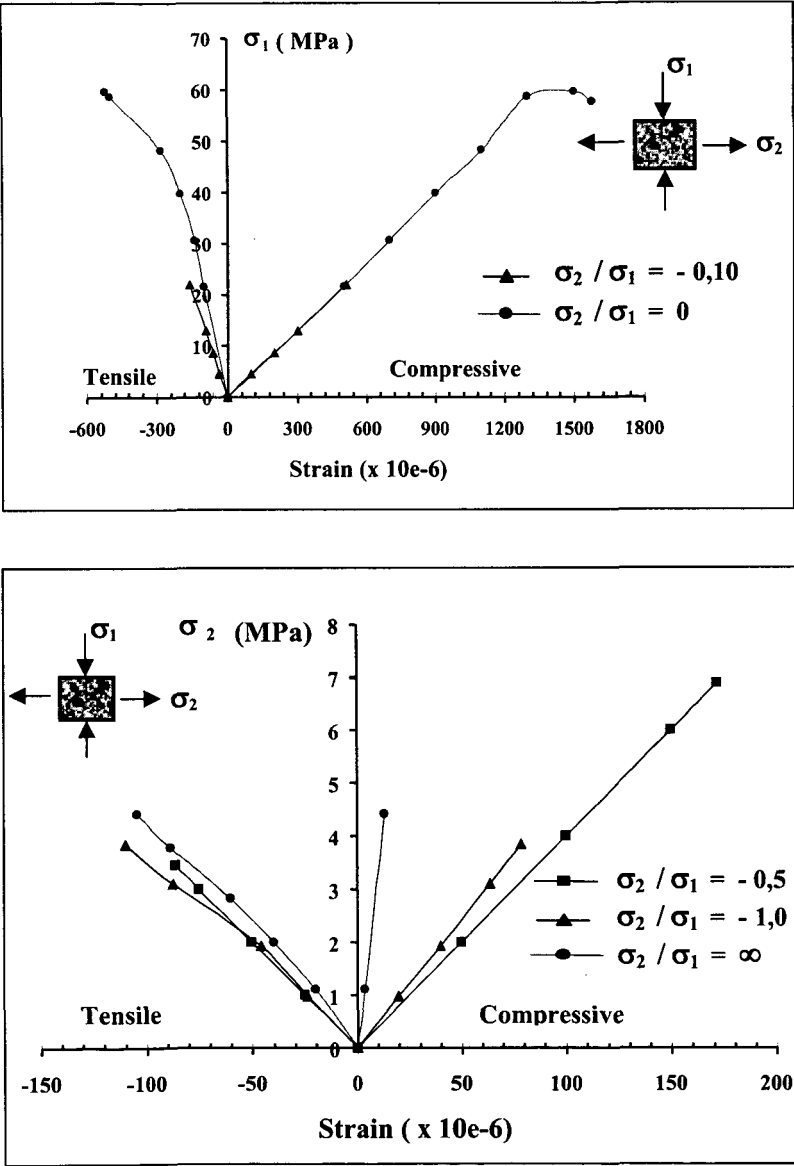


Figure 4 – Stress-Strain Relationships

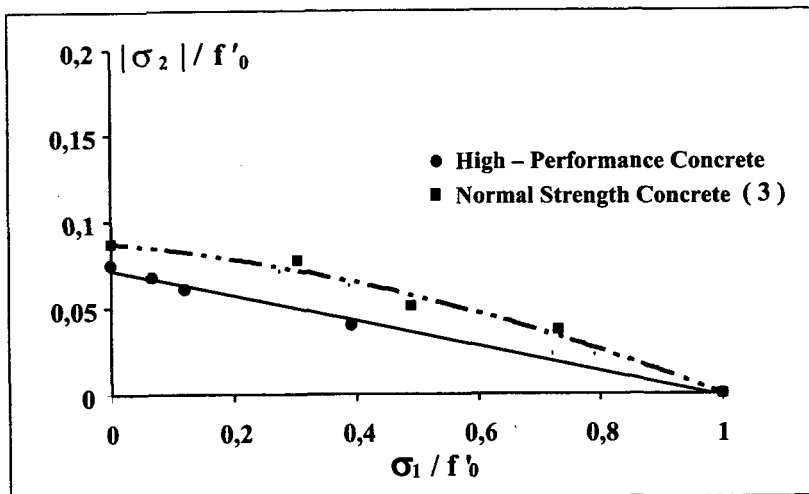


Figure 5 – Strength Envelope of Concrete under Biaxial Tension-Compression Stresses

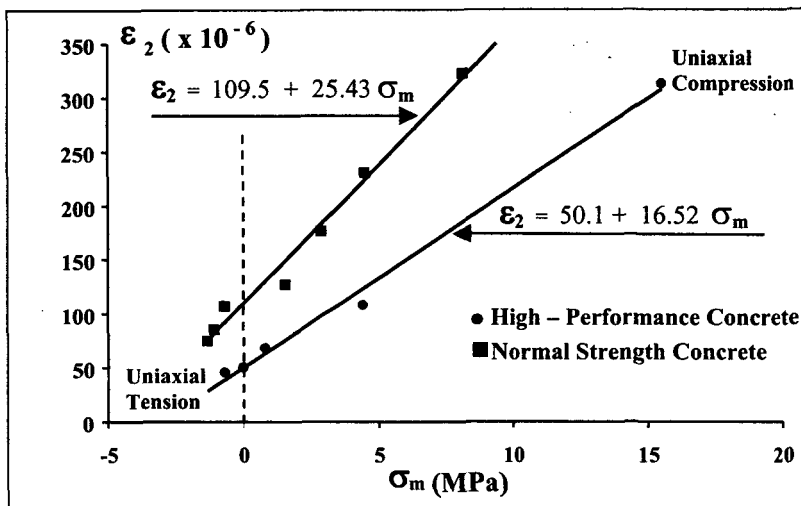


Figure 6 – Principal Tensile Strain versus Mean Normal Stress Relationship at Discontinuity

High-Performance Concrete Using Fly Ash

by A. Camões, P. Rocha, S. Jalali, B. de Aguiar, and R. Delgado

Synopsis: Some of the most recent developments related to the production of concrete have focused on the addition of components which can improve the mechanical, workability and durability properties of concrete and whenever possible, to solve environmental problems in a simple and economical way. This is research work fits in this field, trying to contribute to the clearing up of the advantages and disadvantages of concrete production with the addition of fly ash (FA).

High-performance concrete (HPC) is usually produced using high quality materials. These constituents drastically increase the initial cost of HPC, thus hindering its more widespread usage. This research work intends to investigate the possibility of producing low cost enhanced performance concrete or even low cost HPC, with 90 day strengths in the range of up to 60 MPa, using low quality fly ash and locally available crushed aggregates.

The effect of the amount of fly ash was evaluated using 0, 20%, 40% and 60% cement replacement with different quantities of total binder of 400 kg/m³, 500 kg/m³ and 600 kg/m³. Workability, mechanical and durability properties were also studied.

The results obtained indicate that it is possible to, produce HPC with up to 60 MPa by replacing up to 40% of cement by fly ash and using local available crushed granite aggregates. Furthermore, it was observed that the workability and the durability, as measured by the chloride-ion diffusion coefficient, increased drastically when fly ash partially replaced Portland cement.

Based on the results obtained, it is possible to conclude that the use of fly ash in concrete is beneficial in terms of the workability and durability properties but was some disadvantages because early strengths are reduced.

Keywords: compressive strength; durability; fly ash; HPC; low cost; workability

A. Camões is a Research Assistant in the Department of Civil Engineering at the University of Minho, Guimarães, Portugal.

P. Rocha is a Research Assistant at the Superior School of Technology and Administration, Viana do Castelo, Portugal

S. Jalali is an Associate Professor in the Department of Civil Engineering at the University of Minho, Guimarães, Portugal.

B. de Aguiar is an Associate Professor in the Department of Civil Engineering at the University of Minho, Guimarães, Portugal.

R. Delgado is a Full Professor in the Department of Civil Engineering at the University of Porto, FEUP, Porto, Portugal.

INTRODUCTION

The use and range of applications of high performance concrete (HPC) have increased in the last few years. The growth of this material's usage is due to recent developments in admixtures and additions, namely superplasticizers (SP) and supplementary cementitious materials.

HPC is usually produced using silica fume, high quality fly ash and carefully chosen aggregates. These constituents drastically increase the initial cost of HPC, hence hindering its more widespread usage. In many countries the cost of silica fume is several times the cost of normal Portland cement (NPC). In Portugal, for example, the silica fume is ten times more expensive than NPC and a 10% addition of silica fume would mean doubling the price of the cementitious material. Thus, this research work has focused on the possibility of using locally available fly ash and other aggregates in the production of low cost enhanced performance concrete and low cost HPC.

In order to achieve the aims of the research work, an experimental program was performed which included tests to evaluate the mechanical, workability and durability properties of the proposed concretes. The effect of the amount of cement replacement by mass was evaluated using 0, 20%, 40% and 60% C replacement with different quantities of total binder of 400, 500 and 600 kg/m³.

The mechanical properties studied were uniaxial compression strength and splitting-tensile strength. Further, three-point bend beam tests were also

performed using a closed-loop servocontrolled compression-testing machine. Workability of fresh concrete and binder pastes were evaluated through slump and flow tests for the different mixes studied. The durability of the concretes produced was estimated through capillary water absorption, total open porosity by immersion and apparent coefficient of diffusion using non-steady state chloride migration tests.

MATERIALS AND MIXTURE PORPORTIONS

All aggregates used in the mixes were locally available crushed granite from de same quarry. It is noted that two different crushed sands ($D_{\max} = 2.38$ mm and $D_{\max} = 4.76$ mm) and a crushed coarse aggregate ($D_{\max} = 9.53$ mm) were used as received, this is, without any previous treatment such as washing. The used cement was a normal Portland cement of the Type I, class 42.5R. The used FA was of class F, according to ASTM standards, and was produced by the Portuguese Thermoelectric Power Plant of Pego, with quantities of loss on ignition (LOI) varying between 6% and 9%. The LOI value is more than the maximum allowed by the European Standard EN450 (1) and the ASTM C618 (2), which limit LOI to 5% and 6% respectively. The European norm admits a maximum limit of 7% at national level provided local regulation and arrangements exist. The superplasticizer (SP) used was a naphthalene sulphonate formaldehyde condensates. The quantities of SP used in the mixes were calculated using the results of several tests in pastes and mortars, where it is possible to identify the saturation point (optimum SP solid content dosage) between the values of 0.5% to 1.0% of the binder weight (3). For economical reasons, the value of the SP adopted in this research work was 0.5% by weight of cement.

For more detail of the main characteristics of these materials please refer to references (3-5).

Twelve different kinds of mix proportions were used corresponding to three doses of binder ($B = C + FA$). The quantities of the binder were fixed at 400, 500 and 600 kg/m³ and the corresponding water/binder ratio (W/B) at 0.4, 0.3 and 0.25. The amount of the aggregates was estimated using the Faury method (6). The mix design of the studied concretes is presented in Table 1.

The abbreviation mnemonic used has the following meaning: the first number refers to the amount of binder and the number that follows FA represents the percentage of cement replaced by FA.

Cubic (100x100x100 mm³) and cylindrical (150 mm diameter and 300 mm height) specimens were moulded in order to evaluate the compressive strength gain of the mixes. The splitting-tensile strength was evaluated using cylindrical

specimens of 150 mm diameter and approximately 96.5 mm height. The flexural behaviour was assessed using $850 \times 100 \times 100 \text{ mm}^3$ prisms specimens at a span of 800 mm. As far as the evaluation of this aspect is concerned further test results and analysis are presented in reference (7).

The curing of the specimens was done in a wet chamber at a temperature of 21°C and an 80% RH. One day after the moulding, the specimens were removed from the metallic moulds and immersed into water, where they remained until the day of the tests. One end of the cylindrical specimens used for determining the compressive strength was prepared with a sulphur-capping compound. The compressive strength of this compound was tested according to ASTM C617 (8) using a 2-inch cube, having obtained a result in excess of 55 MPa after 2 hours. Other cylinders were divided into three parts with similar dimensions in order to be tested for splitting-tensile strength. The samples used in the durability tests were obtained from the half prisms obtained from the flexural test with approximately $100 \times 100 \times 422.5 \text{ mm}^3$. For each composition six specimens were prepared by wet cutting with the approximate measures of $100 \times 100 \times 50 \text{ mm}^3$. These samples were used for the non-steady state chloride migration test. Three nominally identical cubic specimens with a 100 mm edge were used for the water absorption tests by immersion and capillarity.

PROPERTIES OF FRESH CONCRETE

The workability of the concrete mixtures was determined using the slump test (9) and by the use of the flow table test (10).

In Figs. 1 and 2, the variation of the slump and the flow with the percentage of the FA is presented. The slump and flow classes are also illustrated in accordance ENV206 (11). The results presented are the average values obtained for different mixes.

An analysis of these figures shows that:

- The enclosure of FA considerably improves the workability of the concrete allowing, even, a change of concretes slump class S1 to S4, or from the class F1 to F4 in flow table test;
- The favourable effect of FA is hardly noticeable in mixes with 400 kg/m^3 binder. This feature can be justified by the higher W/B ratio present in such concrete;
- The favourable action of the FA is less significant in mixes with percentage of FA higher than 40%. An explanation for this fact can be found in higher % of FA water demand, which negatively influences the workability opposing the beneficial effect caused by the spherical shape;
- The differences in the consistency of the mixes with 500 and 600 kg/m^3 binder can be considered minor;

- The simultaneous effects of the W/B and the reduction of the amount of binder are more relevant in the improvement of the workability properties up to 20% of FA content. The consistency of the mixes with 40% and 60% substitution by FA is almost independent of the W/B ratio and the corresponding amount of binder.

MECHANICAL PROPERTIES

Compressive Strength

The effect of the addition of FA on compressive strength was evaluated using tests performed by a closed-loop servo controlled compression-testing machine and using a 5 mm field LVDT with a 0.09% precision. The displacement velocity was of 0.36 mm/min.

The evolution of concrete compressive strength with time with a binder content of 500 kg/m³ and 600 kg/m³, used cylinders of $\phi 150 \times 300$ mm³, and results are presented in Figs.3 and 4. The curves correspond to the application of a numerical model, which predicts the compressive strength with time, proposed by Jalali (12).

The results shown on Figs. 3 and 4 allow the following conclusions:

- Concretes with higher percentage of NPC replacement by FA indicate a smaller rate of strength gain at early ages;
- The percentage FA up to 40% positively influences compressive strength at long curing times, i.e. at about 150 to 200 days;
- Mixes with 60% FA present a significant reduction in strength compared to other mixes, however, in global terms, they achieve values of compressive strength in the range of 60 MPa, given the reduced amount of cement used (200 kg/m³ in the 500FA60);
- At 90 days, it is possible to produce concretes with FA content up to 40% and with compressive strength measured in $\phi 150 \times 300$ mm³ cylindrical specimens in the range of 55 MPa (500 kg/m³ binder) and 60 MPa (600 kg/m³ binder).

The above-mentioned conclusions are also valid for results obtained on 100 mm cubic specimens. It is also possible to confirm that the compressive strength on cylinders is about 80% of the results obtained on cubes.

Modulus of Elasticity

The modulus of elasticity in compression (E_c) for the different concrete mixtures was estimated by the compressive strength tests on $\phi 150 \times 300$ mm³ cylindrical specimens. A LVDT with 5 mm field and 0.05% of precision was placed in the central area of the sample to register the variation in length of a 60 mm distance. Figs. 5 and 6 present the increase of E_c with curing time for the different mixtures. Each curve represents the average value (E_{cm}) determined on two samples.

An analysis of Figs. 5 and 6 yields the following conclusions:

- The mixtures made without FA present a marginal variation in E_{cm} before the age of 56 days;
- At an age of about 425 days the E_{cm} is similar for all mixtures (approximately 32 GPa for 500 kg/m³ binder and around 36 GPa for 600 kg/m³ binder), except for the mixtures 500FA40 and 600FA20, which comparatively present higher values than the expected;
- Despite tending to the same value, the E_{cm} development with time decreased as the FA content increased;
- For a 20% increase of the binder, from 500 to 600 kg/m³, corresponds to an increase of approx 14% in the E_{cm} at 425 days.

The E_c of the concrete is controlled by the modulus of elasticity of its components, that is, of the hydrated binder paste and of the aggregates. The E_c of concrete can be estimated from empirical expressions related to the compressive strength of the concrete. Such is justifiable on the basis of the influence of the modulus of hydrated binder paste, which depends mainly on the capillary porosity of the paste. In a similar way the capillary porosity influences the compressive strength of the paste (13). In this context, Figs. 7 and 8 show the possible relations between the average compressive strengths, determined in concrete cylinders, f_{cm} , and their E_{cm} . These Figs. also present the curves which result from the application of the suggested expressions by the ACI 363 committee (14), by CEB-FIP (13) and by the Portuguese code REBAP (15). The relationship proposed by the ACI committee 363 is restricted to concretes with compressive strength between 21 MPa and 83 MPa; the CEB-FIP formula is applicable whatever the f_{cm} of the concretes may be and REBAP is applicable for concretes up to 50 MPa characteristic compressive strength (to which correspond a f_{cm} of approximately 58 MPa).

The results show that the values of E_{cm} are in general lower than those obtained by the application of the codes. So for the mixes studied the applicability of the proposed expressions whether by ACI 363, by CEB-FIP or by REBAP, underestimates the E_{cm} value. The equation that comes closer to the experimental results is the one by the ACI 363. However, the CEB-FIP suggests the adoption of a coefficient, which translates the influence of the type of

aggregates. As recommended for quartzitic aggregates, a unit value for this coefficient (α_E) was used which enhanced significantly the prediction of E_{cm} . Nevertheless, the reduced values obtained from mixtures with 60% FA (up to 56 days of age) and with 40%FA with 400 kg/m³ binder, will not be justifiable by the adoption of a coefficient that takes into account the influence of the aggregates used.

Splitting-Tensile Strength

Figs 9 and 10 show the results obtained from the splitting-tensile strength ($f_{ct,sp}$) tests obtained for mixtures with 500 and 600 kg/m³ of binder. Results are the average of tests on three nominally identical specimens ($f_{ctm,sp}$).

Despite the occurrence of non-typical results, which make their analysis more difficult, Figs. 9 and 10 draw attention to the following tendencies:

- After the age of 56 days, an increase in the amount of binder does not seem to influence the $f_{ctm,sp}$;
- In FA concrete mixtures, up to an age of 56 days, an increase in the amount of binder causes a slower development of the $f_{ctm,sp}$.

DURABILITY PROPERTIES

Non-Steady State Chloride-ion Migration Diffusion Coefficient

The evaluation of the resistance to the migration of chloride-ion into concrete was made through Chalmers Tekniska Högskola (CTH) rapid test method. This test is performed in a non-steady state regime based on the procedure developed by Luping (16). The test consists of submitting 100x100x50 mm³ specimens, at an age of approximately one year, to an electrical potential difference of $40 \pm 0.2V$, for a period of time calculated according to the intensity of the initial current, as suggested by the author. The coefficient of diffusion was obtained measuring the depth of chlorides penetration using a colourimetric process (16). The results of the coefficient of chloride-ion diffusion (D_{ns}) are presented in Figs. 11 and 12. Results are the average values obtained from six nominally identical specimens ($D_{ns,m}$).

The results obtained indicate a favourable effect of FA addition:

- All the mixtures with FA presented a smaller $D_{ns,m}$ compared to those obtained from mixes without FA, irrespective to the binder content and W/B ratio;
- Even for the mixtures with the poorest binder, i.e 400FA60, the $D_{ns,m}$ obtained is lower than that of the mixture with higher cement content, 600FA0;

- The favourable result of increasing the binder quantity becomes unimportant for mixes with 500 kg/m^3 or more. This is even more obvious in compositions without FA.

Water Absorption Tests by Immersion and by Capillarity

The capillary water absorption tests followed the LNEC specification E393 (17), which is based on the RILEM CPC11.2 draft recommendation. This test consists of registering the increase in mass of a specimen at given intervals of time when permitted to absorb water by capillary suction. The specimen is usually placed on saturated sand or is in contact with water along one surface. The values of the coefficient of capillarity absorption, S_{ca} , shown in Figs 13 and 14, correspond to the slope of the curves representing the water absorbed by unit area versus square root of time, during the initial four hours of testing (18).

The results presented in Figs 13 and 14 show that:

- The addition of FA causes a reduction in S_{ca} ;
- An increase in the quantity of binder, in conjunction with the reduction of W/B ratio, leads to a decrease in S_{ca} , particularly in mixes with up to 500 kg/m^3 of binder;
- The S_{ca} does not seem to change significantly when binder content is increased from 500 to 600 kg/m^3 .

Water absorption by immersion tests according to LNEC specification E394 (19), based on the RILEM CPC 11.1 recommendation, were also performed and led to identical conclusions.

CONCLUSIONS

For the Type of concrete mixtures studied, the mixture containing 500 kg/m^3 of binder revealed better potential for usage as HPC, as they presented an important gain in performance compared to mixes with 400 kg/m^3 of binder and small difference with mixture with 600 kg/m^3 of binder.

Generally speaking at long curing times, mixtures with FA contents up to 40% seem to improve all the properties studied in this research work. Thus, considering the economic factor, the 40% FA mixture indicate a better performance.

Adding 60% FA renders concrete with considerably lower mechanical characteristics, in comparison with other mixes studied here. However, given

the low quantities of cement these mixtures demonstrate good economic performance.

When evaluating the durability of concrete through the chloride ion diffusion coefficient, the coefficient of capillary absorption and the water absorption by immersion, the addition of FA is beneficial, leading to more durable concrete. This effect is particularly noticeable with a substantial decrease in the chloride-ion diffusion coefficient, indicating its special aptitude for environments that promote degradation of reinforced concrete due to chloride ion penetration.

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Table 1 – Concrete mix proportions

Concrete	W/B	C kg/m ³	FA kg/m ³	Sand 1 kg/m ³	Sand 2 kg/m ³	Coarse aggregate kg/m ³
400FA0	0.4	400	0	613.56	233.55	857.45
400FA20	0.4	320	80	591.96	262.38	878.58
400FA40	0.4	240	160	552.99	284.75	875.65
400FA60	0.4	160	240	503.44	300.96	855.01
500FA0	0.3	500	0	502.92	308.43	865.61
500FA20	0.3	400	100	461.85	334.01	869.82
500FA40	0.3	300	200	406.91	349.01	847.11
500FA60	0.3	200	300	364.24	373.70	848.70
600FA0	0.25	600	0	377.30	367.85	850.73
600FA20	0.25	480	120	326.57	399.51	856.01
600FA40	0.25	360	240	271.28	407.93	832.76
600FA60	0.25	240	360	223.26	421.23	824.23

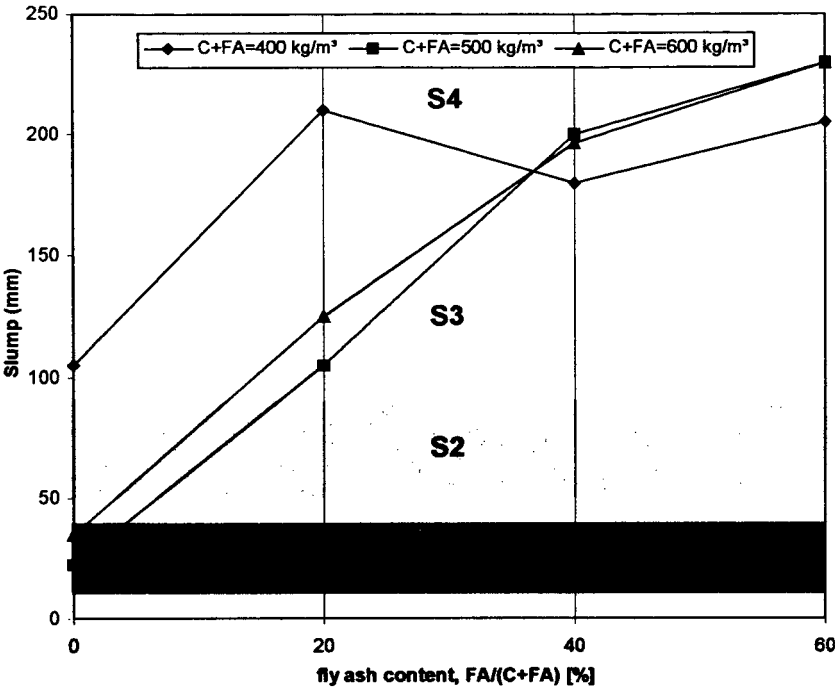


Fig. 1 – Slump versus fly ash content.

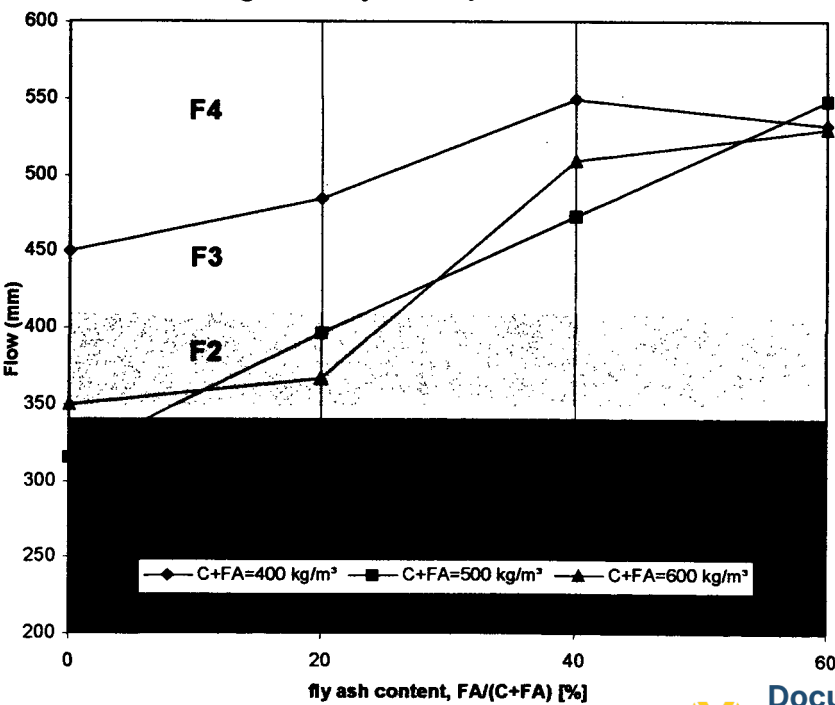


Fig. 2 – Flow versus fly ash content.

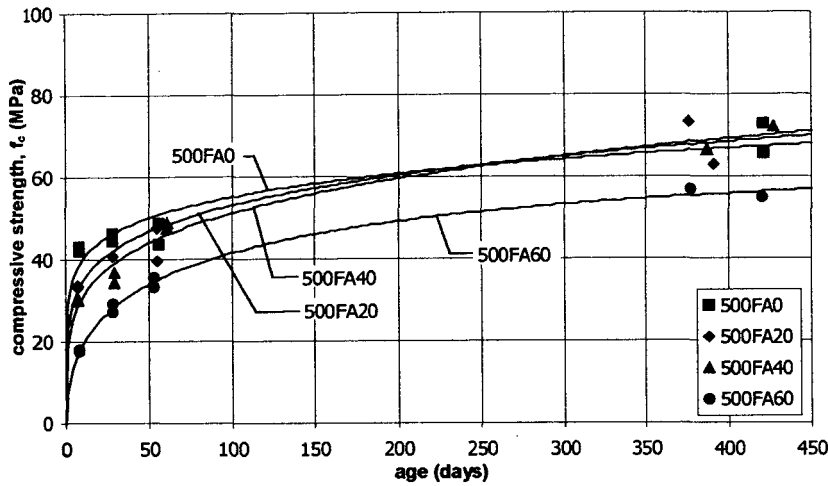


Fig. 3 – Compressive strength development with time ($B=500 \text{ kg/m}^3$).

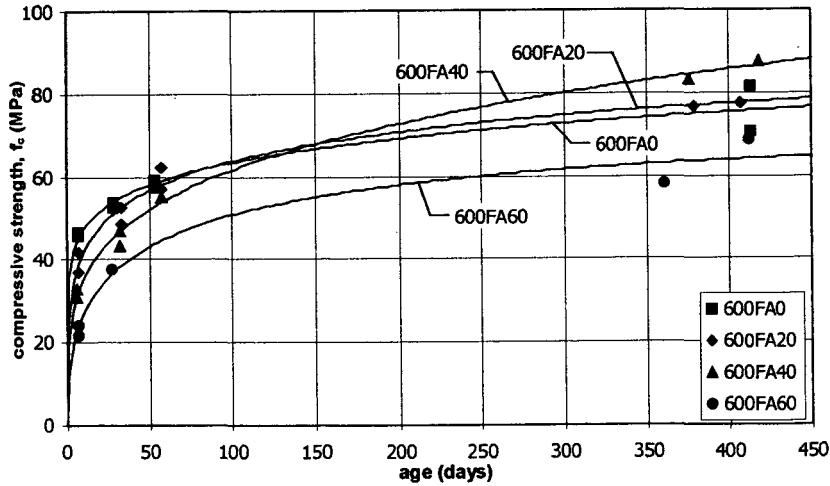


Fig. 4 – Compressive strength development with time ($B=600 \text{ kg/m}^3$).

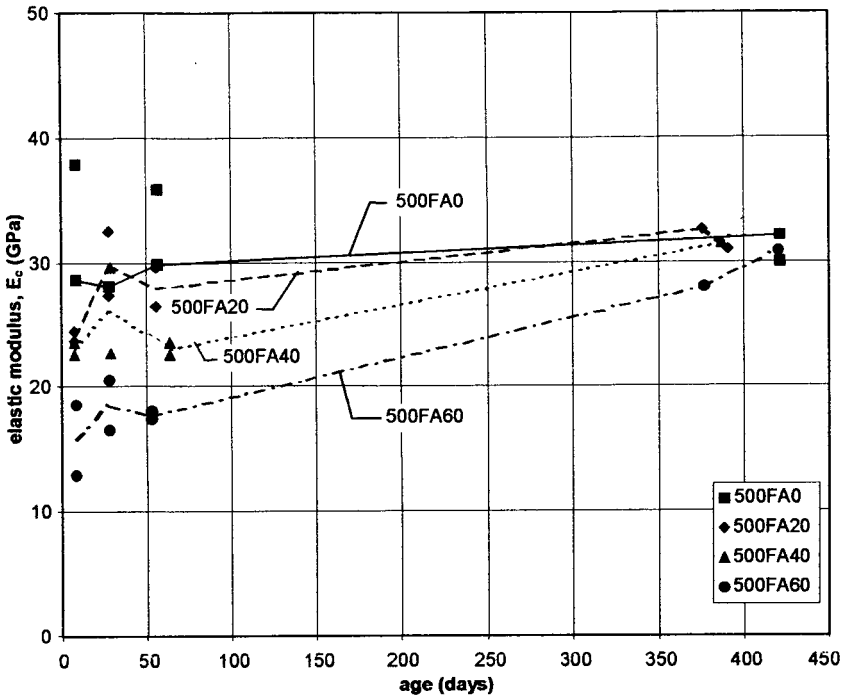


Fig. 5 – Elastic modulus development with time ($B=500 \text{ kg/m}^3$).

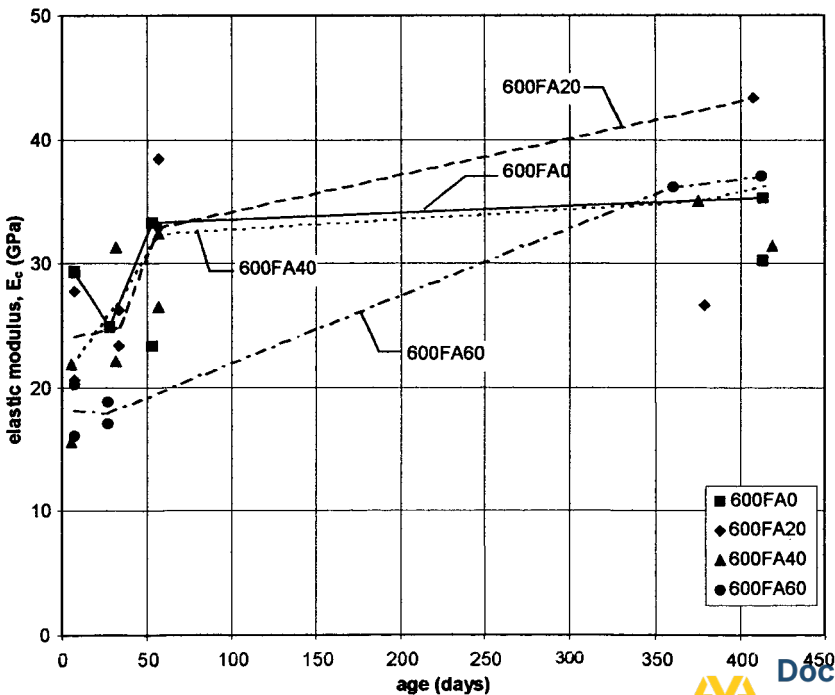


Fig. 6 – Elastic modulus development with time ($B=600 \text{ kg/m}^3$).

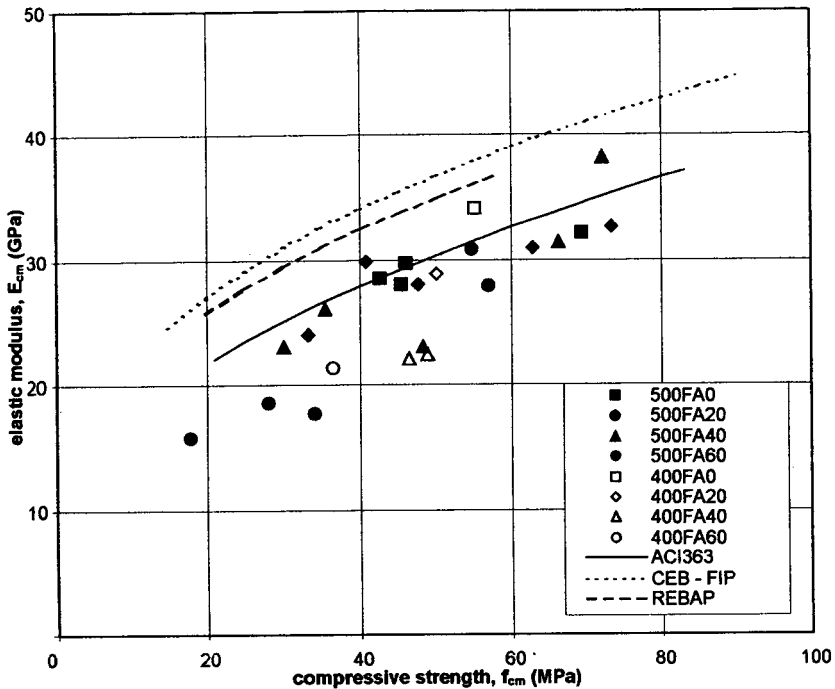


Fig. 7 – Relation between compressive strength and elastic modulus ($B = 400 \text{ kg/m}^3$ and $B = 500 \text{ kg/m}^3$).

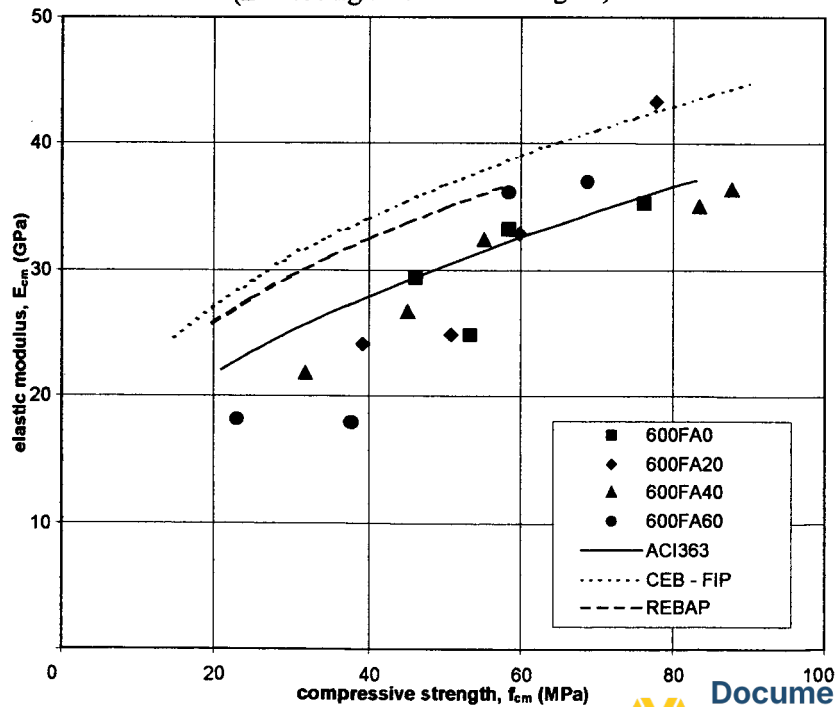


Fig. 8 – Relation between compressive strength and elastic modulus ($B = 600 \text{ kg/m}^3$).

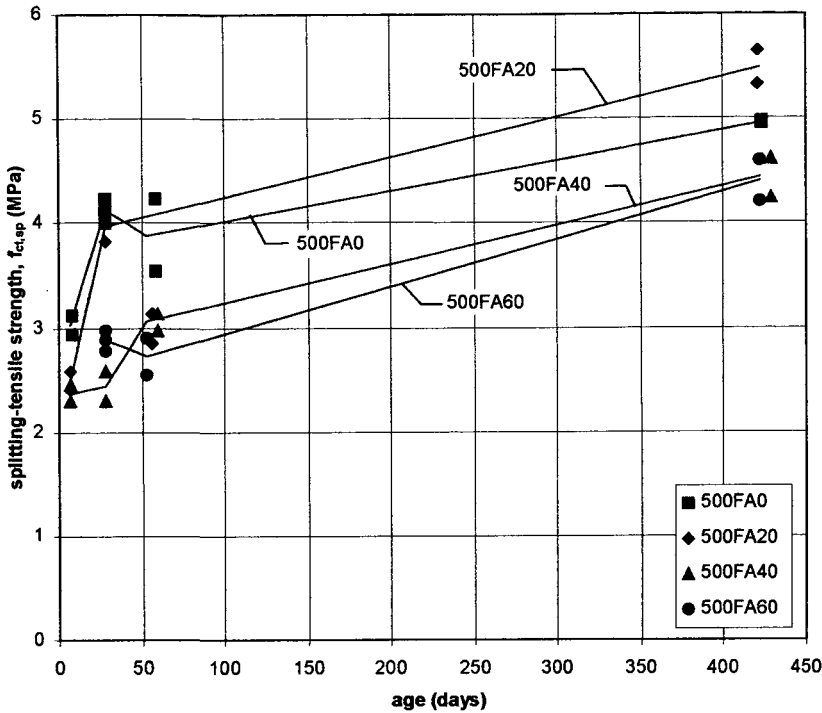


Fig. 9 – Development of splitting-tensile strength with time ($B = 500 \text{ kg/m}^3$).

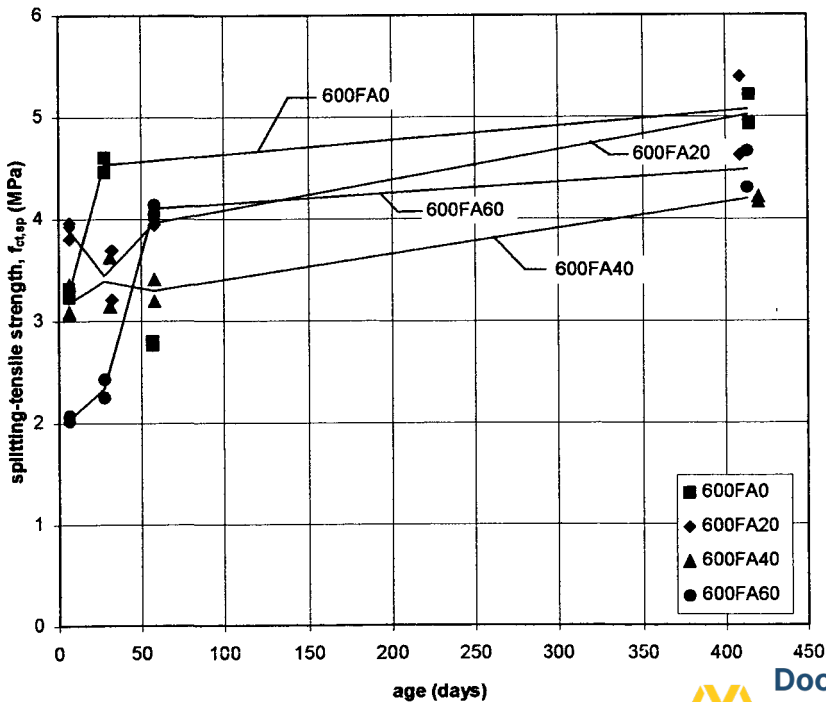


Fig. 10 – Development of splitting-tensile strength with time ($B = 600 \text{ kg/m}^3$).

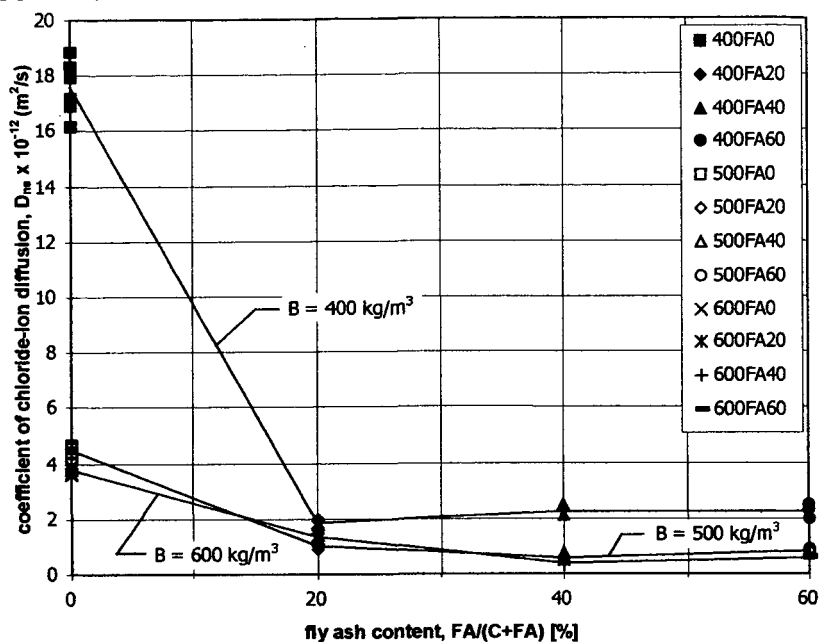


Fig. 11 – Relation between coefficient of chloride-ion diffusion and fly ash content.

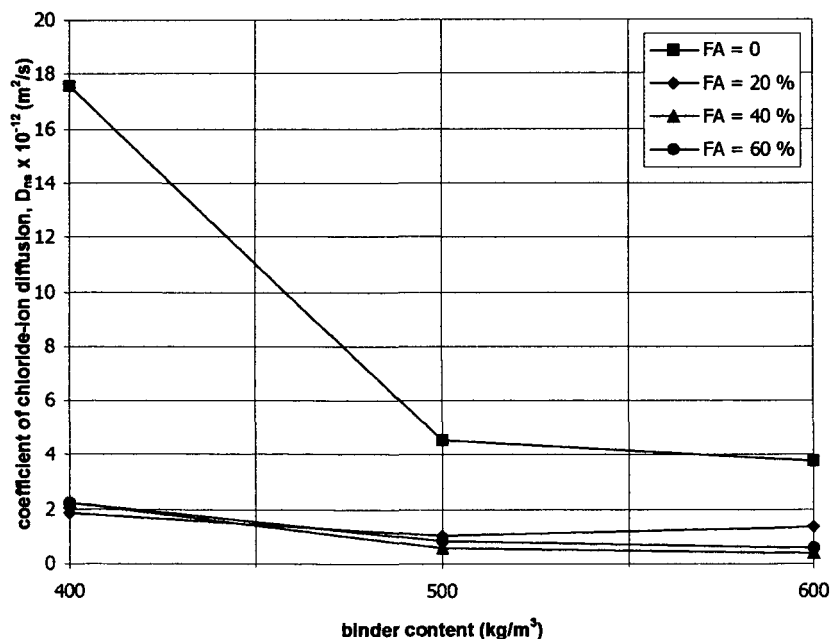


Fig. 12 – Relation between coefficient of chloride-ion diffusion and binder content.

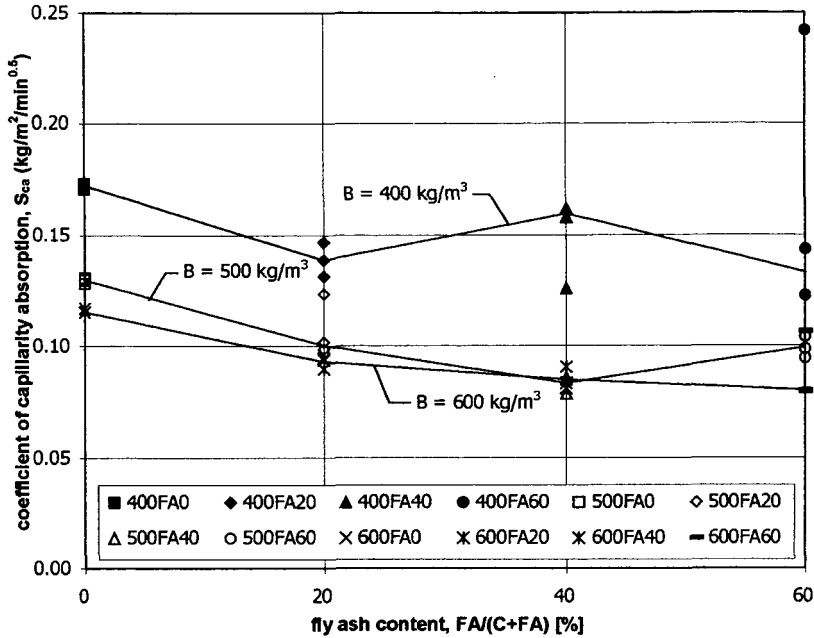


Fig. 13 – Relation between coefficient of capillarity absorption and fly ash content.

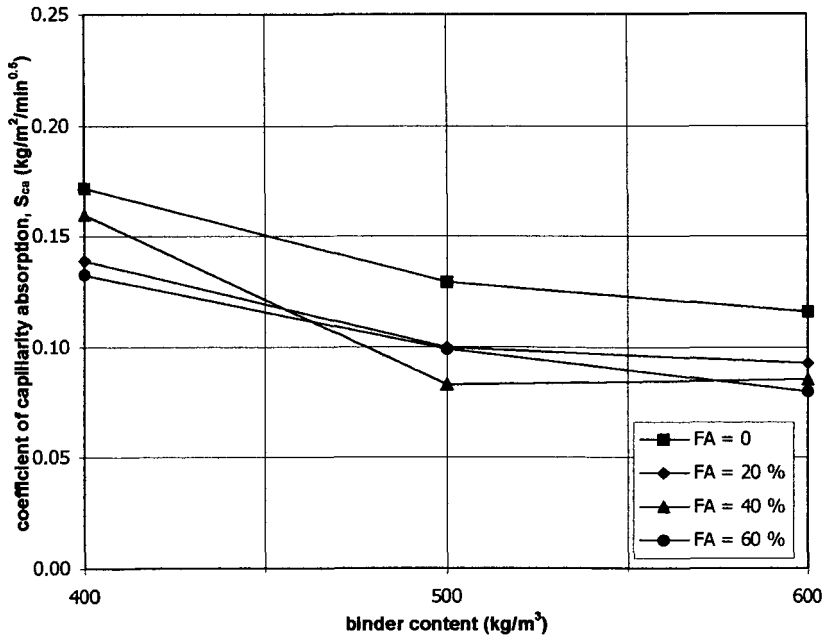


Fig. 14 – Relation between coefficient of capillarity absorption and binder content.

Strength and Chemical Resistance of Mortars Containing Brick Manufacturing Clays Subjected to Different Heat Treatments

by M. O'Farrell, B. B. Sabir, and S. Wild

Synopsis:

The use of calcined clay as a pozzolanic partial cement replacement material in concrete is currently receiving increased attention. Previous work by the authors has demonstrated the effectiveness of utilising ground waste clay bricks in mortar and concrete. This paper presents the results of an investigation of the properties of mortar in which a calcined clay was employed as a pozzolan. The clay was heated in crucibles to 800°C with a heating ramp of 100°C per hour. The furnace environment was then kept constant at 800°C for a period of two hours. After heat treatment the clay was cooled in two different ways. One batch was allowed to cool naturally to room temperature and the second was water quenched with the crucible lid on. Mortars were prepared using either the heat treated clay or ground waste clay bricks (from the same clay subjected to 1000°C calcining) as a pozzolanic partial replacement for cement at replacement levels of 10, 20 and 30%. The compressive strengths of the mortars were monitored up to 90 days and the resistance to sodium sulfate solution and synthetic seawater was monitored up to 300 days. The specimens were also monitored for weight changes. Partially replacing cement by ground brick or heat-treated brick clay gives early strengths that are lower than that of the control. At 90 days, however, the strengths are the same as or greater than that of the control. Heat-treated clay is effective in reducing expansion during exposure of the mortar to sulfate solution and synthetic seawater. The rapidly cooled clay gives better performance, in terms of strength development and resistance to harmful solutions, than the slow cooled clay.

Keywords: chemical resistance; compressive strength; ground brick; heat treated clay

Dr Martin O'Farrell is a Lecturer in the School of Technology, University of Glamorgan. His main interest is the durability of pozzolan containing and waste containing concretes and mortars including sulfate resistance and sorptivity.

Professor Stan Wild is Professor of Construction Materials in the School of Technology, University of Glamorgan. He specialises in the use of pozzolans and novel waste materials in mortars and concretes.

Dr Bahir Sabir is Reader in Engineering Mechanics in the School of Technology, University of Glamorgan. His research interests are in partial cement replacement materials, fibre reinforced concrete and fracture mechanics of concrete.

INTRODUCTION

Because of the technical, and sometimes economic, advantages that can be accrued by the use of pozzolans there has, over the last few decades, been increasing and widespread utilisation of fly ash, silica fume, natural zeolite and other pozzolans in concrete. There are compelling long-term environmental reasons for extending the practice of partially replacing cement in concrete with waste and other less energy intensive processed materials, which have pozzolanic properties. A possible source for a pozzolan is calcined clay. Natural pozzolans in the form of calcined earths blended with lime (calcium hydroxide) have been used to produce cementitious materials since ancient times. In the absence of a natural volcanic pozzolan, the Romans used ground waste bricks to manufacture hydraulic lime mortars. Recent years have seen revived interest in the use of such materials as pozzolans in concrete. For example a recent review (1) demonstrates the effectiveness of using metakaolin as a pozzolanic cement replacement in concrete in a range of applications such as the containment of hazardous wastes.

The continuing search for partial cement replacement materials has led the authors to investigate the utilisation of waste fired clay bricks as a pozzolan for concrete and mortar (2). Use of crushed brick (surkhi) as a pozzolan has been practised in India for many years. When used as a partial cement replacement to produce mortar or concrete, crushed brick results in improved properties including reduction in permeability and improved resistance to sulfate attack and alkali-aggregate reaction (3). The calcining temperature of clay has great influence on its action as a pozzolan. The clay is in its most reactive state when the temperature of calcining results in loss of hydroxyl and a collapsed and distorted clay structure (4). The calcining temperature required to produce this active state is usually in the range 600-900°C (5,6). Above this temperature

crystallisation occurs and activity declines. However at even higher firing temperatures liquid phase forms which, on cooling, solidifies into an amorphous glass phase which also shows pozzolanic activity. In a previous study (2) the authors have demonstrated, by examining ground clay bricks fired at 6 different temperatures in the range 600-1100°C, that the optimum firing temperature of lower Oxford clay to produce the most effective pozzolan is about 1000°C. This work has led to consideration of producing a low-cost manufactured pozzolan from the clay used for brick production. This paper reports the results of a study conducted on using the brick clay, subjected to different heat treatments, as a pozzolan in mortar

EXPERIMENTAL DETAILS

Materials

The portland cement (PC) used has the phase composition 50.9% C₃S, 20.7% C₂S, 11.6% C₃A and 6.4% C₄AF and complies with EN 197-1. The mineralogy of the clay is Illite (23%), Kaolinite (10%), Calcite (10%), Chlorite (7%), Quartz (29%), Gypsum (2%), Pyrite (4%), Feldspar (8%) and organics (7%). The sand used was "Normensand", a standard European sand complying with EN 196-1.

Unlike brick production, where the clay is heated to temperatures in excess of 1000°C, the clay used in the present investigation was heated in crucibles to 800°C with a heating ramp of 100°C per h. The furnace environment was kept constant at 800°C for a further 2 hours. After the heat treatment, one batch of calcined clay was allowed to cool to room temperature and a second was water-quenched with a lid placed over the crucible. The heat-treated brick clays (HTC) were then used as partial cement replacement materials to produce mortar specimens for assessment of strength development and performance in salt solutions. Alongside the HTC specimens, samples in which the cement was partially replaced by ground waste clay bricks (GB) were also prepared and subjected to the same tests. Table 1 gives the chemical compositions of the PC and pozzolan used here. The HTC and GB had the same composition.

Mixture Details

The mortar mixtures had proportions of 1:3:0.5 (cementitious material:sand:water). The cementitious material consisted of PC and GB or HTC. Ten different mixture compositions were examined in the present study. The PC was partially replaced by 0, 10, 20 and 30% GB or HTC. The designations used for the two heat-treated clays are QC (quenched) and SC (slow cooled). Mixture identification is made by the designation letters followed by the percentage replacement of PC by the pozzolan, e.g. QC20 and GB10 represent

mixtures in which the PC was replaced by 20% quenched clay and 10% ground waste brick, respectively.

Specimen Preparation, Curing and Testing

The compressive strength was determined by crushing 100 mm mortar cubes prepared in steel moulds. Mortar bars of dimensions 20 x 20 x 160 mm were used for the expansion and weight change measurements. After casting, the steel moulds containing the specimens were covered with plastic wrap to prevent loss of water by evaporation. After 24 h the specimens were stored in water at $20 \pm 2^\circ\text{C}$. The expansion bars had stainless steel inserts cast into their ends to facilitate accurate measurement of changes in length. After 28 days water curing, some bars were immersed in sodium sulfate solution ($\approx 2.4\%$ concentration) while the remainder were immersed in synthetic seawater. Concentrations of the various salts used to manufacture the synthetic seawater can be found in Table 2. The monitoring of specimens subjected to sulfate solution and synthetic seawater was carried out in accordance with PR ENV 196-X (7). The changes in length and mass of the specimens were monitored regularly for a period of 300 days. The salt solutions were renewed every 28 days. Failure, indicated by F, was taken as the point at which the specimen disintegrated or had expanded sufficiently so that it could no longer be measured with the comparator apparatus. Each experimental measurement given in this paper is the average of those taken from three samples.

RESULTS

Compressive Strength

Tables 3 and 4 give the compressive strengths of mortars containing GB and the heat-treated clays QC and SC, respectively. All results shown are the average of those obtained from three tests. Partial PC replacement with GB or HTC results in compressive strengths at early ages that are considerably below those of the control concrete, but at 90 days the strengths are more or less unaffected. Figure 1 gives the strength development in terms of relative strength, which is the strength at a particular age divided by the strength of the control concrete at the same age. Increasing replacement level results in decreasing relative strength. This is attributed to the dilution effect. As the PC hydrates pozzolanic activity occurs and strength increases with time such that at 90 days, with up to 20% PC replacements by GB or QC, strengths are the same, or in excess of that of the control. On comparing the performance of GB and HTC mortars it is apparent that at 7 days the relative compressive strength of HTC mortars are generally greater than those for GB mortars. As curing time increases beyond 28 days, the rate of increase of relative strength of HTC mortar decreases considerably, whilst

those of GB mortars continue to increase at approximately similar rates to those observed up to 28 days. The reduced rate of relative compressive strength increase at 28 days was a phenomenon observed in an earlier study (8) and attributed to an increased level of sulfate in clays calcined at lower temperatures. An analysis of the sulfate level was, therefore, conducted for all three clays. The acid (HCl) soluble sulfate contents for GB, QC and SC pozzolans were found to be 3.05, 3.0 and 3.94%, respectively. The relatively higher level of sulfate in the SC pozzolan may be responsible for the decline in relative strength beyond 28 days. This behaviour, however, cannot be attributed to only the sulfate level and other factors must be considered. For example it is established that pozzolanicity is very much affected by the glass content, which in turn is dependent on the firing temperature. Vitrification (glass formation) of brick clay occurs at firing temperatures in excess of 900°C and it can thus be deduced that HTC, which was fired at 800°C, has negligible glassy phase relative to GB, which is fired at temperatures in excess of 1000°C. Thus the pozzolanic activity must be a function of the distorted dehydroxylated clay. Quenching would be expected to freeze in a more distorted structure with greater stored energy relative to slow cooling which promotes equilibrium and crystallisation. Therefore, this effect might be expected to give a more reactive material when quenching as opposed to slow cooling.

Chemical Resistance

Sodium Sulfate Solution--Figure 2 shows the expansions of the control (CTRL) mortar and GB and HTC (QC and SC) mortars, with PC replacement levels of 10, 20 and 30%, when exposed to sodium sulfate solution. In all cases, increasing replacement level gives increasing resistance to expansion and at 30% replacement both the GB and QC mortars show very small expansion relative to the control. In terms of providing resistance to expansion, particularly at low replacement levels, the GB mortar experiences the lowest expansion and the SC mortar the highest. In fact at 10% replacement, expansion of the SC mortar exceeds that of the control. Thus similar to strength development, the QC mortars generally perform better than the SC mortars, which again may be attributed to greater pozzolanic activity.

The increase in weight accompanying expansion was measured and the results are presented in Fig. 3. The weight increase shows very similar trends to those of expansion. Figure 4 gives the relationship between weight increase and expansion in sulfate solution for all mortars investigated. The results show a strong direct correlation. The increase in weight that corresponds to very little or zero expansion (see Fig. 4 (b), represented by the intercept, is indicative of the filling of voids (pores or air voids) in the mortar by expansive products. Figure 5 shows the condition of the mortar bars after 190 days exposure to sodium sulfate solution. Whilst the control mortar disintegrated, the HTC mortars generally

showed much improved performance, particularly the QC mortars at high PC replacement levels.

Synthetic Seawater--Figure 6 gives the expansions of the mortar bars subjected to synthetic seawater. Comparison with the results in Fig. 2 shows that the mortars expand much less than those subjected to sulfate solution. This agrees with observations by Lea who states that the lack of expansion in seawater can be attributed to the increased solubility of calcium hydroxide in seawater and that the presence of chlorides in the seawater inhibits the expansive reactions commonly associated with sulfate attack [9]. The low expansion rate may also be attributed in part to the formation of a skin of magnesium hydroxide (brucite), around the mortar prism, reducing ingress of the aggressive seawater solution. GB and HTC mortars expand less than the control mortar and, as with exposure to sulfates, expansion is reduced as the percentage replacement increases. A significant observation is that the HTC mortars give expansions that are significantly smaller than those of GB mortars, particularly at 10% and 20% replacements during early exposure. At 10% PC replacement, the QC mortars give smaller expansions than those of the SC mortars at all exposure periods. There seems to be little difference between the performances of these two mortars as the percentage replacement increases. As with expansion, the weight increase (see Fig. 7) reduces as the percentage replacement increases. Figure 8 shows the relationship between weight increase and expansion. Again there is a distinct change in the gradient which, in this case, occurs at a weight increase of about 2.5%. This figure is about 5 times that obtained for the mortars exposed to sulfate solution and is indicative of a much greater volume of reaction products being formed prior to expansion taking place. This suggests that the reaction products formed may be less expansive or may form extensively as surface deposit rather than in the internal pores (compare Fig. 4 and 8).

DISCUSSION

The HTCs examined in this investigation were manufactured from the same clay as that used in the production of bricks. Mortars containing HTC exhibited greater early relative compressive strengths than those of GB mortars. As curing time increases beyond 28 days, the rate of increase of relative strength of HTC mortars decreases considerably, whilst that of GB mortar continues to increase at similar rates to those observed up to 28 days. Analysis has shown that the QC (quenched cooled) HTC possessed a similar level of sulfate as that of GB, but the SC (slow cooled) HTC showed a higher level of sulfate. Although the higher level of sulfate in the SC pozzolan may be partly responsible for the decline in relative strength beyond 28 days, this does not explain fully the general performance of HTC observed here. Earlier work (8) has shown that the glass content of GB pozzolans appears to be the key factor in strength development, especially at curing times beyond 28 days. The firing temperatures experienced

by the HTC ($> 800^{\circ}\text{C}$) and by the GB ($>1000^{\circ}\text{C}$) lie on either side of the vitrification point of brick clay. Sintering and vitrification (glassy phase production) of brick making clay occurs at temperatures exceeding 900°C . It is, therefore, reasonable to assume that the HTC will contain very little glassy phase. As the HTC has not undergone vitrification but was treated at temperatures exceeding those required for the bound mineral water to be removed ($150 - 600^{\circ}\text{C}$), its structure will be similar to that of an activated clay pozzolan such as MK. Metakaolin has been shown to enhance early strength of mortar and the its effects on strength at extended curing periods (> 28 days) (10) also corroborates the trends observed for HTC mortars. This is evidence that the glass content of pozzolans is of primary importance with respect to long term strength development, most probably due to prolonged C-S-H gel production as a result of the glassy phase-calcium hydroxide (CH) interaction. The improvement in performance of QC mortars over that of SC mortars is attributed to the more distorted structure, due to quenching, resulting in a more reactive material.

Although the performance of HTC mortars in sulfate solution is superior to that of the control concrete, it is somewhat inferior to that of GB mortar. This is again attributed to glass content. Because of the differing firing temperatures of HTC and GB pozzolans, it is expected that GB would contain more of the glass phase. At all replacement levels the QC mortars showed better performance in sulfate solution to that of SC mortars.

It has previously been reported (11) that as the firing temperature of clay brick production decreases then so does its effectiveness at resisting sulfate induced expansion when used in a finely divided state as a pozzolan. It is considered that the moderate firing temperature of 800°C experienced by HTC has the effect of producing a material similar, in its physical state, to MK. It has also been shown (12) that MK reduces expansion as the level of PC replacement increases and that this is attributed to reduction of the CH content (10) thus reducing the amount available for ettringite formation and growth. In acting as an active pozzolan, HTC consumes CH early in the hydration process resulting in less CH being available for gypsum and ettringite formation on exposure to sodium sulfate solution. Experience has shown that MK is considerably more active as a pozzolan than HTC. The difference in performance between HTC and MK is attributed to the type of clay used. The principal clay mineral in brick clay is illite (23%) whereas MK is derived from kaolinite. The temperatures that the HTC experienced were not sufficient to activate the illite, which requires temperatures in excess of 930°C , and even then results in a generally a poor pozzolan (13). However lower Oxford clay also contains kaolinite (10%) and it is the dehydroxylation of this phase that is considered to be the cause of the pozzolanic activity in the HTC mortars.

The more effective role played by GB, when compared to HTC, is attributed to the increased glass content of GB. Without the additional C-S-H gel from

glassy phase dissolution, gel from hydration and initial reaction between HTC and CH will gradually decalcify due to the production of ettringite. With the decalcification of C-S-H gel, the strength of the HTC mortars will be somewhat reduced thus allowing more expansion than in the case of GB mortars.

CONCLUSIONS

1. Partially replacing portland cement with ground brick or heat-treated brick clay gives early compressive strengths that are lower than that of the control. However, up to 20% replacements give strengths at 90 days that are the same as or greater than that of the control.
2. Relative strength decreases as the level of replacement increases. Although heat-treated clays give relative strengths at early ages that are greater than those of ground brick mortars, the latter give better performance at extended ages. This is attributed to the higher glass content of ground brick.
3. Although the sulfate resistance of heat-treated brick clay mortars is somewhat less than that of ground brick mortar, heat-treated brick clays give superior performance to that of the control mortar, particularly at replacement levels 20 or 30%. At all replacement levels the quenched clay gives better resistance than the slow cooled clay.
4. Heat-treated clay mortars are effective in reducing the expansion due to exposure to synthetic seawater. At 10 and 20% replacements the heat treated clay mortars give expansions at early exposure that are considerably lower than those of the ground brick mortars.

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Table 1 – Chemical composition of PC and HTC

	PC	HTC
Specific surface (Blaine)(m ² /kg)	350-500	320-350
Composition (%)		
SiO ₂	20.6	54.8
Fe ₂ O ₃	2.1	6.0
Al ₂ O ₃	5.7	19.1
CaO	65.0	9.4
MgO	2.3	1.8
SO ₃	3.3	2.9
Na ₂ O	0.14	0.5
K ₂ O	0.67	3.2
TiO ₂	---	1.0
LOI	2.81	0.6

Table 2 – Composition of synthetic seawater (8)

Salt	Quantity (g/l)
Sodium chloride	30.0
Magnesium chloride	6.0
Magnesium sulfate	5.0
Calcium sulfate	1.5
Potassium hydrogen carbonate	0.2

Table 3 – Compressive strength of GB mortars (MPa)

	7d	28d	90d
CTRL	52.2	65.3	67.0
GB10	43.8	61.2	74.1
GB20	39.4	55.4	66.2
GB30	31.3	45.2	55.7

Table 4 – Compressive strength of HTC mortars (MPa)

	7d	28d	90d
CTRL	39.6	52.5	61.3
QC10	38.7	52.5	62.1
QC20	32.2	46.8	59.3
QC30	24.9	39.1	53.1
SC10	38.2	52.5	58.1
SC20	32.1	46.7	52.1
SC30	25.1	40.8	49.7