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EXPERIMENTAL STUDY ON MICROSTRUCTURE AND STRUCTURAL BEHAVIOUR OF RECYCLED AGGREGATE CONCRETE

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ABSTRACT

The use of recycled aggregates in concrete opens a whole new range of possibilities in the reuse of materials in the building industry. This could be an important breakthrough for our society in our endeavours towards sustainable development.

The trend of the utilisation of recycled aggregates is the solution to the problem of an excess of waste material, not forgetting the parallel trend of improvement of final product quality. The utilisation of waste construction materials has to be related to the application of quality guarantee systems in order to achieve suitable product properties. Therefore the complete understanding of the characteristics of new material becomes so important in order to point out its real possibilities.

The studies on the use of recycled aggregates have been going on for 50 years. In fact, none of the results showed that recycled aggregates are unsuitable for structural use. Only having inadequate number of studies in durability aspects, made recycled aggregates to be preferred just as stuffing material for road construction.

My thesis, aimed to focus on the possibility of the structural use of recycled aggregate concrete based on a better understanding of its microstructure.

To begin with the characteristics of the aggregates were established, to study their possible application in concrete production. After analysis, the dosage procedure was carried out in order to produce four concrete mixtures using different percentages of recycled coarse aggregates (0% (HC), 25% (HR25), 50% (HR50) and 100% (HR100)) with the same compression strength. Raw coarse aggregates (granite) and sand (crushed limestone) were used in the different concrete mixes.

Macroscopic and microscopic examination were carried out in HC, HR25, HR50 and HR100 concretes in order to observe the durability effects. The macroscopic examination determined the aggregates distribution, composition, the contaminants and aureoles around adhered mortar. Microscopic examination was carried out by Optical light transmitted microscope Leica Leitz DM-RXP, using Fluorescence Thin Sections, in order to analyse the cement paste, the new and old interfacial transition zones, secondary reactions as well as damage. Original aggregates and cement paste, interfacial transition zones and alkali silica gel produced due to reactive aggregates present in adhered mortar were analysed by SEM and EDX-maps.

Beside macro and micro observations, shear failure behaviour of recycled aggregate concretes was studied. Shear failure test was found more appropriate, as concrete properties had more influence in this type of failure behaviour compared to the flexural failure where the reinforcement plays the important role. Sixteen beam specimens were cast and the structural behaviour of these beams was analysed using four different transversal reinforcements for each kind of concrete. An analytical prediction of the experimental results are carried out using a numerical model (Modified Compression Field Theory), using the codes AASHTO LRFD, CSA, Eurocode-2 and expressions proposed in the Spanish code EHE-99.

Organic and inorganic compounds were found to be released from waste materials through leaching and dispersed into the soil and surface water. The leaching of these compounds were measured employing different codes, the two Dutch codes (NEN 7341 and 7345) and the European Normative (EN 12457-2).

Some recommendations are given as to the aggregates characteristics to be used in concrete mixes, taking into account the European standards for recycled aggregates. Also suggestions are made for the production process of concrete using recycled aggregate. Mechanical properties of recycled aggregate concrete are studied and they are compared with that of conventional concrete. Based on the durability of the concrete, some suggestions are proposed with respect to possible alkali silica reaction between new cement and original fine aggregates. It is also determined that the effect of the use of recycled aggregate on the beams' shear strength depend on the percentage of coarse aggregate substituted. The applicability of concrete recycled aggregate with respect to its environmental behaviour is demonstrated. In conclusion, some suggestions for future studies are made which would help us in the evolution of our understanding in this field.

RESUMEN

El empleo de áridos reciclados en el hormigón abre nuevas posibilidades de reutilización de estos materiales en la industria de la construcción. Esto podría dirigir a nuestra sociedad hacia el desarrollo sostenible.

A la vez que la búsqueda de alternativas para la reintroducción en el ciclo productivo de materiales de deshecho surge una búsqueda en la mejora de la calidad de vida, es precisamente ésta búsqueda la que impide su introducción de forma no lo suficientemente meditada, que permita asegurar, de forma adecuada, las prestaciones que ofrecen los materiales naturales en sus diversos usos. Así pues, es necesario conocer con profundidad las características del nuevo material, que permita su adecuado uso a cada circunstancia.

El reciclaje del residuo de construcción y demolición viene siendo estudiado desde los años 50. En particular, no existen claros apuntes que lo señalen como un elemento a ser desechado de funciones resistentes, sin embargo debido a la gran escasez experimental sobre las características estructurales y de durabilidad de que dispone dicho material, su empleo ha venido limitado a usos carentes de solicitación significativa del material, como puede ser la ejecución de viales de tráfico.

Esta tesis, enfoca la posibilidad de utilizar hormigón de deshecho triturado como árido reciclado en hormigón estructural, basado en un mejor entendimiento de su microestructura.

Se establecieron las características de los áridos reciclados para estudiar su posible aplicación en la producción del hormigón. Después de dicho análisis, se estudió la dosificación idónea de cuatro hormigones fabricados con diferentes porcentajes de árido grueso reciclado (0 % (HC), el 25 % (HR25), el 50 % (HR50) y el 100 % (HR100)) con la misma resistencia a compresión. En todos los hormigones se utilizó arena natural (caliza) y el árido grueso natural utilizado fue granítico.

Para observar los efectos de la durabilidad se realizaron análisis macroscópicos y microscópicos en los cuatro hormigones (HC, HR25, HR50 y HR100). Mediante el análisis macroscópico se determinó la distribución de los áridos, la composición, los contaminantes y las aureoles alrededor del mortero adherido. El análisis microscópico se realizó mediante el microscopio óptico Leica Leitz el DM-RXP, se fabricaron láminas delgadas de fluorescencia para cada hormigón y se analizó la pasta de cemento, las nuevas y ya existentes (en los áridos reciclados) interfases, reacciones secundarias así como puntos de deterioro del material. Los áridos originales y la pasta de cemento, las zonas de interfase y el gel de álcali sílice producido debido a la presencia de áridos reactivos en el mortero adherido fueron analizados utilizando SEM y mapas-EDX.

Además del análisis macroscópico y microscópico, se estudió el comportamiento del hormigón con áridos reciclados frente a esfuerzos de cortante. La principal ventaja de dicho análisis frente a otros ensayos de carga como puede ser el de flexión reside en el hecho de que en este último la capacidad última del elemento estructural se encuentra definida en gran parte a partir del armado dispuesto a tal efecto.

Se fabricaron dieciséis vigas, fruto de la combinación entre cuatro cuantías de armado transversales y de cuatro tipos de hormigones (de diferentes porcentajes de árido reciclado). Se llevó a cabo una predicción analítica de los resultados experimentales utilizando un modelo numérico (Modified Compression Field Theory), y utilizando las normas AASHTO LRFD, CSA, Eurocode-2 y las expresiones propuestas en la instrucción Española EHE-99.

Se analizaron los compuestos orgánicos e inorgánicos liberados de los residuos por lixiviación. La lixiviación de las substancias fue determinada utilizando dos normas, la norma Holandesa (NEN 7341 y 7345) y la norma Europea (EN 12457-2).

Se dan algunas recomendaciones en cuanto a las características de los áridos para ser utilizados en hormigones, teniendo en cuenta las normas europeas de áridos reciclados. También se hacen sugerencias respecto al proceso de producción de hormigones con árido reciclado. Son analizadas las propiedades mecánicas del hormigón con árido reciclado y comparadas con las de un hormigón convencional.

Basado en la durabilidad del hormigón, se proponen algunas sugerencias en lo que concierne a la posible reacción álcali sílice entre el nuevo cemento y los áridos finos presentes en el mortero adherido del árido reciclado. También se determina que el empleo de áridos reciclados afecta en la resistencia de elementos estructurales sometidos a esfuerzos de cortante y que este depende del porcentaje de árido reciclado que se utiliza. Se demuestra la aplicabilidad del hormigón con árido reciclado en lo que concierne a su comportamiento ambiental. Para concluir se dan algunas sugerencias para futuras investigaciones que nos ayudarán en la evolución de nuestro entendimiento en este campo.

Contents

Chapter 1
Introduction

1.1 GENERAL ASPECTS

For thousands of years, the improvement of the quality of life has been the indicator of any developed society. This indicator has always been associated to the presence of elements and infrastructures, which facilitate the development of daily activities without taking into account the impact that they could have. History has taught us that society has made the recovery and use of rejected elements a habitual practice. Numerous civilisations have used and reused building materials of earlier civilizations or their own destroyed architecture (either through war or natural causes) to construct new buildings. The remains of ruined Romanesque churches supplied the stone for various farmhouses. In a similar form the Roman quarry was used to supply the stone for the Vatican Renaissance basilica.

Present day construction is derived from a legacy of quick and sophisticated processes and uses of materials initiated in the industrial revolution. An indiscriminate use of primary resources threatens “the dream” in which society has immersed itself: the technical evolution bears upon its shoulders a more responsible use of materials, guiding “the sleepwalkers” by hand towards a sustainable development idea.

The building industry has the ability to act in a bilateral manner. On the one hand it has to be considered as a clear generator of a great quantity of residues and on the other, its long tradition defines its capability of re-using not only its own waste but also the waste from other branches of industry. This capacity of recovery is precisely what is needed to promote in our current society.

Construction and demolition waste is not dangerous from an environmental point of view, the control of this becomes indispensable from the moment that statistics refer to the waste’s volume approaching an unsustainable level. According to information obtained from the Association of demolition companies in Europe, 1.6 kg of construction and demolition waste per habitant is generated daily, in comparison with the daily production of 1.1 kg of solid urban waste per habitant. From this information we can deduce that in Catalonia the annual quantity of construction and demolition waste is approximately of 3.600.000 tons.

The reutilization and recycling of construction and demolition waste is necessary, as it seems the only way to decrease it. Following the II world war certain countries for example Germany were interested in the recycling of “dumping materials”. Other countries, namely the Netherlands, Denmark, Belgium and a few French regions, all of which have a lack of granulated materials followed Germany’s example in the interest shown in waste material recycling.

The situation in Spain with its geographical make up and lacking the clear and necessary motivation shown by Germany has resulted in an evident delay in the use of recycled material at market level.

The trend of the utilisation of recycled aggregates is the solution to the problem of an excess of waste material, not forgetting the parallel trend of improvement of final product quality. The utilisation of waste construction materials has to be related to the application of quality guarantee systems in order to achieve suitable product properties.

The differing typology and quality of construction and demolition waste in each European State is also an added problem. A systematic use of waste materials involves the classification of the mentioned waste materials according to their strength capacity, durability and utility.

The successful recovery and recycling of construction waste should be conducted, in order to satisfy suitable future expectations, without assuming unnecessary risks and using the adequate processes available.

The ideal process should be as follows “to construct for demolishing, to demolish for recycling, to recycle for constructing”. If this process was strictly complied with residues would not be a future problem as their dimensions would be controlled and limited.

Present situation

Europe

Nowadays, the annual quantity of construction and demolition waste in The European Community is over 450 million tons (Symonds European Commission, 2000). If one excludes the soil, the amount of construction and demolition waste amounts to 180 million tons.

The basic components obtained from this type of waste are illustrated in table 1.1:

Table 1.1: Construction and demolition waste in European Community. Average values of European dates

Materials	% Of total weigh
Concrete, masonry, mortar	50
Wood	5
Paper, carton and other fuels	1-2
Plastic	1-2
Metals including steel	5
Soil of excavations, gravel of pavements	20-25
Asphalt	5-10
Muds and other non-fuels	5-10

The European Community states (2000) defined the European catalogue of residues (CER) as an initiative towards a methodology of recycling. Different typologies of materials are presented according to their condition. Actually very few European countries achieve the separation of residues illustrated in the catalogue. However in countries where selective demolition is used, for example in the Netherlands, they obtain a very high level of recycling.

The actual situation of different European Community members is shown in table 1.2. Denmark is one of the countries with better management of construction and demolition waste. Spain is more or less the same as Ireland. A clear trend of common performance between countries does not exist.

Table 1.2: Nations with aptitudes to manage construction and demolition wastes.
D: Germany, BE: Belgium, ÖS: Austria, DK: Denmark, FI: Finland, IR: Ireland, LX: Luxembourg

Code	**Description**	**Estates members**						
		D	**BE**	**ÖS**	**DK**	**FI**	**IR**	**LX**
17 00 00	Construction and demolition waste	√	√	√	√	√	√	√
17 01 00	Inert concrete, brick, tile and ceramic material	√	√	√		√		√
17 01 01	Concrete (included reinforced concrete)				√			
17 01 03	Tiles and ceramics				√			
17 01 05	Construction asbestos material			√				
17 02 01	Wood		√		√	√		
17 02 02	Crystal				√			
17 02 03	Plastic		√		√			
17 03 00	Asphalt, Tar, product tarring	√	√	√	√			
17 04 00	Metals		√	√		√		
17 05 01	Soil	√		√	√			
17 07 00	Mixture of demolition and construction waste	√	√	√	√	√		

In order to achieve the recycling of construction and demolition waste, four points must be considered:

1- The dumping of construction and demolition waste must be strictly regulated, strong measures being taken in the form of fines for illegal dumping.
2- The owners of construction and demolition waste must pay reasonable tariffs for waste dumping, although the materials being dumped are not considered dangerous. None classified waste incurring higher tariffs in order to prevent pollution and to discourage the mixture of wastes.
3- Classification and crushing services must be available for the treatment of the inert part of the construction and demolition waste.
4- The acceptance of the potential users of recycled aggregates as secondary raw aggregates must be promoted and there must be no discrimination based on the origin of the aggregates.

The promotional aims of recycled aggregates are:

1- The aim of the 1999/3/EEC Directive is the restriction of waste dumping. Some member countries of the European Community have started the rigorous application of the instructions. The Netherlands has not dumped no-recoverable material since 1997; or Germany, which does not allow the dumping of mixed waste.
2- The selection of the waste is fundamental to guarantee the homogeneity of the material for recycling.

The European Community has tried to look for a common aim for all state members, although these aims can be established at a national, regional or local level. Germany takes as its aim the reduction between 1995 and 2005 of half of its waste materials by recycling. Denmark already recycles 85% of its waste materials. Dublin places the recycling of its waste materials at 82% by 2004.

In order to introduce recycled materials on to the market, the states must give reassurance against the discrimination of these materials. The technical regulation applied to recycled materials must be equal to those applied to raw materials. Therefore with respect to Spain the aggregates used in structural concrete must have the properties required by EHE-99 (Spanish structural concrete code). The products using recycled aggregates must be seen as new products, and they must follow the same technical procedures as the raw materials in order to guarantee their use.

The studies of the European Committee for Normalisation (CEN), must concentrate their attention on the behaviour of the final product: in its constructional usage and not in its composition.

The present situation in Catalonia

Construction and demolition waste production has increased significantly in Catalonia, in unison with the increase of construction activity. The development of the increase usage of construction and demolition waste depends on the fluctuations of the economy. An annual production is estimated at about four million tons per year for the period 2001-2006. In June 2001 the Catalonia government proposed a new recycling construction waste management scheme for the six years, between 2001-2006.

The objective is to increase from 4 recycling plants, to 11 in 2003 and 18 in the year 2006, with a percentage of recycling estimated at 25 % and 50 %, respectively.

A significant portion of recycled aggregates is used in road construction, however recycled aggregates could be used in mixed concrete and concrete products. Code and technical specification writers need therefore to be convinced that the application of recycled aggregates in concrete is technically feasible and without risk, if the appropriate preventive measures are taken.

The basic components obtained from construction and demolition waste in catalonia are illustrated in table 1.3.

Table 1.3: Construction and demolition waste in Catalonia

Materials	**% Of total weigh**
Concrete, masonry, mortar	54
Wood	5
Paper, carton and other fuels	3
Plastic	3
Metals including steel	2
Soil of excavations, gravel of pavements	22
Asphalt	7
Mud and other non-fuels	4

1.2 OBJECTIVES

The objectives of this thesis are to analyse and give technical specifications on structural strength, durability and environmental behaviour of concrete made with recycled aggregates.

Recycled aggregates are obtained from the waste crushed concretes. From a quality point of view, the original aggregates are heterogeneous in composition being derived from different minerals and adhered mortar. The properties of recycled aggregates must be determined if the aggregates are to be used in concretes, therefore an attempt will be made to give recommendations as to the aggregates characteristics to be employed in concrete mixes, taking into account the European standards for recycled aggregates.

Four different concretes were produced with different percentages of recycled aggregates. Concrete made with 0% of recycled aggregates (HC), concrete made with 25% of recycled coarse aggregate (HR25), concrete made with 50% of recycled coarse aggregates (HR50) and concrete made with 100% of recycled aggregates (HR100). Concretes made with recycled aggregates are compared in all aspects with conventional concrete (HC). Coarse aggregate was the only recycled material employed in the study. The use of fine recycled aggregate could result in a loss of durability and resistance. According to the state of the art (chapter 2), test elements of concrete made with recycled coarse aggregates obtain acceptable results. However in its bibliography few references exist with respect to the structural behaviour of recycled aggregate concrete. Japanese investigators were the first to test the use of recycled aggregate at the end of the nineties.

One of the main objectives of this thesis is to analyse the structural behaviour of concretes made with different percentages of recycled coarse aggregates. Another is to break down the barriers that exist against the use of recycled aggregates in concretes. Up to now only one structure a footbridge (Barcelona, Forum 2004) using 20% of recycled aggregates has been built in Spain. The beam specimens were cast with different amounts of web reinforcement and different percentages of recycled aggregate. They were tested to shear stress to identify the behaviour of the beam specimens and the influence of these parameters.

Flexural strength of beam specimens can be approximately determined by mechanical properties obtained from test elements. Concrete strength has little influence with respect to flexural strength, therefore it is sufficient to assure the compression strength. However, it is not the same when shear stress exists. In this case crack-friction, tensile strength or loss of anchorage by insufficient bond, influence the structural strength and they are unknown in these kinds of concretes. In order to make a precise study of shear stress, it will be necessary to obtain additional information: displacements, appearance and evolution of the cracks, bond between the concrete and reinforcement, and also the flexural strength if shear failure is produced.

An analytical prediction of the experimental results are carried out using a numerical model (Modified Compression field Theory, developed in Toronto, Canada), and also by some codes based on this model, AASHTO LRFD and CSA. The predictions of beam specimens failure are also calculated by The Eurocode-2, as well as by the expressions proposed in the Spanish structural concrete code EHE-99. The approach of analytical failure predictions to reality behaviour will be analysed. Most material and construction product codes do not encourage the use of recycled materials on construction sites. In this thesis the influence of recycled aggregates in concrete will be defined and some recommendations will be given to its utilisation. As the CEN states, the instructions for use will have to concentrate on the behaviour of the final product and not on its composition.

Another objective of this thesis is to define the macrostructure and microstructure differences of concrete made with different percentage of recycled coarse aggregates (0%, 25%, 50% and 100%) and their durability behaviour. It is based on aggregate distribution, distribution and amounts of pores in the cement paste, effective w/c ratio, the interfacial transition zone and the composition of recycled aggregates employing the optical stereo light microscope Leica MZ6, the optical microscope transmitted light microscope Leica Leitz DM-RXP and Scanning electronic microscope (SEM and EDX-maps).

Another objective of the thesis is to measure the environmental behaviour of this material, based on the leaching capacity of waste material in its life. The leaching materials are measured by Dutch and European methods.

1.2 STRUCTURE OF THE THESIS

The structure of the thesis is based on different analysis carried out during research work. As mentioned above the objective is to analyse the structural, durability and environmental behaviour of recycled aggregate concrete.

In chapter 2 the state of art of recycled aggregate concretes is presented and also the analytical methods to determine the failure value of beam specimens caused by shear stress.

In the next chapters the experimental works of this thesis are shown. In chapter 3, properties of recycled aggregates and concretes made with recycled aggregates are represented. Four concretes were produced with different percentages of recycled aggregates. Conventional concrete with 0% of recycled aggregate concrete (HC), concrete made with 25% of recycled aggregate (HR25), concrete made with 50% of recycled aggregate (HR50) and concrete made with 100% of recycled aggregate concrete (HR100). The four concretes had the same compression strength with different dosages.

In chapter 4, the microstructure of four concretes is analysed. The influence of recycled aggregates in the durability of the concrete is explained. And attempts will also be made to provide recommendations regarding their application.

In chapter 5, the test method of reinforced beams specimens is described. The beam specimens were tested on shear stress.

In chapter 6, the results and discussion of structural behaviour of beam specimens with four different web reinforcements and four different types of concretes are given. The results carried out are based on percentages of recycled aggregates and the distribution of transversal reinforcement. The results of each beam are presented in annex A.

In chapter 7, the environmental behaviour of these concretes is illustrated. The four concretes studied are compared with the maximum values required by Dutch and European Codes.

In chapter 8, the general conclusions obtained for experimental work are presented and future research is proposed as a continuation of this work.

Chapter 2

State of the art

2.1 INTRODUCTION

In recent years certain countries have considered the reutilization of construction and demolition waste as a new construction material as being one of the main objectives with respect to sustainable construction activities. This thesis focuses on recycling of concrete waste as an aggregate in structural concrete. From the mid 70s many researchers have dedicated their work to describe the properties of these kinds of aggregates, the minimum requirements for their utilisation in concrete and the properties of concretes made with recycled aggregates. However, minor attention has been paid to both the structural behaviour of recycled aggregate concretes and their durability.

Topics addressed in this chapter include:

- Properties of recycled aggregates
- Recommendations about their applicability
- Mechanical properties of recycled aggregates concrete (RAC)

- Durability of RAC
- Structural behaviour of concrete made with recycled aggregates
- Shear Theory
- Analytical design method about this phenomena
- Code reviews

2.2 AGGREGATES OBTAINED FROM RECYCLED CONCRETE

According to European recommendations, *RILEM-1989, DIN 4226-100, 2000* and *prEN 13242:2002 (final Draft),* the recycled aggregates are classified according to their composition.

RILEM-1989 proposed the classification of recycled coarse aggregates into the three following categories:

- Type I: aggregates that are implicitly understood to originate primarily from masonry rubble.
- Type II: aggregates that are implicitly understood to originate primarily from concrete rubble.
- Type III: aggregates that are implicitly understood to consist of a blend of recycled aggregates and natural aggregates. As minim of 80% of raw material and a maxim of 10% of type I.

In accordance with *DIN 4226-100, 2000,* diverse types of materials are defined according to their composition:

- Type 1: Gravel of concrete / Sand of concrete.
- Type 2: Gravel of construction / Sand of construction
- Type 3: Gravel of masonry / sand of masonry
- Type 4: Gravel of mixture / Sand of mixture

In accordance with *prEN 13242:2002 (final Draft)*, there is no classification of different types of materials according to their composition. The mechanical or physical properties of recycled aggregates are defined in categories.

According to RILEM and DIN 4226-100, the properties of recycled aggregates obtained by crushed concrete and used in concrete production are Type II and Type 1 respectively.

2.2.1 Adhered paste and mortar

In recycled aggregates used and analysed in Europe, the adhered mortar and paste are always present. In Japan some researches developed a method based on "cyclite", which grinds crushed concrete lumps together using a newly-developed processing method, consisting on the removal of mortar from the aggregates surface without crushing the aggregates (Takenaka Corporation, 1999).

The main factors which influence the quantity of adhered mortar in recycled aggregate crushed (with the same machine and power) are as follows: a) water/cement ratio b) original concrete strength and c) Aggregate size. The grinding process has an influence on the amount of adhered mortar and the quality of recycled aggregates.

a) <u>*Water/cement ratio.*</u> The water/cement ratio of the original concrete influences the amount of adhered mortar to original aggregates when the concrete is crushed with the same grinding machine and employing the same power.

The quantity of adhered mortar increases with the decrease of the size of the aggregate (with the same machine and same power), see figure 2.1 (Hansen and Narud, 1988 and Hedengaard, 1981) .

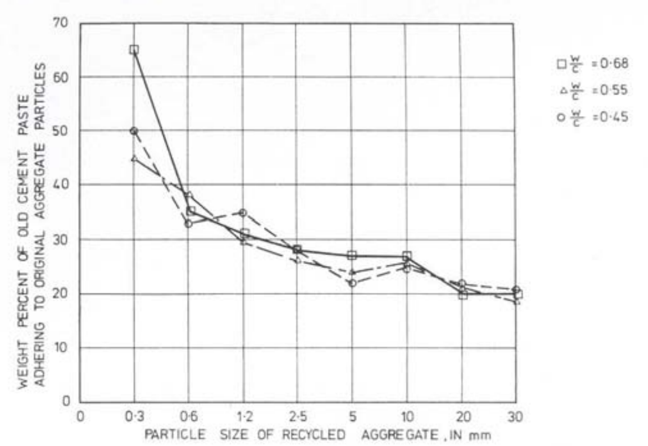


Fig.2.1: Weight percent of cement paste adhering to original aggregate particles in recycled aggregate produced from original concretes made with different water-cement ratios. BCSJ, 1978

b) <u>*Strength of original concrete*</u>. The quantity of adhered mortar in the original aggregate is proportional to the strength of the original concrete (Hasaba, 1981), when the crushed concrete is ground with the same type of machine and the same energy is applied.

Takeshi Yamato, Masashi Soeda (2000), stated that the recycled aggregates which originated from the low strength concrete had less mortar adhered than when the different strength concretes were ground with the same grinding machine and employing the same power.

2.2.2 Properties of recycled aggregates obtained from crushed concrete

Density

In general, the saturated surface density (SSD) of recycled aggregates is lower than that of natural aggregates, due to the low density of the mortar that is adhered to the original aggregate. It depends on the following:

a) *Strength of original concrete*. Nagataki (2000), concludes that with the same quantity of mortar, a recycled aggregate that has been obtained from a concrete of higher strength will have a higher density.

b) *Size of aggregate*. The saturated surface density (SSD) of the aggregates depends on their quality. The aggregates with a higher amount of adhered mortar will have a lower density. According to Hansen (1985) the density changes with the size of the aggregate when concrete is ground with the same grinding machine employing the same amount of energy in the grinding process. Many studies report on intervals of densities (2290 to 2490 kg/m^3) depending on the size of the aggregates. The density (SSD) of recycled aggregate concrete reduces with smaller sizes of aggregates.

As mentioned previously the SSD density will not only depend on original concrete strength but also on the kind of crushing or grinding machine employed and the energy used. Hansen y Narud (1983), Hasaba et al (1981) and BCSJ (1978).

In order to calculate the density of the recycled aggregate the same standard as that of raw aggregates is used, namely the ASTM C-127 standard (Standard Test Method for Specific Gravity and Absorption of Coarse (or Fine) aggregate). In Spain the UNE-EN 1097.6 Code "Ensayos para determinar las propiedades mecánicas y físicas de los áridos. Determinación de la densidad y absorción". According to EHE-98 (Spanish Instruction for structural concretes) the aggregates must have major density to 2000 kg/m^3. The same value is required by RILEM and DIN 4226-100 for recycled aggregates which are derived from crushed concrete.

Water absorption

The water absorption capacity of the recycled aggregate in the mixture, represents one of the main differences between recycled and raw aggregate. It is reported to depend on:

a) *Size of aggregate*. The capacity of absorption of the aggregate increases with its size. The smaller size aggregates having a greater water absorption capacity.

- Hansen y Narud (1983) found that the absorption capacity of recycled aggregate increased with a higher amount of adhered mortar. The high amount of adhered mortar in recycled aggregate also produced a decrease in density. The relationship between the original concrete strength, recycled aggregate size, amount of adhered mortar, density and absorption of recycled aggregates is shown in table 2.1.
- They found the percentage of absorption capacity related to the dimensions of the recycled aggregate to be the following: 4-8 mm- 8.7 %, 16-32 mm- 3.7% and 9.8% for fine recycled aggregate. This value was obtained by crushing concrete with a 0.7 of w/c ratio.
- Hasaba (1981) defined an absorption of 7 % for a size of 25-5 mm
- Japanese investigators report (1978) established it between 3.6 and 8 %.

In all cases it was accepted that absorption capacity was not dependent on the strength of the original concrete.

Table 2.1: Properties of natural gravel and recycled aggregates according to Hansen and Narud (1983).

Type of Aggregate	Size Fraction in mm	Specific Gravity SSD cond.	Water Absorption in percentage	Los Angeles Abrasion Loss Percentage	Volume percent of mortar attached to natural gravel particles
Original natural gravel	4- 8 8- 16 16- 32	2500 2620 2610	3.7 1.8 0.8	25.9 22.7 18.8	0 0 0
Recycled aggregate (H) (w/c = 0.40)	4- 8 8- 16 16- 32	2340 2450 2490	8.5 5.0 3.8	30.1 26.7 22.4	58 38 35
Recycled aggregate (M) (w/c = 0.70)	4- 8 8- 16 16- 32	2350 2440 2480	8.7 5.4 4.0	32.6 29.2 25.4	64 39 28
Recycled aggregate (L) (w/c = 1.20)	4- 8 8- 16 16- 32	2340 2420 2490	8.7 5.7 3.7	41.4 37.0 31.5	61 39 25
Recycled aggregate (M) (w/c = 0.70)	< 5	2280	9.8	-	-

b) Quantity of adhered mortar. There is a relationship between absorption and amount of adhered mortar.

- Ravindrarajah (2000) demonstrated with 15 samples, that the average value of the water absorption in recycled aggregate was 6.35 %, whereas in raw aggregate it was 0.90 %. The absorption capacity of recycled aggregates depends on the quantity and quality of adhered mortar.

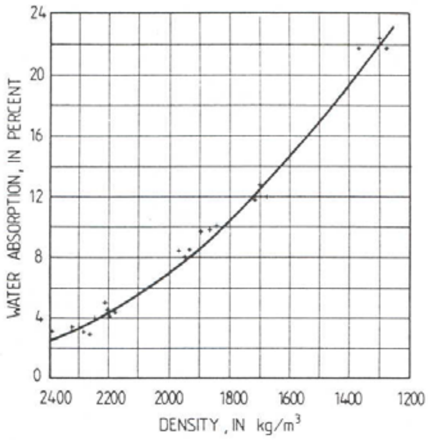


Fig.2.2: Water absorption depending on the density of recycled aggregate, Kreijger 1983

c) Density. There is dependence between density and absorption capacity. Recycled aggregates with adhered mortar have lower density and higher absorption capacity. There are several authors who tried to find a correlation:

- Kreijger (1983) found a parabolic relation between the absorption and density of the recycled aggregate, as shown in figure 2.2.

According to Sanchez de Juan M. (2002), of the 11 samples of recycled aggregates taken from a recycling plant in Madrid, 4 had an absorption capacity of more than 7 % and all of them had an absorption capacity of more than 5%. It was concluded that due to the aggregates' high absorption capacity, concrete should be produced with a maximum amount of 20% of recycled aggregates. Furthermore, due to the high absorption capacity of recycled aggregates, the raw aggregates' absorption capacity is also a requirement for concrete production in order to maintain the EHE requirements of 5% for aggregates maximum absorption capacity for structural concrete production.

According to Gonzalez B. (2002), the absorption capacity of 6-12 mm and 12-25 mm of recycled coarse aggregates obtained by grinding crushed concrete in a recycling plant in Barcelona, is 4,82% and 4,59% respectively, consequently they can be employed in concrete production according to EHE.

The absorption in aspects of production

Prior to the utilization of recycled aggregates in concrete production, the density and absorption capacity of recycled aggregates must be known. ASTM-127 standard and

UNE-EN 1097-6 standard are used to determine them. The EHE-99 (Spanish Instruction of structural concrete) requires that the absorption capacity should not be higher than 5 % for conventional concretes. Probably, this value must be modified with respect to recycled aggregates concretes.

According to the Japanese norm, "*Recycled Aggregate and Recycled Aggregate Concrete*" (1977), neither recycled coarse aggregate with an absorption capacity higher than 7% or fine aggregate with an absorption capacity higher than 13% should be used in concrete production.

Due to the high water absorption capacity of recycled aggregates, the use of presoaked aggregates is suggested in the production of recycled aggregate concrete in order to maintain a uniform quality during concrete production. Some researchers suggest that the aggregate should have an average absorption capacity of 5 % as a maximum in 30 % of the aggregates employed.

M. Barra (1998), uses the effective coefficient of absorption to analyze the absorption of the recycled aggregate. Variation of the water content does not occur in fresh mortar due to the low effective coefficient of absorption of the raw aggregates. In the case of recycled aggregate it is different, recycled aggregates have a much larger effective absorption capacity and it can affect the workability of fresh concrete. The effective absorption capacity is also variable depending on the order of concrete production.

Los Angeles Abrasion

With respect to recycled aggregates the value of Los Angeles abrasion changes depending on the strength of the original concrete, the amount of adhered mortar and the original aggregate quality. Concrete with a higher strength suffers less abrasion. In recycled aggregates the "Los Angeles Abrasion loss percentage" depends not only on the amount of adhered mortar in the original aggregate but also in the form that the original concrete is crushed.

According to the Los Angeles Abrasion test recycled aggregates obtained by grinding a 40 MPa strength concrete have lower abrasion than aggregates obtained by 16 MPa strength concrete. As in table 2.2 and 2.3 it is shown.

The crushing machine and the power employed in the crushing used by each researcher is unknown, so we have to be careful in comparing the aggregates of different authors.

Table 2.2: Los Angeles Abrasion Loss percentage of recycled aggregates obtained by grinding 40 MPa Strength Concrete.

	Recycled aggregates LA abrasion according to several investigators					
	Hansen and Narud (1983)			**Hasaba (1981)**	**Japanese Investigator (1978)**	**Yoshikane (2000)**
Size Fraction	4-8 mm	8-16 mm	16-32 mm	5-25 mm	(*)	5-13 mm
Los Angeles Abrasion Loss Percentage	30.1	26.7	22.4	23.0	25.1-35.1	20.1

(*) For recycled aggregates, according to 15 different concretes crushed by different way.

Table 2.3: Los Angeles Abrasion Loss percentage of recycled aggregates obtained by grinding 16 MPa Strength Concrete.

	Recycled aggregates LA abrasion according to several investigators				
	Hansen and Narud (1983)			**Hasaba (1981)**	**Yoshikane (2000)**
Size Fraction	4-8 mm	8-16 mm	16-32 mm	5-25 mm	5-13 mm
Los Angeles Abrasion Loss Percentage	41.4	37.0	31.5	24.6	28.7

(*) For recycled aggregates, according to 15 different concretes crushed by different way.

Los Angeles abrasion is higher when the strength of the original concrete is lower due to the lower strength of adhered mortar.

According to ASTM C-33 standard "*Standard Specification for Concrete Aggregates"*, the aggregates will be valid to use in concrete production if the loss determined by the "Los Angeles Abrasion test" is less than 50 %. According to the British Code 882, 1201, report 2 1973, the loss according to the "Los Angeles Abrasion test" must not exceed 45 %. According to EHE-98 (Spanish Structural Concrete Code), the gravel's resistance to the "Los Angeles Abrasion test" must be less than 40 %. It will be determined by Code EN 1097-2.

The “Los Angeles Abrasion test” does not represent the real situation undergone by the aggregates during their integrated life within the concrete, therefore it is an unsuitable test for raw aggregates. However, in the case of recycled aggregates the Los Angeles abrasion test is appropriate in order to verify if the crushed concrete is or not damaged by fire.

Sulphate Soundness

The sulphate soundness guarantees the aggregates’ resistance to freezing and thawing cycles. The percentage of loss of weight of recycled aggregates exposed to sulphate solution depends to a great extent on the composition of the tested aggregates, as well as the type of original concrete and method of crushing.

- BCSJ (1978) verified that the loss of weight after five cycles changed from 18.4 to 58.9 % with respect to coarse recycled aggregates. This was analysed employing 15 different concretes, of different strengths and crushed with different methods. In the same analysis using fine recycled aggregates values were obtained from 7.4 to 20.8 %.
- Fergus (1981) found the loss of sulphate weight to be between 0.9 to 2.0 % with respect to coarse recycled aggregates, and 6.8 to 8.8 % with regard to fine recycled aggregates.

The uncertainty of the recycled aggregates aptness for concrete production in terms of durability has become clear from consulting the conclusions of several authors:

- American results indicate that the recycled aggregates generally improve in time with respect to the raw aggregate. Japanese investigators report’s mention that the opposite results might be true.
- However, Kasai (1985) concludes that the sulphate soundness test is unsuitable for evaluation of the durability of recycled concrete aggregates. As durability properties of exposed recycled aggregate concretes are of utmost practical importance, it is suggested that additional studies be made of the durability characteristics of recycled aggregates versus original aggregates.

ASTM C-33, “*Standard Specification for Concrete Aggregate”* limits the loss of weight of the aggregates submitted to five cycles of treatment with solutions of 15% of sulphate magnesium for fine aggregates and 18 % for coarse aggregate. The Spanish code EHE-98 requires the same quantities (test method is UNE EN 1367-2:99).

Contaminants

The presence of contaminants in recycled aggregate influence the strength and durability of concrete made with these aggregates. Nowadays recommendations exist with respect to the limitation of the diverse components which can be presented in recycled aggregates.

BCSJ (1978) and Mukai (1979) introduced the percentage of the contaminants which can produce a reduction of 15 % in compressive strength (table 2.4).

Table 2.4: % de contaminants for a reduction of 15% in the compression strength. BCSJ, 1977

Impurities	**Gypsum**	**Soil**	**Wood**	**plaster**	**Asphalt**	**Paint**
Percentage in aggregate	7	5	4	3	2	0.2

It is imperative to emphasize that the compression strength of concrete which employs 3% of plaster is 15% lower than that without plaster. If the cured concrete follows a humid route it obtains a reduction of 50 %. Since the plaster is softer and weaker in water. Sulphate resistant Portland cement should be used to produce concrete with contaminated recycled aggregate containing plaster or gypsum. However, the aggregates must be cleaned before they are used, which could result in an expensive and difficult production of recycled aggregates. Selective Demolition could be seen as an effective alternative.

According to the results of the Building Contractors Society of Japan (BCSJ, 1977) contaminant's limitations appear in table 2.5.

The possible effects found in the recycled aggregate concrete derived from different pollutants is described in table 2.6 (Hansen, 1985).

Table 2.5: Limitation of contaminants in the recycled aggregates. BCSJ, 1977

Type of aggregate	**plaster, clay and other Impurities of density<1950 Kg/m^3**	**Asphalt, Plastic, Paint, fabric, paper, wood and materials with similar density<1200 Kg/m^3**
Coarse recycled	10 Kg/m^3	2 Kg/m^3
Fine recicled	10 Kg/m^3	2 Kg/m^3

Chlorides

It is recommended that standard specifications for recycled aggregate and recycled aggregate concrete should impose stringent limits on chloride contents of such aggregates and concretes. However, the threshold chloride concentration, below which there is no risk of reinforcement corrosion, remains a controversial issue. It may be concluded that in many cases new concrete produced from recycled coarse aggregate derived from old chloride-contaminated concrete would fail to meet current recommended limits for chloride content in concrete.

Table 2.6: Relation of contaminants in recycled aggregates (Hansen, 1985)

Contaminant	Possible effect in the mix	Recommendation
Bituminous	-Considerable Diminution of the resistance of the concrete.	- Elimination of the material before recycling. - In any case, limit to 1% of asphalt in the aggregates.
Gypsum	-Damage by sulphate expansion	- SR Portland cement should be used.
Organic sulphate	- Decrement of strength	- Limitation at 2 Kg/m^3 of lightweight particles with of density<1200 Kg/m^3
Chlorides	-Reinforcement Corrosion	
Chemical and minerals additives	-No apparent effect.	
Soil	-No apparent effect.	
Metals	-Its presence is not common due to facility of removing	
Crystal	-Possible Alkali-Silika reaction	
Brick and lightweight concrete	-Effect in Durability.	- ≥5% of weight of recycled aggregates, as masonry aggregates are considered.
Aggregates damage by fire	-Out of law with respect to abrasion LA and loss of weight for sulphate.	- There are no present studies.
Susceptible aggregates to freezing and thawing	-Possible improvement to freezing strength	
Reactive aggregates to alkali	-Studies do not exist	
Chemical and radioactive substances	- Studies do not exist	- The use of aggregates from chemical plants is not permitted.

2.2.3 Recommendations of recycled aggregates

Due to the recycled aggregate's properties and heterogeneity certain recommendations have to be followed for their utilisation in new construction material.

Recommendations RILEM, 1989

Recycled coarse aggregates are classified into three categories (see section 2.2.).
To define the typologies, the requirements of each type of aggregates are established in table 2.7. In order to use the recycled aggregates in concretes, the physical properties described in table 2.8 have to be known.

According to RILEM, for the application of the recycled aggregates in the production of concrete, besides fulfilling all the specifications that have been defined in table 2.7 and 2.8, the provisions defined in the table 2.9 must also be considered.

Table 2.7: Mandatory requirement by RILEM

Mandatory Requirements	**Type I**	**Type II**	**Type III**	**Test method**
Min. dry particle Density (Kg/m^3)	1500	2000	2400	ISO 6783 & 7033
Max. water Absorption (% m/m)	20	10	3	ISO 6783 & 7033
Max. Content of material with SSD < 2200 Kg/m^3 (% m/m)	-	10	10	ASTM C123
Max. Content of material with SSD < 1800 Kg/m^3 (% m/m)	10	1	1	ASTM C123
Max. Content of material with SSD < 1000 Kg/m^3 (% m/m and %v/v)	1	0.5	0.5	ASTM C123
Max. Content of foreign materials (metals, glass, soft material, bitumen) (% m/m)	5	1	1	Visual
Max. Content of metals (% m/m)	1	1	1	Visual
Max. Content of organic material (% m/m)	1	0.5	0.5	NEN 5933
Max. Content de filler (< 0.063mm) (% m/m)	3	2	2	PrEN 933-1
Max. Content of sand (< 4mm) (% m/m)	5	5	5	PrEN 933-1
Max. Content of sulfate (% m/m)	1	1	1	BS 812, part 118

Table.2.8: Properties of the recycled material which have to comply to any standard by RILEM

Properties of the recycled material which have to comply to any standard
Grading
Static strength
Form index
Abrasion value
Chloride content
Iron and vanadium content for clean concrete applications
Pop-out potential (Ca, Fe content)
Content of swelling clay
Frost resistance

Table 2.9: Provisions for the use of recycled concrete according to RILEM

Provisions for the use of recycled concrete			
Recycled aggregates	**RCAC Type I**	**RCAC Type II**	**RCAC Type III**
Max. allowable strength class	C16/20	C50/60	No limit
Additional testing required when used in exposure class 1[b]	None	None	None
Additional testing required when used in exposure classes 2[a], 4[a]	ASR Expansion test Use in class 4[a] not allowed.	ASR expansion test	ASR expansion test
Additional testing required when used in exposure classes 2b, 4b	Use in classes 2b, 4b not allowed	ASR expansion test. Bulk freeze-thaw test.	ASR expansion test. Bulk freeze-thaw test.
Additional testing required when used in exposure class 3	Use in class 3 not allowed.	ASR expansion test. Bulk freeze-thaw test. Deicing salt test	ASR expansion test. Bulk freeze-thaw test. Deicing salt test

DIN 4226-100, 2000 (Deutsches Institut für Normung)

In September, 2000, requirements for recycled aggregates were described for their application in concrete and mortar. The requirements are also included for the quality certificate and a system to assure a suitable internal control.

The recycled mineral aggregates have to fulfil the requirements of the DIN 4226-1, unless other codes are established.

Diverse types of materials are defined depending on their composition (see section 2.2.). The composition of the material has to be determined by a representative sample. A minimum of 1000 g of a representative sample has to be used with respect to aggregates less than 8 mm in size and a minimum of 2500 g with aggregates larger than 8 mm.

Table 2.10: Composition of the material of the different types according to DIN 4226-100

Elements		**Composition of the material % Proportion in mass**			
		Type 1	**Type 2**	**Type 3**	**Type 4**
Concrete, mineral aggregates in conformity with E DIN 4226-1		≥ 90	≥ 70	≤ 20	≥ 80
Clinker, brick without pores		≤ 10	≤ 30	≥ 80	
Calcareous sandstone				≤ 5	
Other mineral contents [a])		≤ 2	≤ 3	≤ 5	≤ 20
External contents	**Asphalt**	≤ 1	≤ 1	≤ 1	
	Mineral [b])	≤ 2	≤ 2	≤ 2	
	No mineral [c])	≤ 0,5	≤ 0,5	≤ 0,5	≤ 1
[a]) Other mineral contents are for example: Brick with pores, light concrete, concrete with pores, concrete with pores obtained from rubble, stucco, cement, dross with porous, pumice [b]) External mineral contents are for example: Glass, ceramics, Metal dross NE-, plaster of stucco [c]) External contents not minerals are for example: Rubber(gum) (rubber), plastic artificial matter, metal, wood, remains of plants(floors), paper(role), other materials					

The applicability of the material is determined for:

- Composition (see Table 2.10).
- Physical properties of the aggregates. Minimal densities and the maximum absorption needed for recycled aggregates in order to be used in concrete and mortar (see in table 2.11).
- Fine quantity. The content of fine parts is measured by DIN EN 933-1. Depending on the fine quantity, the aggregates are classified into categories (see table 2.12).
- Quantity of chlorides. The content of acid chlorides in recycled mineral aggregates has to be decided in agreement with each national standards. The value of acid or salt chlorides of types 1, 2 and 3 cannot pass the value of 0.04 % ($ACL_{0,04}$) of the aggregate's mass.

  The value of acid chlorides of the type 4 cannot pass the value of 0.15 % ($ACL_{0,15}$) of the aggregates' mass.

In addition the Code DIN 4226-100, a quality certificate for aggregates is required. The manufacturer has to assure that the aggregates fulfill the values demanded by the code (table 2.13). Production control according to DIN 18200, and the quality certificate will be provided after the external controller verification.

Table 2.11: Density of the particles and water absorption after 10 minutes for minerals recycled aggregates by DIN 4226-100

	Recycled mineral aggregates			
	Type 1	**Type 2**	**Type 3**	**Type 4**
Minimum density of the particles kg/m^3	2000		1800	1500
Value of the fluctuation of the density of the particles kg/m^3	±150			Without requirements
Maximum water absorption after 10 minutes % content in mass	10	15	20	Without requirements

Table 2.12: Categories for maximum values of fine content according to DIN 4226-100

Minerals aggregates	Maximum fraction which pass throw the sieve of 0,063 mm % Percentage of the fraction	Categories f
Coarse mineral aggregate	1,0 1,5 3 4	$f_{1,0}$ $f_{1,5}$ f_3 f_4
Mixture of gravels	2 11	f_2 f_{11}
Fine minerals aggregates (sand)	4 10 16 22 none requirements	f_4 f_{10} f_{16} f_{22} f_{NR}

Table 2.13: Requirements of the Code

Characteristic	Paragraph	Requirements for types 1, 2, 3	4
Denomination of sieve group	E DIN 4226-1:1999-12, 4.2	-	
Composition of the gravel **Coarse mineral aggregates with D/d ≤ 2 or D ≤ 11,2**	E DIN 4226-1:1999-12, 4.3.2.	G_{D85}	G_{D80}
Coarse mineral aggregates with D/d > 2 y D > 11,2	E DIN 4226-1:1999-12, 4.3.2.	G_{D90}	
Fine mineral aggregates	E DIN 4226-1:1999-12, 4.3.3.	Tolerancia de acuerdo con tabla 4	
Mixture of gravels	E DIN 4226-1:1999-12, 4.3.5	G_{D90}	G_{D85}
Shape of the grain	E DIN 4226-1:1999-12, 4.4	SI_{55}	
Resistance against fragmentation	E DIN 4226-1:1999-12, 5.2	LA_{NR} o SZ_{NR}	
Resistance against wear of coarse mineral aggregates	E DIN 4226-1:1999-12, 5.4	$M_{DE}NR$	
Resistance against polished	E DIN 4226-1:1999-12, 5.5.1	PSV_{NR}	
Resistance against abrasion	E DIN 4226-1:1999-12, 5.5.2	AAV_{NR}	
Resistance against graze of studded tires	E DIN 4226-1:1999-12, 5.5.3	A_NVNR	
Resistance against frost	E DIN 4226-1:1999-12, 5.8.1	F_{NR}	
Resistance against frost and thaw with salt	E DIN 4226-1:1999-12, 5.8.1	MS_{NR}	
Volumetric stability	E DIN 4226-1:1999-12, 5.8.2	None requirements	
Fine parts **Fine mineral aggregates**	4.5	f_{10}	f_{16}
Coarse minerals aggregates	4.5	f_3	f_4
Chlorides	4.6	$ACl_{0,04}$	$ACl_{0,15}$
Soluble sulphate in acid	E DIN 4226-1: 1999-12, 6.3.1	$AS_{0,8}$	None requirements

prEN 13242:2002 (final Draft)

This European Standard specifies the properties of aggregates obtained by processing natural, manufactured or recycled materials for hydraulically bound and unbound materials for civil engineering work and road construction.

Regarding geometrical requirements the aggregates general grading requirements are given by dividing them into different categories. It also divides into different categories the overall limits and tolerances for coarse aggregate at mid-size sieves.

This final draft described the requirements for aggregates with large applicability. In this thesis the recycled aggregates are used in concrete production. The original concrete is crushed employing an impact crusher, usually the quality of the aggregates obtained by the impact crusher enables the mentioned aggregates to be used in concrete production. Therefore, it is not considered necessary to expose all the values of this draft.

On the other hand the shape of coarse aggregates must be determined by EN 933-4 and the values described by this draft are defined in table 2.14.

Table 2.14: Categories for maximum values of shape Index

Shape index	Category (Sl)
≤ 20	Sl_{20}
≤ 40	Sl_{40}
≤ 55	Sl_{55}
> 55	$Sl_{Declared}$
No requirement	Sl_{NR}

This Draft separates the material into different categories depending on the mass fraction of crushed or broken particles, as table 2.15 shows.

Table 2.15: Categories for percentage of crushed or broken particles and totally rounded particles in coarse aggregates

Mass fraction of crushed or broken particles %	Mass fraction of totally rounded particles	Category (C)
90 to 100	0 to 3	$C_{90/3}$
50 to 100	0 to 10	$C_{50/10}$
50 to 100	0 to 30	$C_{50/30}$
-	0 to 50	$C_{NR/50}$
-	0 to 70	$C_{NR/70}$
Declared value	Declared value	$C_{Declared}$
No requirement	No requirement	C_{NR}

The categories for fine material content (< 63μm) in the aggregates are also defined. In table 2.16 all the categories depending on the fine material content are determined.

The necessity for testing and declaring all properties shall be limited according to the particular application or final use or origin of the aggregate. The test specified shall be carried out to determine appropriate physical properties when they are required.

The resistance to fragmentation shall be determined in terms of the Los Angeles coefficient, as specified in EN 1097-2: 1998 when it is required. And this draft gives categories for maximum values of Los Angeles coefficients, as table 2.17 shows. The Particle density shall be determined in accordance with EN 1097-6:2000. Water absorption shall be determined in accordance with EN 1097-6:2000.

In order to use the recycled aggregates chemical requirements have to be considered. The necessity for testing and declaring all properties shall be limited according to the particular end use or origin of the aggregate.

Table 2.16: Categories for maximum values of fines content

Aggregate	**Mass fraction of passing 0,063 mm sieve, %**	**Category F**
Coarse	≤2 ≤4 >4	f_2 f_4 $f_{Declared}$
	No requirement	f_{NR}
Fine	≤3 ≤7 ≤10 ≤16 ≤22 >22	f_3 f_7 f_{10} f_{16} f_{22} $f_{Declared}$
	No requirement	f_{NR}
Al-in	≤3 ≤5 ≤7 ≤9 ≤12 ≤15 >15	f_3 f_5 f_7 f_9 f_{12} f_{15} $f_{Declared}$
	No requirement	f_{NR}

Table 2.17: Categories for maximum values of Los Angeles coefficient

Los Angeles coefficient	**Category LA**
≤20	LA_{20}
≤25	LA_{25}
≤30	LA_{30}
≤40	LA_{40}
≤50	LA_{50}
≤60	LA_{60}
>60	$LA_{Declared}$
No requirement	LA_{NR}

The Acid-soluble sulphate content is determined by EN 1744-1:1998 and the maximum values of acid-soluble sulphate content are shown in table 2.18.

Table 2.18: Categories for maximum values of acid-soluble sulphate content

Aggregate	**Mass fraction of SO_3 %**	**Category AS**
Aggregates other than air-cooled blastfurnace slag	≤0,2%	$AS_{0,2}$
	≤0,8%	$AS_{0,8}$
	>0,8%	$AS_{Declared}$
	No requirement	AS_{NR}

The total sulphur content of the aggregate, determined in accordance with EN 1744-1:1998, are distributed in categories in table 2.19.

Table 2.19: Categories for maximum values of total sulfur content

Aggregate	**Mass fraction in %**	**Category *S***
Aggregates other than air-cooled blastfurnace slag	≤ 1	S_1
	>1	$S_{Declared}$
	No requirement	S_{NR}

The prEN 13242:2002 (final draft) give some durability requirements, without defining the applicability of them.

There are different tests to identify if the aggregate is resistant to freezing and thawing or not. One of the properties which determined the resistance to freezing and thawing is the water absorption capacity of the aggregates. The categories for maximum values of water absorption to measure the frost resistance are defined in table 2.20.

Table 2.20: Categories for maximum values of water absorption (EN 1097-6:2000, clause 7)

Water absorption Mass fraction %	**Category WA_{24}**
≤ 1 ≤ 2	$WA_{24}1$ $WA_{24}2$

If the water absorption capacity of aggregates is higher than the values specified in table 2.20, the resistance to freezing and thawing, determined in accordance with EN 1367-2 shall not be greater than the maximum value selected as one of the categories specified in Table 2.21.

Table 2.21: Categories for maximum magnesium sulphate soundness according to prEN 13242:2002

Magnesium sulphate value Loss of mass fraction %	**Category *MS***
≤ 18 ≤ 25 ≤ 35 > 35	MS_{18} MS_{25} MS_{35} $MS_{Declared}$
No requirement	MS_{NR}

2.3 CONCRETE MADE WITH RECYCLED AGGREGATE

Recycled aggregates used in concrete production have less density and more absorption capacity than conventional aggregates due to the adhered mortar. Consequently, in concrete made with recycled aggregates two interfacial transition zones are present: the existing interface between the original aggregate and adhered mortar, and the new interface between old and new mortar. The existing interface cannot be improved, and it is very important to achieve an effective new interface.

In the next sections concrete with 100% of recycled coarse aggregates and conventional sand is discussed. There are some studies with respect to varying the amount of recycled coarse aggregates and how the concrete's strength can be influenced by maintaining all the other conditions constant.

According to B. Gonzalez (2002) concrete made with 50% of dry recycled coarse aggregates with a total w/c ratio of 0.55 achieves the same compression strength as conventional concrete.

According to a comparison of concretes made by Dutch investigators (2000) the use of 20% of recycled aggregates in concrete has no negative influence in concrete strength with respect to conventional concrete.

2.3.1 Properties and design of fresh concrete

Water demand and workability

In Accordance with Mukai (1979), Buck (1973), Frondistou-Yannas (1977), Malhotra (1978), Hansen and Narud (1983) and Ravindrarajah and Tam (1985), recycled aggregate concrete made with recycled coarse aggregates and natural sand needs 5% more water than conventional concrete in order to obtain the same workability. If the sand is also recycled, 15% more amount of water is necessary to obtain the same workability.

If recycled aggregates are employed in dry conditions concrete's workability is greatly reduced due to their absorption capacity. Therefore the recycled aggregates should be saturated or have a high humidity (Nealen and Schenik, 1997).

According to M.Barra (1998), when recycled aggregates are saturated the interface between the aggregates and new paste is not effective, therefore recycled aggregates' humidity should be 80-90% in order to achieve an effective interface.

In general the workability of recycled aggregate concretes is affected by the absorption capacity of recycled aggregates. However, the shape and texture of the aggregates can also affect the workability of the mentioned concretes. This depends on which type of crusher is used (Shokry Rashwan and Simaan AbouRizk 1997).

Recycled aggregates in concrete production must be used in a condition of near saturation point to decrease the absorption capacity. Hansen (1986) concluded that the recycled concrete can be dosed, mixed, transported, placed, compacted in the same way as conventional concrete.

According to Andrew Nealen and Marcus Rühl (1997), recycled aggregates depending on water demand, absorb the water thus decreasing the workability of the fresh concrete (see figure 2.3).

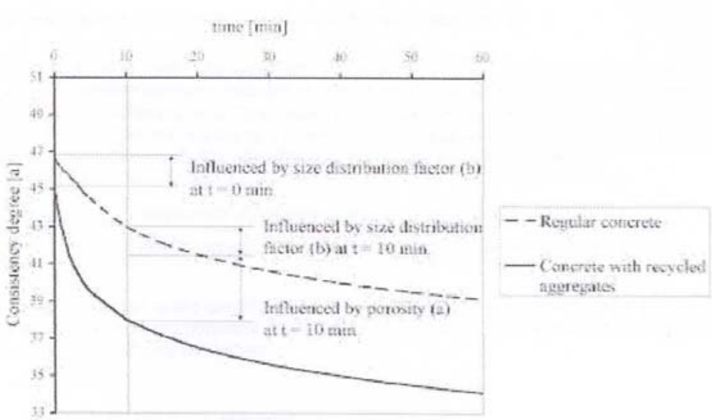


Fig.2.3: Development of rigidity

Two methods were employed for the addition of superplasticizer in the mixing process and the results analysed. In method one, the superplasticizer was added in time zero and the workability of the fresh concrete was similar to conventional concrete as shown in figure 2.4. In method two, the superplasticizer was added at 30 minutes and recycled aggregates absorbed all the water that they could, producing a concrete with the same workability as conventional concrete and with lower w/c ratio as shown in figure 2.5.

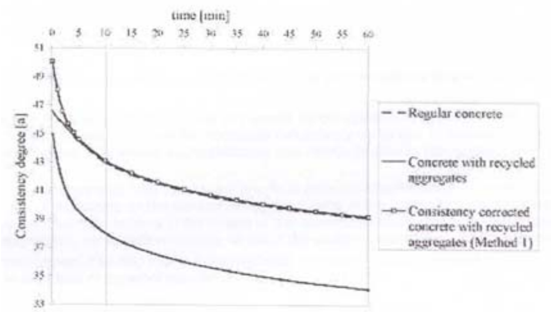


Fig. 2.4: Addition of superplasticizer at zero minute

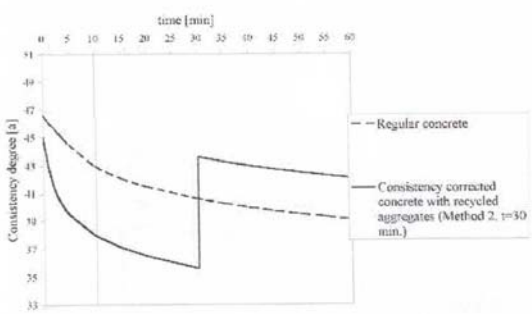


Fig. 2.5: Addition of superplasticizer at 30 minutes

Water/ cement ratio

Mukai (1979) found an excellent relation between the w/c ratio and the compression strength depending on the percentage of recycled aggregates employed, see figure 2.6.

M.Tavakoli and P. Soroushian (1996) demostrated that concrete made with 100% of recycled aggregate with lower w/c ratio than the conventional concrete can have a larger compression strength. When the w/c ratio is the same the compression strength of concrete made with 100% of recycled aggregate is lower.

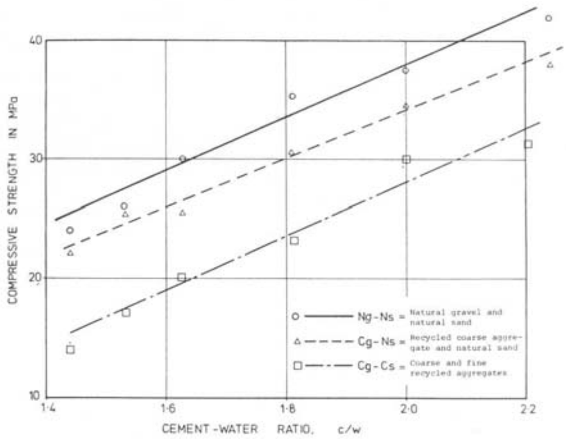


Figure 2.6: Relation between w/c ratio and compressive strength. Mukai (1979)

Cement quantity

In accordance with Hansen (1985) and other researches in order to achieve the same compression strength as in conventional concrete it is necessary to use more cement (5-9%) in concrete made with 100% of recycled aggregates. The values depend on the quality of the aggregate. When recycled fine aggregates are also used 15-20% more cement could be necessary.

Therefore, in order to achieve the same workability and compression strength as conventional concrete it will be necessary to add more cement in concrete made with 100% of recycled aggregate. When recycled sand is used in concrete, much more cement is necessary in order to achieve the same properties as conventional concrete. This makes the use of such concretes uneconomical.

Density and air content

Fresh concrete made with 100% of recycled aggregates have higher and more varied natural air contents than conventional fresh concrete, Mukai (1978), Hansen and Narud (1983).

Hansen (1985) concluded that the natural air content of recycled aggregate concrete may be slightly higher than that of control concretes made with conventional concrete. But it is certainly possible to produce recycled aggregate concrete in the laboratory with no significant increase in air content compared with control mixed.

2.3.2 New Interfacial Transition Zone, ITZ

In conventional concrete the unique interfacial Transition Zone is presented between the mortar paste and the aggregates.

Concrete made with recycled aggregate have an additional interfacial zone between the old adhered mortar to the original aggregate and the new mortar. These zones have to be considered when the concrete's permeability and strength are studied.

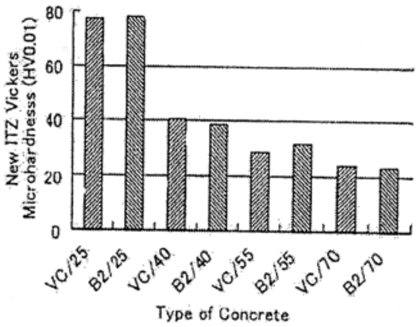


Fig.2.7: Vickers Hardness of the new interfacial transition zone, ITZ

a) *Hardness.* The hardness (Vicker microhardness) in the new interface, between old mortar and new mortar (in recycled aggregate concrete), and in a concrete with raw aggregates is very similar, according to N.Otsuki and S. Miyazato (2000). According to Yamato T., Soeda M. (2000), the hardness in the old transition zone increases with the strength of adhered mortar to aggregate. Besides, the hardness, it is not influenced by the adhered mortar quantity, only by strength.

b) *Capacity of improvement* . According to Otsuki (2000) when a low water/cement ratio occurs the old transition zone, the permeability and the strength of original concrete are difficult to improve. However, a new stronger transition zone can be created with double mixes in the manufacturing process. In the case of the double mix method the addition of water to the mix is divided into two stages. In the first stage the water is poured into the mixer 30 seconds after the adding and mixing of the fine and coarse aggregates. The mixture (water, fine and coarse aggregates) is mixed for a further 30 seconds before the process is stopped to allow the addition of the cement. The cement, water, fine and coarse aggregate mass is then mixed for 60 seconds by mixing machine. In this first mixing stage the concrete has a 0.2 water/cement ratio. After which, the final part of water is added and the mixing continues for the last 90 seconds. With this methodology, an improvement of ITZ, better permeability and strength is achieved.

2.3.3 Mechanical properties of recycled aggregate concrete

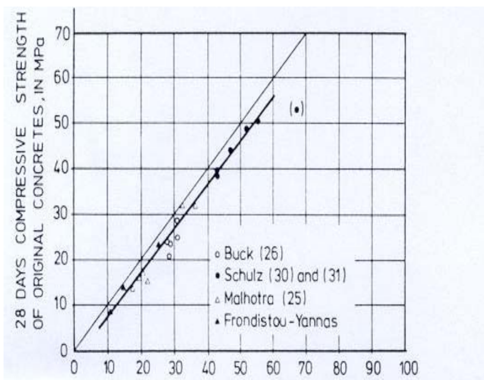
Influence of adhered mortar in strength of Recycled aggregate concrete

The quantity of attached mortar to the recycled aggregates has a negative influence on the long term strength of recycled aggregate concrete. Kokubu (2000) determined that if the water/cement ratio of the original concrete is less or equal to the recycled aggregate concrete, the attached mortar in the aggregates does not influence greatly in the long term strength. Nevertheless, the quantity of adhered mortar to the aggregate influences in the flexural strength and in the fracture energy. The flexural strength and fracture energy decrease is directly related to the quantity (increase) of adhered mortar attached to the recycled aggregates.

Compression Strength

Interests of the natural sand:

Concrete produced with recycled sand may behave differently from conventional concrete. When the entire natural sand is replaced by recycled sand part of the compression strength is lost with respect to conventional concrete. Fine recycled aggregate (sand) reduces the freezing and thawing resistance. Therefore according to European researchers, it is recommended to avoid the utilisation of recycled aggregates smaller than 4-5 mm. In accordance with Japanese investigators the finest aggregates limit is 2 mm.



COMPRESSIVE STRENGTH TO 28 DAYS OF CONCRETE MADE WITH RECYCLED AGGREGATES IN MPa

Fig.2.8: Compressive strength of concrete with recycled aggregates according to the strength of the original concrete, Hansen (1985)

Recycled aggregate concrete produced with coarse recycled aggregate and natural sand:

Many authors have presented conclusions regarding the variation of the compression strength of concrete made with 100% of recycled aggregates with respect to conventional concrete. According to some researchers the value has a decrease of 20% (Nixon, 1978), decrease from 14 to 32% (BCSJ, 1978), decrease of 10% (see figure 2.8. Hansen, 1985), decrease of

12% (Turanti, 1993) and according to Adjukiewicz (2000) observed that the variation of resistance was major in medium strength concretes than in high strength concretes. According to different authors, the different percentage of resistance decrease in concrete made with 100% of recycled aggregates depends on the quality of recycled aggregates.

Compression strength of concrete made with 100% of recycled aggregate is in relation to the following factors:

a) *W/c ratio.*

The compression strength of a conventional concrete will increase with the decrease of the w/c ratio. Nevertheless, Hansen and Narud (1983) concluded that, not only the w/c ratio influences on compression strength of concrete made with 100% of recycled aggregate, but the compression strength of the recycled aggregate concrete also depends on the strength of the original concrete. The compression strength of recycled aggregate concrete is strongly controlled by the combination of the water/cement ratio of the original concrete, when other factors are essentially equal.

Therefore a dependence exists with respect to the new-old w/c ratio. When the w/c ratio of the original concrete is equal or lower than that of the recycled aggregate concrete, the resistance of the recycled concrete can be equal to or larger than the original one. However, when the w/c ratio of the original concrete is high, the original concrete strength will determine the new concrete strength.

In accordance with Rasheeduzzafar and Khan (1984) the compression strength of a concrete made with 100% of recycled aggregates and having a water/cement ratio lower than 0.4 could not increase due to the strength of original concrete. In Concrete made with 100% of recycled aggregate the compression strength depends on the recycled aggregates strength more than on the cement paste strength.

The compression strength of the conventional concrete and concrete made with different percentages of recycled aggregates and w/c ratio can be seen in table 2.22, of BSCJ (1978).

Table 2. 22: Compression Strength of different concretes (BSCJ, 1978)

W/c	Compression Strength of Concrete (MPa)			
	Raw aggregate (original concrete)	**Concrete made with Recycled aggregate 100% and natural sand**	**Concrete made with Recycled aggregate, 50% recycled sand and natural sand.**	**Concrete made with Recycled sand and 100% recycled sand.**
0.45	37.5	37.0	34.0	30.0
0.55	28.9	28.5	25.0	21.5
0.68	22.0	21.0	17.5	13.0

b) Compression strength of the concrete over time (days).

In general, the deferred behaviour of the recycled aggregate concrete is similar to that of conventional concrete (Malhotra, 1978, Buck, 1973 y Tavindrarajah y Tam, 1985). Japanese investigations agree that more than 30 % of the raw aggregates can be replaced by recycled aggregate without producing a significant change in properties with respect to conventional concrete.

In particular, the tests conducted by Rohi M. Salem and Edwin G. Burdette (1998) concluded that the compression strength of concrete made with 100% of recycled aggregate increases by 2% from 7 to 28 days with respect to the 16 % increase in conventional concrete. This could be due to either the absorption capacity of the recycled aggregates or to the bad adherence of the aggregate with the cement paste.

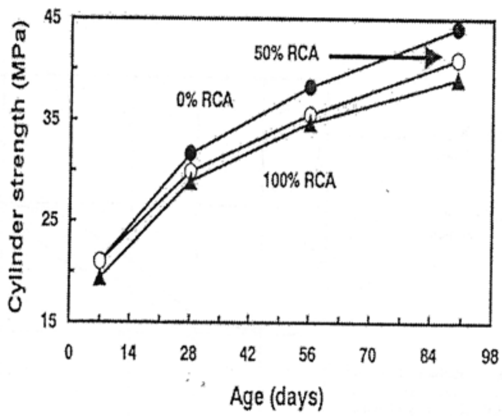


Fig.2.9: Compression Strength over time of concrete of different quantities of recycled aggregate. Ravindrarajah (2000)

Ravindrarajah (2000) found, that at 28 days conventional concrete has 5.4% more compression strength than concrete made with 50% of recycled aggregate and 8,9% more strength than concrete made with 100% of recycled coarse aggregate. (See figure 2.9).

Recycled aggregate concrete produced with coarse recycled aggregate and recycled sand:

The presence of recycled sand suggests an additional loss of strength in recycled aggregate concrete. Gerardu y Hendricks (1985) concluded that concrete produced with coarse and fine recycled aggregate is 15% lower in compression strength than conventional concrete.

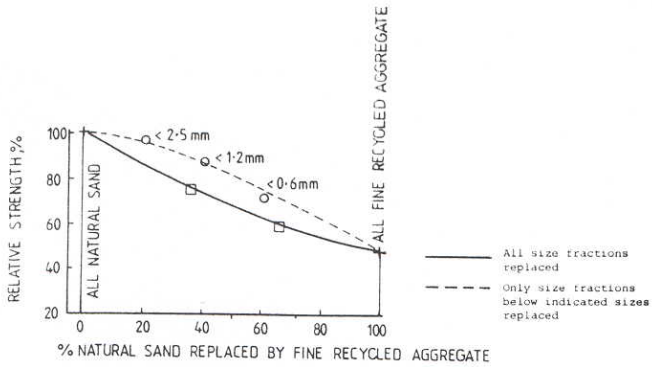


Fig. 2.10: Compressive strength of concrete made with recycled aggregate of w/c ratio of 0.65 replacing some percentage of natural sand by recycled sand

Soshiroda (1983) for BCSJ, obtained the compression strength trend loss by increasing the recycled sand quantity in concrete. In general, the same recycled concrete loses half of its compression strength when the entire natural sand is replaced with recycled sand. Moreover when the recycled sand is smaller than 2 mm more loss of strength is produced. Furthermore, this recycled sand also has a tendency to diminish frost resistance. It is not recommended to use any recycled aggregates smaller than 2 mm in Japan (Hansen, 1983). However in Europe the limit is set at 4 mm.

Compression strength variation coefficient of recycled aggregate concrete

The coefficient variation of the compression strength of a recycled aggregate concrete does not defer too much from the established conventional concrete behaviours (BCSJ (1978), Hansen y Narud (1983) and Coquillat (1982)). However, it must be noted that in practice these results are not easily demonstrated. Since the w/c ratio is difficult to determine.

Any variation in concrete production, or in the properties used, produces a variation of strength in the resultant concrete. The employment of different qualities of recycled aggregate in concrete production, brings about an increase of the coefficient variation (Hansen, 1986). Hendriks found variations of compression strength to be between 41 and 50.6 MPa for concretes employing an identical proportion of mixture, when the recycled aggregates employed were produced in different plants.

In practice, in order to assure the desired compression strength demanded in every case, it is necessary to increase the quantity of cement employed. In the majority of the cases, due to a clearly economic factor, this restricts the use of recycled aggregate in the production of concrete with low-medium strength.

Modulus of Elasticity and damping Capacity

The old mortar which is adhered to the recycled aggregates has a low modulus of elasticity, consequently concrete made with recycled aggregates will always have a lower modulus of elasticity than that of conventional concrete. There are several studies in which the percentage of reduction of the modulus of elasticity has been determined:

- Frondistou-Yannas (1977) found that the modulus of elasticity of concrete made with recycled aggregate and raw sand is 33 % lower than conventional concrete.
- Gerardu and Hendriks (1985) estimated the decrease to be of 15 %. They also estimated a decrease of 40% when recycled coarse and fine aggregates were used in concrete production.
- Coquillat (1982) established that a concrete using recycled coarse and fine aggregate had a 28 % lower modulus elasticity than that of a conventional concrete employing raw aggregates.

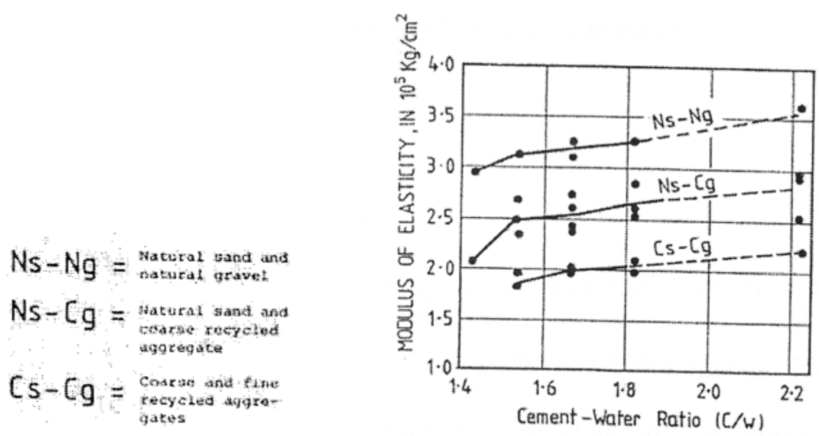


Fig.2.11: Modulus of elasticity in function of the w/c ratio of the conventional and the recycled concrete. Japanese Investigators (1978)

- Ajdukiewicz (2000), in a series of 12 beams, observed that the strain on recycled aggregate concrete beams was larger than on conventional concrete beams. But the differences were much smaller than in the modulus of elasticity.

The Japanese investigators' results can be seen in figure 2.11. High strength original concrete achieves a higher modulus elasticity than low strength original concrete. In concrete made with recycled aggregate crushed from low strength original concrete the modulus of elasticity is always lower than that of concrete made with high strength recycled aggregates or conventional concrete. (Hansen and Boegh, 1985).

The differences between the modulus of elasticity of recycled aggregate concrete and conventional concrete are larger if the recycled aggregate concrete has been produced with coarse and fine recycled aggregates.

Certain methods on how to calculate the modulus of elasticity depending on the concrete strength have been defined by several researchers. Ravindrarajah and Tam (1987), defined some methods for recycled aggregate concretes. In chapter 3, with the experimental results, it will be established which of the presented models adjusts more to reality.

1) Ravindrarajah y Tam (1985), for concretes made with recycled aggregates defined;

Static Modulus of elasticity (E): $E=4.63f_{cy}^{0.50}$ (2.1)

$E=7.77f_{cu}^{0.33}$ (2.2)

Dynamic Modulus elastic (E_D): $E_D=6.19f_{cy}^{0.50}$ (2.3)

$E_D=13.05+3.48f_{cu}^{0.50}$ (2.4)

Being f_{cy} and f_{cu} cylindrical and cubic test elements' compression strength respectively, in N/mm2.

For the case of concrete produced with raw aggregates:

CEB-FIB recommendation (1978): $E=6.6f_{cy}^{0.50}$ (2.5)

Teychennè et al. (1978): $E=9.1f_{cu}^{0.33}$ (2.6)

CPI 10 (1972): $E_D=22+2.8f_{cu}^{0.50}$ (2.7)

Later Ravindrarajah (1987) defined the modulus of elasticity as:

$E=5{,}31f^{0{,}50}+5{,}83$ (for conventional concrete) (2.8)

$E=3{,}02f^{0{,}50}+10{,}67$ (recycled concrete) (2.9)

According to M. Kakizaki (1988), the modulus of elasticity can be derived empirically as follows,

$$E_c = 2.1 * \left(\frac{d_s}{2.3}\right)^{1.5} * \left(\frac{f_c}{200}\right)^{0.5} \qquad (2.10)$$

Where,

d_s: density of concrete

f_c: compression strength of concrete

Tensile and flexural strength

Tensile

- BCSJ (1978), Mukai (1978) y Ravindrarajah y Tam (1985) demonstrated that there are no great differences in the tensile strength of recycled coarse and natural sand concrete with respect to conventional concrete. However if recycled sand replaces the natural sand used in the concrete employing recycled coarse aggregates then the tensile strength diminishes 20 % with respect to conventional concrete.
- Gerardu y Hendricks (1985), stated that the decrease of tensile strength in a concrete made with coarse recycled aggregates and natural sand is less than 10% and that when coarse and fine recycled aggregates are used the decrease is not larger than 20%.

Flexural

- According to BCSJ (1978) the flexural strength of concrete made with recycled aggregate is approximately 1/5 to 1/8 of the compression strength of conventional concrete, however no experimental data is presented.
- Ravindrarajah and Tam (1985), thought that there was no great difference between the flexural strength of concrete made with recycled coarse aggregate and natural sand or conventional concrete. Their conclusions can be seen in table 2.23. This

information contradicts Malhotra (1978), who found a reduction with respect to that of conventional concrete.

Table 2.23: Percentages of strength reduction according to recycled aggregate used (1985)

	Reduction of tensile strength	**Reduction of tension strength**	**Reduction of shear strength.**
Concrete with coarse recycled aggregates and natural sand	6 %	0%	26%
Concrete with coarse and fine recycled aggregates.	10%	7%	32%

- M.Tavakoli y P. Soroushian (1996), established a correlation between flexural strength of concretes made with coarse recycled aggregate and conventional concretes according to the Code 318 of the ACI:

$$f_r = 0.62(f'_c)^{1/2} \text{ Mpa, with } f'_c \text{ en Mpa.} \quad (2.11)$$

This equation is debatable with respect not only to concretes made with recycled aggregate but also conventional concrete, especially in concretes of high strength. The value of the test obtained by recycled aggregate concrete is lower than the results obtained by the equation. The difference is larger when a higher w/c ratio is used. The value of the equation in conventional concrete is smaller than obtained experimentally.

- M. Barra de Olivera and E. Vázquez (1996), note a slight decrease in the strength of the concrete made from dry and saturated recycled aggregates. This decrease is especially noticeable in the flexural strength of concrete using saturated recycled aggregates.

2.3.4 Durability of recycled aggregate concrete

Nowadays, durability of concrete made with recycled aggregates is undoubtedly one of the lines more open to investigation. Some aspects of durability are already studied.

Permeability and water absorption

Permeability of the concrete is one of the basic properties that can cause durability problems in the concrete.

There is no difference in the absorption capacity of recycled aggregate and conventional concretes when the concretes are produced approximately with the same w/c ratio of the original concrete. However, the absorption capacity of the concrete made with recycled aggregates increases with respect to conventional concrete when the w/c ratio used in concrete production is lower than that of the original concrete (Rasheeduzzafar and Khan, 1984).

The same permeability is needed in order to achieve the same compressive strength in concrete made with recycled aggregate and conventional concretes. Therefore, the recycled aggregate concrete needs to have lower w/c ratio when the percentage of recycled aggregate in concrete production is high.

Frost resistance

Several authors affirmed that the frost resistance of the recycled aggregate concrete (using only coarse recycled aggregates) is similar to that of conventional concrete. However when fine recycled aggregates are used the frost resistance decreases substantially.

According to Rohi M. Salem and Edwin G. Burdette (1998), the air entrained method is the best way to improve the frost resistance of recycled aggregate concrete, however this method decreases some of the concretes physical properties.

M. Barra de Olivera and E. Vazquez (1996), observed that the poor results in frost resistance of concretes using saturated and dry recycled aggregates and the good results of those made from semi-saturated aggregates can be explained as being caused by the formation of a more solid and denser interface in these conditions.

Carbonation, chloride penetration and reinforcement corrosion

According to BCSJ (1978) and Khan and Rasheeduzafar (1984), concrete made with already carbonated recycled aggregate suffers 65% more of carbonation than conventional concrete. Rust occurs in the steel reinforcement with 2-3 mm of clean cover at 2 months. The rust risk in reinforced recycled aggregate concrete is higher than conventional concrete. However this risk is possible to decrease with lower w/c ratio in recycled aggregate concrete than conventional concrete.

According to M. Barra, E. Vazquez, (1997) the carbonation risk of recycled aggregate concrete using a higher amount of cement than 400 kg/m^3 of concrete mix is larger than in conventional concretes. The carbonation depth in recycled aggregate concrete and conventional concrete is similar when the amount of cement employed in the mix is between 300 kg/m^3 and 400 kg/m^3. This occurs when the cement is added, the aggregates are saturated or very humid. In poor concretes, using less than 300 kg/m^3 of cement, the carbonation depth is similar in both concretes.

Alkali Silica reaction

The rubble processed at recycling plants may originate from structures which were attacked by ASR or which were potentially reactive, but did not react due to a lack of favourable conditions (such as humidity).

According to J. Desmyter and S. Blockmans (2000), preventive measures such as the use of low alkali portland or blast furnace slag cement, may increase the durability of the recycled concrete as far as ASR is concerned.

2.4 STRUCTURAL BEHAVIOUR OF REINFORCED RECYCLED AGGREGATE CONCRETE BEAMS

According to Miyazawa S. Kuroi T. y Sato, R. (2000), the mechanical properties of reinforced concrete beam specimens made with coarse recycled aggregates under static load are similar to those made of reinforced concrete made with raw aggregates, even when the adhered mortar in coarse recycled aggregates is high.

Flexural Strength

Sato, R., Kawai, K. y Baba Y.(2000) discovered through a series of experiments that the flexural strength of concrete beam specimens made with coarse or fine recycled aggregates have a very similar behaviour pattern when compared with conventional concrete beam specimens.

According to Mukai et al (1988), the failure in low reinforced concrete beam specimens made with recycled aggregate or conventional aggregates occurs when reinforcement yields. However in high reinforced beam specimens the failure occurs by compression of top part.

On subjecting both the reinforced recycled aggregate concrete (RAC) and conventional concrete beam specimens to the same load conditions it was discovered that cracking first appeared in the reinforced RAC beam specimens, however the ultimate load is similar in both beams. With respect to low reinforced concrete beam specimens made with recycled aggregates the displacement is larger than in conventional concrete. However there is no difference when the specimens are strongly reinforced.

Shear Strength

The ratio of shear cracking strength of a reinforced concrete beam specimen made with recycled coarse aggregate is approximately 0.9-1.0 with respect to that of a conventional concrete reinforced beam specimen.

According to Mukai et al (1988), the shear strength of a low reinforced concrete beam specimen made with recycled aggregate is 10% lower than that of a conventional concrete beam. However with high reinforcement, the recycled aggregate concrete beam specimens achieve the same and sometimes even larger strength than conventional ones.

Recycled aggregate concrete beam specimens with low transversal reinforcement have less ductility than conventional concrete specimens. However this can change when the beam specimens are strongly reinforced.

In accordance with Yagishita (1993), in concrete beam specimens made with recycled aggregate the first diagonal crack occurs before than that of conventional concrete. However, the ultimate shear load is similar in both recycled aggregate concrete and conventional concrete beam specimens. The cracks widths are larger in recycled aggregate concrete beam specimens. The bond between reinforcement and concrete is lower in recycled aggregate concretes than in conventional concretes.

According to B. Gonzalez (2002) on comparing concrete beam specimens made with recycled aggregate to those made of conventional concrete it was discovered that the first diagonal crack occurs with lower load.

The diagonal crack load in beam specimens with transversal reinforcement is approximate to collapse load in non-reinforced beam specimens.

The splitting crack is more relevant in recycled aggregate concretes than conventional ones, however this phenomenon is less significant with the presence of transversal reinforcement.

2.5 ULTIMATE LIMIT STATE (US) OF SHEAR. BASIC THEORY

2.5.1 Shear failure mechanism in beams without stirrups

The maximum shear strength, in the case of non-existent transversal reinforcement, occurs when first diagonal crack is produced or immediately afterwards. For this reason, generally, the shear capacity of these elements is taken as the initial shear cracking load. It depends basically on five parameters.

a) *Concrete tensile strength*. The cracking load is a function of the concrete tensile strength (concretes produced with raw aggregates). The diagonal crack occurred with an applied force approximately three times smaller to that of the concrete tensile strength.

b) *Quantity of longitudinal reinforcement*. Wide cracks (less friction between cracks), less dowel action and smaller compression block occurs in a beam specimen with a low quantity of longitudinal reinforcement. In beam specimens with 0.0075% to 0.025% amount of longitudinal reinforcement present failures by shear.

c) *Arch action a/d*. It is widely demonstrated that the tangential average stresses increase significantly in beam specimens with a shear span/ depth (a/d) ratio less than 2.5. For elements with ratios a/d<1 a strut and tie model method must be used.

d) *Shape effect*. The strength of the beam specimen with respect to tangential stresses increases as the depth of the beam specimen decreases. This effect is produced because the cracks are wider when the depth of the beam is larger. Several experimental studies (Collins et al. 1993) verified that the depth effect disappears in beams specimens without transversal reinforcement when they present well distributed longitudinal reinforcement.

e) *Axil load*. The tensile forces tend to decrease the cracking load, while the compression forces tend to increase it.

f) *Residual tensile stresses between cracks* . When the crack is produced, there are small parts of concrete which in effect sew the crack. The concrete continues transmitting tensile force until the cracks widths are around 0.05- 0.15 mm. In elements with small depth, this effect is more important.

The *failure process* in a structural specimen without transverse reinforcement is described as follows:

1. Modification of the elastic stress field is produced before the formation of shear cracks and when flexural cracks occur.

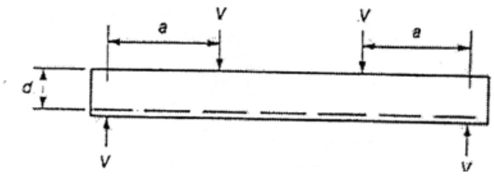


Fig.2.12: Useful depth/ load point ratio

When the shear crack occurs, the concrete between cracks remains isolated, cutting the incremental tensile flow in the longitudinal reinforcement. At that moment a T tensile force is existed on one side of the longitudinal bar and a T+ΔT force on the other. The magnitude of the variation of T will depend on the type of beam.

2. The ΔT is more important in small depth beams (at the instant that the shear crack is produced) than it is in large depth beams. In general the beams collapse by cracking when the a/d ratio is between 2.5 and 6.

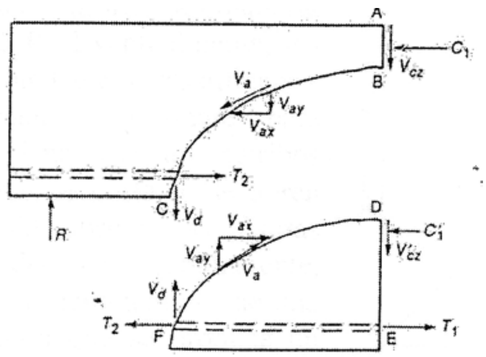


Fig.2.13: Internal forces on a cracked beam without stirrups

In the majority of cases the failure is produced by the so-called *splitting crack*, the failure of anchorage or bond. The failure can also occur by compression failure at the top of the beam. When the crack is opened V_d and V_{cz} are only working (see figure 2.13). The type of beam specimen collapse depends on which resists the most the longitudinal reinforcement bond or the compression crack.

3-When the concrete is isolated between the cracks, the compression stress passes through the crack (Va) due to aggregate interlock, see figure 2.13. The bond between concrete and longitudinal bars is necessary in order to maintain the equilibrium between T and ΔT. As the compression stress increases, the aggregate interlock effect decreases (Va=0). When this occurs the longitudinal reinforcement dowel effect and the compressive top part only work. There is an instant in which the bond fails, leaving the top compression part supporting the shear load, finally resulting in the crushing of the top zone of the beam.

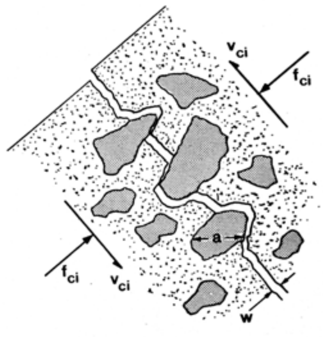


Fig.2.14: Aggregate interlock effect

2.5.2 Shear failure mechanism in beams with stirrups

The intention of shear reinforcement is to assure that structural element can obtain its whole flexural strength.

The presence of transversal reinforcement does not prevent the formation of inclined cracks, because transversal reinforcement only starts to work when the cracks have already been produced.

The shear load history of a reinforced beam specimen can be summarized as:

- The load is supported by the concrete before cracking.
- The section of the structural specimen resists the external shear after the flexural cracking and before the inclined cracking.
- The stirrups start working when shear crack is produced,
- The stirrups yield with the increment of load, after which all progressive increases of load have to be compensated without the help of the transversal reinforcement. This quickens the increase in the width of the cracks. When this occurs the effect is the same as that of beams without transversal reinforcement.

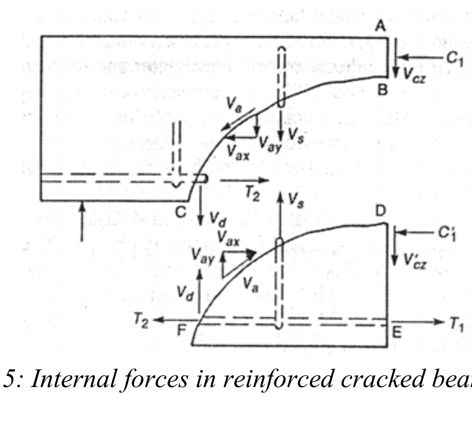


Fig.2.15: Internal forces in reinforced cracked beam

- The compression of aggregate interlock decreases with the increase of the cracks, resulting in the transference of the entire load to the compression top part, where the load is situated and the dowel effect of the longitudinal reinforcement occurs. Finally, the failure is produced when one of these two resistant mechanism fails: *splitting failure*, by longitudinal reinforcement bond or *shear compression,* due to the failure of the concrete compression zone.

2.5.2.1 The Ritter-Mörsch model

In the early XX century, truss models were used as conceptual tools in the analysis and design of reinforced concrete beams. In 1899 and 1902, both the Swiss engineer Ritter and the German Mörsch independently postulated that after cracking (due to diagonal tension stresses) the concrete beam could be idealized as a parallel chord truss with the compression diagonal inclined at 45º with respect to the longitudinal axis of the beam.

The truss analogy proposes that a cracked reinforced concrete beam acts like a truss with parallel longitudinal chords, a web composed of diagonal concrete struts, and transversal steel ties (see fig 2.16). When shear is applied to this truss, the concrete struts are placed in compression, while tensile stress is produced in the transverse ties and in the longitudinal chords.

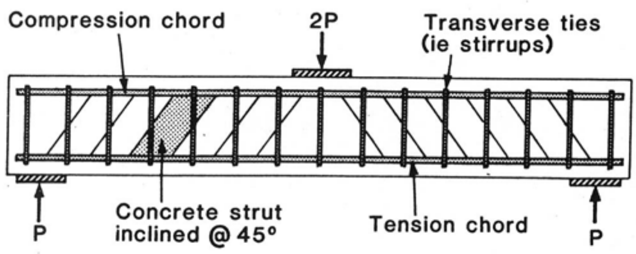


Fig. 2.16: Model of Ritter-Mörsch

The principal simplifications introduced in the model are:

- The compression force supported by the concrete is due to the shear compression placed on the top part of the beam, dowel effect and aggregates interlock along the crack.
- A vertical resistant element is defined.

The ideal design is when the stirrups' distribution corresponds to them yielding with the ultimate load. In the case of assuming the hypothesis that all the stirrups yield and that each one transmits a force $A_y f_y$, the system remains statically determined. For this reason it is named the yield truss model.

Definition of internal forces in a beam with web reinforcement

According to the previously explained model, the force in a tie, $A_y f_y$, can be calculated as:

$$A_y f_y = \frac{Vs}{jd / \tan\theta} \tag{2.11}$$

The average compression force of the diagonals is:

$$f_{cd} = \frac{V}{b_w jd \cos\theta sin\theta} = \frac{V}{b_w jd}\left(tan\theta + \frac{1}{tan\theta}\right) \tag{2.12}$$

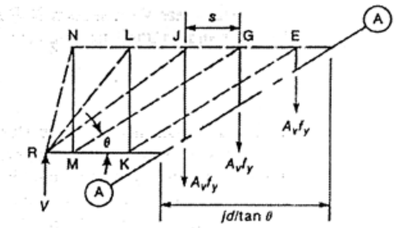


Fig. 2.17: Determination of forces in the stirrups according to the model.

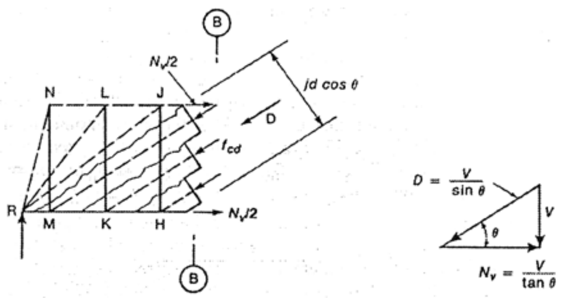


Fig. 2.18: Determination of forces in the diagonals of compressions

2.5.2.2 Shear design of beam specimens. Analytical Methods

The inclination of shear cracks is a principal parameter evaluating the beams' response to shear stress. The transverse ties will only contribute to the resistance of the element if the cracks cross them. The number of effective reinforcements, and an exact forecast of the response of the structure will depend directly on the correct evaluation of the cracks inclination.

The Ritter-Mörsch's models assume 45º of the crack inclination with respect to longitudinal axis. However, the results obtained from laboratory tests, present minor inclinations at 45º.

At present there are models which not only study the reinforced concretes' shear behaviour but also determine the inclination of the compressed strut, consider transverse and longitudinal reinforcements' strain and compressed concrete. These models are known as "*Compression field approaches"*. In these models the equilibrium equations, compatibility and stress –strain relations of the reinforcement and cracked concrete are used to predict the response of a section submitted to shear stress. Of all the models studied the most acceptable model was one proposed by Collins and Mitchell named *"Compression field theory"* (1974) which has already been modified and now it is known as *"Modified Compression Field Theory"*.

Modified Compression-Field Theory (MCFT)

Mörsch (1922) concluded that it was mathematically impossible to determine the crack inclination, considering 45º to be a conservative value. The experience gained with the truss model has revealed that its results are sufficiently conservative, particularly with respect to beams with small amounts of web reinforcement. The assumption that θ was equal to 45º is, in general, incorrect.

The first methods based on Wagner's procedure for determining the θ applicable over the full load range were developed by Mitchell and Collins (1974) with respect to members in torsion. Further developments led to the Modified Compression Field Theory (Vecchio and Collins, 1986).

The MCFT is a general model for the load-deformation behaviour of two-dimensional cracked reinforced concrete subjected to shear. It is a model which considers the concrete with concrete stresses in principal directions plus reinforcing stresses which are assumed to be only axial. The concrete stress-strain behaviour with respect to compression and tension was derived originally from Vecchio's tests (Vecchio and Collins, 1982).

The key simplifying assumption of the MCFT is that the principal strain directions coincide with the principal stress directions. This assumption is justified by experimental measurements, which show that the principal directions of stress and strain are parallel within ±10º.

The concrete struts are also at a shallower angle to those of the cracks, and the compressive stress field must be transferred across the cracks, thus reducing concrete strength from its uncracked state and inducing shear stress across the crack faces. This produces tensile stresses in the cracked concrete.

It is recognised that the local stresses in concrete and the reinforcement vary from point to point in the cracked concrete. High reinforcement stresses as well as low concrete tensile stresses occur at crack location. The compatibility conditions of MCFT are expressed employing the average strains on cracked concrete and its reinforcement, where the strain is measured using greater lengths than those of the cracked spacing. The equilibrium conditions, which relate to the concrete stresses and the reinforcement stresses with the applied loads, are also expressed in terms of average stresses.

In a similar manner, the strains used for stress-strain relationships are based on average strains. Together they consider the combined effects of local strains at cracks, strains between cracks, bond-slip, and crack slip. The calculated stresses are also average stresses in so much as they implicitly include stresses between cracks, stresses at cracks, interface shear on cracks, and dowel action. In this model, the cracked concrete in reinforced concrete is treated as a new material with empirically defined stress-strain behaviour.

The equilibrium equations, the compatibility relationships, the reinforcement stress-strain relationships, and the stress-strain relationships for the cracked concrete undergoing compression and tension enable the average stresses, the average strains, and the angle θ to be determined for any load level up to the failure.

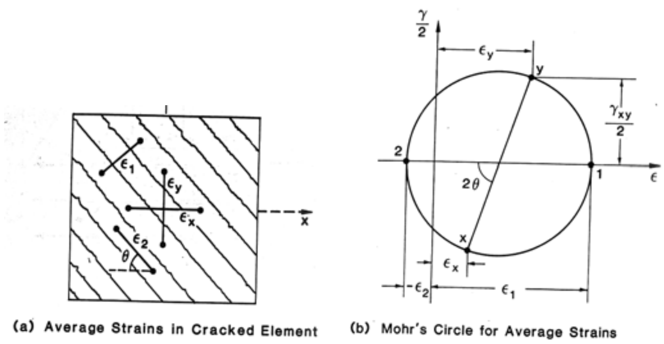


Fig.2.19: Conditions of compatibility for cracked element

Failure of the reinforced concrete element may be governed not by average stresses, but rather by local stresses that occur at a crack. This so-called crack check is a critical part of MCFT as are the theories derived from it. The crack check involves limiting the average principal tensile stress placed on the concrete to a maximum allowable value determined by considering the steel at a crack and the ability of the crack surface to resist shear stresses.

Compatibility conditions

$\varepsilon_1 + \varepsilon_2 = \varepsilon_x + \varepsilon_y$ (2.12)

And for geometric relations

$\gamma_{xy} = \dfrac{2\,(\varepsilon_x - \varepsilon_2)}{\tan\theta}$ (2.13)

$\tan^2\theta = \dfrac{(\varepsilon_x - \varepsilon_2)}{(\varepsilon_y - \varepsilon_2)}$ (2.14)

Equilibrium Conditions

$f_x = f_{cx} + \rho_{sx} f_{sx}$ (2.15); Horizontal Equilibrium.

$f_y = f_{cy} + \rho_{sy} f_{sy}$ (2.16); Vertical Equilibrium.

and

$\nu_{xy} = \nu_{cy} + \rho_{sy} \nu_{sy}$ (2.17), assuming

$\nu_{cx} = \nu_{cy} = \nu_{cxy}$.

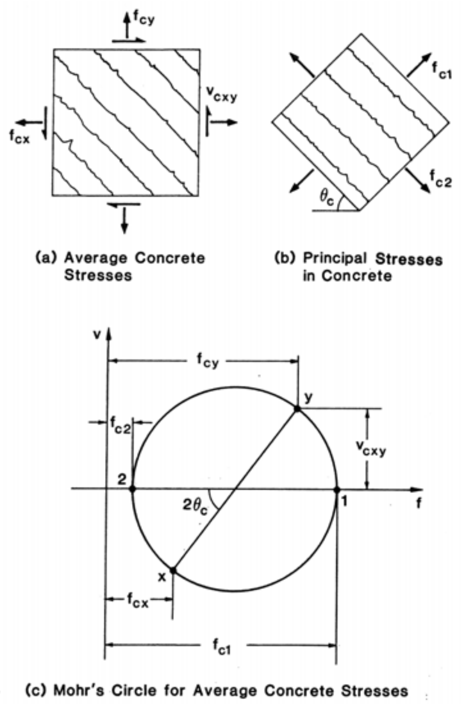


Fig.2.20: Conditions of equilibrium for cracked element

Stress- strain relationships

Steel:

$f_{sx} = E_s \cdot \varepsilon_x \leq f_{yx}$ (2.18)

$f_{sy} = E_s \cdot \varepsilon_y \leq f_{yx}$ (2.19)

$\nu_{sx} = \nu_{sy} = 0$

Concrete: The principal strains' directions in the concrete do not exactly coincide with the principal stresses' directions in the concrete.

Nevertheless, the simplification of assuming the coincidence of both directions seems to be reasonable:

$$\theta_c = \theta$$

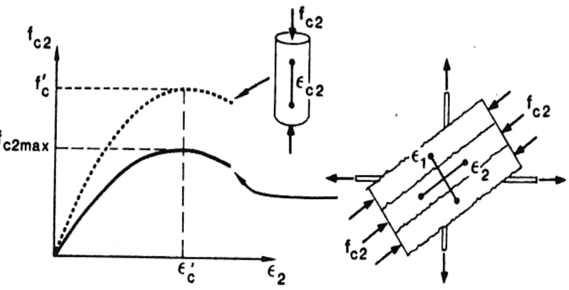


Fig. 2.21: Stress-strain relationship for cracked concrete by compression

Based on experimental results, Collins (1978) suggested that the relation between the principal compression stress, f_{c2}, and principal compression strain, ε_{c2}, for a diagonally cracked concrete differed from that of the curved stress-strain derived from the compression failure of a cylindrical test element.

The proposed function for f_{c2} results in being not only a function of the compression's principal strain ε_{c2}, but also of the coexistent tensile's principal strain ε_{c1}.

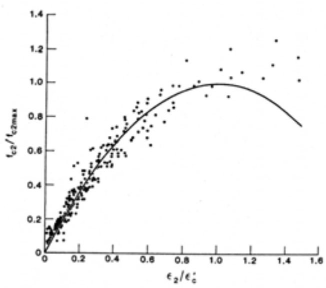


Fig. 2.22: Relation of the maximum compression strength of the element, f'c, with the tensile principal strain

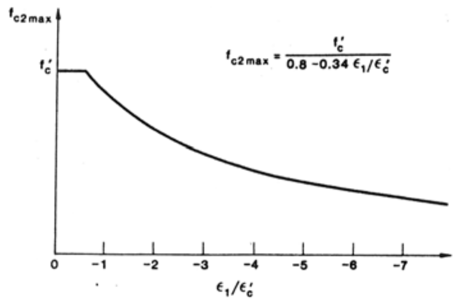


Fig. 2.23: Relation of the maximum compression strength of the element, f'c, with the compressive principal strain

Therefore, the cracked concrete suffering tensile strains in normal direction to compression has a lower strength than that obtained by the cylindrical test element. The suggested relation is the following one:

$$f_{c2} = f_{c2max}\left(2\left(\frac{\varepsilon_2}{\varepsilon'_c}\right) - \left(\frac{\varepsilon_2}{\varepsilon'_c}\right)\right) \qquad (2.20)$$

$$\frac{f_{2max}}{f'_c} = \frac{1}{0.8 - 0.34\frac{\varepsilon_1}{\varepsilon'_c}} \leq 1.0 \qquad (2.21)$$

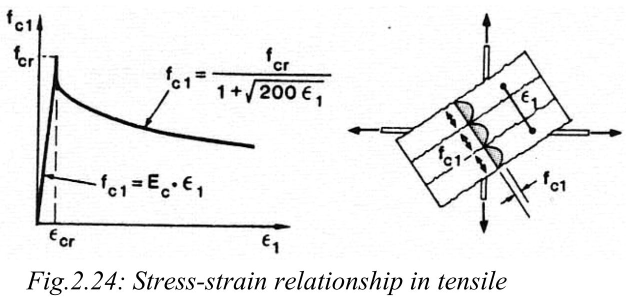


Fig.2.24: Stress-strain relationship in tensile

The relation between average tensile principal stress in the concrete and the average tensile principal strain can be assumed lineal before the cracking. After cracking, f_{c1} decreases, with increment of strain ε_1. The proposed relation is:

$f_{c1} = E_c \cdot \varepsilon_1$, for $\varepsilon_1 \leq \varepsilon_{cr}$ (2.22)

$f_{c1} = \dfrac{f_{cr}}{1+\sqrt{200\varepsilon_1}}$ after cracking. (2.23)

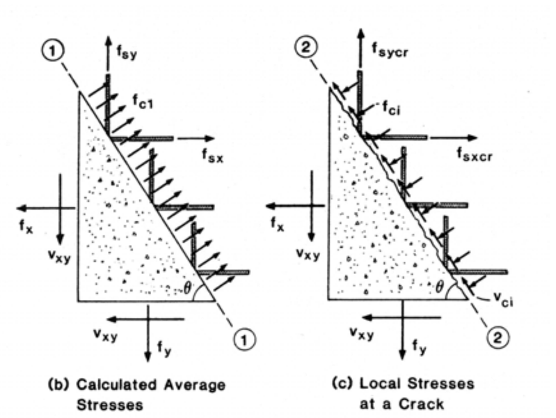


Fig.2.25: Comparison of local stresses in crack with its average values

Value in the plane of the crack

Until now the theory has developed a response in average values (considering several cracks). Nevertheless, the average values do not provide information relating to local variations. In a crack, the tensile's stress in the reinforcement will be superior to that obtained using average values, however between the cracks the average value will be higher than existing one. These local variations are important because the ultimate capacity of the element could be governed by the capacity to transmit stresses across the cracks.

The exterior forces f_x, f_y and ν_{xy} are fixed in each case. Due to external-internal equilibrium the stresses in the crack and their average values must be statically equivalent.

As seen in figure 2.22, the tangential average stress is zero in plane 1 and it exists in plane 2. These stresses ν_{ci} must always exist with small compression stress, f_{ci}, along the crack.

The equilibrium of stresses between both planes according to both directions:

$\rho_{sy}(f_{sycr} - f_{sy}) = f_{c1} + f_{ci} - \nu_{ci}\tan\theta$ (2.24); Vertical equilibrium.

$\rho_{sx}(f_{sxcr} - f_{sx}) = f_{c1} + f_{ci} - \nu_{ci}/\tan\theta$ (2.25); Horizontal equilibrium.

The equilibrium can only be satisfied without the presence of tangential and normal stresses in the crack given the case that the reinforcement has not yielded, so that:

$f_{sxcr} \leq f_{yx}$ and $f_{sycr} \leq f_{yy}$

The presence of tangential and normal stresses in the crack will be guaranteed in the case where either a) the reinforcements do not exist in one or in both directions or when b) the reinforcements cannot follow the principal average stresses due to them having yielded.

The relations between stresses ν_{ci}, the width of the crack *w*, and the compression strength resultant by the equilibrium in the crack f_{ci}, have been experimentally studied. According to Walraven (1981), the relation is the following one:

$$\nu_{ci} = 0.18\ \nu_{ci\ max} + 1.64\ f_{ci} - 0.82 \cdot \frac{f^2_{ci}}{\nu_{ci\ max}} \quad (2.26)$$

$$\text{where: } \nu_{ci\ max} = \frac{\sqrt{-f'_c}}{0.31+24w/(a+16)} \quad (2.27),$$

being α the maximum size of aggregates in millimetres and the stresses in MPa.

The crack's width *w* should be the average width along the surface of the crack. This average width equal to the product of the principal stress of tensile and the spacing between cracks s_θ; that is

$$w = \varepsilon_1 \cdot s_\theta \quad (2.28)$$

$$s_\theta = \frac{1}{\frac{\sin\theta}{s_{mx}} + \frac{\cos\theta}{s_{my}}} \quad (2.29)$$

taking s_{mi} =1.5·maximum distance between reinforcement bars of y direction.

The used variables are summarized:

$$\nu = \frac{V}{b_w\, jd}\text{, shear stress} \quad (2.30)$$

ρ_x, longitudinal reinforcement ratio.

$\rho_{y,}$ transversal reinforcement ratio.

θ, inclination struts of compression.

f_2, stress in concrete's struts.

$f_{sy,}$ stress in the transversal reinforcement

$f_{sx,}$ stress in the longitudinal reinforcement.

ε_x, longitudinal strain.

ε_y, transversal strain.

ε_2, principal strain of the compression struts.

ε_1, principal average strain normal to compression struts, measured in lengths that contain several cracks.

From these equations, it can be obtained θ, f_2, f_{sy}, f_{sx}, ε_x , ε_y , ε_2 y ε_1.

2.5.3 Code review

Spanish Code EHE-99

For concrete without web reinforcement, the EHE code of practice adopted the CM-90 formula with a minor variation:

$$V_c = \left[0{,}12\xi\left(100\rho_s f_{ck}\right)^{1/3} - 0{,}15\sigma'_{cd}\right] b_o d \qquad (2.31)$$

where, f_{ck} is in MPa and $f_{ck} \leq 60$ MPa,

ξ $= 1 + \sqrt{\dfrac{200}{d}}$ with d in mm

ρ_1 $= \dfrac{A_{sl}}{b_w d} \leq 0{,}02$

A_{sl} is the area of the anchored tensile reinforcement

b_0 is the width of the cross-section (mm)

σ'_{cd} $=N_d/A_c$, being N_d the factored axial force including the prestress force (tensile positive) and A_c the cross sectional area of concrete

V_{Rd} is in Newtons

The concrete safety factor is also included in equation 2.41. Factor 0,12 of that equation should be changed by 0,15 not to consider it.

For concrete with web reinforcement The EHE code of practice assumes that a concrete contribution, V_c, can be added to the steel contribution. Hence:

$$V = V_s + \beta V_c \qquad (2.32)$$

$$V_c = \left[0{,}10\left(1+\sqrt{\frac{200}{d}}\right)\left(100\rho_s f_{ck}\right)^{1/3} - 0.15\sigma'_{cd} \right] b_o d \qquad (2.33)$$

where all the parameters have the same meaning than for members without web reinforcement. The steel contribution is given by:

$$V_s = \frac{A_{sw}}{s} z f_{ywd} \cot\vartheta \qquad (2.34)$$

with cot θ compressed between 0,5 and 2. For non-prestressed members without axial force β equals 1 if θ is taken as 45º. If cot θ is assumed to be equal to 2 (hence, θ ≈ 26,6º) then β= 0.

Eurocode 2: Final Draft April 2003

For concrete without web reinforcement, the final version of the new draft of the Eurocode 2 presents a different shear procedure to that of the current version. It is based on the MC-90 equation with certain variations. The design value for the shear resistance for non-prestressed members which do not require design shear reinforcement is given by:

$$V_{Rd,c} = \left[\frac{0{,}18}{\gamma_c} k\left(100\rho_1 f_{ck}\right)^{1/3} + 0{,}15\sigma_{cp} \right] b_w d \qquad (2.35)$$

with a minimum of

$$V_{Rd,\min} = \left[0{,}035 k^{3/2} f_{ck}^{1/2}\right] b_w d \qquad (2.36)$$

where,

f_{ck} is in MPa and $f_{ck} \leq 100$ MPa

k $= 1+\sqrt{\frac{200}{d}} \leq 2{,}0$ with d in mm

ρ_1 $= \frac{A_{sl}}{b_w d} \leq 0{,}02$

A_{sl} is the area of the anchored tensile reinforcement

b_w is the smallest width of the cross-section in the tensile area (mm)

σ_{cp} $=N_{Ed}/A_c < 0{,}2\ f_{cd}$ (MPa). N_{Ed} is the axial force in the cross-section due to loading or prestressing in Newton ($N_{Ed} > 0$ for the compression). The influence of imposed deformations on N_E may be ignored. Ac is the area of concrete cross section (mm^2).

V_{Rd} is in Newtons

For members requiring design shear reinforcement, their design is based on a truss model. For members with vertical shear reinforcement, the shear resistance, $V_{Rd,s}$ should be taken as the lesser of:

$$V_{Rd,s} = \frac{A_{sw}}{s} z f_{ywd} \cot\theta \qquad (2.37)$$

and

$$V_{Rd,\max} = \alpha_c b_w z \nu f_{cd} / (\cot\theta + \tan\theta) \qquad (2.38)$$

The limiting recommended values of cotθ are given in the next expression

$$1 \leq \cot\theta \leq 2,5$$

Where,

A_{sw}	is the cross-sectional area of the shear reinforcement
s	is the spacing of the stirrups
f_{ywd}	is the yield strength of the shear reinforcement
ν	may be taken as 0,6 for $f_{ck} \leq 60$ Mpa, and 0,9-f_{ck}/200 for high-strength concrete beams
α_c	= 1, for non-prestressed structures.

AASHTO LRFD 2000

The AASHTO-LRFD shear design procedure is based on the modified compression field theory. The nominal shear resistance for a non-prestressed member without shear reinforcement is given by:

$$V_c = \beta \sqrt{f'_c} b_v d_v \qquad (2.39)$$

the values of β and θ, depend on the equivalent crack spacing parameter s_{xe}, where

$$s_{xe} = \frac{35}{a + 16} s_x \qquad (2.40)$$

where *a* is the maximum aggregate size, and s_x is the crack spacing parameter as it is defined in figure 2.26.

The longitudinal strain in the web, ε_x, can be found from the longitudinal strain in the flexural tension flange, ε_t, where

$$\varepsilon_t = \frac{\dfrac{M_f}{d_v} + V_f - \phi_p V_p + 0.5 N_f - A_p f_{p0}}{E_s A_s + E_p A_p} \tag{2.41}$$

and f_{p0} can be taken as $0,7f_{pu}$ for usual levels of prestress. For members without stirrups ε_x can be taken as ε_t (see fig 2.26).

The shear strength of the *reinforced concrete* section is expressed as

$$V_n = \beta \sqrt{f_c^{'}} b_v d_v + \frac{A_v f_y}{s} d_v \cot\theta \tag{2.42}$$

The values of β and θ listed in the figure 2.27 are based on calculating the stress that can be transmitted across diagonally cracked concrete which contains at least the minimum amount of transverse reinforcement required for crack control.
The shear stress in figure 2.27 can be defined as

$$v = \frac{V_n - V_p}{b_v d_v} \tag{2.43}$$

For members with stirrups ε_x can be taken as $0,5 \cdot \varepsilon_t$, as justified in figure 2.29. ε_t is calculated by the equation 2.51.

CSA (2004)

The CSA shear design procedure in flexural region is based on the modified compression field theory. Required Shear resistance is factored

$$Vr=Vc+Vs \tag{2.44}$$

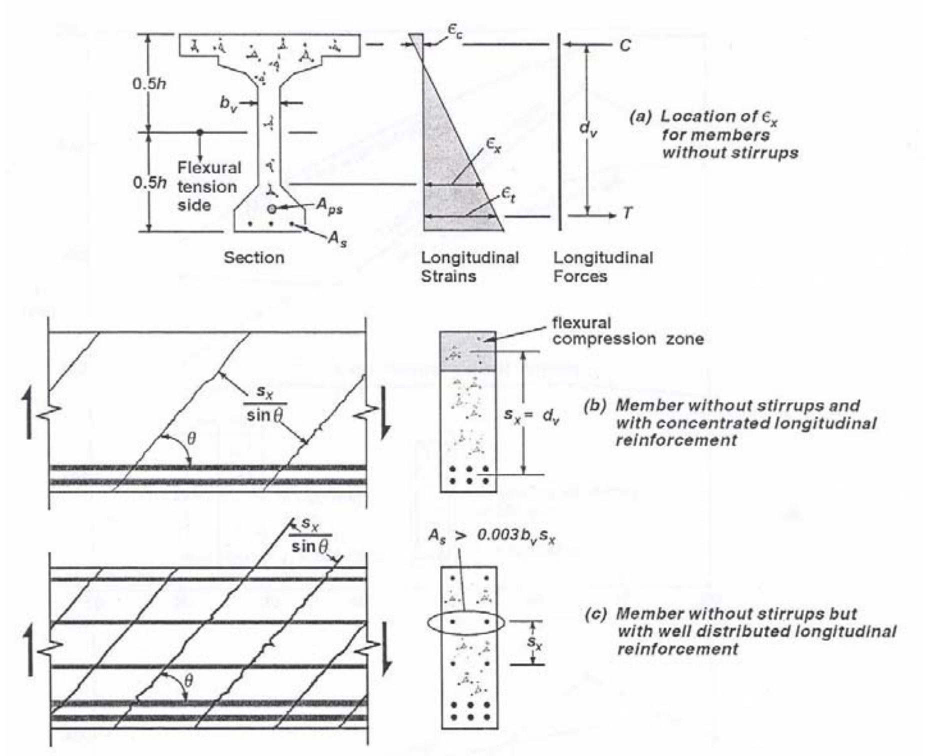
$$Vr \leq 0,25\phi_c f_c' b_w d_v \tag{2.45}$$

Determination of V_c

$$V_c = \phi_c \lambda \beta \sqrt{f_c^{'}} b_w d_v \tag{2.46}$$

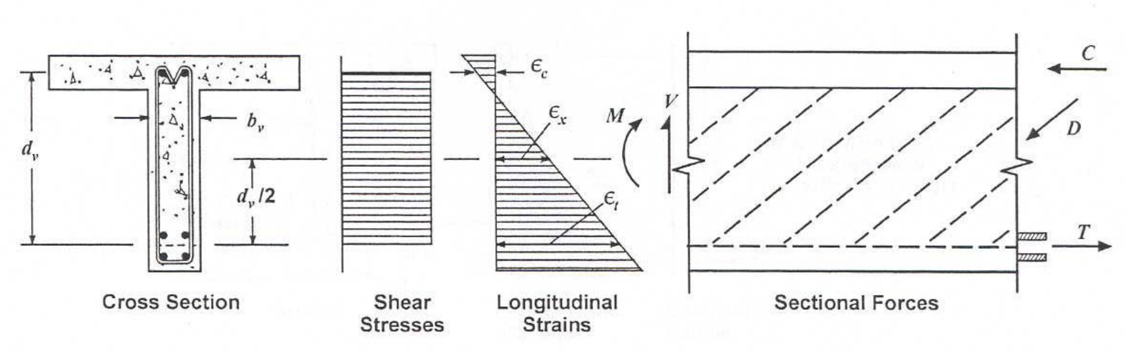
where

$$\beta = \frac{0.40}{(1+1500\varepsilon_x)} \cdot \frac{1300}{(1000+s_{ze})} \tag{2.47}$$



S_z		ε_xx 1000							
		≤ 0.00	≤ 0.125	≤ 0.25	≤ 0.50	≤ 0.75	≤ 1.00	≤ 1.50	≤ 2.00
≤ 125	β	0,428	0,366	0,325	0,271	0,238	0,214	0,184	0,163
	θ	26,4	27,7	28,9	30,9	32,4	33,7	35,6	37,2
≤ 250	β	0,406	0,336	0,292	0,239	0,208	0,185	0,156	0,137
	θ	29,3	31,6	33,5	36,3	38,4	40,1	42,7	44,7
≤ 375	β	0,393	0,317	0,272	0,219	0,188	0,167	0,140	0,121
	θ	31,1	34,1	36,5	39,9	42,4	44,4	47,4	49,7
≤ 500	β	0,383	0,303	0,257	0,204	0,174	0,154	0,126	0,109
	θ	32,3	36	38,8	42,7	45,5	47,6	50,9	53,4
≤ 750	β	0,368	0,282	0,234	0,182	0,153	0,133	0,108	0,091
	θ	34,2	38,9	42,3	46,9	50,1	52,6	56,3	59
≤ 1000	β	0,337	0,266	0,218	0,166	0,138	0,119	0,095	0,079
	θ	36,6	41,1	45	50,2	53,7	56,3	60,2	63
≤ 1500	β	0,291	0,242	0,193	0,143	0,116	0,098	0,076	0,062
	θ	40,8	44,5	49,2	55,1	58,9	61,8	65,8	68,6
≤ 2000	β	0,257	0,225	0,175	0,126	0,100	0,084	0,063	0,051
	θ	44,3	47,1	52,3	58,7	62,8	65,7	69,7	72,4

Fig. 2.26: Values of β *and* θ *for section without shear reinforcement*



$\frac{v_f}{\phi_c f_c'}$		ε_xx 1000					
		≤ 0.00	≤ 0.125	≤ 0.25	≤ 0.50	≤ 0.75	≤ 1.00
≤ 0,075	β θ	0,311 21,8	0,269 24,3	0,244 26,6	0,215 30,5	0,198 33,7	0,185 36,4
≤ 0,100	β θ	0,261 22,5	0,242 24,9	0,228 27,1	0,208 30,8	0,193 34	0,181 36,7
≤ 0,125	β θ	0,238 23,7	0,228 25,9	0,218 27,9	0,201 31,4	0,188 34,4	0,177 37
≤ 0,150	β θ	0,226 25	0,216 26,9	0,209 28,8	0,196 32,1	0,184 34,9	0,173 37,3
≤ 0,175	β θ	0,21 26,26	0,209 28	0,203 29,7	0,189 32,7	0,178 35,2	0,163 36,8
≤ 0,200	β θ	0,208 27,4	0,202 29,0	0,197 30,6	0,178 32,8	0,161 34,5	0,149 36,1
≤ 0,225	β θ	0,199 28,5	0,194 30,0	0,178 30,8	0,154 32,3	0,144 34,0	0,136 35,7
≤ 0,250	β θ	0,178 29,7	0,176 30,6	0,160 31,3	0,141 32,8	0,131 34,3	0,125 35,8

Fig. 2.27: Values of β *and* θ *for section containing at least the minimum amount of shear reinforcement*

For sections containing at least minimum transversal reinforcement, the equivalent crack spacing parameter, s_{ze} shall be taken as equal to 300 mm, otherwise s_{ze} shall be computed by eq. 2.58.

$$s_{ze} = \frac{35 s_z}{15 + a_g} \qquad (2.48)$$

Where,

S_z, is the distance between reinforcements
A_g, is the maximum aggregate size of coarse aggregate. If f_c'≥70 MPa, take a_g=0

The angle of inclination θ of the diagonal compressive stresses shall be calculated as

$$\theta = 29 + 7000\varepsilon_x \qquad (2.49)$$

In lieu of more accurate calculations, the longitudinal strain, ε_x, shall be computed from

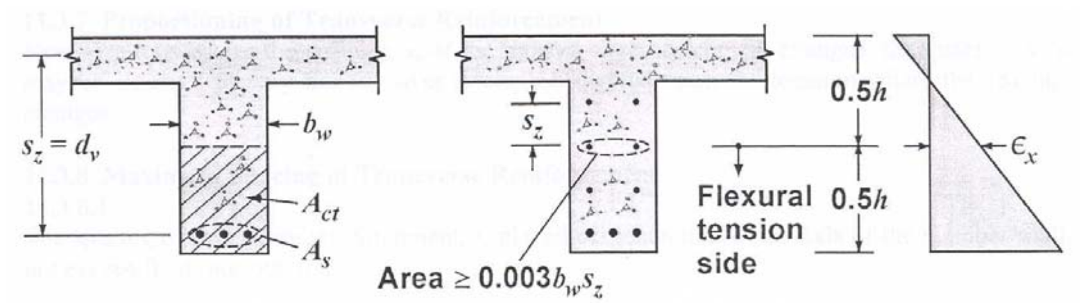
$$\varepsilon_x = \frac{M_f / d_v + V_f}{2 E_s A_s} \tag{2.50}$$

Determination of Vs

For members with transverse reinforcement perpendicular to the longitudinal axis, V_s shall be computed from

$$V_s = \frac{\phi_s A_v f_y d_v \cot\theta}{s} \tag{2.51}$$

where θ is determined in accordance with eq. 2.59.



2.5.4 Method proposed by Cladera and Marí

Cladera and Marí (2003), in their recent research carried out in the *Universitat Politecnica de Catalunya* proposed a new method of determining the ultimate shear stress in concrete beams with and without transversal reinforcement.

Beams without web reinforcement. The simplified shear design method adopts a size effect term similar to that of the EHE one, and it limits the concrete compressive strength to 60 MPa.

$$V_c = \left[0{,}225\xi\left(100\rho_s\right)^{1/2} f_c^{0,2}\right] b_w d \tag{2.52}$$

where

$\xi = 1 + \sqrt{\frac{200}{s_x}} \leq 2{,}75$ is the size effect;

s_x is whichever is smaller, d_v or the vertical between longitudinal distributed reinforcement.

d_v is the mechanical depth. Taken to be 0,9·d,

d is the effective depth in mm,

$\rho_s = \frac{A_{sl}}{b_w d} \le 0{,}02\left(1 + \frac{f_c}{100}\right)$ is the amount of longitudinal reinforcement,

$f_c \le 60$ MPa and

b_w is the smallest width of the cross-section area in mm.

Beams with web reinforcement

For members with web reinforcement, the failure shear strength is given by:

$V = V_c + V_s$

$$V_c = \left[0{,}17\xi\left(100\rho_s\right)^{1/2} f_c^{0,2} \tau^{1/3}\right] b_w d \qquad (2.53)$$

where

$\xi = 1 + \sqrt{\frac{200}{s_x}} \le 2{,}75$ is the size effect

s_x is whichever is smaller, d_v or the vertical distance between longitudinal distributed reinforcement.

d_v is the mechanical depth which can be taken as 0,9·d,

d is the effective depth in mm,

$\rho_s = \frac{A_{sl}}{b_w d} \le 0{,}04$, is the amount of longitudinal reinforcement,

fc≤ 100 MPa,

$\tau = \frac{V_d}{b_w d_v} \le 3$ MPa, and

b_w, the smallest width of the cross-section area in mm.

And,

$$V_s = d_v \frac{A_{sw}}{s} f_{ywd} \cot\theta \qquad (2.54)$$

where

A_{sw} is the cross-sectional area of the shear reinforcement,

S is the spacing of the stirrups,

f_{ywd} is the design yielding strength of the shear reinforcement, and

θ is the angle of the compression struts, derived as follows:

$$\theta = 20 + 15\varepsilon_x + 45\frac{\tau}{f_{ck}} \leq 45^{\mathrm{o}} \tag{2.55}$$

where

ε_x is the longitudinal strain in the web, expressed in 1/1000, calculated by the following expression:

$$\varepsilon_x \approx 0{,}5\frac{\frac{M_d}{d_v} + V_d}{E_s A_{sl}} \cdot 1000 \leq 1 \tag{2.56}$$

$$\frac{\tau}{f_{cd}} \geq 0{,}05$$

The expression of the longitudinal strain in the web is a conservative simplification of the real strain. It assumes that in the web the strain is equal to one half the strain in the tension reinforcement, and that the maximum longitudinal strain of the reinforcement is 0,002.

Simplified shear design method

To apply the procedure presented above, it is necessary to evaluate the shear strength in different sections of the beam, due to the interaction between the bending moment and the shear strength. The simplified shear design method assumes that the longitudinal strain in the web, ε_x, is equal to 1, and therefore that the longitudinal reinforcement yields; this is the worst condition under which to calculate the shear strength. Hence:

$$\theta = 35 + 45\frac{\tau}{f_{ck}} \leq 45^{\mathrm{o}} \tag{2.57}$$

where

$$\frac{\tau}{f_{ck}} \geq 0{,}05$$

2.6 CONCLUSIONS

From the mid-seventies onwards the properties of recycled aggregates and their application have been studied throughout the world. My conclusions obtained from the research and investigation carried out are the following.

- The recycled aggregates obtained from crushed concrete consists of adhered mortar and original aggregates. The quantity of adhered mortar in recycled aggregates is higher in small size aggregates. Due to the adhered mortar in original aggregates mechanical and physical properties of recycled aggregates are worse than those of raw aggregates. Recycled aggregates' properties: density, absorption, porosity, Los Angeles abrasion (this test is appropriate to verify that the crushed concrete is not damaged by fire), freezing and thawing resistance are inferior in quality to those of raw aggregates.

- According to RILEM recommendations, the recycled aggregates obtained from crushed concrete, should be defined as type II. Type II is a material that originated primarily from concrete rubble. The recycled aggregate must have a lower than 10% water absorption capacity and a minimal dry particle density of 2000 kg/ m^3. Recycled aggregate concrete is allowed to achieve 50/60 MPa. It does not require an additional test to be used in exposure class 1. In order to use in other exposure classes ASR expansion and Bulk freeze-thaw test are required.

- According to DIN 4226-100, 2000; aggregates obtained from crushed concretes are defined as Type 1. 90% of the material is crushed concrete. 2000 kg/m^3 is the minimal density required with 10% of maximum water absorption capacity. The value of acid or salt chlorides of all types of aggregates employed cannot pass the value of 0,04% of the mass.

- According DIN 4226-100, 2000 recycled aggregates require a quality certificate. The recycled aggregate must be certified, all their properties being defined by the DIN Code for aggregates for use in concrete production. Production control is also required and the certificate of quality will be provided only after external controller verification.

- According to prEN13242: 2002, the properties of recycled aggregates are defined by categories. The properties of recycled aggregates should be checked against the Codes of each country and their applicability. Aggregates with absorption capacity higher than 2% are not resistant to freezing and thawing. Therefore the percentage of lost mass by magnesium sulphate attack must be determined. But according to many authors the sulphate soundness test is unsuitable for evaluating the aggregates within the cement paste.

- The water absorption capacity of recycled aggregates has to be taken into account when using recycled aggregate in concrete production. The recycled coarse aggregates used in concrete manufacture should be kept in humid conditions. This will ensure not only concrete's workability but also the effective w/c ratio. If the recycled aggregates are used in this condition the new interface transition zone can be effective, producing better properties, and prevention to freezing and thawing. The new interfacial transition zone also depends on the concrete production process. Although it is not possible to improve the old interface transition zone it is possible to achieve an effective new transition zone which produces a low w/c ratio cement paste on the interface.

- In concrete made with 100% of recycled coarse aggregates the effective w/c ratio must be lower than that of conventional concrete in order to obtain the same compression strength. Therefore, in recycled aggregate concretes (using more than 50% of recycled coarse aggregates) more cement than conventional concretes is necessary to achieve the same workability and compression strength.

- The compression strength of recycled aggregate concrete depends on the strength of the original concrete. The adhered mortar of recycled aggregates can be the weakest point in the concrete.

- There is not a significant change in the properties of concrete made with 20-30% of recycled coarse aggregates with respect to that of conventional concrete.

- Concretes made with 50 and 100% of recycled aggregate's strength have a lower increase in compression strength from 7 to 28 days than those of conventional concrete employing only raw aggregates.

- The variation coefficient of recycled aggregate concrete is higher than conventional concrete.

- The tension strength of concrete made with recycled aggregates and natural sand is similar to conventional concrete. However, if recycled aggregates are saturated at concrete production, the tension strength of recycled aggregate concrete decreases.

- The modulus elasticity of recycled aggregate concrete is always lower than conventional concrete.

- With respect to durability;

    - Concrete made with recycled aggregates needs to have a lower effective w/c ratio to achieve lower permeability.
    - The freezing and thawing resistance is lower in recycled aggregates concrete than in conventional concrete. However it can be improved if the recycled aggregates are humid and the air-entrained is used at concrete production.
    - A lower w/c ratio can improve rust risk in recycled aggregate concrete, decreasing its permeability.
    - The rubble processed at recycling plants may originate from structures which were attacked by ASR or which were potentially reactive, but did not react due to a lack of favourable conditions (such as humidity). Preventive measures such as the use of low alkali portland or blast furnace slag cement, may increase the durability of the recycled concrete as far as ASR is concerned.

- With respect to the structural behaviour of reinforced recycled aggregate concrete;

    - The first cracking load is lower in recycled aggregate concrete specimens than that of conventional concrete.
    - According to flexural and shear behaviour, the ultimate load is similar in reinforced recycled aggregate concrete specimens and conventional concretes.
    - The bond resistance in recycled aggregate concrete is lower than that of conventional concrete.

Great progress has been made with respect to the analytical solution of shear problems in concrete since Mörsh and Ritter postulated the earliest truss models at the beginning of the XX century. However, most of the high-sophisticated tools employed in determining failure shear load require considerable simplification to make it suitable for code formulation. All Codes are used in conventional concrete. There is no adaptation to concrete made with recycled aggregates.

For beam specimens with web reinforcement, the lack of studies carried out with respect to the concrete's contribution (85%) in a beam with stirrups equal to the same beam without transverse reinforcement should be corrected. The AASHTO procedure and CSA (2004) which are based on the MCFT, present a more rational approach to that of the EHE or the Eurocode-2. The use of AASHTO is by tables whereas the CSA (2004) method is analytical and the results are rather good.

Chapter 3

Material properties and dosage election of recycled aggregate concrete

3.1 INTRODUCTION

One of the objectives of the experimental phase was to analyse the structural behaviour of reinforced concrete made with different percentages of recycled aggregate, all of which had the same compressive strength. In this chapter all the elements of the concrete are analysed and the different mix proportions of concrete are determined to achieve the same compression strength.

The recycled aggregates employed to produce the concrete were taken from a waste-recycling area. The origins of the concretes were unknown therefore the composition was evidently heterogeneous. A raw coarse aggregate (granite) and sand (crushed limestone) were used in different concrete mixes.

The characteristics of the aggregates were established in order to study their possible application in concrete production. After analysis, the dosage procedure was carried out.

In order to produce the following four concretes; control concrete (HC), recycled aggregate concrete with 25% of recycled coarse aggregates (HR25), recycled aggregate concrete with 50% of recycled coarse aggregates (HR50) and recycled aggregate concrete with 100% of recycled coarse aggregates (HR100), different dosages were used to get the same compression strength in all of the concretes.

In this chapter the recycling plant, properties of aggregates, dosages of these four concretes and mechanical properties of different concretes are studied to obtain the same compression strength at 28 days.

3.2 RECYCLED AGGREGATES

3.2.1 Recycling plant

The recycled aggregates were obtained by use of a mobile crusher. Its lay-out is defined in figure 3.1. This mobile crusher was situated on site inside the waste area and only clean concrete was crushed. The 0/40 mm fraction was obtained, as seen in figure 3.1. Varying qualities of crushed concretes were accumulated together, without checking their quality or if they had suffered from some unknown pathology.

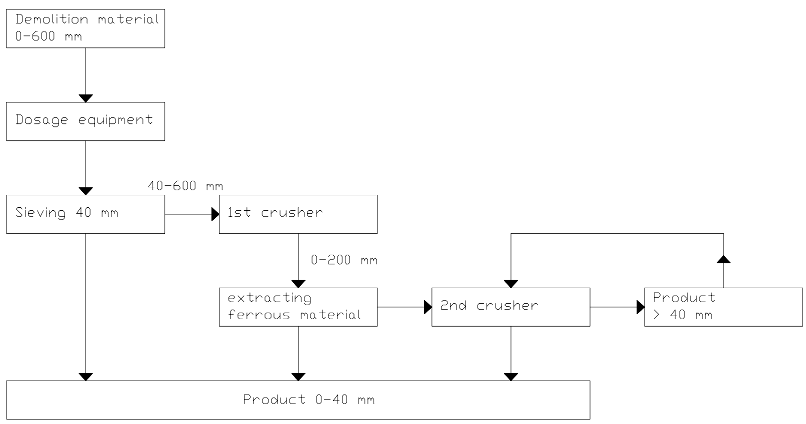


Fig. 3.1: Existing Recycling Plant

Impact crushers were used in the production of recycled aggregates, as not only a high productivity but also a high standard in shape of the aggregates is achieved with this kind of crusher. However, the quantity of fine material (<5mm) produced is high (almost 50%) but in this case, as mentioned in chapter 2 and in general this fine material is not used in recycled aggregate concrete production.

Fig. 3.2: Recycling plant

3.2.2 Grading and composition of natural and recycled aggregates

The quality of the aggregates used to produce concrete had to be determined. For conventional concrete production, each country has its own standard with respect to these materials' requirements. In order to use the recycled aggregates, it is imperative that their properties be determined, due to their heterogeneous nature and because they are already a manufactured material which has a life expectancy, and as in this case, their life expectancy and condition is unknown.

As mentioned above, coarse aggregates (granite) were used as natural aggregates, and limestone sand was used in all the concrete mixes studied. The aggregates were transported to the laboratory in large bags. The sample testing of the aggregates was conducted according to UNE 932-1 and their properties were later determined. This was carried out according to the quantity of materials required in each test.

Aggregate sieves

In all cases it is necessary to identify the grading of aggregates in order for employment in concrete production. It is also important to use an aggregate with a grading that ensures reasonable workability, minimum segregation and minimum obtaining of air voids. Recycled and natural aggregates had the same fraction size, 4/10 mm, 10/16 mm and 16/25 mm. The same sieving machine was employed for both types of aggregates sieved, although different machines were used in the crushing process. The sieve distribution was kept within the margins of the determined standard employed. The fractional size of the limestone sand employed in all the concrete mixes was 0/4 mm.

Aggregate sieve distribution was in accordance with code UNE-EN 933-1,2. Coarse aggregates sieve distribution was compared with the limits referred to by the ASTM Standard. Figure 3.3 shows that the recycled coarse aggregates and raw coarse aggregate were within the limits required by the standards given. The quantity of recycled aggregates smaller than 4 mm were not within the given limits, however as this material was not employed, it was not considered in our findings. As mentioned above, the recycled aggregates were crushed by an impact crusher. This kind of crusher produces rather large quantities of fine materials, however, the quality of the coarse aggregates, independent of the concrete's quality is highly acceptable.

According to prEN 13242 (Final Draft) depending on general grading requirements the coarse recycled aggregates have a G 85-15 category, consequently the grading was adapted for use in concrete.

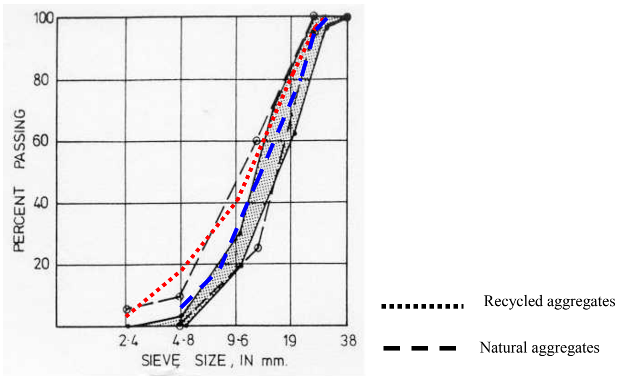


Fig.3.3: Sieve distribution according to ASTM

The natural coarse aggregate produced from the crushing was completely within the margins detailed by ASTM, so the sieve distribution was perfect for use in concrete mixes.

According to EHE-99 (Spanish structural concrete code) the sand grading was within the grading limits, as figure 3.4 shows.

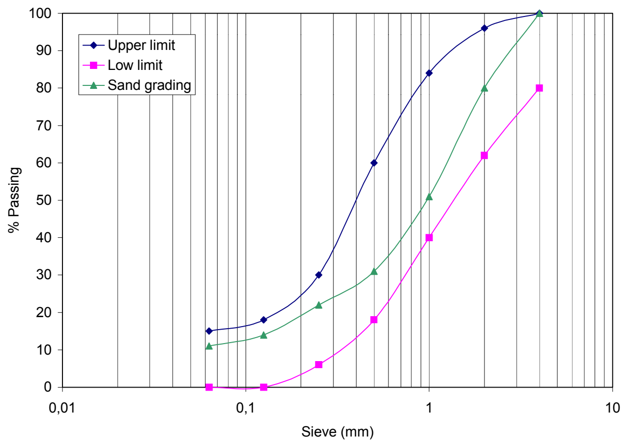


Fig.3.4: Sand grading

Material smaller than 63 µm was determined according to Normative UNE-EN 833-1. The quantity of this material was 1,09% and 1,42% in recycled and natural coarse aggregates, respectively. The recycled material was cleaner than the natural material employed. In the sand obtained from the crushed limestone the material smaller than 63 µm was 10.88%.

According to EHE-99, the coarse aggregates employed in concrete mixes must not exceed 1% of the finer material (63 µm) of any type of coarse aggregate except in limestone coarse aggregates where the value is 2%.

In the case in question, it can be stated that the recycled aggregates were within the legal requirements detailed in the normative previously mentioned, however the natural aggregates should have been cleaned before use. As the limestone aggregates had a higher limit than that of 1%, the granite natural coarse aggregates used must be considered as being dirty.

According to EHE-99 limestone sand has a 15% (63 µm) limit, consequently in this case the material smaller than 63µm was 10.88%, and could be used. According to prEN 13242:2002, the category of the recycled coarse aggregates is f_2 because the mass fraction after being passed through the 63 µm sieve was less than 2%.

According to normative DIN 4226-100 the category of the recycled coarse aggregates is also f_2 (defined as a mixture of gravels), and in this case the mass fraction after being passed through the 63 µm sieve was also less than 2%. The same values were considered in RILEM.

As stated above, limestone sand was used in all concrete mix production studied in this thesis. The sand equivalent was calculated to determine the relative proportions of detrimental fine dust or clay-like material in fine aggregates. The result was 77%, and according to EHE, it had to be larger than 75 to be employed in I and II exposure class.

Recycled aggregates composition

It is imperative to determine the composition of recycled aggregates if they are to be employed in concrete production. They have to have minimum property requirements to assure the concretes mechanical properties, low permeability and also durability. The recommendations existing regarding the composition of recycled aggregates that could be employed in the production of concrete are defined in chapter 2.

A representative sampling was taken to determine the composition of recycling aggregates. According to normative DIN 4226-100, a mass of 25 kg of each fraction of aggregates was taken. Four materials dominated in the composition of the recycled aggregates see figure 3.5.

Figure 3.5 is divided into 4 graphics; three of them (a), b) and c)) are of different fractions and the fourth one, d), is the average composition determined with participation of each fraction percentage in the concrete.

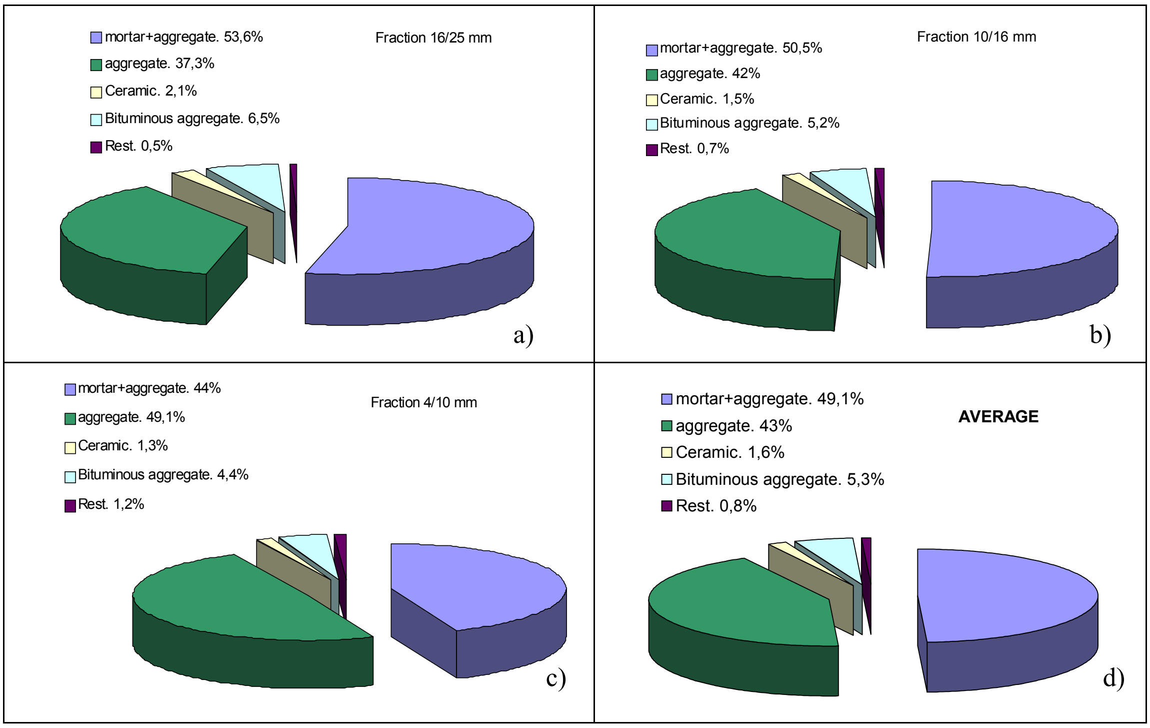


Fig.3.5: Recycled aggregate composition

The most important difference between the three fractions shows on a), b) and c) is that which occurred when the aggregate was finer, the rest of the composition (impure material) was higher, although the bituminous and ceramic material quantity was lower. In all the fractions, the quantity of aggregates and aggregates with adhered old mortar was higher than 90%, although the clean aggregates without mortar was higher when the aggregate was smaller. Similarly the quantity of aggregates with adhered old mortar was smaller.

The quantity of aggregates devoid of adhered old mortar was high due to the fact that either the impact crusher produced very acceptable quality aggregates or that probably, the original concrete was not a high strength concrete.

The type of original coarse aggregates was determined, due to their high percentage. A representative sample of the original coarse aggregates was taken and the types were classified as; 74% of crushed limestone rock, 10% granite, 10,6% metamorphics and 5,2% of quartz.

In chapter 2, some recommendations concerning the composition of recycled aggregates are given by RILEM and DIN 4226-100. According to RILEM this material is considered as Type II. Whereas by DIN 4226-100 (which has a more detailed description), classify the recycled aggregates as type 1: where used recycled aggregate had more than 90% of aggregates and concrete, and less than 2% of ceramic material and a little more than 1 % of bituminous material. In the case of the recycled aggregate employed in the concrete mixes studied the bituminous quantity was higher (5.0%). However considering that this bituminous material only affects the resistance of the concrete and not its durability or its interface quality, we considered the recycled aggregate acceptable for concrete production. Also, the percentage of any other material was within the limit defined by DIN 4226-100 as type 1.

Adhered mortar

There was no determining of existing adhered mortar in the recycled aggregates employed. The only applicable method to determine the amount of adhered mortar was through “thermal shock”. This method consists of putting the recycled aggregates (with the adhered mortar in a saturated condition) in an oven and heating to 500ºC. This produces internal forces which break the adhered mortar. It was impossible to employ the method of attacking the mortar with chloride acid, due to the heterogeneity of the material.

In principle the original concretes appeared as low strength concrete due to the high percentage of clean aggregates (without adhered mortar) after crushing. It was assumed that the quantity of adhered mortar was approximately 20% for fraction 10/25mm and approximately 40% to 4/10 mm fraction.

3.2.3. Mechanical and chemical properties of natural and recycled aggregates

In this section all the mechanical properties of the recycled coarse aggregates, natural coarse aggregates and also of limestone sand are determined by required tests. These aggregates should be used in concrete production, and the values of their mechanical properties must be in accordance with instruction EHE-99 requirements (Spanish structural concrete code) and European standards for recycled aggregates to be used in concretes.

Density

The density is the ratio between the mass and the volume of the material. In this case the recycled aggregate was a porous material, so it was very important to identify the volume (with or without pores), because the water trapped inside of the pores changed the mass of the aggregates. The properties were determined according to EN 1097-6:2000; Determination of particle density and water absorption.

It was necessary to know the density of the aggregates in order to determine the mix proportions of the concrete to be produced and discover the volume that they occupied in the mentioned concrete. The existing accessible and inaccessible pores were considered in the volume.

All the necessary tests were carried out for all fractions of recycled coarse aggregates, raw coarse aggregates and limestone sand. The values of the test are shown in table 3.1.

Table 3.1: Properties of the aggregates

Test EN 1097-6:2000	**EHE-99**	**Conventional aggregate**				**Recycled aggregate**		
	Aggregates	**Sand (0-4)**	**Coarse (4/10)**	**Coarse (10/16)**	**Coarse (16/25)**	**Coarse (4/10)**	**Coarse (10/16)**	**Coarse (16/25)**
Density (kg/m^3)	Density (kg/m^3)							
$\mathbf{d_{rd}}$	>2,0	2,495	2,634	2,655	2,663	2,306	2,327	2,361
$\mathbf{d_{ssd}}$	>2,0	2,560	2,662	2,679	2,682	2,415	2,427	2,452
$\mathbf{d_A}$	>2,0	2,669	2,710	2,719	2,714	2,589	2,586	2,598
Porosity (%)								
P		6,53	2,807	2,373	1,864	10,938	9,994	9,134
Absorption (%)								
$\mathbf{A_p}$	<5	2,619	1,066	0,894	0,700	4,743	4,296	4,296

Evidently, as defined in chapter 2, the density of recycled aggregate was lower than that of conventional aggregates. The average value of density of recycled coarse aggregate in a saturated and surface-dry condition was 2,430 kg/m^3. This value equated with the information contained in chapter 2 and was consequently expected.

The density values of recycled coarse and conventional aggregates were quite similar. Recycled aggregates having less density due to the adhered mortar. However the most important difference between these two aggregates is their absorption capacity. The recycled aggregates have mortar adhered to them and this material is porous, so the capacity to absorb water is much higher. This high absorption of water is one of the most important factors to take into consideration in the determining of material mix proportions in concrete production.

If we check the aggregates properties with RILEM and DIN 4226-100 requirements, according to RILEM the recycled aggregates are defined as Type II and according to DIN 4266-100 recycled aggregates are defined as type 1. Both types of the recycled aggregate described are appropriate for their use in concrete.

As previously mentioned, the sand and conventional aggregates were much denser and therefore had less absorption capacity than recycled coarse aggregate.

Porosity

The porosity is defined as the ratio between the accessible pore volume in the grains and the total volume of the aggregate sample including all the pores.

As shown in table 3.1, the porosity of the recycled aggregates is much larger. Evidently this is due to two factors, firstly because the recycled aggregates have some adhered mortar which is extremely porous, and secondly having suffered from “manufacturing” the quality of the original aggregates has been reduced (negatively affected).

Water content

It is imperative to ascertain the aggregates water content for concrete dosage and its production. This value is more important in recycled aggregates due to their absorption capacity. The water content or the humidity of the aggregates was measured according to EN 1097-5:2000. The mass of water content is the difference between the material mass in the situation of using and the dry mass.

It is normal for weather conditions to affect the humidity of the aggregates if they are kept outside, which is the normal case. The aggregates used in this thesis were transported to the university in winter and they were left in the open-air. The humidity of the aggregates was continually changing therefore the humidity levels had to be checked before each use. This factor is very important and it has to be considered in concrete production. Furthermore, in order to produce a well controlled concrete, the humidity has to be high. This means that the humidity of recycled aggregates is recommended to be between 3-3,5%, so most of the time the aggregates had to be wet. In this way the absorption capacity of the aggregates is reduced and the concrete dosage is controlled, also the w/c ratio and the fresh concrete workability too. Moreover in the concrete mixing process when the water is added the cement forms an effective interfacing bond to the wet aggregate which still has some absorption capacity.

In most cases of concrete production the aggregates have to be damp, but if they are already wet, due to being stored outside in the rain, then the humidity has be measured before mixing.

The humidity of the conventional aggregates and the limestone sand was also measured, because it too has an influence on the fresh concrete. Fine sand absorbs 100% of its capacity, therefore it was necessary to determine its humidity level. Likewise the humidity of the conventional coarse aggregates had to be measured. Their absorption capacity was low and it was necessary to wet them down.

Absorption

The water absorption of aggregate is determined by measuring the increase in mass of an oven-dried sample when immersed in water for 24 hours (the surface water being removed). The ratio of the increase in mass to the mass of the dry sample, expressed as a percentage, is termed absorption.

The aggregates absorption capacity was determined by keeping the sample submerged in water for 24 h. However when the aggregates are used for concrete production, (they have little contact time with water), they are never submerged in water, so the effective absorption is lower and difficult to determine. Consequently it is necessary to wet recycled aggregates in order to reduce the effect of this uncontrolled property. As table 3.1 shows, conventional aggregates had much lower capacity of absorption. The total absorption capacity was not considered in concrete production.

At the moment of concrete production, the recycled aggregates were wet, approximately 3-3.5% of humidity as an average value (not saturated). In this way, the absorption capacity was lower, the w/c ratio was controlled, as was the workability of the fresh concrete.

Table 3.1 shows that the recycled aggregates water absorption was 4.5%, none of the fractions cross the EHE’s limit of 5% with respect to use in structural concrete. The water absorption of the raw coarse aggregates and the sand were 0,8% and 2,6 %, respectively. For concrete dosage, all of the absorption capacities have to be taken into account, besides the humidity of the aggregates.

According to RILEM and DIN 4226-100, the recycled aggregates are type II and type 1 respectively. Both of these types are appropriate for use in concrete.

Normative prEN 13242:2002 (final draft) gives 2% as a maximum value of water absorption to define resistance to freezing and thawing, in this case some more tests were necessary to define this property in recycled coarse aggregates.

Los Angeles Abrasion

The recycled and conventional coarse aggregates were tested in accordance with normative EN 1097-2. Recycled aggregates had less resistance to abrasion than conventional aggregates see table 3.2. This was an expected result and was caused by the adhered mortar on the original aggregates, this material being much less resistant than that of the conventional aggregates.

Table 3.2: Los Angeles Abrasion of aggregates

Test EN 1097-2	EHE-99	Recycled aggregate (All-in)	Raw aggregate (All-in)
Los Angeles abrasion (%)	40	33.5	19.8

The loss of quantity of material after tests was not so high, because there were many original aggregates with little mortar adhered to them. In accordance with EHE-99, the aggregates with <40% of "Los Angeles abrasion" are able to be used in concrete.

According to European requirements for recycled aggregates, RILEM and DIN 4226-100 propose that this property has to be known and analyzed depending on the application of the material and each country's standard. In accordance with prEN 13242:2002, the category of recycled aggregates is LA_{40}.

Shape index

The shape index of the aggregates has to be taken into consideration in order to produce an effective interface between the aggregates and cement paste, as well as to give more or less workability to fresh concrete. This property was determined in accordance with UNE EN 933-3. The test was carried out for recycled and conventional coarse aggregates. While the recycled coarse aggregates were crushed by an impact crusher, the raw aggregates were crushed by a conical one. The impact crusher produces a better result, see table 3.3.

Table 3.3: Shape index of aggregates

	EHE-99	Raw aggregate			Recycled aggregate		
	Any aggregate	6-12 mm	12-18 mm	18-25 mm	6-12 mm	12-18 mm	18-25 mm
Shape coefficient (%)	≥ 20	21	24	29	24	28	31

According to EHE-99, similar levels of raw or recycled coarse aggregates could be used in structural concretes.

European requirements of recycled aggregates, RILEM and DIN 4226-100, propose that their properties have to be determined and analyzed depending on their usage and each country's standards.

In accordance with the normative prEN 13242:2002 the category of the aggregates is Sl_{40}.

Chemical analysis

The chemical analysis was carried out in accordance with the normative EN 1744-1 in each fraction of the three materials employed; recycled coarse aggregate, raw aggregate and limestone sand. Due to the probability of finding soluble elements in aggregates which could create certain durability problems in reinforced concrete. Within this European Standard there are four tests which were considered of extreme importance: 1) Determination of water-soluble chloride salts by potentiometry, 2) Determination of water-soluble sulphates, 3) Determination of acid soluble sulphates. The primary composition of recycled aggregates was also determined. The values obtained were taken from the average of two samples (see table 3.4) they were determined by fluorescent X-rays.

Table 3.4: Primary composition of recycled aggregates

	Fe_2O_3	MnO	TiO_2	CaO	K_2O	P_2O_5	SiO_2	Al_2O_3	MgO	Na_2O
Recycled aggregate	2,51	0,06	0,29	27,92	1,47	0,10	52,95	6,97	4,33	<Limit

As can be seen in table 3.5 the sulphate and chloride salts which were soluble in water were very low and could be considered as acceptable values.

Table 3.5: Chemical analysis

Tests EN 1744-1	EHE-99 Max. quantity in %, of sample's weigh	Raw aggregate Considered average	Sand	Recycled aggregate Considered average
Water-soluble chloride salts (%)	0.05	$0.8 \cdot 10^{-3}$	0.01%	$5.5 \cdot 10^{-3}$
Water-soluble Sulphates (% SO_3)	-	0.001	0.05%	0.055
Hydrochloric acid soluble sulphates (% SO_3)	0,8	0.05	-	0.2

According to RILEM the total admitted quantity of sulphate is 1 % m/m.

According to DIN 4226-100, chlorides must not exceed 0,04 % of the mass, and acid soluble sulphates must not exceed 0,8%.

In accordance with the normative prEN 13242:2002, the category of the recycled aggregate is $AS_{0,8}$, due to the fact that they had less than 0,8 % of acid soluble sulphates, and category S_1, with respect to the sulphur content, because the quantity of sulphurs was lower than 1%.

Weight loss by sulphates

As mentioned, recycled coarse aggregates have a more than 2% water absorption capacity, so it was necessary to determine their resistance to freezing and thawing by another test method. The method employed in accordance with the normative UNE-EN 1367-2 was the determination of the aggregates magnesium sulphate soundness.

Two representative samples were used in accordance with EN 932-1 and both samples gave approximately the same value as presented in table 3.6. Although the values slightly exceeded the maximum value required by Spanish Standards, these aggregates were considered usable in certain exposure conditions where thawing and freezing do not exist.

Table 3.6: Magnesium sulphate soundness

Test. UNE-EN 1367-2	EHE-99	Recycled aggregates Sample 1	Recycled aggregates Sample 2
Magnesium sulphate soundness (%)	18	19.8	19.35

In accordance with prEN 13242:2002, the category of the recycled aggregates is MS_{25}, because the loss mass fraction was lower than 25%. A better category exists which is defined as 18% of lost material.

It is considered that the limit of 18% mentioned is the best, as this provides the best qualification regarding resistance to freezing and thawing. The aggregates that have this category can be used in any exposure class. However, it is not applicable in our specific case, as the objective of this study was to test concretes using recycled aggregates where freezing and thawing did not occur.

The sulphate soundness test is unsuitable for evaluation of frost resistance of aggregates used in concrete as it only evaluates the aggregates themselves not the aggregates within the cement paste.

Alkali silica reactivity

After crushing by a mobile crusher the recycled aggregates obtained from the waste concrete were removed from the dump site. As previously mentioned, the origin of this concrete its age and properties were unknown but it was self evident that it had come from a demolition site. Due to its ambiguous origins it was necessary to conduct an alkali silica reactive test in order to verify the aggregates reactivity.

This test was practised according to UNE 146508:1998 however a change was made in relation to the w/c ratio with respect to the method in which the test was carried out. The test describes that the w/c ratio should be 0.47, however the workability of the fresh mortar applying this ratio was so small that the water quantity was increased to w/c 0.58. This change had little effect on the results obtained.

Recycled aggregates are formed by adhered mortar and original coarse and fine aggregates, the latter being also presented in the adhered mortar. Alkali Aggregate Reactive test of recycled aggregates and adhered mortar were carried out separately. Recycled aggregates suffered an expansion of 0,07% at 14 days, see figure 3.6-a. And the adhered mortar suffered an expansion of 0.1% at 14 days, see figure 3.6-b. According to ASTM C1260, an aggregate is potentially reactive with 0.1% of expansion at 14 days. Therefore, the original fine aggregates (present in adhered mortar) were considered potentially reactive. Consequently, the test was continued until 28 days, in which time the mortar bars achieved an expansion of 0.19% and with value the aggregates possible reactive state was verified, see figure 3.6-b. The aggregates went on expanding, so the test was continued for 56 days. At which time the aggregates reduced their expansion, see figure 3.c).

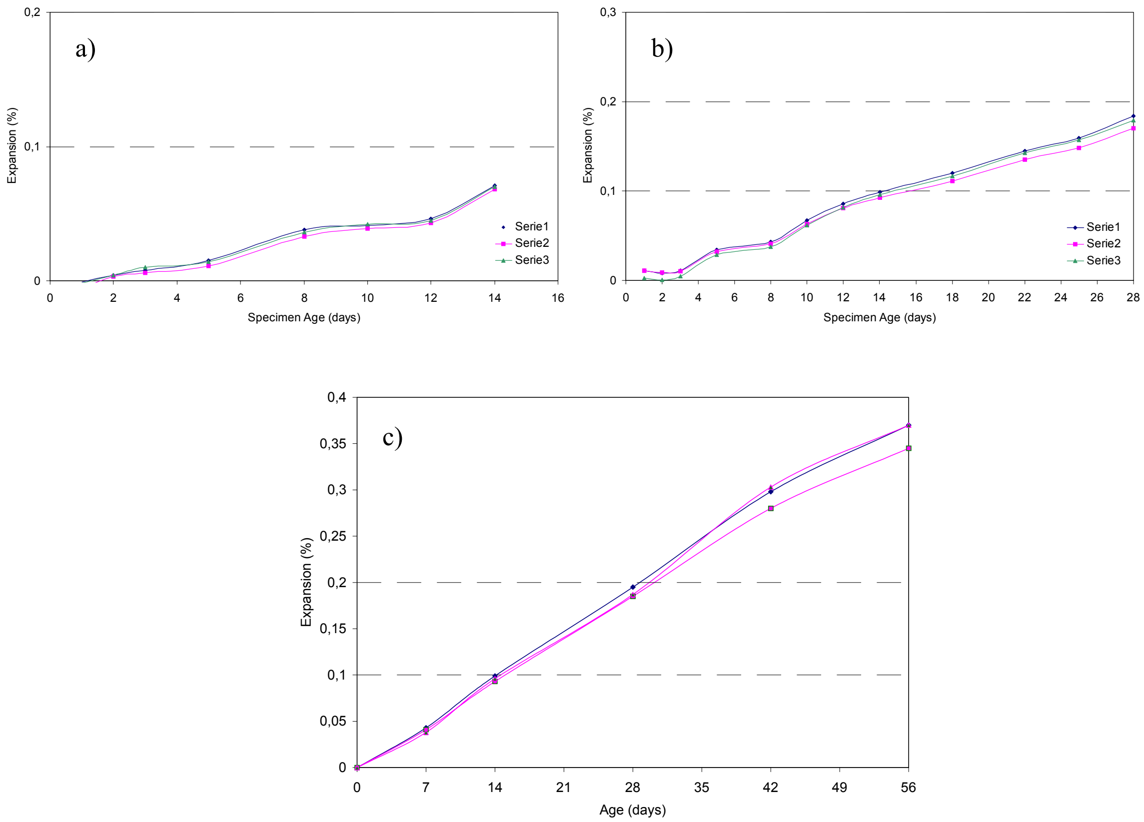


Fig 3.6: a) Expansion versus age for three samples of recycled aggregates. b) Expansion versus age for three samples of adhered mortar (original fine aggregates). c)Expansion versus age for three samples of adhered mortar attacked at 56 days

Contaminants

In the previous section it was mentioned that 5% of bituminous material and 1.5% of ceramic material were discovered in the analysis of the recycled aggregates composition. These materials, in the percentages given, were considered acceptable. A piece of aluminium was encountered during concrete production. This material is an unacceptable contaminate but as no more pieces of aluminium were found in the recycled aggregates and as they were free of other kinds of contaminants the recycled aggregates were considered in condition to be employed in the concrete mix.

Aggregates qualification depending on European requirements

The results of the testing of properties in the recycled aggregates composition were compared with European requirements. It is considered imperative to assess the global properties of the recycled coarse aggregate to be used in concrete production in order that the mentioned aggregate can be evaluated and categorised in accordance with European requirements.

The European requirements considered are, RILEM, DIN 4226-100 and prEN 13242:2002. All the properties discovered are categorised according to the stated requirements. As illustrated in table 3.7, the recycled aggregates are classified as Type II according to RILEM and Type 1 according to DIN 4226-100.

According to the latest final Draft prEN 13242:2002, all the properties have very acceptable qualities except for resistance to freezing and thawing (here the aggregates pass from the first level to be qualified as second level), however the obtained second level category is acceptable.

Table 3.7: Category of the recycling aggregates depending on the different European requirements

	Properties of Recycled aggregates											
	Grading	**<63μm**	**Composition**	**Density**	**Absorption**	**Abrasion**	**Shape Index**	**Total Sulphate**	**Chloride Salts**	**Total sulphur**	**Freeze Thaw**	**ARS**
RILEM	Type II	F_2	Type II	Type II	Type II	(*)	(*)	Type	Type	Type	(*)	(*)
DIN 4226-100	Tipe 1	F_2	Type 1 (**)	Type 1	Type 1	(*)	(*)	Type	Type	Type	(*)	(*)
prEN 13242:2002	G85-15	F_2	-	-	-	LA_{40}	SI_{40}	$AS_{0,8}$		S_1	MS_{25}	(*)

(*) Each country has to fix depending on the application and the standard.

(**) If it is considered the bituminous quantity, it will be defined as Type 2.

3.3 RECYCLED AND CONVENTIONAL CONCRETE CASTING OF TEST SPECIMENS

In order to find appropriate mix proportions it was necessary to produce test elements in the form of different concrete mixes. The objective of the research carried out was to produce four concrete mixes with the same compression strength, each employing a different percentage of recycled coarse aggregates. Several concrete beams were cast to compare the variations in their structural behaviour.

In chapter 2, different properties of concretes produced with recycled aggregates have been analyzed. Unfortunately, due to the fact that this type of research has never been carried out at length in Spain it was essential to define how the replacement of different percentage of raw aggregates for recycled ones could influence in the concrete's strength.

As previously mentioned, limestone sand was used as the fine aggregate in all the concrete mixes. The utilisation of recycled sand was avoided, due to its absorption capacity, which would no doubt produce a shrinkage effect. One must also consider the high probability of contaminants contained in recycled sand which evidently would have the negative effect of creating durability problems.

Four different dosages were employed in the casting of the concrete beams. The first concrete was a control concrete (HC), in this case raw, fine and coarse aggregates were used. In the second concrete, (HR25) 25% of the coarse raw aggregates were substituted for recycled coarse aggregates, in the third concrete (HR50) 50% of the coarse raw aggregates were substituted for recycled coarse aggregates and in the fourth one (HR100) 100% of the raw coarse aggregates were substituted for recycled coarse aggregates. The objective was to obtain the same compression strength in all four concretes.

CEM I 52.5R, a high quality, high strength and rapid-hardening Portland cement was used in all mixes. No other cement was employed. The dosage chosen to initiate the research was according to the instructions laid out in EHE, Spanish Instruction of structural concrete: exposure class II dosage. In order to assure the strength, impermeability and durability of the concrete it was necessary to employ: a minimum of 300 kg of cement/m^3 of concrete mix and maximum 0.55 effective w/c ratio. Concrete with the mentioned requirements is also adequate for use in exposure class I.

3.3.1 Material election

Aggregates

The aggregates are the basic material employed in concrete production. Two kinds of coarse aggregates were used. Raw coarse aggregates (granite aggregate), and recycled coarse aggregates. The recycled coarse aggregates were obtained by crushing different qualities of old concretes (as previously mentioned, in Spain it is very difficult to acquire a clean aggregate from concrete due to the demolition procedures employed). However, the heterogeneous nature of these aggregates was due to the different quality of the original concretes rather than their different components.

The fractions of the recycled aggregates employed in the mixes were the same as those of the raw coarse aggregates; 4/10 mm, 10/16 mm and 16/25 mm. Both materials were sieved in the same mobile sieving machine. As mentioned, all concretes were produced using limestone sand. Its fraction was 0/4 mm.

According to prEN 13242:2002 the recycled coarse aggregates are classified in categories depending on the percentage of crushed (broken particles) and total particles in coarse aggregates. In this case, the mass fraction of crushed or broken particles was between 90 to 100 and the category is $C_{90/3}$.

Cement

High strength and rapid-hardening Portland cement CEM I 52.5 R was the only cement used in the concrete production detailed. It was chosen due to its high tendency of use in Spain and because of the laboratories infrastructure.

Its chemical analysis and constituent are illustrated in table 3.8 and table 3.9, respectively.

Table 3.8: Chemical composition of the cement

Chemical Analysis								
Composition	**SiO2**	**Al_2O_3**	**Fe_2O_3**	**CaO**	**MgO**	**SO_3**	**Na_2O**	**K_2O**
%	20,19	5,25	3,68	62,81	1,80	3,02	0,15	0,86

Table 3.9: Constituent cement

Constituent					
Composition	**C3S**	**C2S**	**C3A**	**C4AF**	**Na_2O equiv.**
%	53,11	17,82	7,69	11,20	0,71

Additives

In order to achieve the same workability in all four different concretes, Glenium C313, superplastificizer was used. Its application and efficiency was guaranteed by the provider.

The specifications of the additive:

- Relative density(20ºC): 1,030±0,02 (g/cm^3)
- pH: 6±1
- Recommended dosage: from 1,0 to 2,5% on the cement weight

Water

Barcelona city's mains water was the only water employed and its purity guarantees its aptness for concrete production.

3.3.2 Dosage system and workability of fresh concrete

As stated in chapter 2, concretes with a certain percentage of recycled coarse aggregates need more quantity of cement and a little less w/c ratio than conventional concrete in order to obtain the same compression strength. In this case, four different concretes were produced. However, in order to define the suitable dosage, the tests were begun with the two most opposed concretes, control concrete (HC) and concrete made with 100% of recycled coarse aggregates (HR100). The acceptable mix proportions were considered when both concretes had the same compressive strength at 28 days. The concretes were used with the same slump which had been acquired with the superplasticizer.

The Bolomey dosage method was used in the mixing of both concretes, the dosage calculations began with the cement quantity and w/c ratio required for exposure class IIa. The aggregates percentage in each dosage was calculated by the Bolomey analytical method (determining the volume of each fraction). The weight of each fraction employed in the concrete mix was calculated by its density. The same method was used in the calculations of both concretes. The humidity of the aggregates was measured and their absorption capacity considered at the moment of concrete production. In the case of the limestone sand (which has a maximum capacity for water absorption) it was imperative to calculate the amount of water to be added to the mix, so as not to affect the effective w/c ratio and maintain the concrete's plasticity.

In concrete made with recycled coarse aggregates, when the w/c ratio is mentioned, it is considered as the effective w/c ratio (the useful w/c ratio in the paste).

This very important point must be taken into consideration, not so much in conventional concretes, but in concretes made with recycled aggregates. It is imperative to know the humidity of the recycled aggregates at the moment of dosing. There are several reasons for believing that recycled coarse aggregate should have high humidity content. Recycled aggregates have a high absorption capacity, if they are not humid their high absorption capacity would reduce the water in the paste thus losing both the workability of the concrete, and the control of the w/c ratio in the paste. For example, if the recycled aggregates contain 80% humidity the absorption capacity is low and both the fresh concrete's workability and its w/c ratio are controlled.

The high humidity of recycled aggregates must be taken into consideration as it is an important factor in the reduction of the influential variables. Consequently, one must take into account the humidity level of the recycled aggregate in order to control the quality in concrete production and achieve an equal compression strength. Recycled coarse aggregates must be wet before their employment in making concrete. An ideal level of humidity would be 80%, however the most important factor is that the aggregates employed are wet in order to reduce their absorption capacity. It is important to note, however, that they should not be saturated, as that would probably result in the failure of an effective interfacial transition zone between the saturated recycled coarse aggregates and the new cement paste. In the production of the recycled aggregate concretes studied, humid recycled aggregates were used.

As stated, in order to begin dosing, the minimum conditions of exposure class IIa were considered. In this class a minimum of 300 kg of cement/m^3 of concrete mix and maximum 0.55 effective w/c ratio are needed. Two stages were needed to achieve the objective of producing a conventional concrete (HC) and a 100% recycled coarse aggregate concrete (HR100) with the same compression strength. In Stage 1, the HC concrete was produced using 325 kg of cement /m^3 with a w/c ratio of 0.50. However, in order for the HR100 concrete to achieve a similar compression strength to that of the HC concrete it was necessary to use for too much cement. Therefore in Stage 2 the cement quantity employed in the conventional concrete (HC) was reduced to 300 kg of cement/m^3 of the concrete mix, with a w/c ratio of 0.55, while the same conditions were maintained for the production of the HR100 concrete as those detailed in stage 1.

The fresh concretes were used with 8-10 cm of slump. All concretes were produced with the same workability which had been attained using a supeplasticizer (Glenium C313). Although the recycled aggregates were humid (normally recycled aggregate concretes have less workability) the amount of additive used was slightly higher than that used for conventional concrete. The workability of fresh concrete was determined by a slump test, in accordance with UNE 83313:90.

The concretes of stage 1 and stage 2 were produced in a vertical axle mixer machine. The same process of adding materials was always used, first the cement and water were added and mixed during 60 seconds. After which the aggregates were added starting with the finer aggregates and terminating with the larger ones, the cement, water, sand and aggregates were mixed for 1 minute before adding the additive. The complete mixture was then mixed for 1 minute.

Cubic test elements were manufactured in these stages. The test elements were placed in the humidity room at 21ºC with 100% humidity for 2 hours before testing them at 7 and 28 days. All the test elements were kept in the same conditions before testing and the compression strength of the concrete was determined according to those laid out in normative UNE 83-304-84.

3.3.3 Production stage 1. Mix proportions for HC and HR100

As mentioned above, it was necessary to carry out the testing in two stages in order to achieve the objective of having the same compression strength in all concretes. Stage 1 dealt with experiments carried out on HC and HR100 concretes. These two concretes were chosen, because of their great comparative qualities.

The dosing of the control concrete, HC, began with 325 kg of cement/m^3 of concrete mix and 0.50 of effective w/c ratio, taking into consideration the density and the humidity of all the aggregates. 49,5 MPa and 60,3 MPa of compression strength were obtained at cubic test elements at 7 days and 28 days respectively. Different dosages were used to achieve the same strength in HR100, see table 3.10. Five dosages were used for 100% recycled aggregate concrete. The aggregates, cement and water are given in mass (kg) of the material to produce 1 m^3 of concrete and the additive is given as the % of the cement weight and the w/c is considered as the effective ratio in the paste.

Table 3.10: Dosages of control (HC) and recycled aggregates concrete (HR100)

	A 0/4 (kg)	G1 4/10 (kg)	G2 10/16 (kg)	G3 16/25 (kg)	Cement (kg/m^3)	Aditive	Water (kg)	Effectiv ew/c
HC	710.5	346.5	290.4	570.0	325	1.28%	178.7	0.50
HR100-1	660.7	422.2	300.7	383.9	325	2%	178.7	0.50
HR100-2	613.9	433.7	296.8	378.9	345	2%	189.7	0.43
HR100-3	586.8	448.5	298.0	380.3	365	2%	186.5	0.40
HR100-4	586.8	448.5	298.0	380.3	365	2%	186.6	0.4
HR100-5	660.7	422.2	300.7	383.9	325	0.58%	178.7	0.52

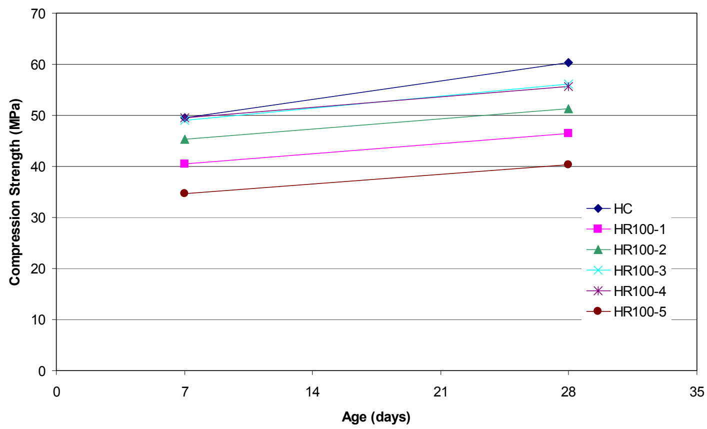


Fig.3.7: Concrete strength's evolution with the age

Table 3.11: Compressive Strength of different concretes. Strength % decrement of recycled aggregate concretes with respect to control concrete. Strength evolution with the age.

	Fc in MPa To 7 days. (fc % decrement respect HC)	Fc in MPa To 28days.(fc % decrement respect HC)	Percentage of increment of the strength from 7 to 28 days
HC	49,5	60,3	21.8%
HR100-1	40,5(18,2%)	46,5 (-22,8%)	14.8%
HR100-2	45,4 (8,3%)	51,3 (-14,9%)	13%
HR100-3	49 (-1%)	56,1 (-7%)	14.5%
HR100-4	49,5 (0%)	55,6 (-7.8%)	12.3%
HR100-5	34,7(-30%)	40,3(-33%)	16.1%

The first dosage of HR100-1 contained the same mix proportion as HC, the same quantity of cement and the same effective water/ cement ratio. It was discovered that after testing, the HR100-1 concrete had a 22.8% reduction of compression strength in comparison with the HC concrete, see figure 3.7 and table 3.11. Therefore it was necessary to increase the cement quantity and reduce the effective w/c ratio in the HR100-1 mix in order to achieve the compression strength of HC. In dosage HR100-2, 345 kg of cement/m^3 of concrete mix was used with an 0.43 effective w/c ratio, but this still gave a compression strength 15% less than the control concrete (HC) at 28 days. In dosage HR100-3, 365 kg of cement/m^3 of concrete mix was used with 0.40 effective w/c ratio, this produced 7% less compression strength than control concrete (HC) at 28 days. In dosage HR100-4, the process employed for the dosage of HR100-3 was repeated (conditions were maintained and no changes were made) and after testing it was discovered that the properties remained the same as those of HR100-3. Consequently it is evident that if concrete production is carried out in a methodical manner, the variables are quite controllable and the concretes' properties are quite easy to repeat.

However it is abundantly clear, that concretes with a high strength level are very difficult to produce employing 100% of recycled coarse aggregates. HR100-3 achieved only 7% lower compression strength than HC. However, HR100-3 needed for more cement to achieve that compression strength. Consequently it was not considered a viable option to utilise such a large amount of cement in the recycled aggregate concrete, due to the increment of cost.

The testing carried out in the first stage left us with some interesting points to consider. Employing the same methods and in exactly the same conditions the HR100 concrete which had the same cement quantity, the same effective w/c ratio and employed 100% recycled aggregate had 20-25% less compression strength than conventional concrete.

A comparison of conventional concrete with 100% recycled aggregate concrete (HR100) revealed that the conventional concretes' strength increased more than that of HR100 in the last 21 days of the 28 days allowed for curing. In figure 3.7 it is possible to verify that the evolution of different recycled concretes strength were almost parallel or at least very similar in the last 21 days of the 28 days allowed for curing. However, the strength evolution in the conventional concrete in the last 21 days mentioned was a little higher. A comparison of HR100 and HC compression strength revealed that there was no notable difference in both concretes after 7 days curing, the great difference in the compression strengths being noted after 28 days due to the time difference.

As previously mentioned, the quantity of cement needed to produce a concrete made with 100% of recycled aggregates with similar properties of the control concrete (HC which was initially made employing 325 kg of cement) was too high. Consequently, the dosage of the control concrete (HC) was changed from 325 kg to 300 kg of cemen/m^3 (of concrete mix) with 0.55 effective w/c ratio.

3.3.4 Production stage 2. Mix proportions of HC, HR25, HR50 and HR100

The control concrete (HC) dosage was changed to 300 kg of cement/m^3 of the concrete mix with an 0.55 effective w/c ratio. This concrete achieved 44.3 MPa of compressive strength at 28 days. This value was comparable with HR100-1's strength, of 46.5 MPa at 28 days. Once these two dosages were considered viable the manufacture of the HR25 and HR50 beams was carried out.

3.3.5 Chosen dosage and mechanical properties of the concretes

The definitive dosages for Control concrete (HC), concrete made with 25% (HR25), 50% (HR50) and 100% (HR100) of recycled coarse aggregate are illustrated in table 3.12. The aggregates, cement and water are given in mass (kg) of the material used to produce 1 m^3 of concrete mix. The additive is given as the % of the cement weight and the w/c is considered as the effective ratio in the paste.

With these dosages, similar compression strengths were obtained for all concretes as illustrated in table 3.13 and figure 3.8.

Several changes were carried out in HR50 and HR100 dosages in order to achieve the same compression strength in all concretes. HR25 was produced with the same proportions of materials as HC. The same quantity of cement was used with the same effective w/c ratio. Therefore, the HR25 achieved the compression strength of HC, without any notable variation with respect to HC. However, the HR50 mix needed 6% more of cement mass than HC and the effective w/c ratio was reduced to 0.52. For

HR100, 8,3% more cement mass was needed to achieve the HC's compression strength with an 0,50 effective w/c ratio.

Table 3.12: Definitive dosage for Control Concretes (HC), 25 % Recycled aggregate concrete (HR25), 50% recycled aggregate concrete (HR50) and 100% recycled aggregate concrete (HR100). Aggregates, cement and water are given in mass (kg)

	A 0/4	G1 4/10	G1R 4/10	G2 10/16	G2R 10/16	G3 16/25	G3 R16/25	C	Adit. %	W	w/c effec.
HC	765,13	332.7		295.07		579.2		300	0.97	165	0.55
HR25	765,1	249,5	72,8	221,3	64,6	434,4	128,3	300	0.79	165	0.55
HR50	739,0	172,1	150,6	147,4	129,2	289,4	256,6	318	0.84	175	0.52
HR100-1	683,2		425,8		306,4		391,2	325	1.38	179	0.50

Table 3.13: Definitive concretes strengths at 7 and 28 days in cubic tests elements

Concretes	Fc (MPa) 7 days	Fc (MPa) 28 days	Percentage of strength increment from 7 to 28 days
HC2	37.13	44.3	19.4%
R1	40.5	46.0	13.5%
R25	37.5	43	14.6%
R50	40.4	46.13	14.1%

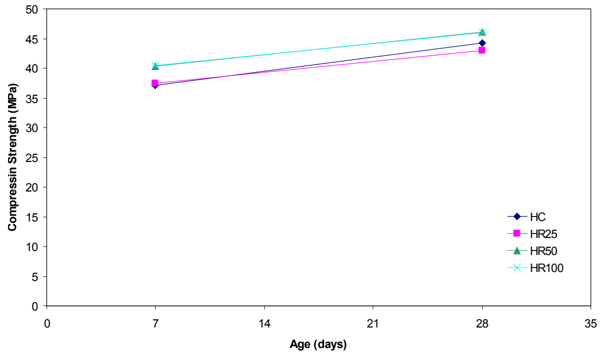


Fig. 3.8: Definitive concretes strengths at 7 and 28 days in cubic tests elements

The compression strengths of concrete mixes HR25, HR50 and HR100 was more or less the same with ageing (see figure 3.8). There was an increase of approximately 12-15% of compression strength of recycled aggregate concrete (HR25, HR50 and HR100) when the conventional concrete (HC) strength increased by approximately 20% in the last 21 days of the 28 day curing period.

Cylinder test elements were produced from defined mix proportions in order to determine the compression strength, tensile strength and modulus of elasticity of the four concretes produced during their 28 days of curing, see table 3.14.

Table 3.14: Mechanical properties of the concretes

	Density (kg/dm^3)	Fc (MPa)	Tensile Strength (MPa)	Modulus of elasticity (MPa)
HC	2.42	29	2,49	32561,7
HR25	2.40	28	2,97	31300,4
HR50	2.39	29	2,70	28591,7
HR100	2.34	28	2,72	27764,0

The tensile strength was similar in all of the concretes produced although as table 3.14 shows, the tensile strength of the recycled aggregate was higher than that of the control concrete (HC). This was probably due to the cleaner surface of the recycle aggregates in comparison to that of conventional raw aggregates.

The tests were conducted according to; UNE 83-304-84: compression strength, UNE 83-306-85: tensile strength and UNE 83-316-1996: modulus of elasticity.

3.3.6 Production of the chosen dosage concretes using an automatic mixer machine

Once the adequate dosage was found in stage 2, the production of different concretes in order to build the beam specimens was carried out. The main objective of this thesis is to analyse the structural behaviour of concrete made with different percentages of recycled coarse aggregates. In stage 1 and 2 a vertical axle mixer machine was used and the material was added manually. This machine which has a capacity of 60 l was the only machine used for the test elements production.

As the beam specimens construction required the automatic adding of material, a large mixer machine was employed. The mixer machine decided upon had a mixing capacity of 250 l. The manner in which the beam specimens were built is illustrated in chapter 5. In this chapter the mechanical properties of these concretes is studied.

The dosage used was the same as showed in table 3.12. It is important to mention that in this case, the machine employed had a much larger capacity for mixing as well as operating at a higher speed. Theoretically with this machine the obtained workability of the fresh concrete should be higher than that of the small machine (where the material was added manually). However a little more superplasticizer was used to maintain the same workability derived from a manual machine. The reason for this was due to the difference encountered in the machine’s order of mixing materials. In the automatic machine, in the first step of operating the fine and coarse aggregates were mixed for 30 seconds. The second step consisted of adding the cement and a further mixing of materials for 30 seconds. The third step consisted of adding water to the cement and aggregate mix and mixing for 1 minute. The fourth and final step consisted of manually adding the superplasticizer and a further mixing of all component materials for 1 minute before the mixing machine was stopped.

In this case the workability of the fresh concrete was also around 8-10 cm and the recycled aggregates which were always employed wet, contained about 80% humidity.

The production of HR25 and HR50 was somewhat complicated due to the manufacturing site. In the university where the beams were cast, there are only 4 silos but 7 silos would be needed in order to carry out concrete production using 7 different aggregates. Consequently, with respect to these concretes the recycled aggregates were weighed manually, instead of by automatic means.

The same procedures were employed in the production of the concrete except for the recycled coarse aggregate which was watered the day before production and its humidity determined. On the day of concrete production the humidity of all aggregates was again measured and its value used to calculate the concrete dosages.

Table 3.15: Mix proportions of beam specimens. The w/c ratio is an effective value in the paste. Aggregates, cement and water are given in mass (kg)

	A 0/4	G1 4/10	G1R 4/10	G2 10/16	G2R 10/16	G3 16/25	G3 R16/25	C	Adit. %	W	w/c effec.
HC	765,13	332.7		295.07		579.2		300	1.40	165	0.55
HR25	765,1	249,5	72,8	221,3	64,6	434,4	128,3	300	1.66	165	0.55
HR50	739,0	172,1	150,6	147,4	129,2	289,4	256,6	318	1.90	175	0.52
HR100	683,2		425,8		306,4		391,2	325	1.90	179	0.50

The fresh concretes' slump was measured on the mix leaving the mixing machine and its value was always found to be between 8 and 10 cm. The concrete beam specimens and cylinder test elements were made after testing the mix's slump value. This chapter deals with the measuring of the concretes properties through the use of cylinder test elements.

The concretes properties were determined at 28 days and at 6 months to analyse their properties during that time.

The properties were defined according to; UNE 83-304-84: compression strength, UNE 83-306-85: tensile strength, UNE 83-316-1996: modulus of elasticity. The tests results are shown in table 3.16.

Table 3.16: Properties of Beam specimens' concrete at 28 and 6 months

	Fc (MPa) 28 days	Fc (MPa) 6 months	Tensile Strength (MPa) 28 days	Tensile Strength (MPa) 6 months	Modulus of elasticity (MPa) 28 days	Modulus of elasticity (MPa) 6 months
HC	35,53	42,54 (+19%)	2,84	3,64 (28%)	32129	32437
HR25	38,79	46,28 (19%)	3,01	3,88 (29%)	32840	31427
HR50	39,42	44,4 (13%)	3,36	3,65 (8,6%)	32505	29758
HR100	38,26	38,66 (1%)	2,79	3.28 (18%)	28635	27063

All test elements had been in the humidity room at 21ºC and at 100% of humidity for 3 hours before they were tested.

The strength values of the concretes produced in the large automatic mixer machine were a little different to the previous findings, however in the next chapter it will be noted that the concretes of the beam specimens analysed had the same strength, consequently these strengths were considered as effective ones.

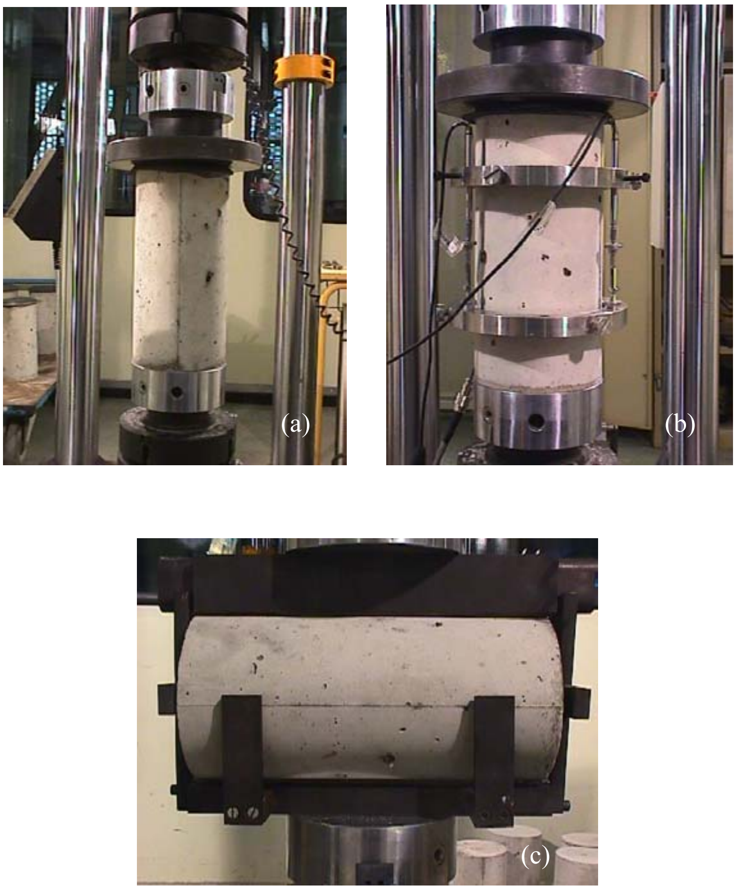


Fig. 3.9: Pictures of the mechanical properties tests. (a) Compression strength, (b) Modulus of elasticity, (c) tensile strength

Both the compressive and tensile strengths had similar values in all the different types of concretes produced. These similar values of compression strength in concretes with different percentage of recycled aggregates, were achieved with varying the quantity of cement employed.

The compression strength's evolution time was different in all concretes. It was discovered that with respect to control concrete (HC) its compression strength value at 28 days increased over the following 5 months and at 6 months there was a 19%

increase. The same increase in compression strength was also obtained in HR25 concrete. However, when the percentage of recycled aggregate employed in the mix was increased the compression strength measured at 6 months was smaller than those mentioned.

The recycled aggregate concretes had a larger tensile strength than those of the control concretes, except for the concrete where a 100% of recycled aggregate was employed. However, in contrast, the modulus of elasticity of the recycled aggregate concretes was reduced when the recycled aggregates percentage was increased. This situation was expected, because recycled aggregates were more prone to deformation than raw aggregates. What was not expected was the fact that the modulus of elasticity was less at 6 months than at 28 days.

Test results of the modulus elasticity values of the different concretes were compared with the values calculated which were based on the equations given by Ravindrarajah and Tam (1983) to determine the modulus elasticity of recycled aggregate concrete, as described in chapter 2. The values are compared in table 3.17.

Table 3.17: Modulus of elasticity by authors (GPa)

	Ravindrarajah y Tam				CEB-FIB			
	HC	**HR25**	**HR50**	**HR100**	**HC**	**HR25**	**HR50**	**HR100**
28 days	27.60	28.84	29.07	28.64	39.34	41.11	41.44	40.82

According to the proposed Ravindrarajah and Tam model, when the percentage of the recycled aggregate was increased in the concrete the values were closer to experimental values, so the model is quite valid with respect to recycled aggregate concretes.

According to the CEB-FIB recommendation, defined for conventional concretes, the E values for recycled aggregate and conventional concretes were overestimated.

The following equations are evaluated:

$E = 5.31f^{0.50}+5.83$ (for conventional concrete)
$E = 3.02f^{0.50}+10.67$ (for recycled concrete)

The values also defined by Ravindrarajah (1987) are represented in table 3.18:

Table 3.18: Modulus of elasticity by Ravindrarajah (GPa) (1987)

	Ravindrarajah			
	HN	**HR25**	**HR50**	**HR100**
28 days	37.48	29.48	29.63	29.35

The proposal models were not improved with respect to the values indicated in table 3.17. It is possible that the last values (table 3.18) are not as reliable as the first ones (table 3.17).

According to Takizaki (1988, see chapter 2 section 2.3.3) his model for calculating the modulus of elasticity is acceptable for both conventional and recycled aggregate concrete. The modulus of elasticity of recycled aggregate concretes are shown in table 3.19.

Table 3.19:Modulus of elasticity by Takizaki (GPa) (1988)

	Takizaki			
	HN	**HR25**	**HR50**	**HR100**
28 days	31.20	31.17	31.22	29,80

The failure of the concrete derives from its weakest point. The weakest point being in these medium strength concrete, the recycled aggregates themselves. In all conventional concretes the interface is the weakest point however this is not the case with concrete employing recycled aggregates, as it is the recycled aggregate itself which is the weakest point.

The weakest point of the control concrete studied was located in the interface between the aggregates and the paste. In figure 3.10 the depicted tensile failure of the test element of HR100 occurs through the use of recycled aggregates (the recycled aggregates being the weakest point) producing two symmetric faces.

Fig. 3.10: Recycled aggregates concrete failure way by tensile

3.3.7 Variability of test result

In order to determine the variability of each concrete, the compressive strength was measured using a minimum of 25 test specimens for each concrete.

On analysing the results, it was deduced that standard deviation increased with the increase of recycled aggregates in the concrete. In this case, all the test elements had been cast in the laboratory, so both their concrete components and their production was well controlled. However, although the increment of the standard deviation is not as great as that at the moment of production at the work site, this value has to be considered as a coefficient security. The standard deviation increased by 18% and 49% in HR25 and HR100 respectively in comparison with that of HC. It is true, that the standard deviation value in all concretes was low, except for that of HR50. This error could be due to the lower number of tests used to determine the value, defined by "Gaussian distribution".

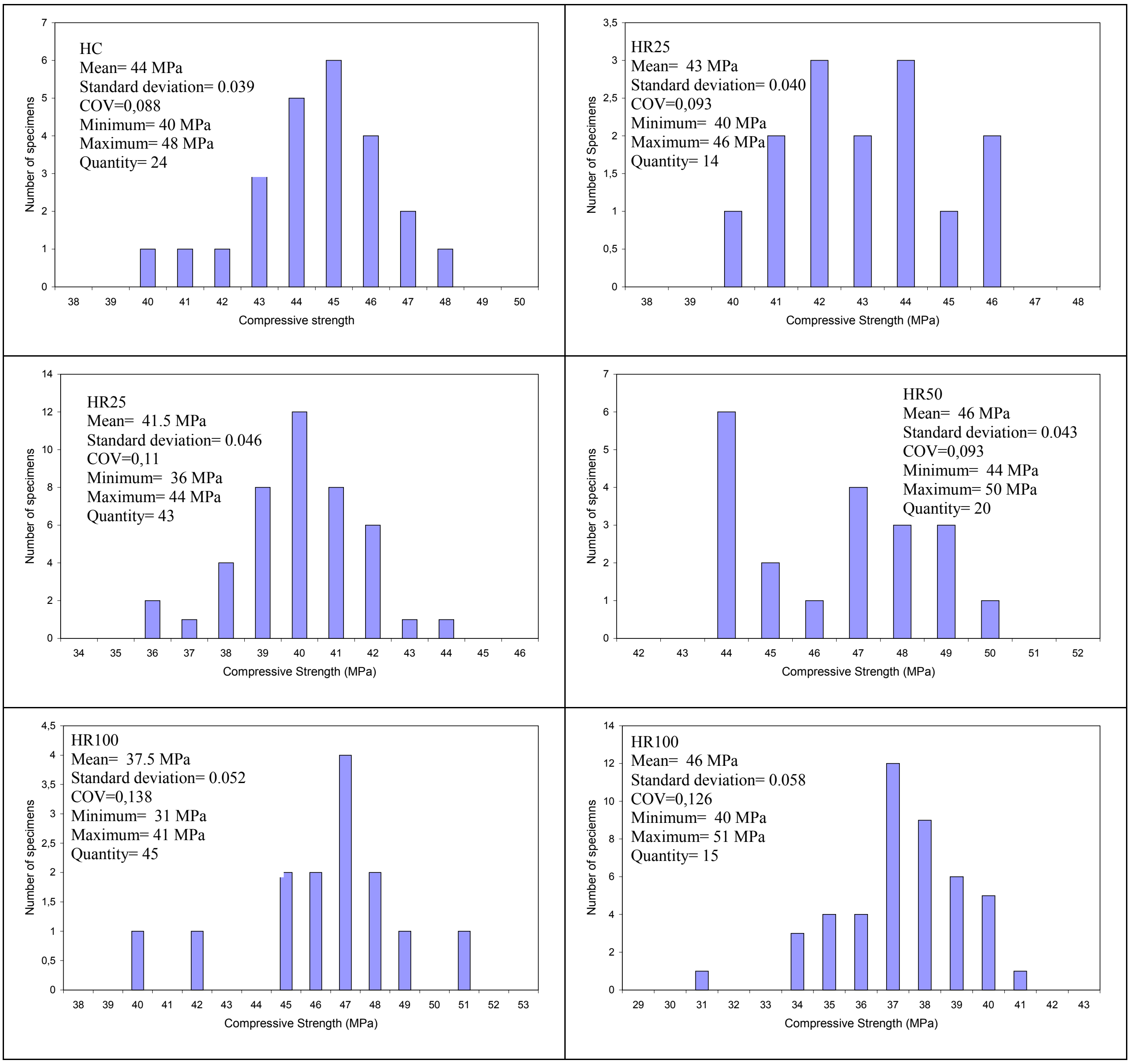


Fig. 3.11: Standard deviation of different type of concretes

3.4 CONCLUSIONS

In this chapter the properties of recycled and raw aggregates are determined. These aggregates are used to make concrete with different percentages of recycled coarse aggregates obtaining the same compressive strength to that of conventional concrete (HC). The concretes were made with different percentages of recycled aggregates, HR25, with 25 % of recycled coarse aggregates. HR50, with 50% of recycled coarse aggregates and HR100, with 100% of recycled coarse aggregates.

The conclusions obtained are with respect to;

Aggregates' properties:

- The grading of recycled aggregates is similar to that of the natural aggregates. In this case, the coarse recycled aggregates had less than 2% of fine material. Therefore they could be used as they were (without cleaning).

- The composition of recycled aggregates used revealed that there was a high percentage of aggregates without adhered mortar due to the low strength of the original concrete. Bituminous aggregates were higher than 5%, however as it only affects the strength of the concrete we considered it appropriate for use in our tests.

- The density of the coarse recycled aggregate is lower than that of conventional coarse aggregates, due to the adhered mortar and the quality of original aggregates. However the greatest difference between these two aggregates is their absorption capacity.

- The absorption capacity of recycled coarse aggregates can be lower to that of the maximum permitted by EHE (Spanish structural concretes code), consequently, according to Spanish requirements they can be used.

- The recycled aggregates had less resistance to abrasion than the conventional aggregates- 33.5% to 19,8% respectively.

- The impact crusher achieved a better shape index than the conical crusher. Recycled aggregates were crushed by the impact crusher whereas the natural aggregates were crushed by the conical one.

- Water-soluble chemical elements (chloride and sulphates) were low in recycled aggregates. Therefore, the probability of them causing durability problems in reinforced concrete was low.

- The recycled coarse aggregates had more than 2% water absorption capacity. It was necessary to determine the frost resistance by employing another test method.

- In accordance with EHE regulations, raw aggregate weight loss due to sulphates must be less than 18% and recycled aggregates lost 19,8%. Therefore, according to this Code these aggregates were considered unable to be used in exposure where a freezing and thawing cycle exists. However more tests are necessary to verify this property. Because it only evaluates the aggregates not aggregates within the cement paste.

- 53% of the recycled aggregates' composition was SiO_2. The original fine aggregates which formed the adhered mortar were potentially reactive. The used cement had more than 0.6% of alkalis, however the structural elements cast were kept dry, therefore the risk of ASR did not exist.

- According to RILEM and DIN 4226-100 the recycled aggregates that come from crushed concrete, are considered as Type II and Type 1, respectively. In accordance with prEN-13242:2002 the aggregates are placed in the best category with respect to all properties except with respect to freezing and thawing resistance.

- DIN 4226-100 has more detailed description (classification) of recycling material than RILEM or prEN –13242:2002.

Recycled aggregate concrete:

- It is essential to know the water content (the moisture absorption) of the recycled aggregate in order to determine the correct concrete dosage. In order to produce well controlled concrete production (the w/c ratio and fresh concrete workability) with respect to coarse recycled aggregates, the humidity of these aggregates must be considered as high, consequently they will be used in concrete production with little water absorption capacity.

- Concrete made with recycled aggregates needs more superplasticizer to achieve the same workability as conventional concrete.

- Concrete made with 100% of recycled coarse aggregates had 22,8% less compression strength than conventional concrete at 28 days, with the same effective w/c ratio (w/c=0,50) and cement quantity (325 kg of cement/m^3).

- Concrete made with 100% of coarse recycled aggregate requires for more (in fact too much) cement in order to achieve a high compression strength.

- It is not cost effective to use concrete made with 100% of recycled aggregate due to the high level of cement used in its manufacture. These aggregates should be used in concretes with low-medium compression strength. In contrast, the adhered mortar in recycled aggregates is lower in strength than conventional aggregates and the new paste. Consequently the weakest point in concretes made with coarse recycled aggregates employing a cement paste of a medium-high strength can be determined by the strength of the recycled aggregates or their adhered mortar.

- Concrete made with 25% of recycled coarse aggregates achieves the same mechanical properties as that of conventional concrete employing the same quantity of cement (300 kg of cement/$m^3$of concrete) and the equal effective w/c ratio (w/c=0,55).

- Concrete made with 50% of recycled coarse aggregates needed a lower effective w/c ratio (w/c=0,52) as well as 6% more cement than conventional concrete (w/c=0,55 and 300 kg of cement/m^3) to achieve the same compression strength at 28 days.

- Medium compressive strength concrete made with 100% of recycled coarse aggregates needed an effective w/c ratio of 0,50 and 8,3% more cement than conventional concrete (w/c=0,55 and 300 kg of cement/m^3) to achieve the same compression strength at 28 days.

- The compression strength of concrete made with recycled aggregates develops less over the last 21 days of its 28 day curing period than that of conventional concrete. The raising of the percentage of recycled aggregate used in concrete, increases its strength to approximately 12-15% when the conventional concrete strength is approximately 20%.

- The tensile strength of recycled aggregate concrete can be higher than conventional concrete (concrete using raw aggregate).

- Modulus elasticity of concretes made with 50% and 100% of recycled aggregates is lower than that of conventional concrete. Concrete made with 25% of recycled aggregates maintains the properties of conventional concrete.

- Ravindrarajah's (1987) proposed method of Modulus elasticity is closer to the experimental value when the percentage of recycled aggregates is increased. Takizaki's (1988) proposed method of Modulus elasticity has a good correlation with experimental values.

- An automatic mixing machine was employed in the production of the beams. The sequence of adding materials to form the concrete mix in this type of machines is the same as usually employed on building sites. These kinds of machines produce better mixes and the concrete's mechanical properties are better.

- With respect to our studies the order of adding materials to the concrete mix employed in the automatic mixing machine resulted in the cement being adhered to the recycled aggregates before the water was added. When the water was added the cement was absorbed into the recycled aggregates, consequently producing an effective interface. The adhered old mortar on the recycled aggregates becoming the weakest point. The fail of the test elements by tensile testing produced cracks through the aggregates as a result of the old attached mortar.

- Standard Deviation increases with the increase of the percentage of recycled aggregates. When HC concrete was compared with HR25 and HR100 concretes it was found that the standard deviation increased by 18% (HR25) and 49% (HR100).

Chapter 4

Microstructure of recycled aggregate concrete

4.1 INTRODUCTION

The need for knowledge about the durability of recycled aggregate concretes is one of today's most important relevant issues.

The aggregates were tested to determine the presence of chemical contaminants, their resistance to freezing and thawing as well as to alkali reactivity. The recycled aggregates' adhered mortar (fine original aggregates) was potentially reactive and the frost resistance determined the weight loss by magnesium sulphate was that of the maximum value permitted by the EHE (Spanish structural concrete code). However this EHE test is not totally appropriate as it only evaluates the aggregates not aggregates within the cement paste. The aggregates did not have either chloride or sulphate contaminants.

Macroscopic and microscopic examination were carried out in HC, HR25, HR50 and HR100 concretes in order to analyse any possible durability problem. The test elements of these four concretes were 8 months old when they were analysed. During this time the test elements were stored in the humidity room at 20ºC with 100% of humidity and with the same degree of hydration. The macroscopic and microscopic examination were carried out in the Materials Science Group of the Faculty of Civil Engineering and Geoscience (Delft University of Technology (NL)) and in the Construction Engineering department of the Polytechnic University of Catalonia.

4.2 VISUAL INSPECTION OF SAMPLES. MACROSCOPIC EXAMINATION

The macroscopic examination was carried out on samples taken from three cylinder tests for each of the concretes under study (HC, HR25, HR50 and HR100).

The visual inspection was useful to determine the homogeneity of the concrete with respect to the distribution of the aggregates in the concrete, and the density of the concretes. The macroscopic examinations were made employing the stereo light microscopes Leica MZ6 and Leica CLS 150.

4.2.1 Aggregates distribution and composition

When the percentage of the recycled aggregate in the concrete was increased the heterogeneous nature of the aggregates also increased as did the colour (which was affected by the different material compositions of the aggregates). More brick and bituminous materials were present. The aggregates distribution in HC, HR25, HR50 and HR100 are shown in figure 4.1.

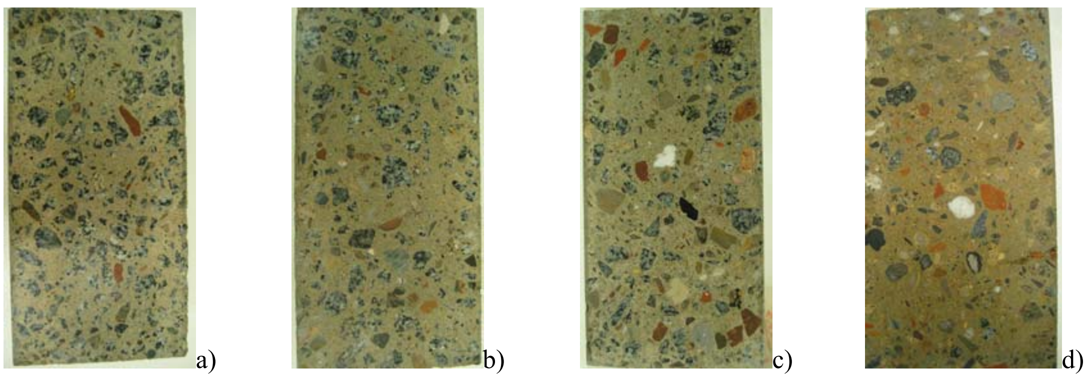


Fig.4.1: Distribution of aggregates in concretes. a) conventional concrete, HC b) concrete with 25% of recycled aggregate, HR25 c) concrete with 50% of recycled aggregates, HR50 d) concrete with 100% of recycled aggregates, HR100. Dimensions of concrete pieces are 30x15 cm

The four concretes (HC, HR25, HR50 and HR100) had a homogeneous density, and a regular distribution of aggregates. The porosity of the new paste appeared to be similar in all concretes, although the recycled aggregates appeared to be far more porous than conventional aggregates, due to the adhered mortar, see figure 4.2.

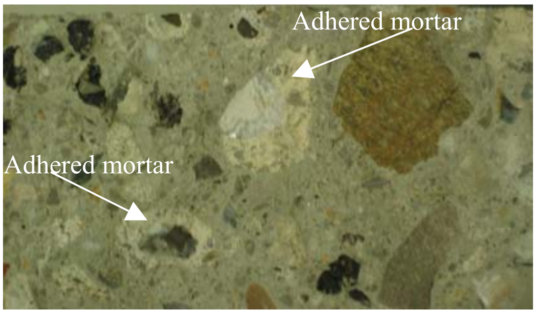


Fig.4.2: Recycled aggregates in the concrete

The old cement paste is not as dense as the new one, however this does not affect the interfacial transition zone between the old and new paste which is effective, see figure 4.2.

The large pore distribution in the new cement paste in the four concretes (HC, HR25, HR50 and HR100) was similar. However a later examination was carried out employing a high precision optical microscope for more precise measurement of the pores.

4.2.2 Contaminants in the recycled aggregate concrete

In the macroscopic analysis of the three samples (cut segments) of the same cylinder test element HR100 (concrete employing 100% recycled aggregate) a crack was detected, see figure 4.3.

The crack was produced by recycled aluminium aggregate, which reacts with cement and water producing hydrogen bubbles when curing. Aluminium is a serious contaminant for concrete production as its corrosion will result in expansion and cracking. The mentioned aluminium contaminant came from the demolition material employed in the production of the recycled aggregates used in the concrete mixes studied. Its negative affect on the concrete produced emphasises the importance assuring the aggregates composition by eliminating any contaminants that maybe discovered whether they be aluminium or other substances/materials. However no more pieces of aluminium found in recycled aggregates employed in our tests.

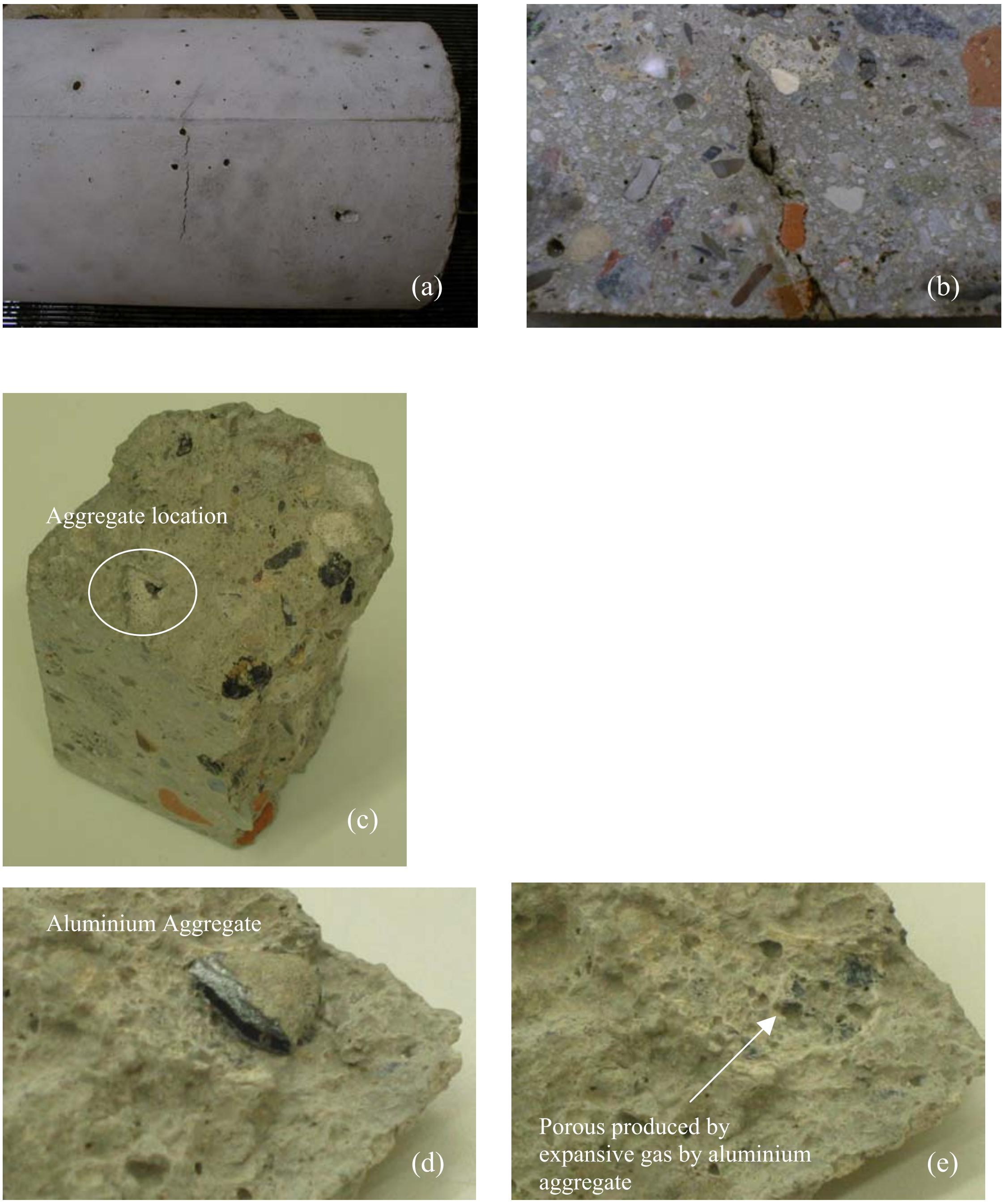


Fig.4.3: Aluminium aggregate effect in the concrete producing hydrogen bubbles. (a) Visual crack in the cylinder test element. (b) The crack inside test element (c) Aggregate localization in the concrete (d) Aluminium aggregate in the concrete. (e) Porosity produced by the aluminium aggregate.

4.2.3 Aureole around concrete recycled aggregate

There were doubts regarding the contact between the new paste and the recycled aggregate. At concrete production the recycled aggregates were humid, cement was added to the mixing machine and with the addition of water it probably accumulated on the border of the aggregate. Aureoles were discovered on the outside of the recycled aggregates, they always occurred around the adhered mortar when the concrete was saturated and at approximately 30-40ºC. (It is explained on section 4.2.3).

When recycled aggregate concretes were saturated and their conditions changed by increasing temperature (but maintaining the saturated condition) an aureole appeared around the recycled aggregates, see figure 4.4.

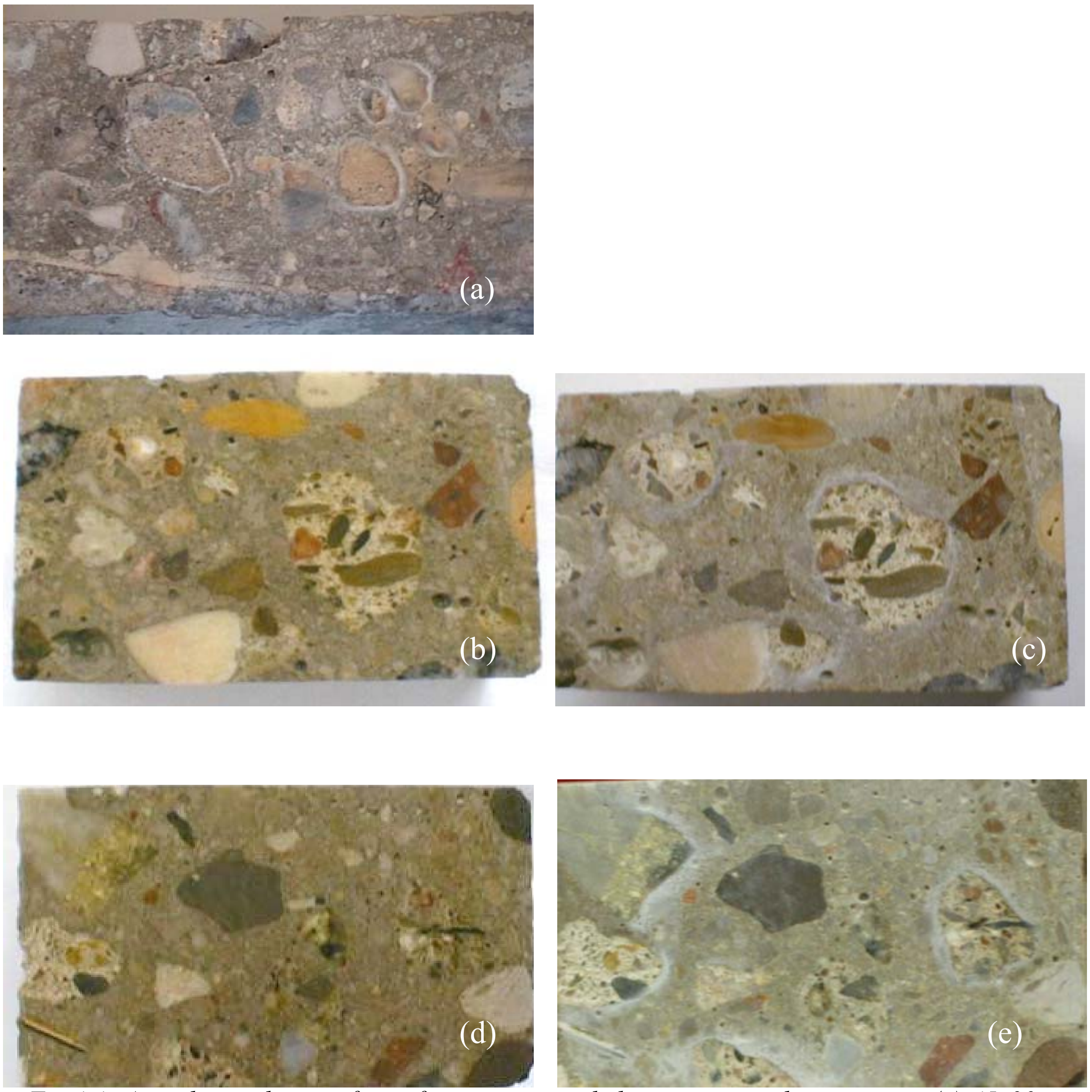


Fig.4.4: Aureoles in the interface of concrete recycled aggregate and new paste. (a) 45x30 mm piece of concrete (b) First small sample of concrete saturated without aureole. (c) Piece b after four days at 40ºC. (d) Second small saturated sample. (e) Sample c after being heated in the oven at 40ºC.

The above results occurred when the recycled aggregate concretes were transported in humid hermetic plastic bags from Barcelona to Delft (NL). The situation was reproduced in the UPC, materials department several months later under controlled conditions. 45x30x110 mm pieces were cut from saturated cylinder test elements, placed in humid hermetic plastic bags and heated in an oven to 40ºC. In these conditions water transportation occurs and almost all the recycled aggregates with adhered mortar were discovered to have aureoles around them. The 'aureoles' phenomenon was later analysed employing the optical transmitted light microscope Leica Leitz DM-RXP and ESEM (Environmental Scanning Electron Microscope) and SEM (Scanning Electron Microscope). The optical transmitted light microscope Leica Leitz DM-RXP was also used to define the quality of interfaces, the w/c ratio, the porosity of different concretes and the aggregates quality.

An ESEM/SEM were later employed in the study of this aureole composition and shape in order to verify the reason for their existence.

4.2.4 Carbonation of recycled aggregates

An analysis of the aggregates was considered important, to determine whether they were carbonated and this was the reason for the origin of the aureoles.

A sample of recycled aggregate concrete containing an aureole was taken and fenoftalein was used to discover if the recycled aggregates were carbonated or not. This test was carried out in order to check if $Ca(OH)_2$ solubility and its efflorescent were possible.

A 45x30x20 mm sample of concrete with aureole around the recycled aggregate was taken and the aggregates were analysed as shown in figure 4.5.

Fig.4.5: Recycled aggregates are not carbonated. The rose colour shows that the alkalinity of the aggregates.

Two representative samples were taken. The 16/25 mm sample was examined employing fenoftaleina and 32% was not carbonated. Using the same process the 10/16 mm sample proved to have 37.5% no carbonated.

4.2.5 Composition of recycled aggregates

As stated in chapter 3, 50% of the recycled aggregates were original aggregates with adhered mortar. In order to analyse existing aureoles, it was considered imperative to determine the class of the original coarse aggregates. A representative sample of coarse original aggregates (from recycled aggregates) were taken and classified as the following: 74% of coarse aggregates were limestone, 10% granite, 10.6% metamorphic and 5.2% quartz.

The mineralogy composition of the fine recycled aggregates was detected employing a SEM and EDX-map, see section 4.4. (In chapter 3, it was measured that 53.4% of recycled aggregates' composition is of Silica, consequently original fine aggregates could be of silica).

4.3 MICROSCOPIC ANALYSIS. QUALITY CONTROL OF HARDENED RECYCLED CONCRETE EMPLOYING AN OPTICAL FLUORESCENCE MICROSCOPE

There are certain advantages of employing an optical microscope with respect to concrete examination. Firstly, the optical microscope offers an extensive field of view which in turn gives an excellent understanding of the microstructure of the concrete and the possible causes of production faults or deterioration mechanisms. Secondly, it is a fast and economic (low cost) examination. The information obtained from the employment of an optical microscope is markedly enhanced by adding a fluorescent light to the two standard light modes of plane polarised and crossed polarised light. The fluorescent illumination provides information concerning the following: the capillary porosity (w/c), cracks, crack pattern, air void system, porosity of aggregates and paste defects, information necessary for evaluation of the quality and the condition of the concrete. The mentioned is an acceptable method for employment in quality control as well as for evaluating deterioration in concrete. In this case it is necessary to check the quality of the concretes due to their short life span.

The water/cement ratio is one of the most important engineering parameters in the use of concrete. The w/c ratio determines the strength and durability of the concrete produced. In hardened concrete there is no direct form of determining the w/c ratio.

However, for more than 20 years optical microscope analysis has been employed as an indirect form of determining the w/c ratio in hardened concrete. Miscellaneous concrete laboratories all over the world apply this method on concrete samples under study.

As there is no standard procedure set out with regard to the fluorescent thin section technique (FTS) each laboratory practices their own method. The concretes were studied in the laboratory of the Civil Engineering Materials Science Group in TUDelft (NL).

4.3.1 Preparation and analysis of samples

In order to carry out the quality control research it was necessary to employ one cylinder test element taken from each of the concrete mixes under study. The four concretes; HC, HR25, HR50 and HR100 were left for almost 8 months in the humid room and the degree of hydration in all concretes was similar. In order to conduct the research three samples were taken from each concrete. A 45 mm slice was taken from the top of the cylinder test element, see figure 4.6. The circular slice-piece was 150 mm in diameter and 45 mm thick. Three small segments were taken from a rectangular segment cut from this slice.
From each rectangular pieces, three slices were taken out, called 1, 2, and 3 see figure 4.7.

Preparation of fluorescent thin sections (FTS)

A Fluorescent Thin Section (FTS) is a very thin slice of concrete glued to an objective glass which in turn is covered by a thin coverglass. The concrete slice has a thickness of 20 μm and is transparent with respect to transmitted light. The thin section has been vacuum impregnated with a yellow fluorescent epoxy resin. The amount of fluorescent dye entering the cement paste depends on two factors: the w/c ratio as determined by the capillary porosity and the degree of hydration. As a consequence of this impregnation all air-filled spaces in the concrete (air-voids, capillary pores and cracks are filled with the fluorescent material) show a large contrast with the adjacent concrete when viewed in the fluorescent transmitted light mode of the microscope.

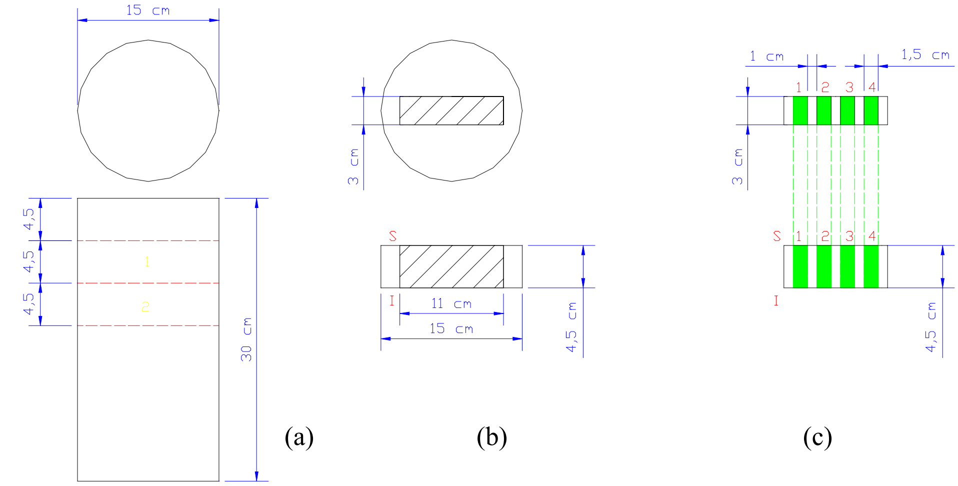


Fig. 4.6: (a) Cylinder test element. (b) a rectangular piece is taken (c) 4 small pieces of each concrete are obtained.

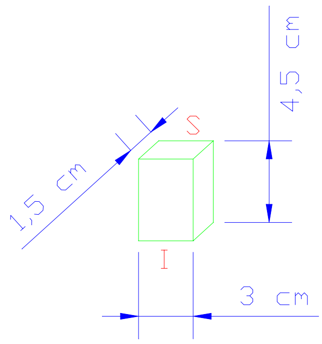


Fig.4.7: The shapes (in cm) to do thin sections for microscope study

A dual purpose machine the DBT Diamond Roller and Grinder/86 was used for the production of the fluorescent thin sections, while the 45x30 mm thick specimens were produced by sawing, the thinner 45x30 mm specimens were produced by grinding. All samples, without exception, were attached to a piece of glass, which could be held by the vacuum holders employed in the machine's grinding or sawing facility. The grinding was carried out by diamond grits. The coarse roller (roller #1) was used to grind a maximum of 0.5 mm from the concrete specimen. Roller #2 and #3 were used to grind another 0.1 and 0.05 mm, respectively. The concrete samples moved back and forward over the grinding roller, until the grinding at a certain stage was finished. After the three rollers grinding a perfectly flat surface was produced.

The following were analysed: aggregates, cement paste, interface between new paste and coarse aggregates, new paste and old paste, and new paste and old recycled aggregate. All of the samples were studied under three different lights, ordinary light, crossed polarized light and fluorescent light.

4.3.2 The shape and distribution of aggregates

The distribution of the aggregates was homogeneous, as coarse or fine aggregates were homogeneously distributed by the concrete. The distribution of the aggregates was appropriate in all kinds of concretes, HC, HR25, HR50 and also in HR100. However we had to take into account that the quality of the aggregates were not the same.

Crossed polarized light was employed to define the natural aggregates mineralogy, the granite aggregates composition was basically the feldspar group and quartz, see figure 4.8. The aggregates obtained from recycled aggregates were of a very heterogeneous nature. There were also bituminous and ceramics aggregates (see section 4.2.5).

The quality of the aggregates were very different, very few natural aggregates were cracked, less than 6 %. The isolated cracks exist within the aggregates, as shown in figure 4.9.

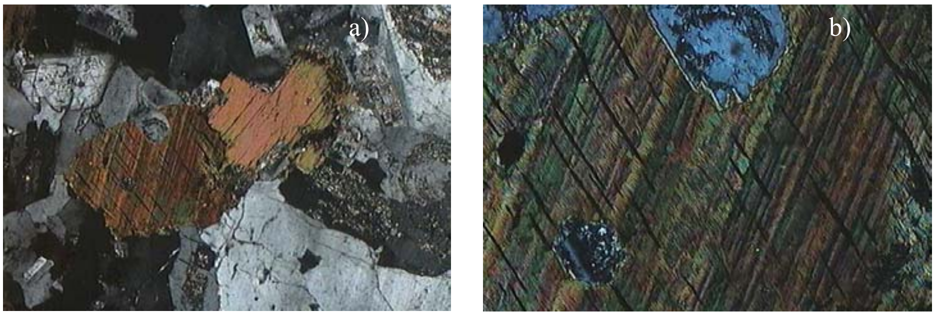


Fig.4.8: Composition of raw granite aggregates. a) Feldspar group and quartz. Width of field 3.95 mm b) Feldspar group. Width of field 1.03 mm

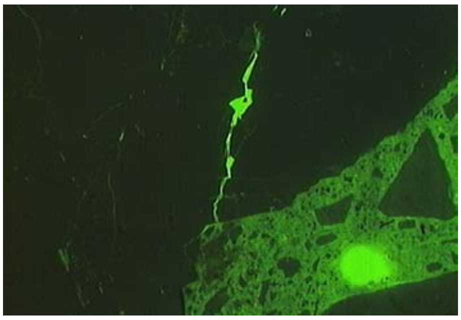


Fig.4.9: Cracks inside of the natural aggregate.
Width of field 3,95 mm

The quality of the original aggregates obtained from recycled aggregate used in our research was inferior to that of conventional coarse aggregates as the majority of the original aggregates were found to have cracks. Consequently the porosity level of the recycled aggregate without adhered mortar was also higher than the conventional aggregates, see figure 4.10.

More than 50% of the recycled aggregates have adhered old concrete paste. In almost all cases this paste was discovered to be in worse quality than the new paste. It is evident that due to their high porosity level the recycled aggregates are far more inferior than those of conventional aggregates, see figure 4.11.

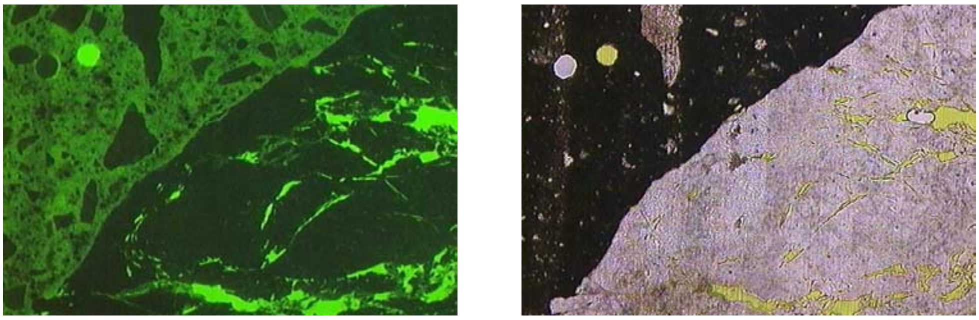


Figure 4.10: High porosity of recycled aggregate by fluorescent and normal light. Width of fields 3,95 mm

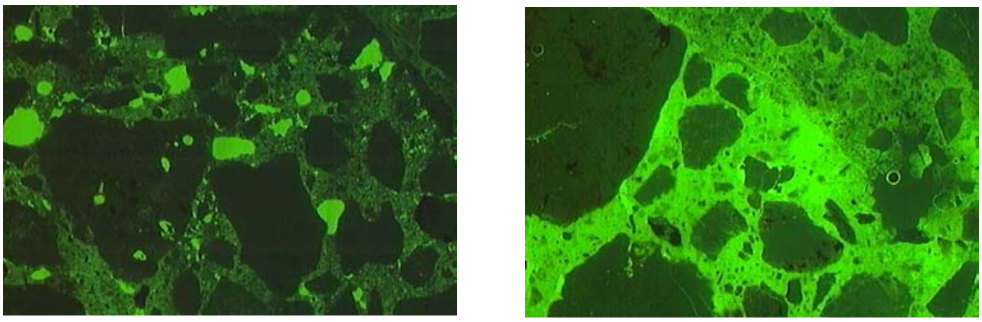


Fig.4.11: Recycled aggregates. High porosity of old paste. Width of fields 3,95 mm

In the analysis of the aggregates' composition (see Chapter 3), it was discovered that 5% of the recycled aggregates were bituminous, and 2% were ceramic.

The bituminous and ceramic recycled aggregates were not cracked (Figure 4.12 and figure 4.13). The effectiveness of the bituminous aggregates interface was acceptable, as was that of the ceramic aggregates interface where no sign of water accumulation were present (in general). Figure 4.13 shows that the interface could be affected by water accumulation.

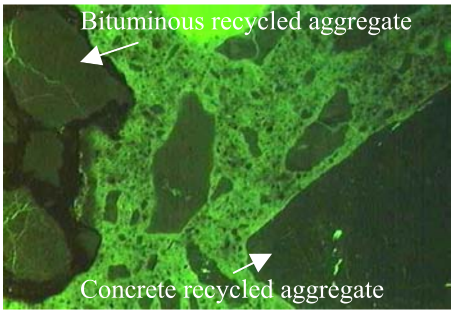


Fig.4.12: Bituminous aggregate. Width of field 3,95 mm

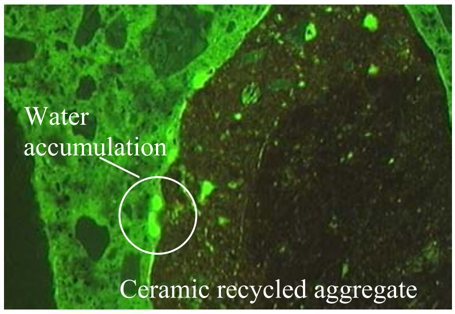


Fig.4.13: Ceramic aggregate. Width of field 3,95 mm

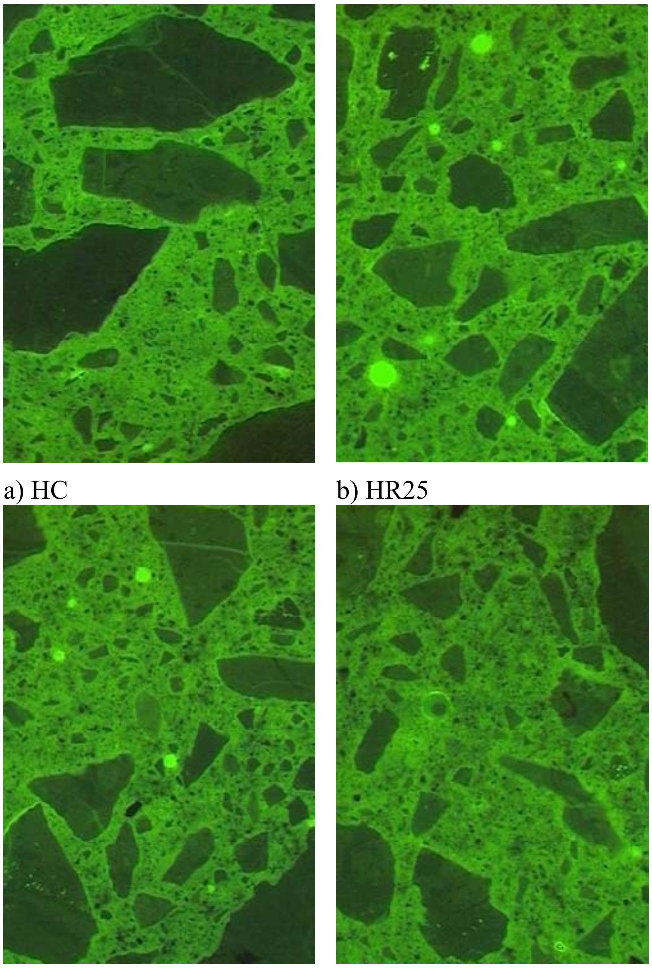
4.3.3 Cement paste

The new cement paste in each concrete was different. To achieve the same compression strength a higher quantity of cement was necessary in the concretes made with 50% and 100% of recycled aggregates. The reference concrete (HC) was produced with 300 kg of cement/m^3 of concrete mix. The concrete mix with 25% of coarse recycled aggregate (HR25) was produced with the same quantity of cement, and water, but 0.26% more of superplasticizer was needed to obtain the same workability (based on the cement content 1.4% of superplasticizer was used in the preparation of HC concrete, 1.66% in HR25). In order to obtain a similar compression strength to that of the reference concrete (HC) the quantity of cement employed in the higher recycled aggregate concrete HR50 and HR100 was increased to 6% in the HR50 and 8.3% in the HR100 mix. The quantity of superplasticizer employed was also higher than the 1.66% used for the HR25 mix. In the HR50 and HR100 concrete mixes 1.90% superplasticizer (depending on cement weight) was added to achieve the same workability (10 cm slump cone).

The recycled aggregates were wetted to approximately 80% of their absorption capacity in order to prevent serious workability problems that would otherwise be produced if the aggregates were drier. In this way not only would water movement be controlled, but also the water/ cement ratio in the paste.

The microscope's fluorescent light was used in order to analyse the w/c ratio in the four concretes under study: HC, HR25, HR50 and HR100. It was not possible to determine exactly the w/c ratio value because there was no reference sample where the w/c ratio had been established as being exact. However if we compare the cement paste of concrete made with recycled aggregates to that of a high quality conventional concrete (HC) it is possible to make a comparison of cement paste through difference in fluorescent light intensity and distribution. The intensity of the colour determines the density of the paste.

The paste density (under fluorescent light intensity) of the four different concretes mentioned above HC, HR25, HR50 and HR100 is shown in figure 4.14. It can be seen that the paste colour is homogeneous in each of the concretes but the colour intensity is darker with respect to concrete where more cement was used, evidently this is to be expected as the concrete employing more cement in its mix is denser than the concrete employing less.



a) HC b) HR25

c) HR50 d) HR100

Fig 4.14: Cement paste of four different concretes. (a) HC, control concrete (b) HR25, 25% of recycled coarse aggregate, (c) HR50, 50% of recycled aggregates (d) HR100, 100% of recycled aggregates. Width of fields 3,95 mm

The control concrete (a) and concrete made with 25% of recycled aggregates (b) have the same cement quantity. The recycled aggregates are wet, therefore in the HR25 concrete there is a higher total water quantity than that of the control concrete. This quantity of water does not affect the quality of the new paste.

In HR50 (c), the recycled aggregates were also humid (approximately 80%) but not saturated. Therefore, in this case too, the total water quantity was higher than the control concrete but the effective w/c ratio was lower in the paste. The cement paste was a homogeneous paste which was a little denser than both the HC and HR25 pastes.

The concrete made with 100% of recycled coarse aggregates (d), the aggregates were humid and the total water quantity in the concrete was higher. The new paste was denser than the other concretes, because of water retention in the pores of the aggregates. Consequently this did not affect the effective w/c ratio.

As previously mentioned, the new paste employed in all of the concrete mixes was acceptable. In figure 4.14 it can be seen how the density in the paste is homogeneous and also that the air voids quantity is more or less the same as in all of the cement pastes employed.

The hardened paste cement content is expressed as a percentage of the volume of cured concrete. It is the sum of the proportional volumes of the cement and the net mixing water (including the liquid portions of any chemical admixtures).

The volume and the percentage of the paste employed is determined in each concrete mix by adding the volume of the cement and the net mixing water. It is imperative to determine the percentage of the paste in the concrete in order to measure the porous quantity in the paste. Three surfaces of each concrete were analysed and the air voids were quantified measuring their size according to ASTM C 457-98; "*Microscopical Determination and Parameters of the Air-Void System in Hardened Concrete".* The values of these calculations are shown in table 4.1.

Table 4.1: Air void percentage in different kind of concretes.

Concrete	Vcement (dm3)	Vwater (dm3)	Paste content (dm3)	Paste in concrete %	Concrete piece (mm^2)	Paste in piece (mm^2)	Air void surface (mm^2)(*)	Air void %
HC	98.3	165	263.3	26.33	1350	355.45	11.28	3.17
HR25	98.3	165	263.3	26.33	1350	355.45	13.27	3.73
HR50	104.26	165.36	269.62	26.96	1350	363.96	13.07	3.59
HR100	106.55	162.5	269.05	26.90	1350	363.15	20.85	5.74

(*) Average value by three different pieces

The air void quantity increased with the percentage of recycled aggregates employed in the concrete mix. Although there is no notable clear difference between HR25 and HR50 concrete, the same cannot be said for the HR100. The increment of the amount of air void in the cement paste is evident when 100% of recycled aggregates are used.

It is imperative to emphasise that the air void % term employed in our research does not refer to voids of submicroscopical dimensions, such as the porosity inherent to the hardened-cement paste. Air voids have a diameter larger than 152 μm.

The distribution of the air voids in each concrete (depending on their size) is also different, see figure 4.15. Concrete with more recycled aggregates not only has more porosity but the air area is also more extensive. This increase of the air surface is due to the increment of the larger sized air voids. Figure 4.15, clearly details the increase in the larger sized air voids with respect to the HR100 concrete.

The percentage of air voids is quite similar in all of the concretes studies, however, HR100 concretes proved to be slightly more porous, its porosity being 5.76%. Under flourescent light analysis the pores are seen to be round, regularly distributed and detached from each other, consequently the permeability of the concrete is quite low.

The air voids are determined in the new paste, without counting the existing pores in the adhered mortar. When the pores of the old concrete are taken into account it can be deduced that the air void quantity increases. In the concrete mix where 25% of recycled coarse aggregates were employed the porosity does not increase. Evidently the recycled aggregate has an influence on the increase of pores in the cement paste, but it is not important in concretes where their use is only minor.

There is a notable increase in pores in concrete when more recycled aggregates are used in concrete manufacture. Evidently it is necessary to analyse the porosity of the concretes under study. Even though the pore quantity increased in HR50 and HR100 concrete due to the existence of adhered mortar, its low density and high permeability was more evident. Taking into account the adhered mortar the increase in porosity with respect to HR100 concrete was 0.34%. Consequently the total porosity of HR100 concrete was 6,08%.

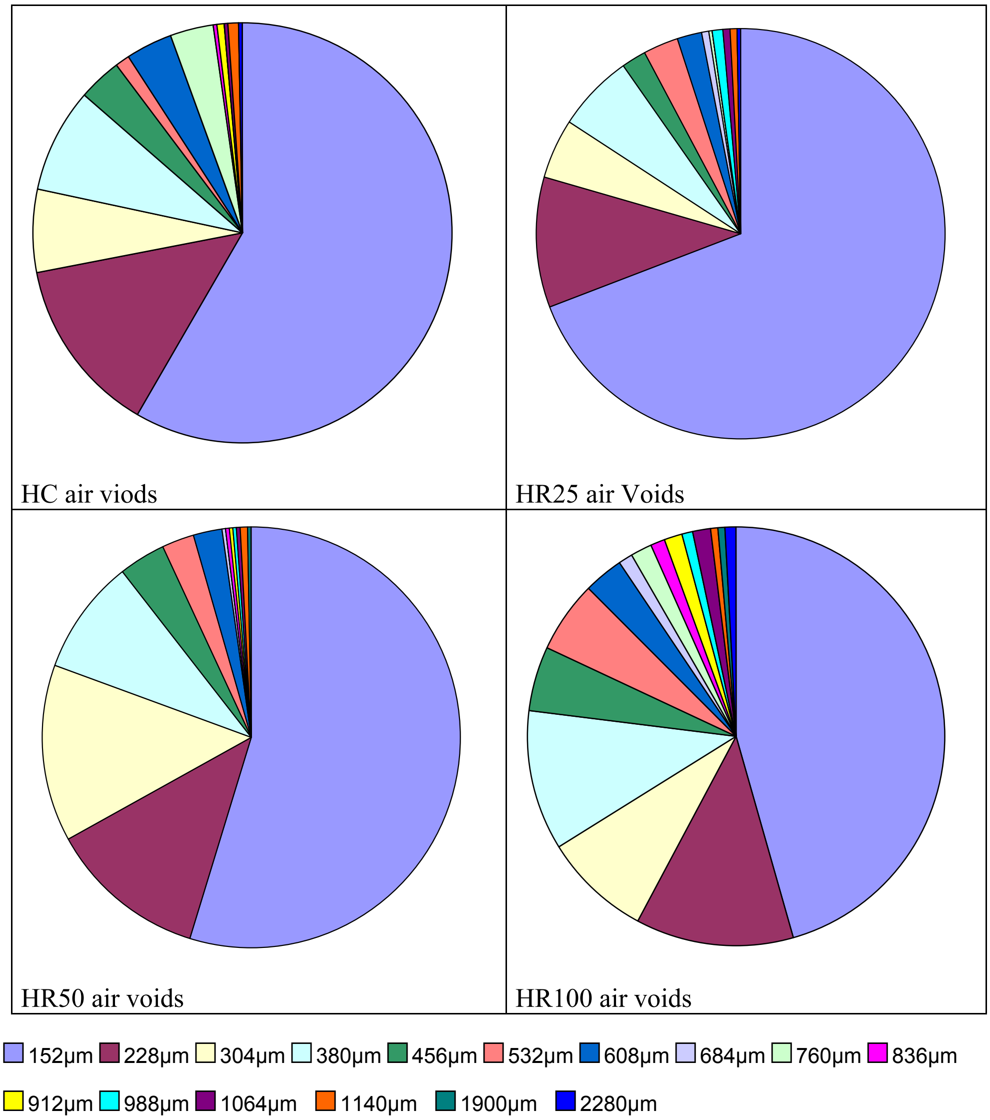


Figure 4.15: Air void in different concretes

The w/c ratio of the old paste is higher than the w/c ratio of the new paste. The volume of capillary voids initially filled with water increases as consequently of the increase in the amount of water.

The water/cement ratio is an important criterion in the composition of concrete, controlling the microporosity of the cement paste and hence having a critical influence on concrete strength properties. Furthermore, the water/cement ratio also has an important bearing upon durability. The durability of concrete generally being related to the water permeability of concrete, which is in turn largely dependent upon cement paste porosity. We can see in the following slide samples, the differences of the w/c ratio respect to new and old paste.

In figures 4.16 and 4.17, the existence of an air void in the old paste can be noted. The air voids which are irregular in form are located between the aggregates.

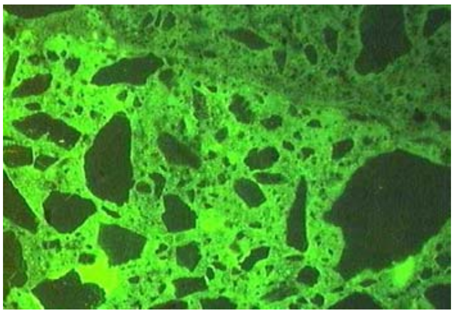


Fig. 4.16: Recycled aggregate. Width of field 3.95 mm

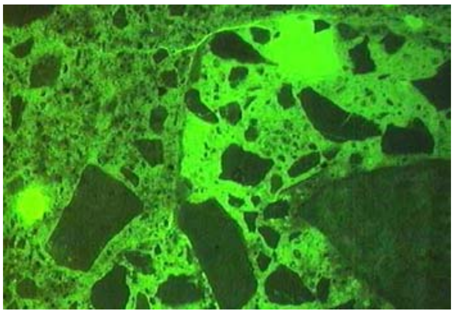


Fig 4.17: Recycled aggregate.Width of field 3,95 mm

Bleeding is a typical feature of all common concrete mixes. The occurrence of excessive bleeding is an indication of the possibility of a high water/cement ratio. It is common for such entrapped bled water to accumulate beneath aggregate particles. This water may also occupy cavities and voids left by incomplete compacting. In the case studied few bleeding points were found in these concretes. There were no differences between the four concretes under study, therefore it can be stated that, the water contained in the aggregates did not play an active part in the hydration of the cement. Some bleeding points were found (under fluorescent analysis), but this occurred in both recycled aggregate concrete and in control concrete (HC), see figures 4.18, 4.19 and 4.20.

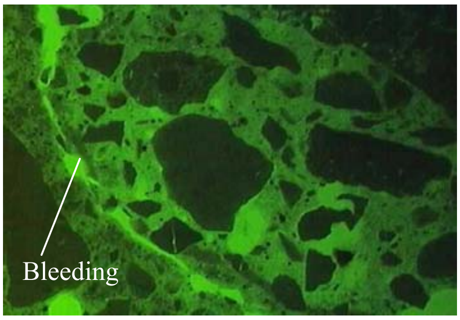


Fig.4.18: Bleeding in 100% recycled concrete. Width of field 3,95 mm

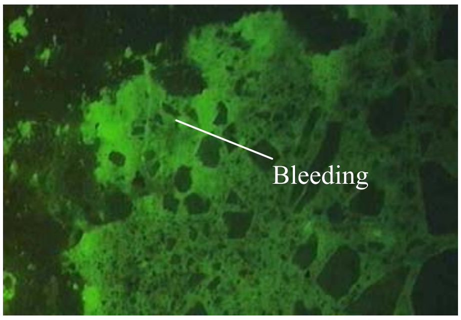


Fig 4.19: Bleeding in 50% recycled concrete. Width of field 3,95 mm

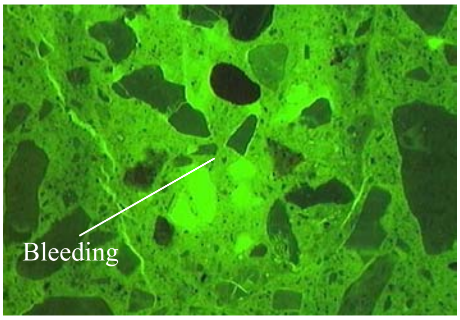


Fig 4.20: Bleeding in reference concrete due to compacting. Width of field 3,95 mm.

The hydration grade of these concretes after 8 months of curing can be qualified as being almost total, the quantity of water in the concrete was high, the water quantity was over the level required for cement hydration.

Portlandite ($Ca(OH)_2$) is formed during the hydration of the calcium in Portland cement and it is thus characteristic of uncarbonated cement paste. Using optical microscopy, Portlandite appears on hardened concrete as small crystals uniformly distributed in the cement paste.

Fig. 4.21: Portlandite distribution in HC. White shining points are portlandite crystals. Width of field 0,5 mm

Fig. 4.22: Portlandite distribution in HR100. Width of field 0,5 mm

The portlandite distribution in the HC and HR100 concretes, which was detected using polarised light, can be seen in figure 4.21 (HC concrete) and figure 4.22 (HR100 concrete) which appears as white shining crystals, was identified by the use of a double cross polarised light. Figure 4.21 depicts the reference concrete (HC) in which more water was used and higher w/c ratio. It can be seen that in this concrete sample there are more white shining points than in the HR100 concrete's cement paste, see figure 4.22. In this case less water was used with lower w/c ratio, therefore the portlandite crystals are smaller and there are fewer. In the reference concrete the portlandite crystal are larger due to a higher w/c ratio.

4.3.4 The New Interfacial Transition Zone

It is convenient to differentiate the intrinsic interfaces that are formed between paste and aggregates. The interface is considered to have a fundamental affect on the strength of concrete.

The nature of the interface is controlled by the properties of both the aggregate and the cement paste. Aggregates used in concrete as well as the possibility of adherent dust and dirt have varying degrees of porosity, shape and surface roughness.

Physical contact and/or degree of separation of the paste from the aggregate will be largely dependent on the amount of water concentrated at the interface and the amount of plastic settlement that takes place before setting.

Both these factors, which are closely interrelated, create space at the interface in which crystallisation from the pore solution can occur.

In recycled aggregate concrete, two interfaces exist. One is the new paste with old aggregate and the other one, old paste with old aggregate. When the old aggregate does not have any old paste adhered to its surface or in its pores, the interface develops in much the same way as with natural aggregate (non recycled aggregates).

A weak point exist in recycled aggregate. When the composition of the recycled aggregate is made up of basically of old paste, this material could be dirty and dusty and its porosity is very high. The capacity of water absorption is high and the doubts about creating an acceptable interface are quite evident. However the interface in almost all of the aggregates studied was effective, the aggregates were wet, with a high amount of humidity but not saturated, they permitted the cement to be absorbed into the aggregate by means of suction and to have very substantial interface. Almost all of the aggregates under study showed a high standard of interface achievement. There are approximately the same amount of cases of failed interface in conventional concrete as those of recycled aggregate concrete.

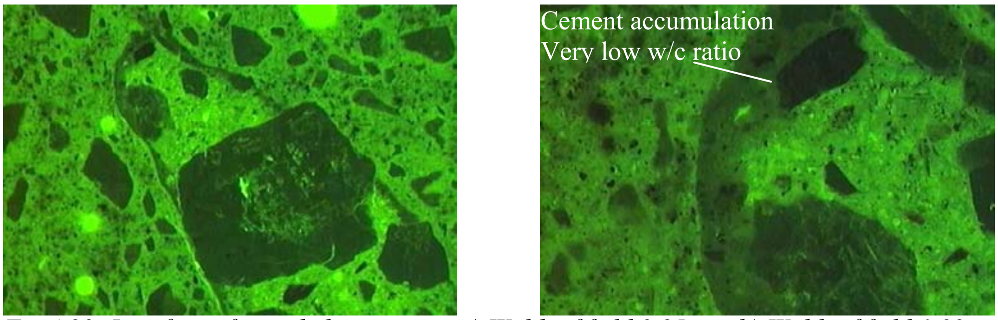


Fig.4.23: Interface of recycled aggregate. a) Width of field 3,95 mm b) Width of field 1,03 mm

The following slides (figure 4.23 a) and b)) show the interface between the paste and the aggregates. The old recycled aggregate is composed of aggregate and old mortar. It is evident that in the old mortar the w/c ratio is higher, so the porosity of the old cement paste is higher than that of the new one. The added mortar of the recycled aggregate could be its weakest point as the interface created by the new cement paste is quite acceptable.

In figures 4.23 a) and b) it can be seen that the cement quantity on the border of the recycled aggregate is high, consequently producing a very dense cement paste with low w/c ratio. The cement was accumulated in the interface, therefore the w/c ratio was low and the cement paste had very high density. However, the possibility of having a certain quantity of unhydrated cement exits due to the low w/c ratio.

Almost certainly the occurrence of the aureole was due to cement accumulation and the concrete production process. These concretes were produced in an automatic mixing machine. The order of materials placed in the mixer machine was always the same; first the aggregates were added and mixed for 30 seconds, before adding the cement. Both aggregates and cement were mixed for 30 seconds. Then the water was added and the mass mixed for a further 30 seconds before finally adding the additive. The complete concrete mix was mixed for a further 2 minutes. The recycled aggregates were wet and the cement adhered to the surface of the aggregates and filled the pores. Therefore it can be stated that when the water was added the cement which was adhered to the aggregate surface was absorbed by the aggregates. The aggregates were wet but they were not saturated, consequently they maintained a part of the absorption capacity.

Actually the interfacial transition zone between the paste and the old aggregates is effective in almost all the aggregates under study, see figures 4.24 and 4.25. In these two images the interface between the new paste and the recycled aggregate is effective and an accumulation of cement was also appreciated in the interfacial transition zone, see figure 4.23.

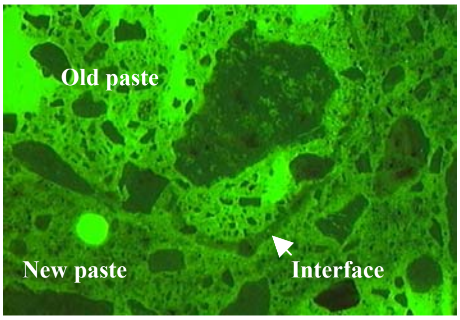


Fig.4.24: Interface. Width of field 3,95 mm

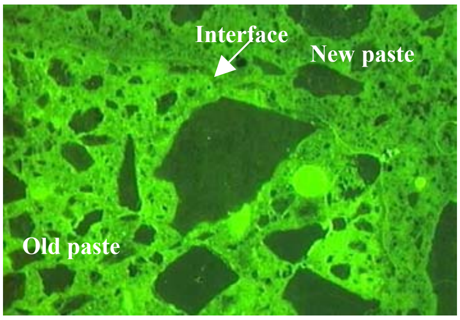


Fig.4.25: Interface. Width of field 3,95 mm

Other recycled aggregate specimens are shown in figure 4.26 and 4.27. The original concrete is less dense than the new paste and has more pores and a higher w/c ratio. The interfacial transition zone is also effective in these aggregates, corroborating our previous findings which indicated the weakest point in concrete made with recycled aggregates could be the recycled aggregates and not the interfacial transition zone as is determined with conventional concretes.

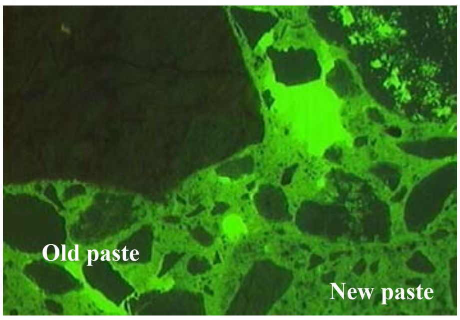


Fig.4.26: Interface. Width of field 3,95 mm

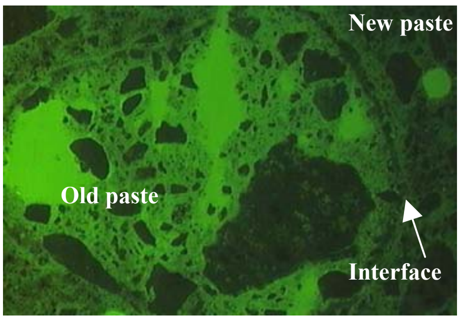


Fig.4.27: Interface Width of field 3,95 mm.

The adhered mortar in the recycled aggregates is more fragile and this proved to be a problem with respect to the samples preparation time.

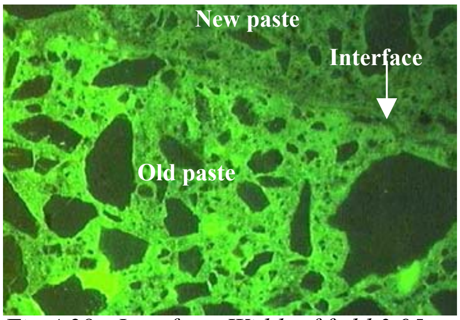


Fig.4.28: Interface. Width of field 3,95 mm

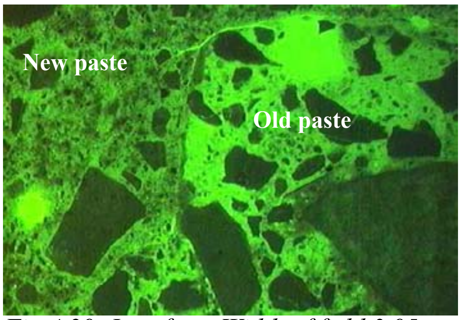


Fig.4.29: Interface. Width of field 3,95 mm.

As figure 4.30 clearly depicts the interface of the natural coarse and fine aggregates with the new paste is acceptable in all types of concretes.

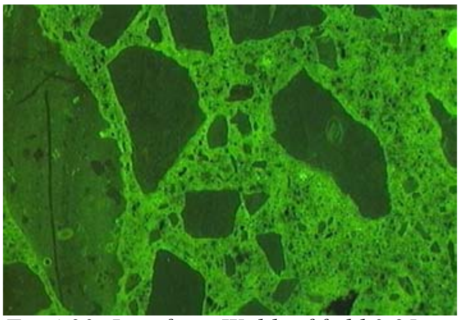
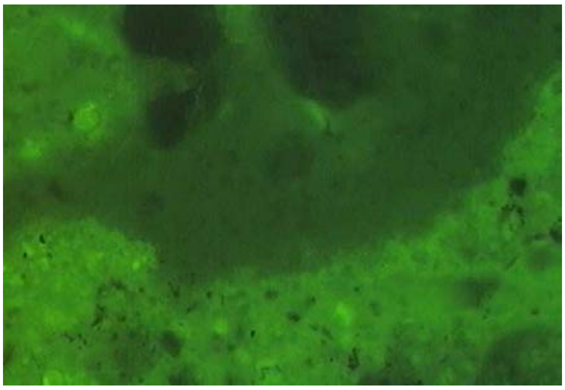


Fig.4.30: Interface. Width of field 3,95 mm

In figure 4.31 it can be seen that the interfacial transition zone was quite effective in recycled aggregates. The accumulation of cement exists in the interface. The interfacial transition zone of the recycled aggregates was denser than that of the new cement paste, see figure 4.31-a). In figure 4.31-c) white shining crystals (Portlandite) cannot be seen in the cement paste of the interface, which means that in this particular interface of the recycled aggregates the water quantity was low, with a very low w/c ratio and therefore the possibility of unhydrated cement exists. This accumulation of cement could be effective if it were supposed that in its life-time the cement could be hydrated. This cement accumulation could cause durability problems and concrete failure due to the cement's alkali concentrations in the interface, if the aggregates were reactive as they were in this case.

In figure 4.32 the existence of the white shining crystal (portlandite crystals due to water at that point) can be seen in the border of the original aggregates (yellow letters). This is a common occurrence with respect to raw aggregates, resulting in the interface being the weakest point of the concrete. In the border of the recycled aggregates (white letters), white shining points, portlandite crystals, cannot be seen due to their minute size or an unhydrated cement accumulation. As mentioned above, this can be acceptable when the original fine and coarse aggregates are not potentially reactive due to the accumulation of alkalis in the interface which can produce an expansive reaction.



a) Flourescent light

b) Ordinary light

c) Polarised ligth

Fig 4.31: Cement accumulation in Recycled aggregate interface. Pictures size, 520x140 µm. a)The interface under fluorescent light examination. b) The interface under ordinary light examination. c) The interface under polarised light examination.

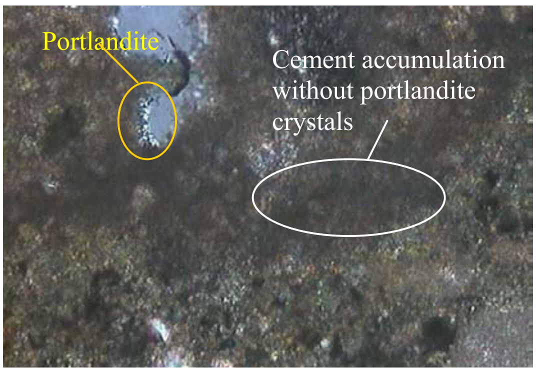


Fig 4.32: Interfaces of aggregates. The interface of original aggregates (yellow letters) there is an accumulation of portlandite crystals. In the interface of recycled aggregates (white letters), there is an accumulation of cement without portlandite crystal.

4.3.5 Damage and secondary reactions

An optical microscope analysis of the different concretes studied revealed no presence whatsoever of any damage. However, as previously mentioned, a change in the environmental condition produced an aureole in the interfacial transition Zone, between the adhered mortar of the recycled aggregates and the new cement paste. This aureole only occurred around the adhered mortar of the recycled aggregates, evidently its presence must be related to the cement which is accumulated in the interface and to the absorption capacity of the aggregates. In order to determine the composition and the shape of the aureoles, the recycled aggregate concrete was analysed under Scanning Electronic Microscope (SEM) achieving composition of aureoles by EDX-maps.

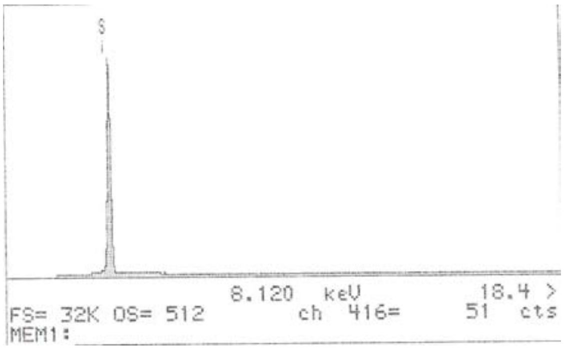
4.4 CONCRETE AND INTERFACIAL TRANSITION ZONE EXAMINED BY SEM AND EDX-MAPS

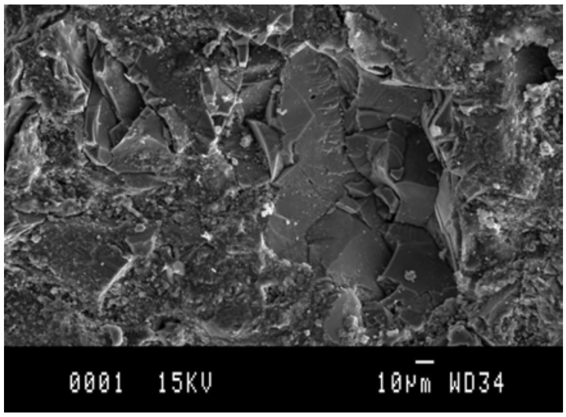
The adhered mortar of the recycled aggregates is potentially reactive see Chapter 3. Consequently, the original fine aggregates of the recycled aggregates must be reactive. The aureoles always occurred round the adhered mortar where the cement was accumulated. Therefore, ASR-gel could occur. In order to verify this possibility, the fine original aggregates, original cement paste and the aureole of the interfacial transition zone of the HR100 were examined under SEM and their EDX maps were determined.

4.4.1 Original Aggregates

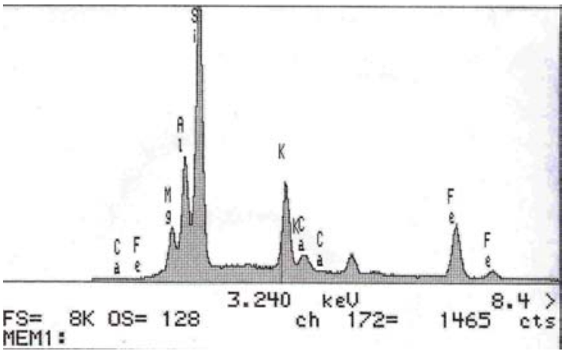
The sample of the HR100 without a polished surface and impregnated by carbon was examined.

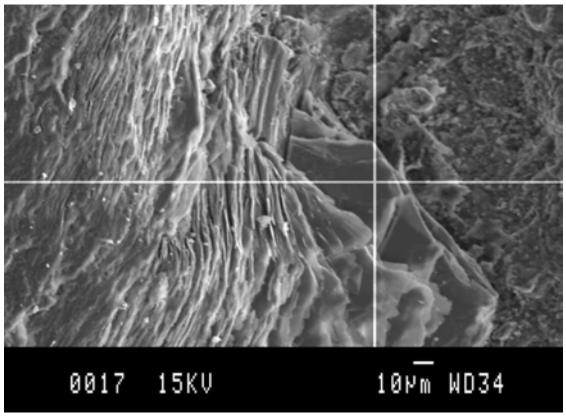
Silica is the principal composition element of the original fine aggregates of the recycled aggregates (see figure 4.33).


Si
8.120 keV 18.4 >
FS= 32K OS= 512 ch 416= 51 cts
MEM1:

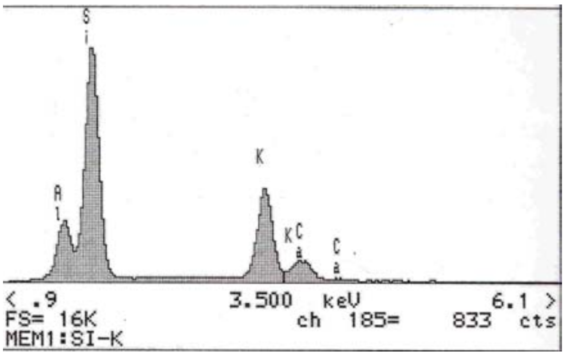

0001 15KV 10μm WD34

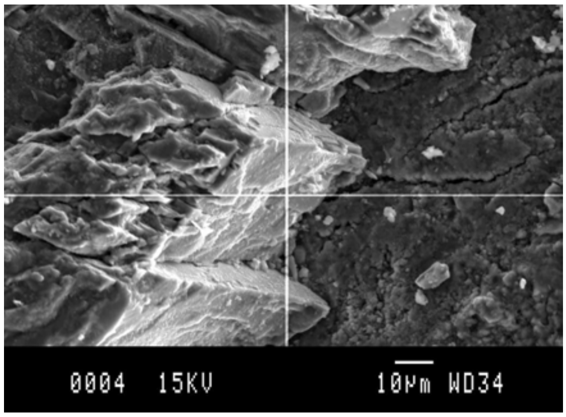
a)


Si
Al
Mg
Ca
Fe
K
KCa
Ca
Fe
Fe
3.240 keV 8.4 >
FS= 8K OS= 128 ch 172= 1465 cts
MEM1:

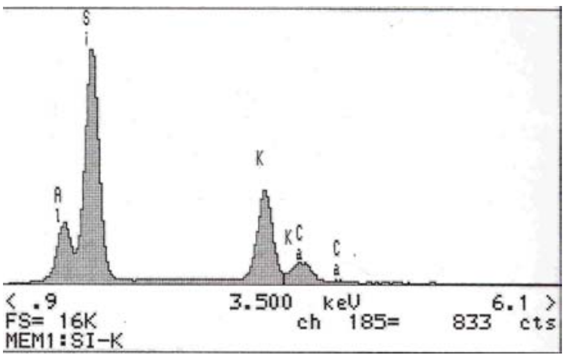

0017 15KV 10μm WD34

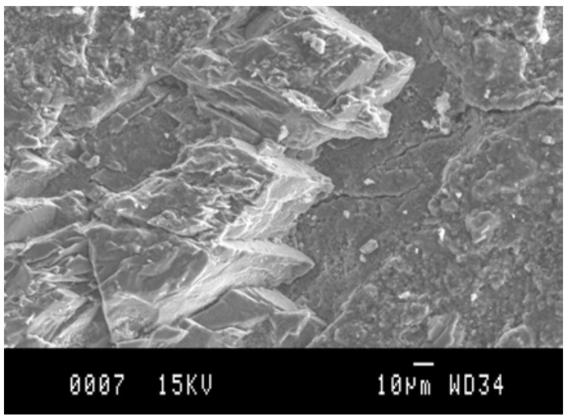
b)


Si
Al
K
KCa
Ca
< .9 3.500 keV 6.1 >
FS= 16K ch 185= 833 cts
MEM1:SI-K

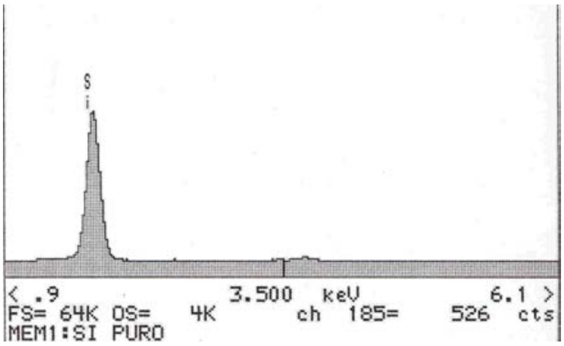

0004 15KV 10μm WD34

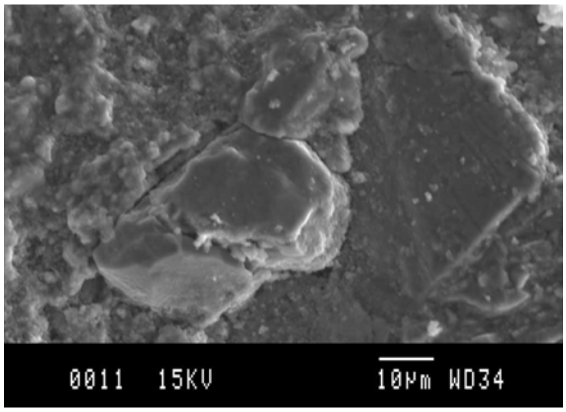
c)


Si
Al
K
KCa
Ca
< .9 3.500 keV 6.1 >
FS= 16K ch 185= 833 cts
MEM1:SI-K


0007 15KV 10μm WD34

d)





e)

Fig.4.33: Spot inside of different aggregates. a) and e) quartz aggregate, b), c) and d) potassium feldspar

The recycled fine aggregates are defined as quartz and potassium feldspar.

The cement paste around the original fine aggregates has the composition CSH of Portland cement CEM I, see figure 4.34.

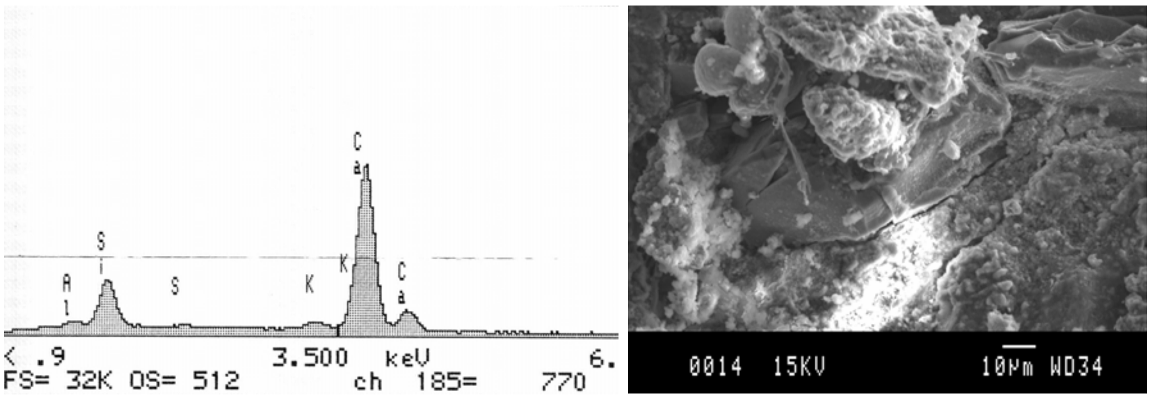


Fig.4.34: Very small area of the cement paste around of the original aggregate

The cement paste of the recycled aggregates is more porous than the new paste but its composition is defined as common CSH in Portland cements. These aggregates were sometimes cracked due to the manufacturing phases and their life span. The cracks always started in the ‘Silica’ original aggregates found in the recycled aggregates, they crossed the original cement paste of recycled aggregate and they ended in the interface with the new cement paste. The cracks were empty, not one reaction product was detected.

4.4.2 Original cement paste

The cracks were clear and were not filled with any reaction product, consequently they were produced in the manufacturing phases and the concrete's life cycle. Figure 4.35, depicts the analysis of the recycled aggregate's cement paste composition.

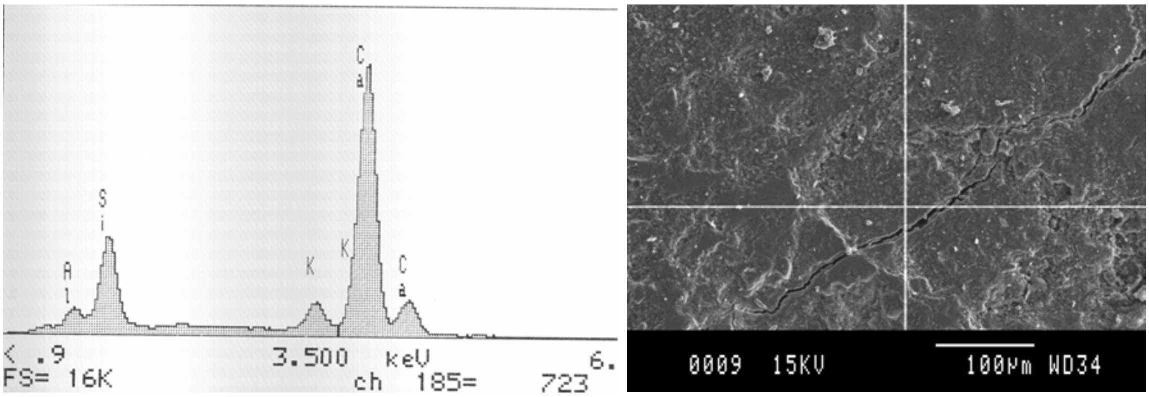


Fig.4.35: Spot in the recycled aggregate cement paste.

The composition of the cement paste of the recycled aggregates was found to be a common one. The original paste was a conventional CSH gel and almost all of the fine aggregates had Si as a main component. However, one of the most important points to analyse was the interface zone of the recycled aggregates and the new paste.

4.4.3 Interfacial transition zone

The interfacial transition zone is the weakest area in conventional concretes due to the accumulation of water at this point. However, an accumulation of the cement in the interface zone can occur in recycled aggregates with the consequent affect of the adhered mortar being the weakest point.

A SEM was employed to analyse "the interface transition zone" of a sample of unpolished concrete where the aureole (dense line in SEM) was detected. The image of the interfacial transition zone is shown as a continuous material, see figure 4.36.

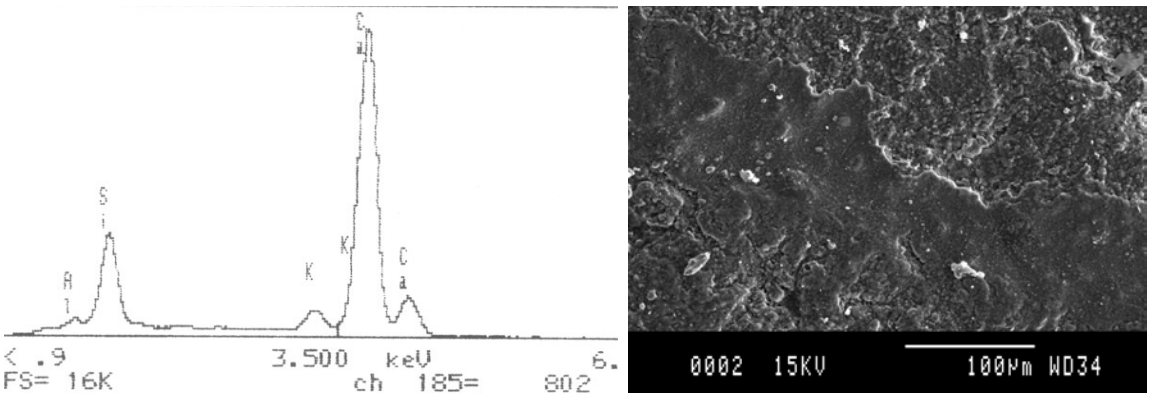


Fig.4.36: Interfacial transition zone between recycled aggregates and new paste

The EDX map defines the total area of the picture, and the general composition of this area is defined as CSH. This resolution was expected due to the fact that the aureoles (dense line in SEM) were above the cement paste. When an area of the interface was examined through increased magnification (from 100µm to 10 µm) the properties changed little with respect to the larger area under study, the amount of Si was higher, see figure 4.37. When a spot in the aureole was marked for study (dense line in SEM), its examination revealed that its composition was different to that of the larger area, with a higher quantity of Si, see figure 4.38.

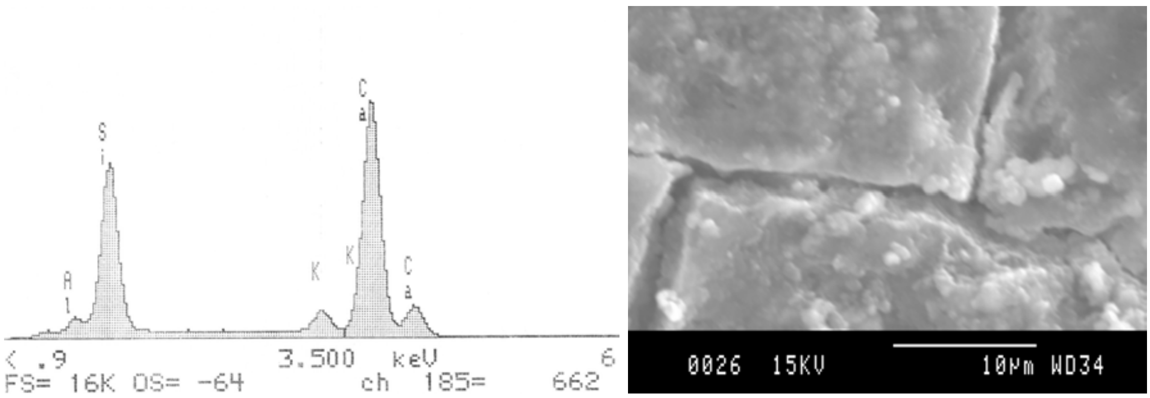


Fig.4.37: Interfacial transition zone between recycled aggregates and new paste. EDX map of the total are of the picture

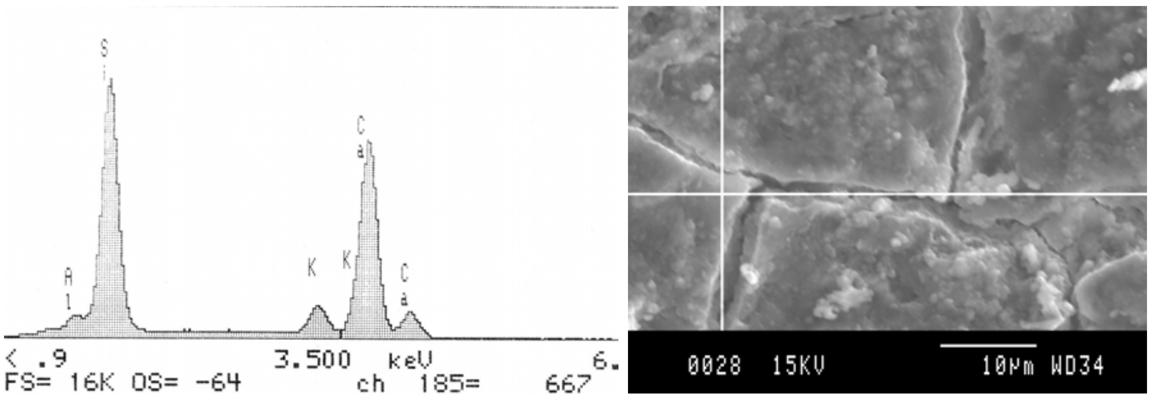


Fig.4.38: Spot in Interfacial transition zone between recycled aggregates and new paste.

The SEM analysis of the aureole spot revealed that although the element composition was similar, the Si quantity was higher, and this is not usual in the CSH composition of hydrated Portland cement. It is a usual composition of ASR, in figure 4.39 there is a general image of the aureole, typical image of ASR.

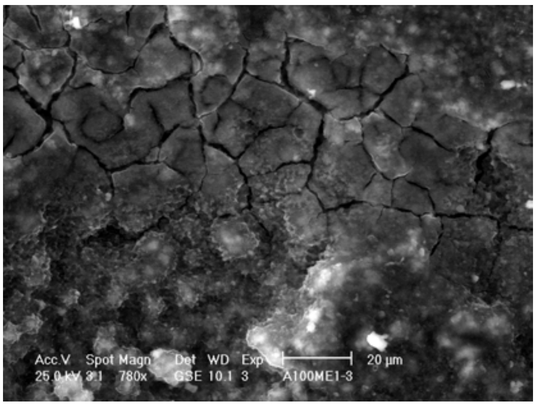
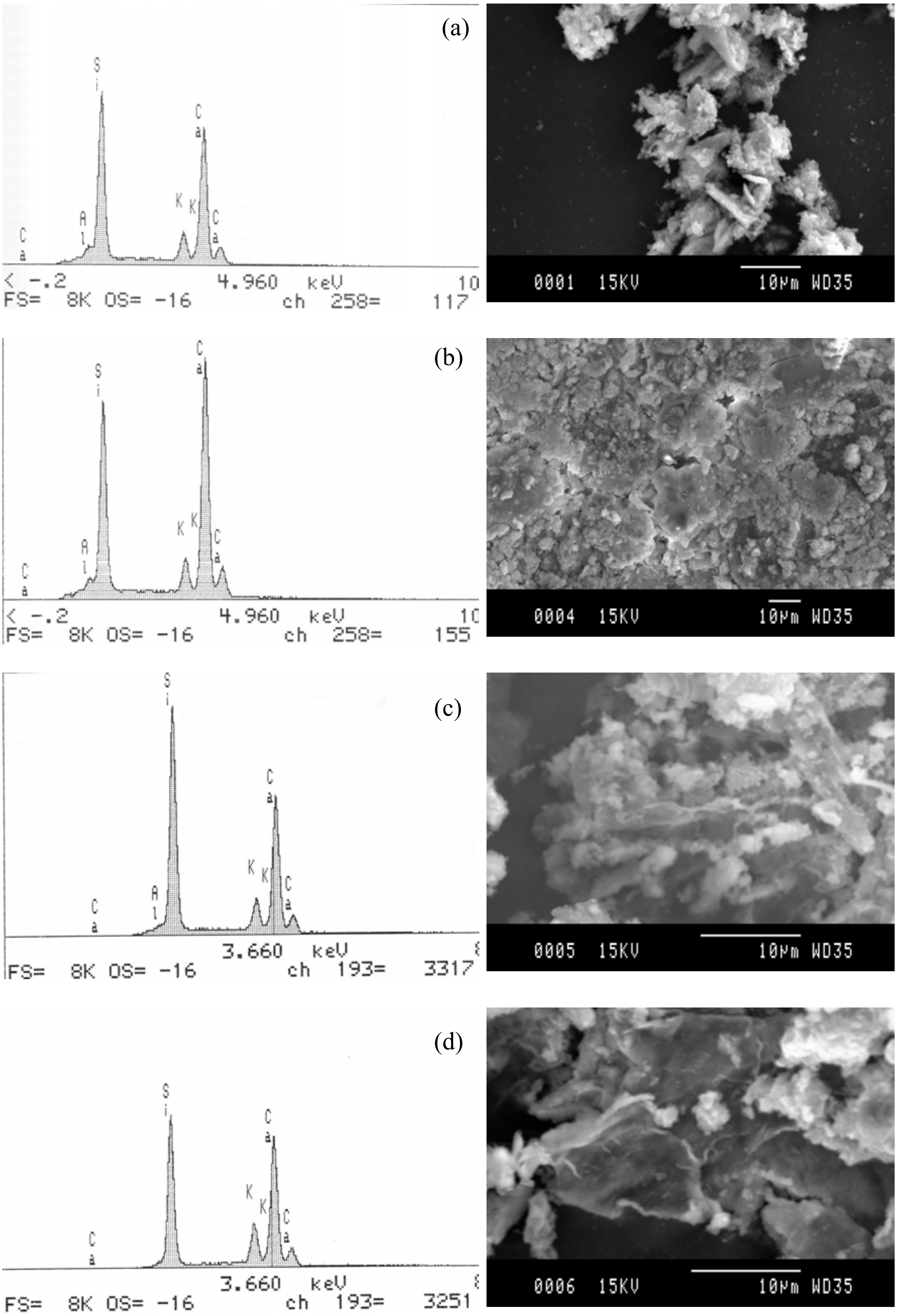


Fig.4.39: General image of the reaction product.

In accordance with figure 4.38, the composition of the aureole was made up of principally Si, Ca and K, with a higher quantity of Si. It was considered necessary to analyse the aureole devoid of the cement paste. This material was removed meticulously to avoid the inclusion of any concrete or other contaminants before its examination under SEM, and some EDX maps were produced in order to determine the aureole composition.

It is possible to view the aureoles pictures with their corresponding EDX maps in the following figure 4.40. The EDX are defined by a numerous amount of spots in the same material, and the most prominent EDX spots are shown as follows.


(a)
Si
Ca
K
Al
< -.2 4.960 keV 10
FS= 8K OS= -16 ch 258= 117
0001 15KV 10μm WD35
(b)
< -.2 4.960 keV 10
FS= 8K OS= -16 ch 258= 155
0004 15KV 10μm WD35
(c)
3.660 keV
FS= 8K OS= -16 ch 193= 3317
0005 15KV 10μm WD35
(d)
3.660 keV
FS= 8K OS= -16 ch 193= 3251
0006 15KV 10μm WD35

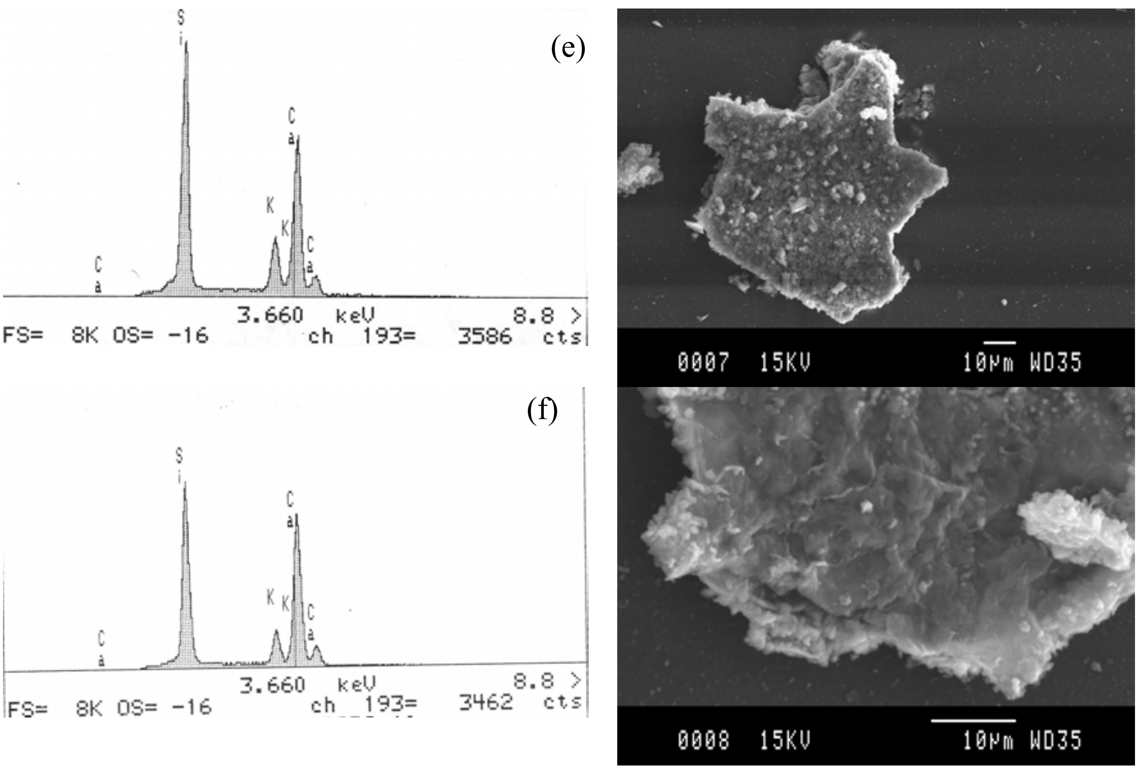


Fig. 4.40: Aureole (ASR gel) composition by spots. From (a) to (d) are different samples. (e), is a different sample (g) is another one.

The aureole was examined employing diffraction and the form of the material is amorphous.

4.4.4 Alkali Silica Gel

The aureole is defined as an Alkali Silica Reaction (ASR) gel. Cement accumulation was detected by optical microscope in the interface of the aggregates. Consequently, alkali accumulation was in the adhered mortar. The original fine aggregates had Si as their principle composition. The solubility of SiO_2 is strongly dependent upon pH. With respect to amorphous silica when the pH is less than 8 the solubility is constant (at $10^{-2.7}$ M) with decreasing pH, while it rises rapidly with respect to higher pH values (Stumm and Morgan, 1970). In this case the large amount of hydroxyl (OH^-) ions present in the pore solution, due to high alkali concentration (potassium and sodium), dissolve the reactive silica on the aggregates surface to form an alkali silica gel.

Mortar bars were cast using the adhered mortar (original fine aggregates) of recycled aggregates and they were submitted to sodium solution for 56 days (ASTM C1260). The mortar bars suffered a substantial expansion (see chapter 3, section 3.2.3) due to the Alkali Silika reaction. The silica aggregates dissolved, see figure 4.41.

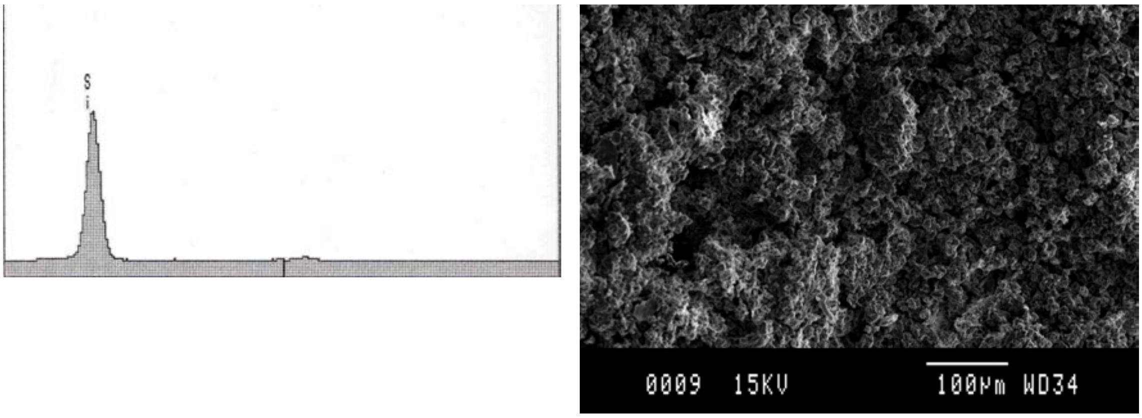
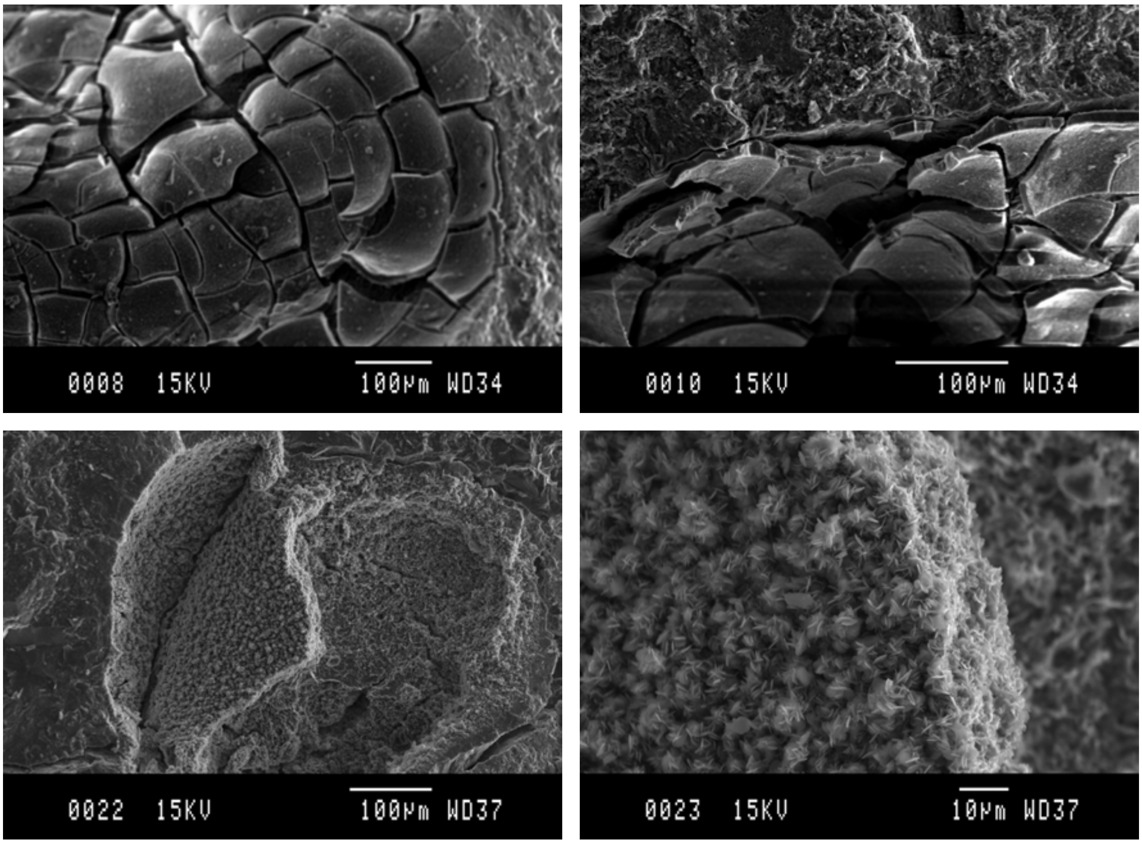


Fig. 4.41: Silica aggregates appear totally dissolved in alkali solution

The alkali silica reaction appears in different shapes, but all the types depicted have the same composition, see figure 4.42.



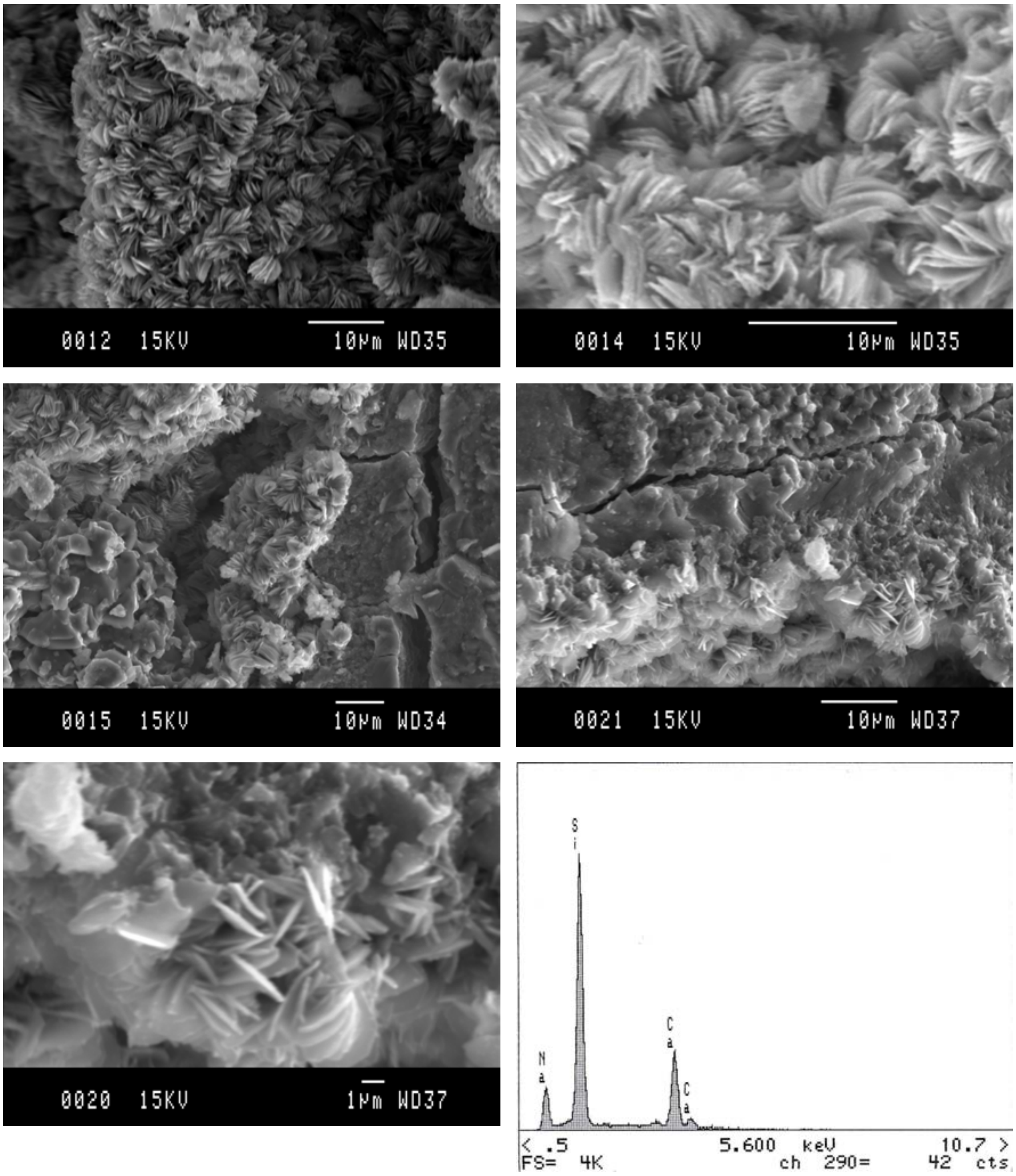


Fig. 4.42: Different shapes of Alkali Silica Reaction. The composition is Si-Ca-Na.

4.5 CONCLUSIONS

Some conclusions are obtained from this research work, the most important ones are:

- Alkali Silica Gel was produced as a result of the alkali contribution from new cement and the reactive fine aggregates of adhered mortar in conditions when the concrete was saturated or almost saturated.

    - The original fine aggregates of adhered mortar were potentially reactive as shown in Chapter 3 and they consisted of quartz and potassium feldspar with silica as the main mineral, SEM and EDX-maps were used to examine them.

    - The used cement, CEM I 52.5R (which is a very fine material) has high amount of alkalis (>0.6%). The cement was found to have accumulated in the interface as a result of the concrete production, the fineness of the cement employed and the recycled aggregates absorption capacity. This accumulation produced an effective interface between the recycled aggregate and the new cement paste, however, alkali accumulation also occurred.

    - The water which is in contact with this accumulated cement reaches a high pH value by dissolving the reactive silica on the aggregates surface to form an alkali silica gel.

    - The Alkali Silica Gel is only appreciated in the interface of recycled aggregates and the cement paste. It does not fill pores and cracks, so it does not affect the concretes' properties.

An analysis of the microstructure of the concrete made with different percentages of recycled aggregates (0%, HC; 25%, HR25; 50%, HR50; 100%, HR100) was carried out and the conclusion obtained are the following:

- According to macrostructure analysis the distribution of the aggregates was homogeneous in all concretes. All the recycled aggregates studied behave similarly from this point of view. The high porosity of the original cement paste is evident.

- Contaminants must be avoided in order to produce good quality concrete. Aluminium in recycled aggregate reacts with the cement producing Hydrogen bubbles and cracking the concrete. The percentage of this class of "aluminium aggregate" in concrete production must be nil (0%).

- The Fluorescent thin-section (FTS) method of analysis by optical microscope is an excellent method to compare the w/c ratio of the paste, the interfacial transition zone and also the quality of the aggregates.

- The original aggregates were of a poorer quantity when compared to those of the raw aggregates regardless of the presence of adhered mortar.

- The adhered mortar of the original concretes is less dense than that of the new cement paste. The porosity is high resulting in the poor quality of the recycled aggregates.

- The effective w/c ratio of the HR50 and HR100 mixes is lower than that of the HR25 and HC mixes. With the lower difference, the water trapped inside of the aggregates did not react with the cement. However the difference is small, which concludes that the effective w/c was the expected one.

- The density of the paste is homogeneous and the air void quantity is more or less the same. Consequently it can be stated that there are a lot of similarities in the different types of concretes studied.

- The air void percentage (larger than 152 μm) is similar to those found in HC, HR25 and HR50 mixes (3,17%, 3,73% and 3,59%, respectively). When a 100% of recycled coarse aggregates are used in concrete, the larger size (850 μm-2000 μm) air void quantity is higher. The percentage of the volume of air void increase to 5,74%. When air voids of adhered mortar are considered, the percentage of porosity increase is around 0,4%. (It does not refer to voids of submicroscopical dimensions).

- Due to the order of the material added to the mixing machine and because the recycled aggregates were humid, the cement adhered to the aggregates before the water was added resulted in the production of an interfacial transition zone with a very low w/c ratio. An accumulation of cement occurred in the interface. There was probably an accumulation of a certain quantity of unhydrated cement.

- The hydration which will occur during the concrete's life guaratees an effective interfacial zone.

- With potentially reactive or reactive original aggregates, it is recommened to use low alkali portland or blast slag cements as they may increase the durability of the recycled concrete.

- With respect to the concrete made with recycled aggregates, the quality of interfacial transition zone is better than that of the old paste, consequently in these concretes, the weakest point could be the adhered mortar. Therefore the adhered mortar strength will be what determines the material strength and behaviour. This is one of the greatest differences with respect to conventional concrete.

Chapter 5

Structural behaviour of beam specimens. Test description

5.1 OBJECTIVES

The objective of this part of the investigation was to analyse how different percentages of coarse recycled aggregates in specimens of the same concrete strength influenced the structural behaviour of beams. Such specimens were considered to be representative of elements subjected to bending or compression but with limited structural responsibility such as prefabricated facade panels or precast elements of one way composite slabs.

For this purpose, shear failure tests were conducted on several reinforced beams in order to identify the behaviour of these elements as well as the influence of these parameters. The beams contained different transversal reinforcement and different percentages of coarse recycled aggregates.

The approximate flexural strength of the beam specimens could be determined by the mechanical properties obtained from test elements. Concrete strength has little influence on flexural strength, therefore it was sufficient to assure the compression strength.

However, it is not the same when shear stress exists. In this case crack-friction, tensile strength or loss of anchorage by insufficient bond influence the structural strength and they are unheard of in these kind of concretes. In order to make a precise study of shear stress, it is necessary to obtain additional information: displacements, appearance and evolution of the cracks, bond between the concrete and reinforcement, and also the flexural strength if shear failure is produced.

In the previous chapter, the appropriate dosages were defined as HC, HR25, HR50 and HR100 with 0%, 25%, 50% and 100% of coarse recycled aggregates, respectively, in order to achieve the same compression strength. The same dosage was used to cast the beams.

The structural behaviour of these beams was analysed using four different transversal reinforcements for each kind of concrete, in all a total of 16 beams.

5.2 DESCRIPTION OF THE TESTED BEAMS

All the beams were reinforced with the same quantity of longitudinal reinforcement. With respect to transversal reinforcement, there were four types of distributions, described as V1, V2, V3 and V4.

5.2.1 Geometry of beam specimens

The geometry used in the V1, V2, V3 and V4 beams is described as follows:

- The beams' chosen lengths were 3,05 m, due to limitations of the test machine.
- The total depth of the beams was 35 cm.
- The a/d (Shear span/depth) ratio chosen was 3,3. So that the shear stress did not affect the D zone.

  The result of the distance of the point of application of the load to the support axis was 1,0 m.

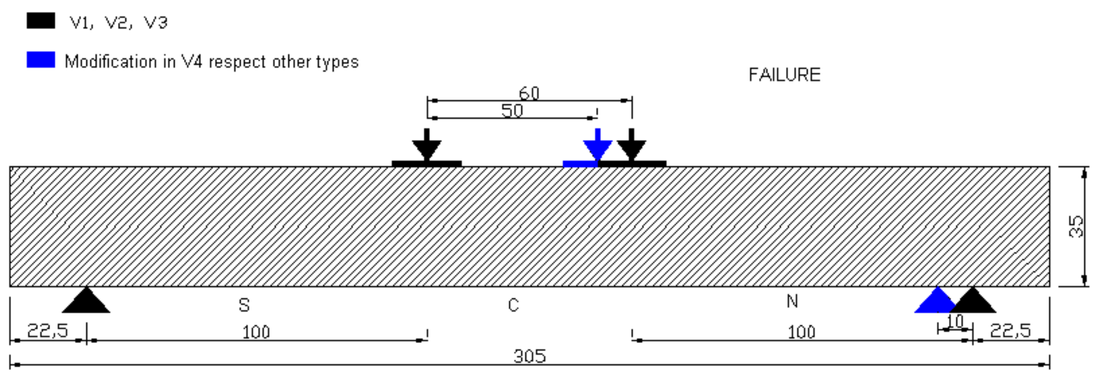


Fig.5.1: Description of support and load point in all beams

Incident. Because of an error in the web reinforcement supply of beams type 4 (V4), the load and support position were modified, as shown in figure 5.1. The new position achieved a comparable and symmetric reinforcement of distribution with respect to the load cell.

5.2.2 Shear reinforcement

According to stress laws, three different zones were defined depending on the reinforcement.

- *North side* (referred to as N, see figure 5.1), failure side with the lowest web reinforcement in beam specimens, where there was an increase of moment law moving away from the support point. The shear stress was constant.

- *South side* (referred to as S, see figure 5.1), symmetric to the north side but with a higher web reinforcement. This side was also placed between support and load points.

- *Central side* (referred to as C, see figure 5.1), constant moment and therefore, zero shear stress. It was placed between two load points and the two sides mentioned above.

The web reinforcement distributions are shown in tables 5.1 and 5.2. The web reinforcement was designed to obtain the failure in the north side. The adopted criteria concerning the reinforcement position in the north side were (they are illustrated in figure 5.2, 5.3, 5.4 and 5.5):

- *Type V1:* 0 cm^2/m of reinforcement. In this series, since there was not transverse reinforcement, the influence of the concrete strength was clearly observed.

- *Type V2:* Higher quantity of reinforcement (0,435 mm^2) than demanded by EHE (Spanish structural concrete code). The transverse reinforcement consisting of stirrups of ϕ6 (6 mm diameter) was separated by 130 mm.

- *Type V3:* Minimum reinforcement demanded by EHE. It was calculated by:

  $$\sum \frac{A_\alpha \cdot f_{yd}}{\operatorname{sen}\alpha} \geq 0{,}02 \cdot f_{cd} \cdot b_o \qquad [5.1]$$

  Where A_α is the area of web reinforcement per unit length, inclined at an angle α with respect to the longitudinal axis of the beam, $f_{y\alpha,d}$ is the design yield strength of the transverse reinforcement inclined α, and f_{cd} the design compression strength of the concrete. For an area of $A_\alpha \approx 0{,}330$ mm^2, the maximum distance between stirrups of 6 mm diameter must be 171 mm. They were situated at a distance of 170 mm.

- *Type V4:* Less amount of reinforcement than required by EHE for this geometry. The reinforcement area was considered in accordance with EHE, with the maximum separation allowed for compressed longitudinal reinforcement. Therefore stirrups of 6 mm diameter were separated by a distance of 240 mm.

The reinforcement amount on the south side was established by changing the diameter of stirrups from 6 to 8 mm to ensure that shear failure did not take place in this zone. Therefore it was also possible to detect the diameter influences in the concrete.
In all cases the steel was B-500S, with f_{yk}=500 N/mm^2.

5.2.3 Longitudinal reinforcement

In order to assure the shear failure, it was necessary to avoid a previous flexural failure. To achieve an ultimate load bending capacity larger than the shear capacity in all cases, the reference beam (V2) had to be the one with the highest shear capacity and, therefore it had to have the highest amount of shear reinforcement.

The flexural strength of the beam was such that the shear produced would be 2.8 times the shear strength, according to EHE. The necessary reinforcement to resist this load consists of 2ϕ32 (2 bars of 32 mm diameter) and 1ϕ16 (1 bar of 16 mm of diameter) in the central zone.

In all cases the steel was B-500S, with f_{yk}=500 N/mm^2.

5.2.4 Geometric details

In accordance with EHE the minimum clean cover was the maximum diameter of the aggregate, which for the beams tested was 25 mm. Therefore a cover r= 25 mm was adopted, an effective depth being 303 mm.

The reinforcement distributions are shown in figures 5.2, 5.3, 5.4 and 5.5. Figure 5.5 shows theoretical and real reinforcement distribution and the position of the support with respect to the load location of beam type 4 (V4).

The north support point was moved 10 cm to the centre side in order to achieve a symmetric distribution of the stirrups in relation to the test load point (see figure 5.1) The anchorage of the beams being 10 cm longer on the north side. The displacement of the support point led to a decrease in the centre side from 60 cm to 50 cm in order to maintain the beam span.

Consequently, the centre length was 50 cm instead of 60 cm. The length of the south side remained the same. The beam was symmetrically maintained with respect to the load cell but the anchorage length on the north side was 10 cm longer than on the south side.

The distance between the end of the beam and the support point was conditioned by the anchorage length necessary for the longitudinal reinforcement. The radius of curvature of a bar with 32 mm diameter was defined by 11,2 cm which was less than the distance to the end-support point (22.5 cm).

The four types of reinforced sections are illustrated in figure 5.6.

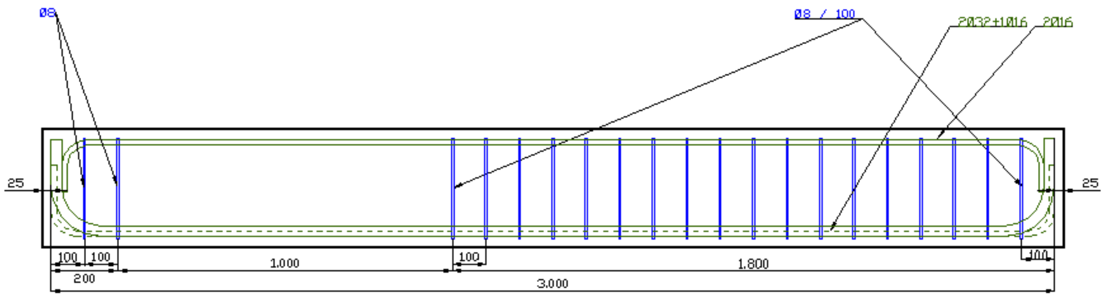


Fig. 5.2: Reinforced Type V1 (mm)

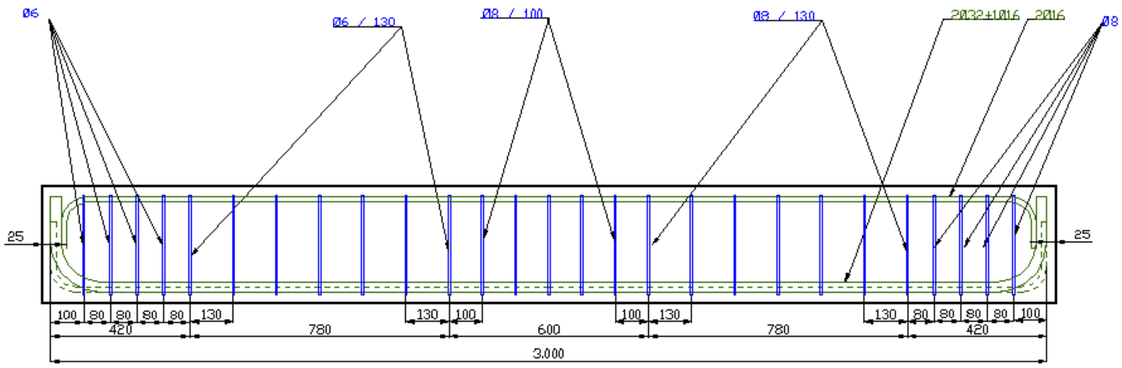


Fig. 5.3: Reinforced Type V2 (mm)

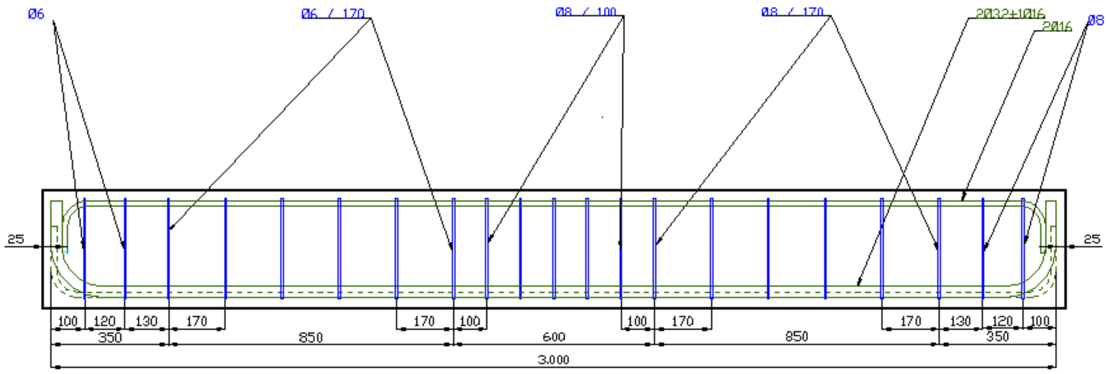


Fig. 5.4: Reinforced Type V3 (mm)

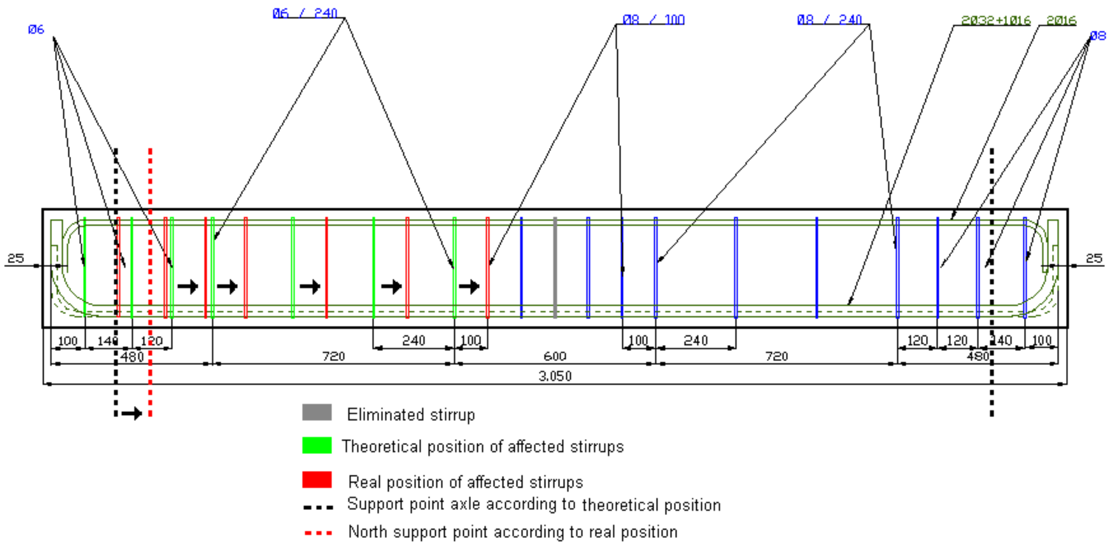
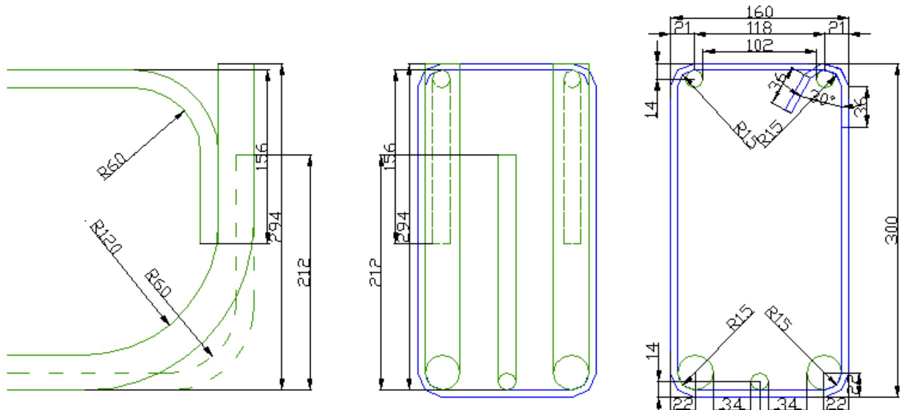
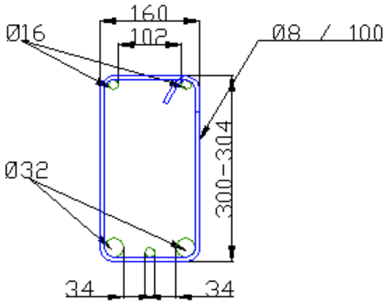


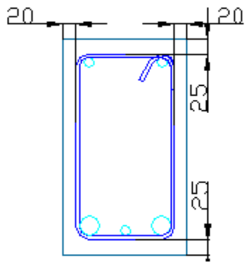
Fig. 5.5: Reinforced beam Type V4 (mm)



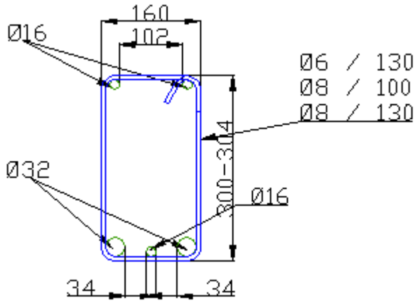
Detail of reinforcement (mm)



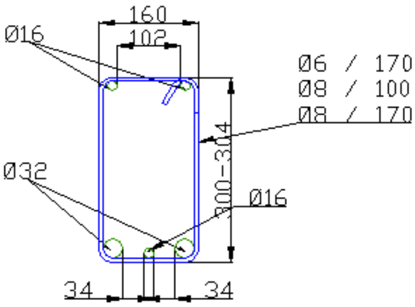
Section V1 (mm)



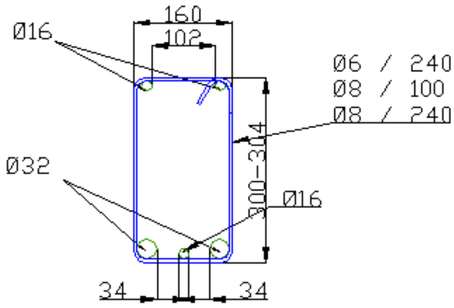
Clean cover (mm)



Section V2. (mm)



Section V3 (mm)



Section V4 (mm)

Fig.5.6: Cross section of the beam specimens

Table 5.1: Details of beam specimens in failure side (North side)

Beam	fc MPa	fc, failure MPa	b mm	d mm	a/d	Shear reinforcement			Long. Reinforcement		Cast Date	Test Date
						Stirrup/ spacing mm	ρ_t%	ρ_t MPa	Longitudinal reinforcement	ρ_l		
HC-1	35.53	41.91	200	309	3.23	-	0	0	2∅32+1∅16	2.92	6.11.01	17.12.01
HC-2	35.53	41.91	200	304	3.29	∅6/130	0.217	1.180	2∅32+1∅16	2.97	6.11.01	20.12.01
HC-3	35.53	41.91	200	304	3.29	∅6/170	0.166	0.903	2∅32+1∅16	2.97	6.11.01	14.12.01
HC-4	35.53	41.91	200	304	3.29	∅6/240	0.117	0.636	2∅32+1∅16	2.97	6.11.01	21.12.01
HR25-1	38.79	42.38	200	309	3.23	-	0	0	2∅32+1∅16	2.92	13.11.01	08.01.02
HR25-2	38.79	42.38	200	304	3.29	∅6/130	0.217	1.180	2∅32+1∅16	2.97	13.11.01	10.01.02
HR25-3	38.79	42.38	200	304	3.29	∅6/170	0.166	0.903	2∅32+1∅16	2.97	13.11.01	09.01.02
HR25-4	38.79	42.38	200	304	3.29	∅6/240	0.117	0.636	2∅32+1∅16	2.97	13.11.01	11.01.02
HR50-1	39.42	41.34	210	309	3.23	-	0	0	2∅32+1∅16	2.92	20.11.01	16.01.02
HR50-2	39.42	41.34	210	304	3.29	∅6/130	0.217	1.180	2∅32+1∅16	2.97	20.11.01	17.01.02
HR50-3	39.42	41.34	210	304	3.29	∅6/170	0.166	0.903	2∅32+1∅16	2.97	20.11.01	15.01.02
HR50-4	39.42	41.34	210	304	3.29	∅6/240	0.117	0.636	2∅32+1∅16	2.97	20.11.01	18.01.02
HR100-1	38.26	39.75	200	309	3.23	-	0	0	2∅32+1∅16	2.92	22.11.01	21.01.02
HR100-2	38.26	39.75	200	304	3.29	∅6/130	0.217	1.180	2∅32+1∅16	2.97	22.11.01	22.01.02
HR100-3	38.26	39.75	200	304	3.29	∅6/170	0.166	0.903	2∅32+1∅16	2.97	22.11.01	22.01.02
HR100-4	38.26	39.75	200	304	3.29	∅6/240	0.117	0.636	2∅32+1∅16	2.97	22.11.01	21.01.02

Table 5.2: Details of beam specimens in No failure side (South side)

Beams	fc,28 days MPa	fc, failure MPa	b mm	d mm	a/d	Shear reinforcement			Long. Reinforcement		Cast Date	Test Date
						Stirrup/ spacing mm	ρ_w%	ρ_w MPa	Longitudinal reinforcement	ρ_l		
HC-1	35.53	41.91	200	301	3.23	∅8/100	0.502	2.791	2∅32+1∅16	2.92	6.11.01	17.12.01
HC-2	35.53	41.91	200	301	3.29	∅8/130	0.386	2.146	2∅32+1∅16	2.97	6.11.01	20.12.01
HC-3	35.53	41.91	200	301	3.29	∅8/170	0.295	1.640	2∅32+1∅16	2.97	6.11.01	14.12.01
HC-4	35.53	41.91	200	301	3.29	∅8/240	0.209	1.162	2∅32+1∅16	2.97	6.11.01	21.12.01
HR25-1	38.79	42.38	200	301	3.23	∅8/100	0.502	2.791	2∅32+1∅16	2.92	13.11.01	08.01.02
HR25-2	38.79	42.38	200	301	3.29	∅8/130	0.386	2.146	2∅32+1∅16	2.97	13.11.01	10.01.02
HR25-3	38.79	42.38	200	301	3.29	∅8/170	0.295	1.640	2∅32+1∅16	2.97	13.11.01	09.01.02
HR25-4	38.79	42.38	200	301	3.29	∅8/240	0.209	1.162	2∅32+1∅16	2.97	13.11.01	11.01.02
HR50-1	39.42	41.34	210	301	3.23	∅8/100	0.502	2.791	2∅32+1∅16	2.92	20.11.01	16.01.02
HR50-2	39.42	41.34	210	301	3.29	∅8/130	0.386	2.146	2∅32+1∅16	2.97	20.11.01	17.01.02
HR50-3	39.42	41.34	210	301	3.29	∅8/170	0.295	1.640	2∅32+1∅16	2.97	20.11.01	15.01.02
HR50-4	39.42	41.34	210	301	3.29	∅8/240	0.209	1.162	2∅32+1∅16	2.97	20.11.01	18.01.02
HR100-1	38.26	39.75	200	301	3.23	∅8/100	0.502	2.791	2∅32+1∅16	2.92	22.11.01	21.01.02
HR100-2	38.26	39.75	200	301	3.29	∅8/130	0.386	2.146	2∅32+1∅16	2.97	22.11.01	22.01.02
HR100-3	38.26	39.75	200	301	3.29	∅8/170	0.295	1.640	2∅32+1∅16	2.97	22.11.01	22.01.02
HR100-4	38.26	39.75	200	301	3.29	∅8/240	0.209	1.162	2∅32+1∅16	2.97	22.11.01	21.01.02

5.3 SPECIMEN DETAILS

5.3.1 Casting of test specimens

All the beam specimens were cast in the Structural Laboratory of The Construction Engineering department of UPC.

The Building process was:

- *Web reinforcement in the specimens*. It was prepared by an external company.

- *Strain gauges placement*. The bars were prepared (grinding and cleaning) and afterwards the gauge was adhered, as shown in figure 5.7.

Fig.5.7: Strain gauges in the reinforcement

- *Concrete*. The beams were cast in four days. Each day a different kind of concrete was produced. An automatic mixer machine with a maximum capacity of 250 l was used, consequently various productions of mix were needed to build the four beams of the same type of concrete, as shown in table 5.3.

Table 5.3: Data of beams production and times of production of each concrete to build 4 beams.

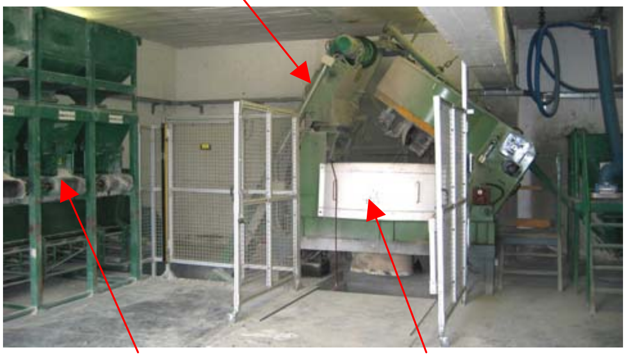
Concrete type	Cast Date	Number of times of production
HN	06/11/01	6
HR25	13/11/01	6
HR50	20/11/01	6
HR100	22/11/01	7

A problem occurred in the casting of the beam specimen HR50. The width of the beams were 210 mm instead of 200 mm this was due to the reuse of wooden moulds which had been deformed through previous casting. The difference produced has been considered in calculations.

Several images of the beam specimens casting are shown in figure 5.8.

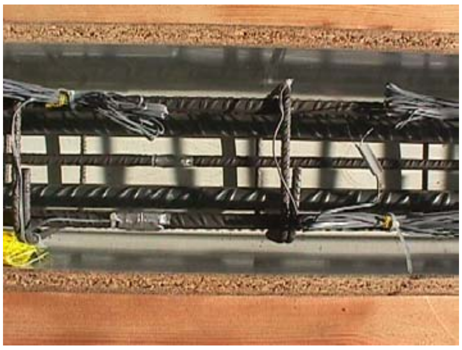
Mix machine Controller

Silos



Automatic mix machine

*Reinforcement inside moulds. Beams dimension is 305 * 20 cm.*



Reinforcement detail with the gauges

Compacting the fresh concrete

Build process of beam specimens

Cylinder test element were made for concrete control

Fig.5.8: Several pictures of casting of beam specimens

5.3.2 Materials properties

Concrete

Test specimens were taken during the casting of the beams. These specimens were used to compare the properties of the different concretes at 28 days and 6 months, as explained in chapter 3. Other test specimens were maintained in the same conditions as the beams in order to make a better comparison of the beams' concrete.

The beams and their test specimens were not only watered twice daily for 15 days to achieve the appropriate hydration of the cement but also protected from rain by a plastic covering which also maintained the humidity.

All beams were tested at between 1 and 2 months of casting. The dosages used for the four concretes designed can be seen in table 5.4.

The properties of the concretes at test date are shown in table 5.5.

Table. 5.4: Dosage of beams' concrete in kg/m^3. The w/c ratio is an effective value in the paste.

	A 0/4	**G1 4/10**	**G1R 4/10**	**G2 10/16**	**G2R 10/16**	**G3 16/25**	**G3 R16/25**	**C**	**Adit. %**	**W**	**w/c**
HC	765,13	332.7		295.07		579.2		300	1.40	165	0.55
HR25	765,1	249,5	72,8	221,3	64,6	434,4	128,3	300	1.66	165	0.55
HR50	739,0	172,1	150,6	147,4	129,2	289,4	256,6	318	1.90	175	0.52
HR100-1	683,2		425,8		306,4		391,2	325	1.90	179	0.50

Table. 5.5: Beams' concrete properties at cast date.

	Fc (MPa)	**Tensile Strength (MPa)**	**Modulus of elasticity (MPa)**
HC	41,91	2,64	33727,5
HR25	42,38	3,13	33233
HR50	41,34	3,22	31805
HR100	39,75	3,28	25391,25

Reinforcing Steel Properties

B 500 S steel reinforcing bars, with characteristic yield stress of 500 MPa, were used. Table 5.6 presents the actual yield stress, f_y, and the ultimate stress, f_u, for the web reinforcing bars which were tested following EN 10002-1, and UNE 4-474-92

Standards. Typical stress-strain curves are shown in Figure 5.9. Longitudinal reinforcement bars were not tested.

Table. 5.6: Properties of web reinforcing bars

Size (mm)	Area (mm^2)	f_y (MPa)	f_u (MPa)
∅6	28.27	544	656
∅8	50.27	556	660

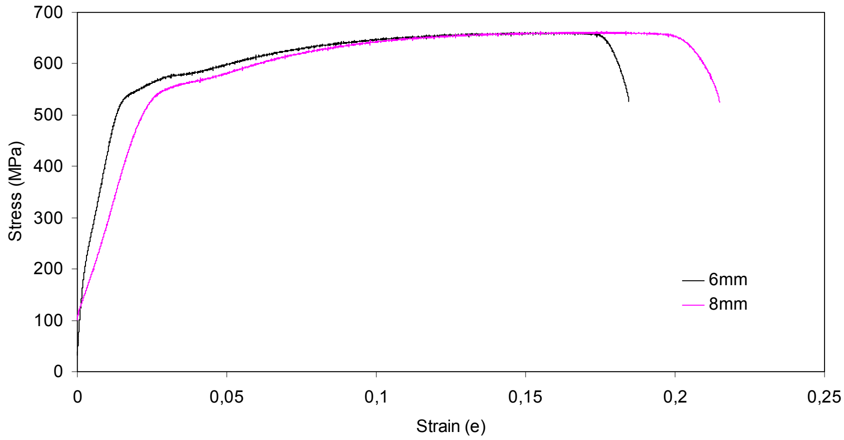


Fig.5.9: Typical stress-strain curves for web reinforcement steel

5.4 INSTRUMENTATION

5.4.1 Strain gauges

In order to measure the strain, gauges were placed on transversal and longitudinal reinforcements. The gauges of Tokyo Sokki Kenkyujo, the type FLA-6-11 were chosen. The gauges are placed differently according to the type of beam being tested. The gauges were bonded to stirrups and longitudinal bars which were considered adequate to measure the strain in the failure area, see figures 5.10 to 5.13.

5.4.2 Magnetostrictive transducers. Localization of the temposonics.

In order to determine the load-deflection curve, displacements were measured in all the beam specimens by means of magnetostrictive transducers, Temposonics ©, located below the loading points (T5 and T6) and support point (north: T2 and T1, south: T3 and T4), see figure 5.14.

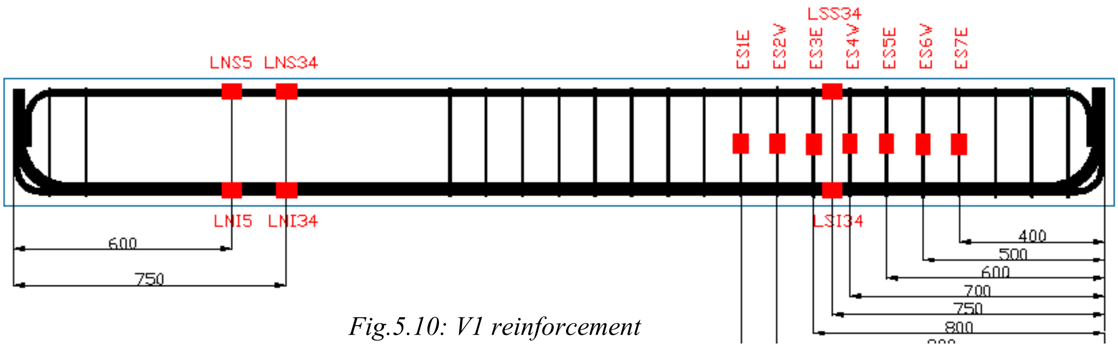


Fig.5.10: V1 reinforcement

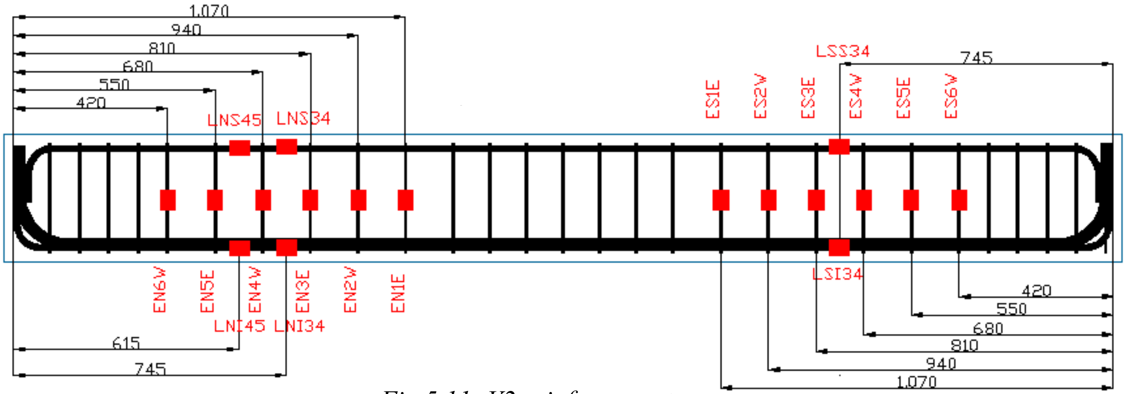


Fig.5.11: V2 reinforcement

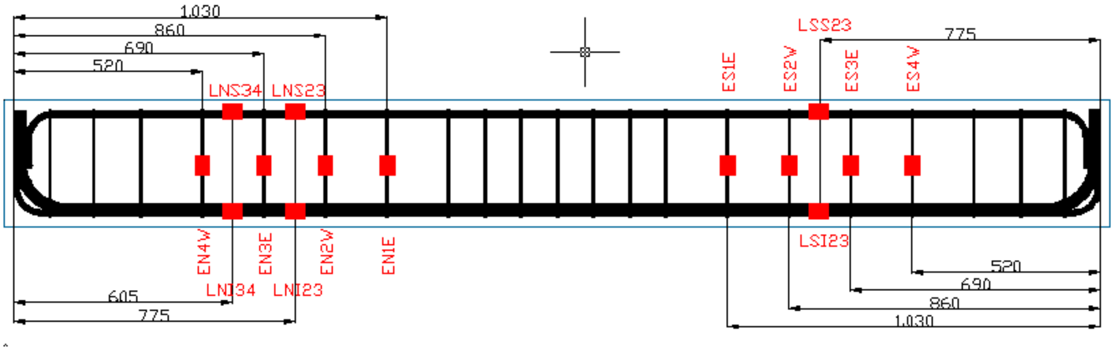


Fig.5.12: V3 reinforcement

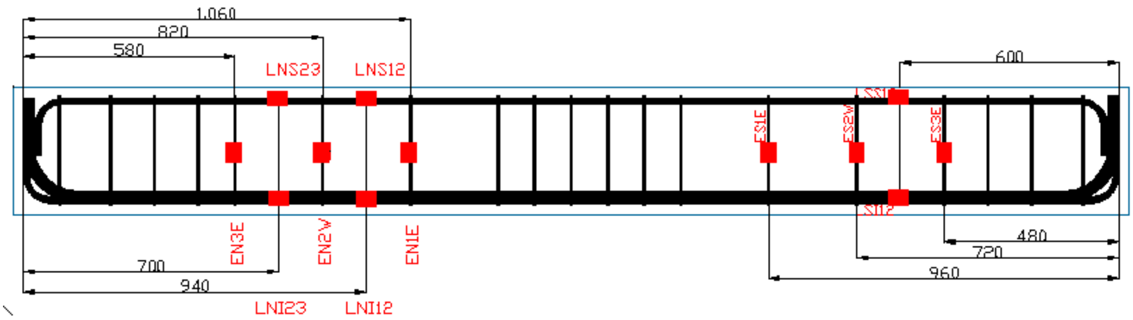


Fig.5.13: V4 reinforcement

Temposonics localization in HC and HR25 beams.

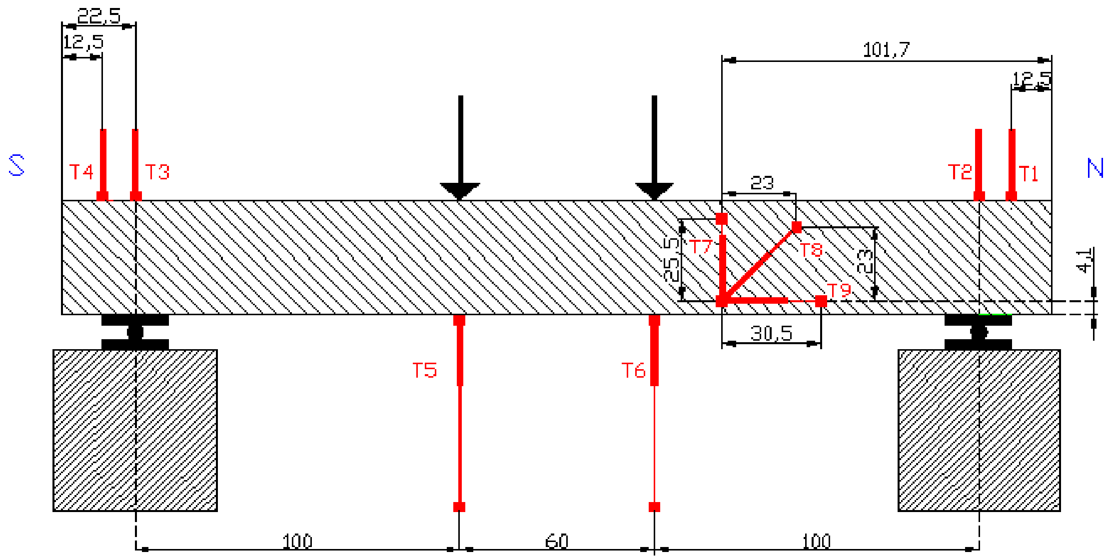


Fig.5.14: Transducers disposition according to triangular scheme for beam V1, V2 and V3 with HC and HR25.

Due to the modification conducted on V4, in order to obtain a symmetric geometry, comparable to that of the other beams, the Temposonic transducer position on the north side was moved, as were those of the stirrups.

On the south side the Temposonic transducers position remained unaltered on the four types of beams. Figure 5.15 shows the position of the Temposonic transducers for beam V4.

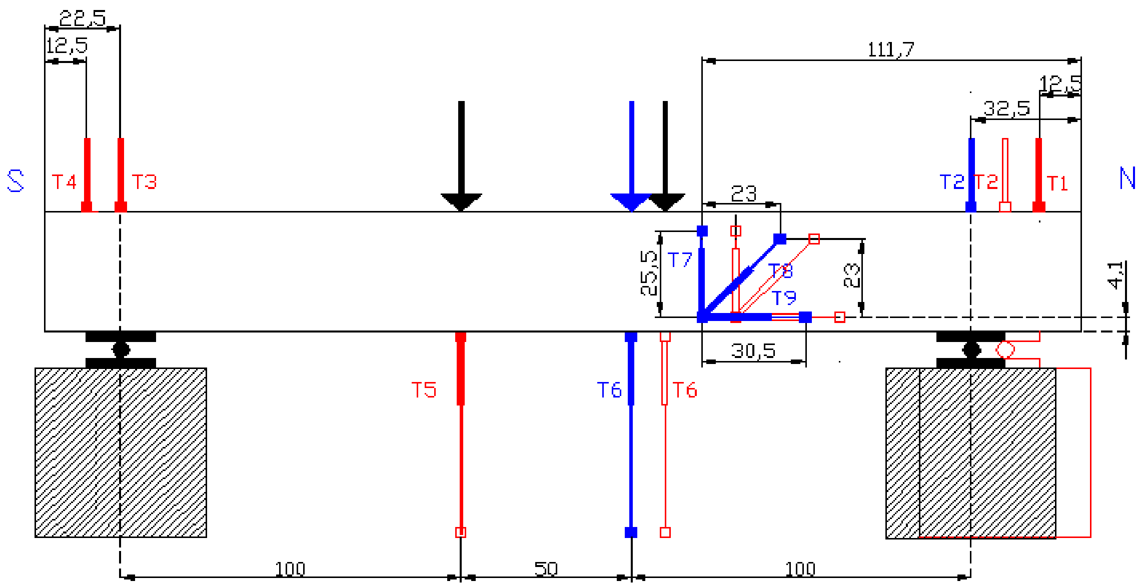


Fig.5.15: Transducers position according to triangular scheme for beam V4 with HC and HR25.

Temposonic tranducers localization in HR50 and HR100 beams.

After half of the beams were tested, it was decided to change the position of the Temposonic transducers. A rosette of three Temposonic transducers was mounted on the north side of the beam to measure the web strain γ_{xy}. Figure 5.16 presents the actual configuration.

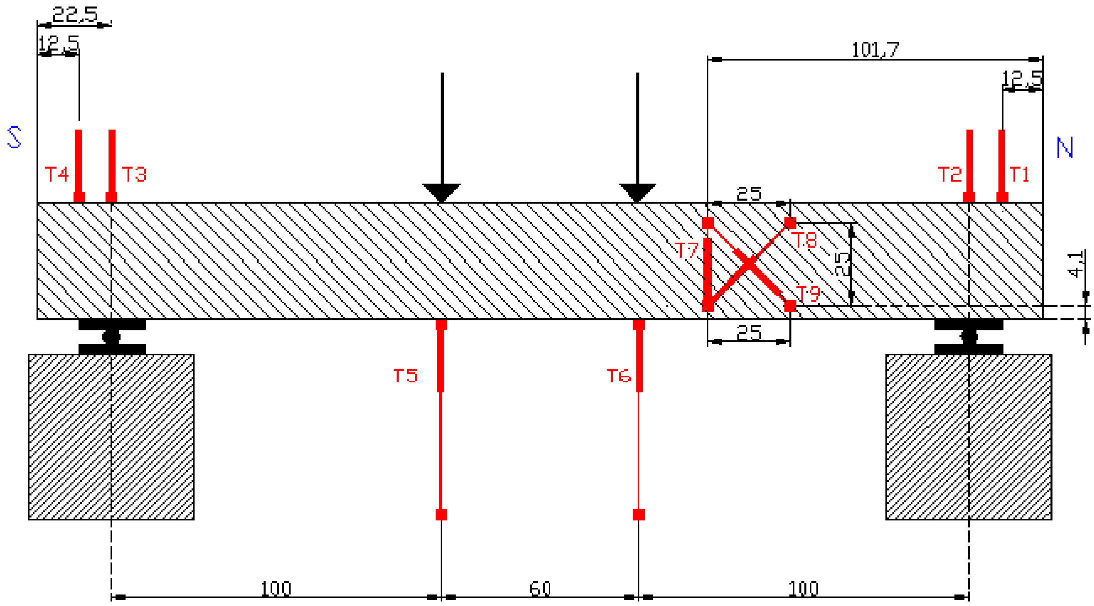


Fig.5.16: Transducers position according to rosette scheme. For beams V1, V2, V3

Due to the modification conducted on V4 to obtain a symmetric geometry comparable to that of the other beams, the Temposonic transducer disposition in the north side was moved as stirrups.

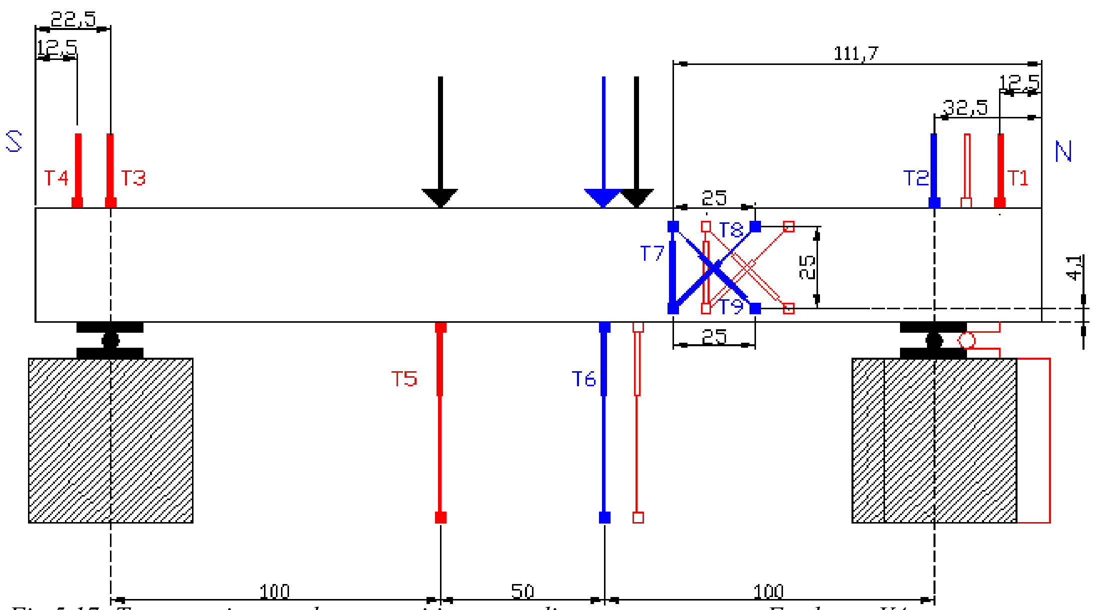


Fig.5.17: Temposonic transducers position according to rosette cross. For beam V4.

Beam specimens V1, V2, V3 and V4 were made with 50 and 100% of recycled aggregates and beam V4 with 25% of recycled aggregates were tested for configuration.

The Mohr's circle in figure 5.18 deduces how the web strain γ_{xy} may be obtained from the readings of transducers T8 and T9. The web strain can be calculated as:

$$0{,}5 \cdot \gamma_{xy} = R \cdot \sin 2\theta \qquad [5.2]$$

where R is the Mohr's circle radius and θ the angle of the principal compression strain. Or as:

$$\frac{\varepsilon_{T9} - \varepsilon_{T8}}{2} = R \cdot \sin 2\theta \qquad [5.3]$$

Where ε_{T8} and ε_{T9} are the strain of transducers T8 and T9 respectively, (see figure 5.16 and 5.17).

Hence:

$$\gamma_{xy} = \varepsilon_{T9} - \varepsilon_{T8} \qquad [5.4]$$

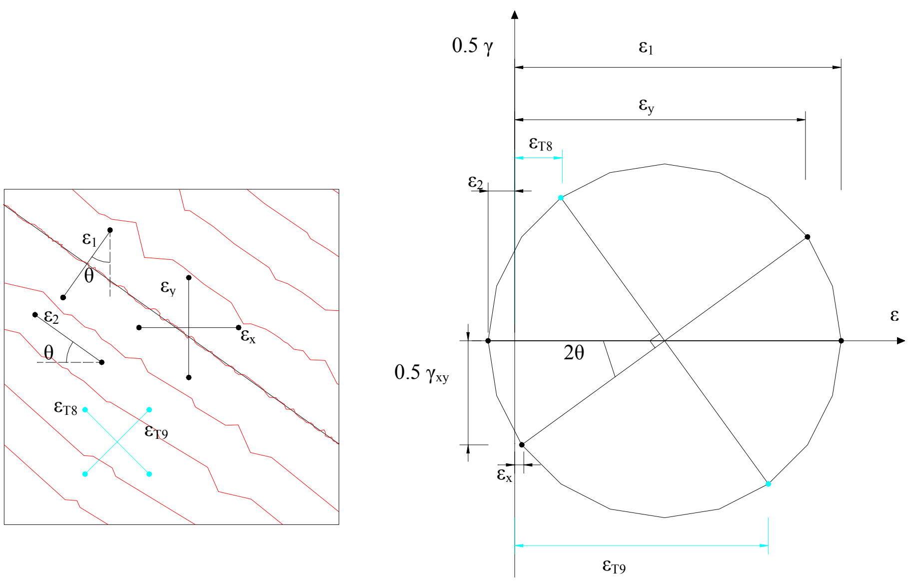


Fig.5.18: Calculation of the shear strain γ_{xy} from Mohr's circle

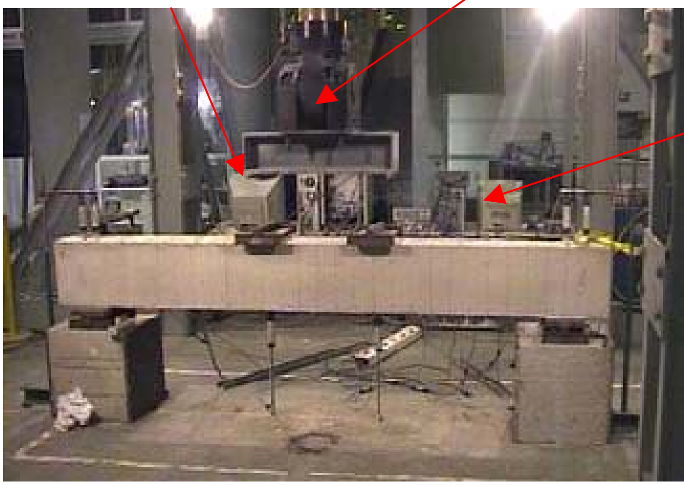
5.5 TECHNICAL EQUIPMENT. DATA ACQUISITION SYSTEM

The data acquisition system utilized was the HP-34970A with 40 analogical inputs. While magnetostrictive transducers were connected directly the strain gauges were first connected to a control module. The software for the data collection was the Data Logger. Readings were taken every 2 or 3 seconds.

5.6 TESTING PROCEDURE

5.6.1 Test configuration

Figure 5.19 and 5.20 show the actual test configuration. The load cell was connected to a steel beam (H300), which distributed the load at two points, each one being located 30 cm from the centre of the concrete beam. A 150 mm wide and 28 mm thick neoprene was placed under a spherical bearing.



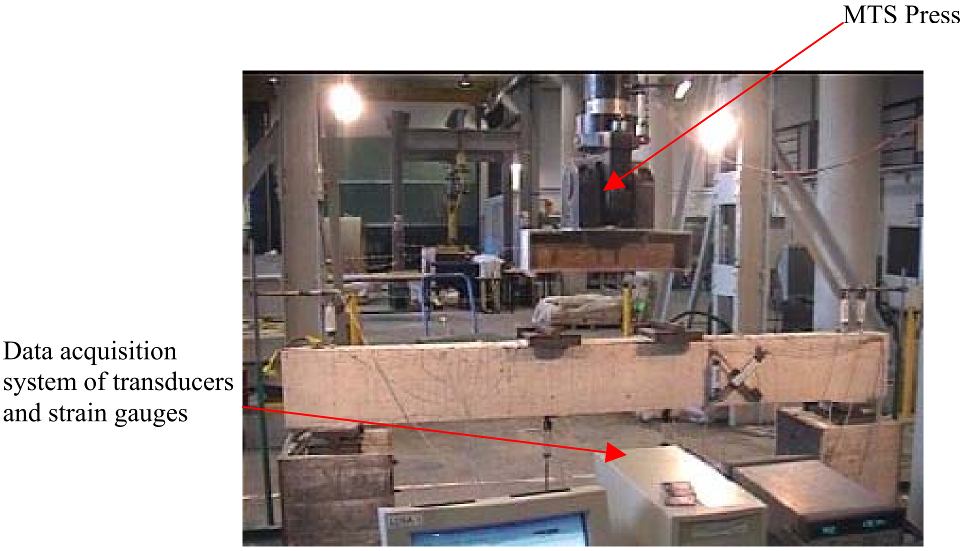


Fig.5.19: Test scheme.

Fig.5.20: Test configuration

All beam specimens were supported by Teflon © (very low friction material). The teflon © was placed between the pin bearing and beam, enabling the beams to move. The supports used were exactly the same, see figure 5.21.

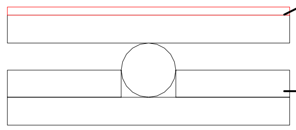


Fig. 5.21: Detail of bearing

5.6.2 Loading Procedure

The tests were carried out under displacement control using a closed-loop hydraulic MTS compression machine with a loading capacity of 4500 kN. The loading rate was slightly varied for each type of beam tested in accordance with the amount of transversal reinforcement used. This allowed for an approximate failure rate of 65 minutes. Table 5.7 shows the loading rate for each beam specimen, and the duration of the test. The tests were maintained once the flexural cracking load was reached. During the testing of the last 8 beams (four of HR50 and four of HR100) the crack development was closely followed to ascertain length of cracking, although crack widths were not controlled.

Table.5.7: Loading procedure and test duration

Beam specimen	**Loading rate mm/s**	**Test duration Min**
HC-1	0.0025	64
HC-2	0.005	74
HC-3	0.004	70
HC-4	0.004	72
HR25-1	0.0025	55
HR25-2	0.005	55
HR25-3	0.004	68
HR25-4	0.004	92

Beam specimen	**Loading rate mm/s**	**Test duration min**
HR50-1	0.0025	59
HR50-2	0.005	68
HR50-3	0.004	66
HR50-4	0.004	67
HR100-1	0.0025	70
HR100-2	0.005	65
HR100-3	0.004	65
HR100-4	0.004	70

Chapter 6

Analysis of structural test results

6.1 GENERAL CONSIDERATIONS

This chapter discusses the results obtained from sixteen beam specimens tested at the Structural Technology Laboratory of The *Universitat Politécnica de Catalunya.* The primary objective of the study was to investigate the behaviour of reinforced recycled aggregates concrete beams failing on shear. The full description of the beam specimens and testing procedure were presented in Chapter 5.

A large amount of data was collected during the experimental investigation. Each test has, usually, more than 15 channels of data acquired from the data acquisition system. Since data was collected every 2-3 seconds, thousands of data were stored for each beam specimen. For brevity, it was decided to review in Annex A each test in 3-6 pages in length.

Table 6.1 summarises the results of the 16 tests of beam specimens subjected to bending moment and shear. The table gives the main characteristics of each beam specimen, the failure shear strength, and the approximate cracking strength.

Table 6.1: Summary of experimental results. Details of beam specimens on failure side. Failure shear load ($V_{failure}$) and approximate cracking load (V_{cr})

Beam	fc MPa	f_{sp} MPa	b mm	d mm	a/d	Shear reinf. Stirrup/ space mm	Shear reinf. ρ_w MPa	Long. Reinforcement Longitudinal Reinforcement	Long. Reinforcement ρ_l	$V_{failure}$ kN	V_{cr} (kN) Aprox.
HC-1	41.91	2.64	200	309	3.32	-	0	2∅32+1∅16	2.92	100.5	96
HC-2	41.91	2.64	200	304	3.32	∅6/130	1.180	2∅32+1∅16	2.97	213	108
HC-3	41.91	2.64	200	304	3.32	∅6/170	0.903	2∅32+1∅16	2.97	177	98
HC-4	41.91	2.64	200	304	3.32	∅6/240	0.636	2∅32+1∅16	2.97	187.5	112.5
HR25-1	42.38	3.13	200	309	3.32	-	0	2∅32+1∅16	2.92	104	85
HR25-2	42.38	3.13	200	304	3.32	∅6/130	1.180	2∅32+1∅16	2.97	186.5	86
HR25-3	42.38	3.13	200	304	3.32	∅6/170	0.903	2∅32+1∅16	2.97	169	100,5
HR25-4	42.38	3.13	200	304	3.32	∅6/240	0.636	2∅32+1∅16	2.97	238	109
HR50-1	41.34	3.22	210	309	3.32	-	0	2∅32+1∅16	2.92	89	82
HR50-2	41.34	3.22	210	304	3.32	∅6/130	1.180	2∅32+1∅16	2.97	220	92,5
HR50-3	41.34	3.22	210	304	3.32	∅6/170	0.903	2∅32+1∅16	2.97	176	93.5
HR50-4	41.34	3.22	210	304	3.32	∅6/240	0.636	2∅32+1∅16	2.97	164	91
HR100-1	39.75	3.28	200	309	3.32	-	0	2∅32+1∅16	2.92	84	80
HR100-2	39.75	3.28	200	304	3.32	∅6/130	1.180	2∅32+1∅16	2.97	189.5	86
HR100-3	39.75	3.28	200	304	3.32	∅6/170	0.903	2∅32+1∅16	2.97	163	90
HR100-4	39.75	3.28	200	304	3.32	∅6/240	0.636	2∅32+1∅16	2.97	168	80

In annex A the experimental data is organised into four sections. The first page is the overall summary page and it describes the details of the specimen, material properties, and reinforcement ratios. There is also a brief paragraph with the summary of test observations.

The summary table lists the significant parameters at selected data sets. In some cases the data sent by strain gauges was unreliable. The date from these strains gauges have been omitted in the table. The third section shows the plots of the previous data.

The fourth section depicts the cracking pattern at different load stages (when it was analysed) with two or three pictures.

All the tests were carried out according to previously determined procedures except for beam type 4 (V4) due to the change of position of the load points and the support point, probably an arch effect was produced.

6.2 ANALYSIS OF BEAMS' FAILURE. MODE OF FAILURE

All beam specimens failed in shear. However, in beam specimens HC-4, HR25-4 and HR100-4 the shear cracks did not cross the compression zone of the beam on the north side.

As discussed in chapter 2, the mode of failure of beam specimens without stirrups is different to the collapse of beams with shear reinforcement. Beams HC-1, HR25-1, HR50-1 and HR100-1 suddenly failed resulting in the appearance of a single shear crack. The shear strength decreased as the percentage of recycled aggregates increased except for HR25-1 which maintained a shear strength of HC-1. Nevertheless the failure mode was similar in all of them. Section 6.3 compares the experimental results for these beam specimens.

Pictures of figure 6.1 show the failure of HC-1 and HR100-1.

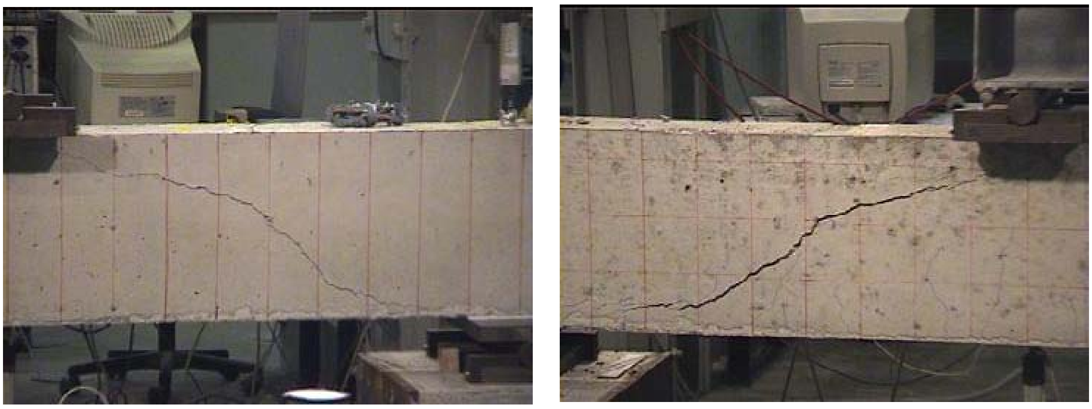


Fig. 6.1: Shear failure of HC-1 and HR100-1

Failure in HC-1 and HR25-1 resulted from a splitting crack, and in these cases the compression area was not cracked. However, in HR50-1 and HR100-1 the failure took place at the maximum shear stress by a splitting crack and crushing of compressed concrete at the top due to a combination of compression and shear stresses. One crack appeared in all beams, except in HR50-1 where 2 cracks appeared before the beams' failure.

In contrast, beams containing stirrups (section 6.4) presented a more ductile response. After the formation of the first shear crack, stirrups started working and further shear cracks developed. At failure, the compressed top part of the beam crushed under the combination of compression and shear stresses, there were no large differences between the behaviour of beams produced from different types of concrete.

The typical failure mode of beams V1, V2 and V3 are illustrated in figure 6.2.

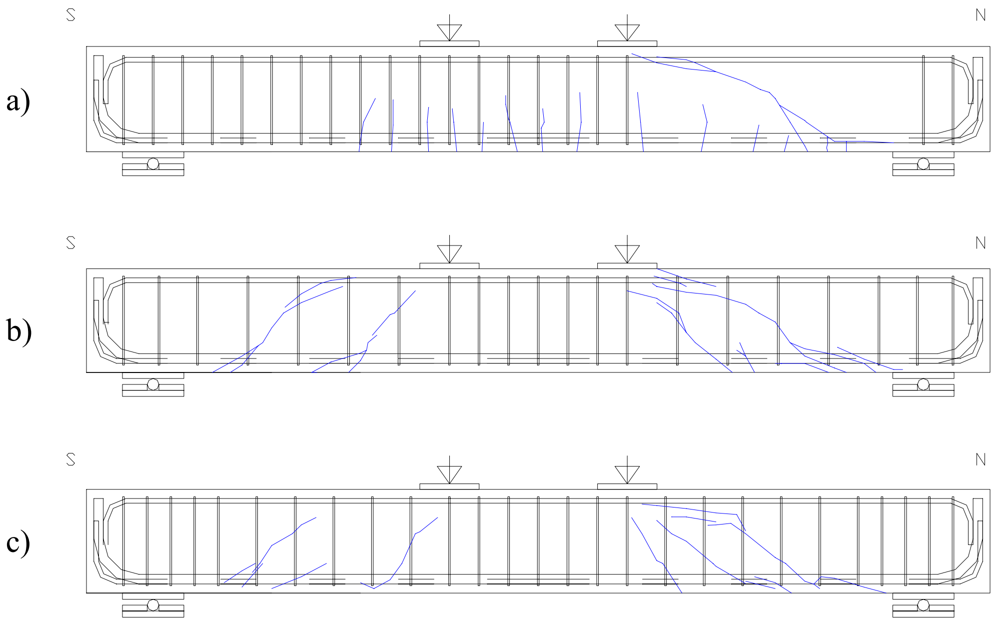


Fig.6.2: Failure of three different reinforcements. a) Beam type V1 b) Beam type V3 and c) Beam type V2.

In beams specimen type V4 (HC-4, HR25-4, HR50-4 and HR100-4), as described in the previous chapter, the modification of the space between load points was carried out (10 cm smaller than the rest of the beams). It led to the changing of one support to maintain the same a/d ratio as the other beams. Therefore on the north side (where the collapse occurred), the beam had a 10 cm longer anchorage. This action resulted in the shear cracks not always crossing the compression zone of the beam in the north side. In some cases the top compressed south zone is crushed, as illustrated in figure 6.3.

Although the reinforcement between the load and support points of the beam is the same in all cases, the failure mode is rather different. In HC-4 and HR50-4 the failure occurred in the north side but in HR25-4 and HR100-4 the failure took place in the south side. The collapse did not occur by a splitting crack as in the other beams, so in beams V4 (HC-4, HR25-4, HR50-4 and HR100-4) the ultimate load was much larger than expected at the beginning for beams with these amounts of reinforcement, probably due to arch effect.

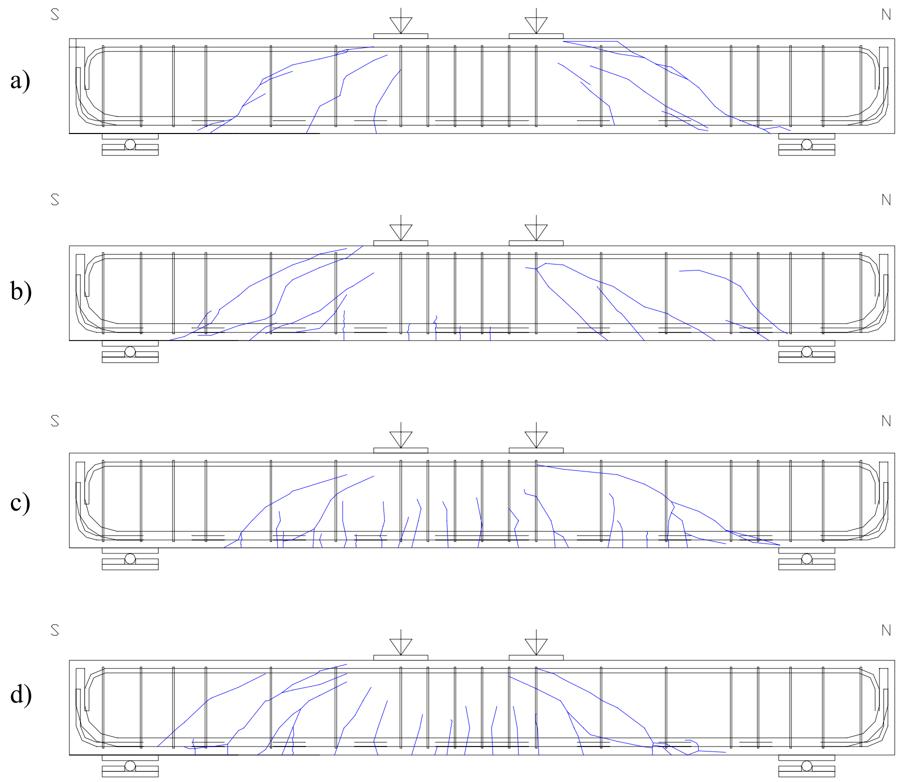


Fig.6.3: Failure of the type V4 beams. a) HC-4, b) HR25-4, c) HR50-4 and d) HR100-4

6.3 BEAM SPECIMENS WITHOUT WEB REINFORCEMENT

Specimens HC-1, HR25-1, HR50-1 and HR100-1 did not contain shear reinforcement in the north side. The only parameter which varied for all these beams was the concrete mix. Longitudinal reinforcement was constant and equal to 2.92%. Failure shear strength was, respectively, 1.62 MPa, 1.68 MPa, 1.37 MPa and 1.36 MPa. In figure 6.4 the shear force- displacement of each beam is plotted. The HR25-1, HR50-1 and HR100-1 recovered the cracked load. This is a result of the cell load being controlled by displacement. As commented before, the failure in HC-1 and HR25-1 occurred by a splitting crack, and in these cases the compression area is not cracked. However, in HR50-1 and HR100-1 the failure occurred at the maximum shear stress by a splitting crack and the crushing of the compressed top part under the combination of compression and shear stresses.

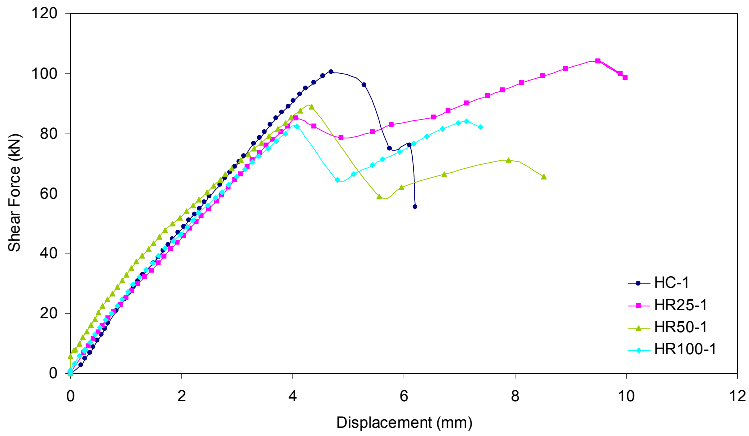


Fig.6.4: Load-deflection response for type V1 beams

Only one crack appeared in all the beams tested, except for HR50-1 where 2 cracks appeared before the beams' failure.

The ratio of each beam specimen's shear strength with the conventional concrete shear strength (HC-1) is shown in figure 6.5. The shear stress decreased when the percentage of recycled aggregate was higher than 25%. A serious decrease of strength was produced when 50% and 100% of recycled aggregates were used.

HC-1 and HR25-1 had the same shear strength, and the longitudinal reinforcement strain was very similar. HR50-1 and HR100 had the same shear strength and the longitudinal reinforcement strain also was similar, as illustrated in annex A.

Test Residual tensile factor ($\beta_{test} = Vu_{test} /(b * d * \sqrt{fcd})$) decreased in recycled aggregate concretes with percentages of recycled aggregates higher than 25%. The test residual tensile factor of HC-1, HR25-1, HR50-1 and HR100-1 was 0.25, 0.26, 0.21 and 0.21, respectively.

The strain of the longitudinal web reinforcement in the failure side of the beam specimens made with conventional concrete (HC) was similar to that of the beam specimens employing recycled aggregates. The bars' strain depended on the load.

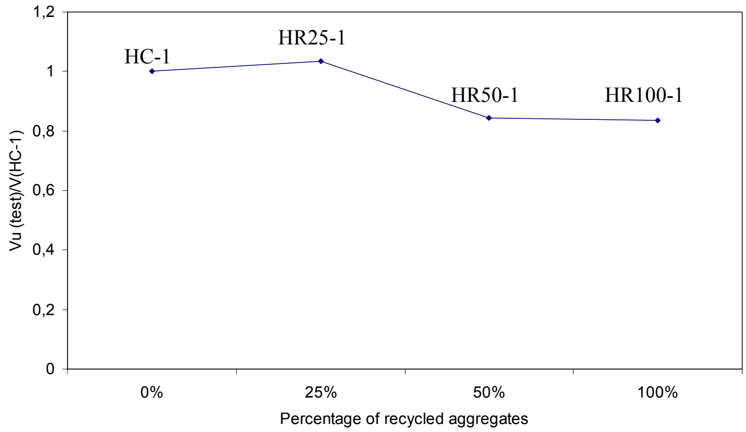


Fig.6.5: Beam specimens without web reinforcement. Influence of the recycled aggregate percentage in shear strength.

With respect to the south side, which did not collapse, the strain in stirrups placed in beams made with a concrete containing a higher amount of recycled aggregate is larger. Although the shear strength was lower, shear cracks occurred and the gauges could detect these see figure 6.6.

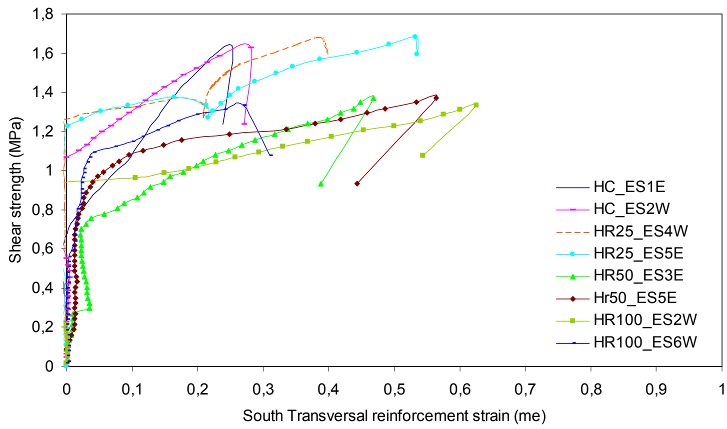


Fig.6.6: Strain of South transversal stirrups- Shear strength of different concretes

6.4 BEAM SPECIMENS WITH STIRRUPS

In the previous chapter three kinds of beam stirrup distribution are described. Beam specimens V4 (HC-4, HR25-4, HR50-4 and HR100-4), were designed with less stirrups than required by the EHE. As explained earlier, these beam specimens behaved differently to the other specimens tested.

Beam specimens V3 (HC-3, HR25-3, HR50-3 and HR100-3) were reinforced with the minimum amount required by EHE, and beam specimens V2 (HC-2, HR25-2. HR50-3 and HR100-3) had more reinforcement than the V3 specimens.

Minimum reinforcement in accordance with EHE

The minimum amount of shear reinforcement was provided not only to ensure that the beams did not collapse just after the formation of the first shear crack but also to control the diagonal cracks at service load levels. The minimum amount of shear reinforcement provided by the EHE formulation is proportional to the concrete's compressive strength ($\sum \frac{A_{\alpha} f_{y\alpha,d}}{\mathrm{sen}\alpha} \geq 0{,}02 f_{cd} b_o$). Most Codes suggest that the inclined cracking load of beams increase proportionally to the tensile strength of the concrete. This assumption, in accordance with these tests, is not real in the case of recycled aggregate concretes. In the case of recycled aggregate concretes the tensile strength can be higher than conventional ones, as occurred in the concretes described in this thesis. However as shown in the previous section, the inclined cracking load and the collapse load decreased with respect to the conventional ones. The diagonal cracking was produced later in the beam specimen employing the higher tensile strength conventional concrete and it required more stirrups to absorb the load. This was not the case with regards to the beam specimens employing recycled aggregates.

Beam specimens HC-3, HR25-3, HR50-3 and HR100-3 were provided with the minimum amount of web reinforcement proposed by EHE. Table 6.2. summarises the provided web reinforcement.

The value of the measured shear at stirrup yielding, V_y in table 6.2, was taken as the shear strength when the second stirrup crossing the crack yielded. Table 6.2. describes the behaviour of these beams. In terms of load-displacement, figure 6.7 shows the high ductility of the beam specimen (employing the minimum reinforcement required by EHE) after cracking.

Table 6.2: Minimum amount of web reinforcement according to EHE for each specimen, and observed failure, yielding and cracking shear

Beam	fc Mpa	Provided Shear reinf. (EHEmin)		Collapse side (north side ϕ6)							ϕ8
		Stirrup/ Spacing Mm	ρ_w # Mpa	$V_{failure}$ (kN)	V_y (kN)	V^*_{cr} (kN)	V_y / V_{cr}	V_{fail} / V_{cr}	V_{serv} EHE	V_{serv} CSA	V^*_{cr} (kN)
HC-3	41.91	∅6/170	0.903	177	157.5	98	1.60	1.81	54.8	84.5	108,5
HR25-3	42.38	∅6/170	0.903	169	140.5	100,5	1.40	1.68	54.9	84.5	97
HR50-3	41.34	∅6/170	0.903	176	163.5	93.5	1.74	1.88	54.7	84.5	86,5
HR100-3	39.75	∅6/170	0.903	163	152.5	90	1,69	1.81	54.4	84.5	85

 # Calculated using the real yielding stress of the stirrups
** Approximate cracking shear force*

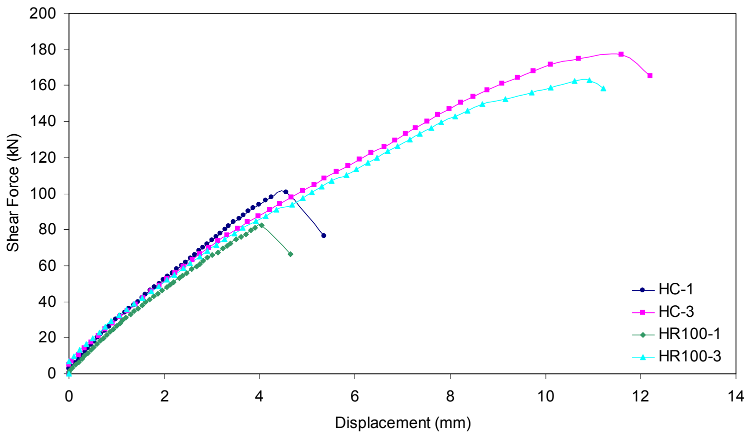


Fig. 6.7: Displacement in beams without shear reinforcement and with the minimum amount in accordance with EHE

The shear force at service loads is estimated as the failure shear strength divided by 1.80, to take into account the load (1.50) and material partial safety factors, and it is given in the table 6.2 for EHE and CSA (2004) failure shear strength.

According to an analysis of both codes, there was no diagonal cracking for the service load, consequently it is evident that the minimum reinforcement employed did not have to control crack widths.

6.4.1 Influence of recycled aggregates quantity

As previously illustrated, different percentages of recycled aggregates were used in concrete beam specimens using the same quantity of stirrups. The minimum amount of web reinforcement was used in the V3 beam specimens. As explained the four V3 type beam specimens were produced using a different concrete mix for each beam: HC-3, HR25-3, HR50-3 and HR100-3.

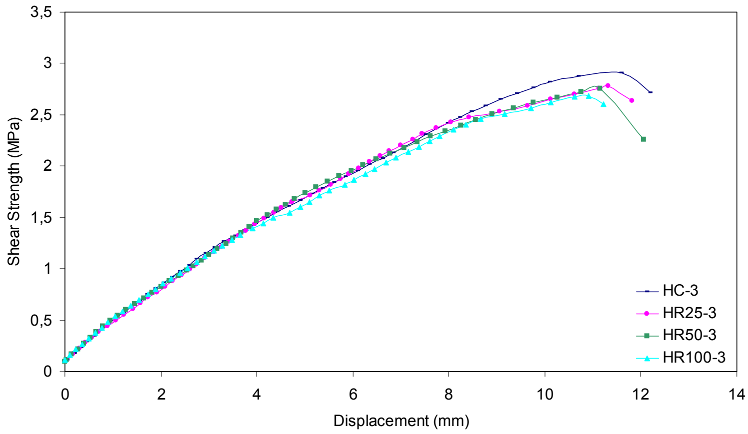


Fig.6.8: Shear Strength- Displacement of V3 beam specimens with different percentages of recycled aggregate in concrete

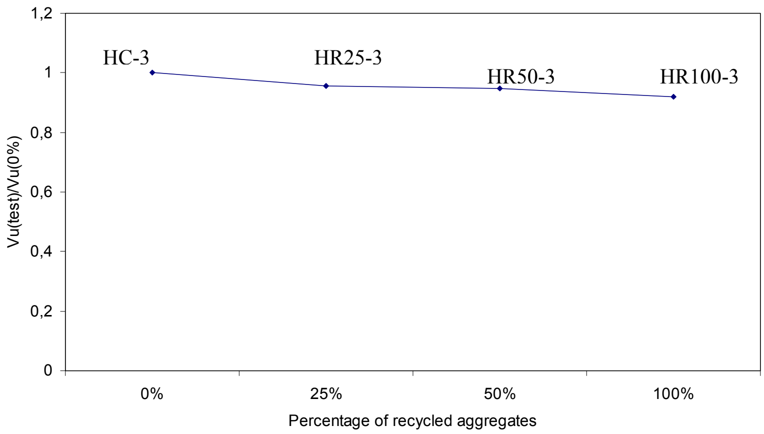


Fig.6.9: Beam specimens with minimum reinforcement required by EHE. Influence of the recycled aggregate percentage in shear strength

In Figure 6.8 it can be seen that the displacement-Shear Strength curve is very similar in all of the concretes. A larger displacement existed when the shear strength was greater, there was practically no differences amongst beams made with different percentages of recycled aggregate. As shown in figure 6.9 the shear strength decreased when the percentage of aggregate was increased, although only a 10% decrease was observed.

The behaviour of these four different beam specimens is summarised in table 6.2. The cracking load for beam specimens with recycled aggregates was lower, however the failure load was very similar to conventional and recycled aggregate concrete. The quick yielding of stirrups in HR25-3 (after the occurrence of diagonal cracks) had to be taken into account as the crack aggregate interlock could have been very low or had inadequate bond thus producing a "brittle failure". It is interesting to mention that in HR25-3, three stirrups on the failure side yielded.

As summarised in table 6.3, the beam specimens V2 (HC-2, HR25-2, HR50-2 and HR100-2) were reinforced with more quantity of reinforcement than required by EHE. Figure 6.10 illustrates the shear Strength- displacement of beam V2. The behaviour of beams was as expected, except in the case of HR25-2 which achieved less load than any of the other beam specimens tested. In this case the yield load of HR25-2 was also lower than any of the other concrete in the beam specimens tested.

Table 6.3: Provided reinforcement for each specimen, and observed failure, yielding and cracking shear

Beam	fc	Provided Shear reinf.		Collapse side (north side ϕ6)					ϕ8
	Mpa	Stirrup/spacing mm	ρ_w # Mpa	$V_{failure}$ (kN)	V_y (kN)	V^*_{cr} (kN)	V_y / V_{cr}	V_{fail} / V_{cr}	V^*_{cr} (kN)
HC-2	41.91	∅6/130	1.180	213	Error	108	Error	1.86	Error
HR25-2	42.38	∅6/130	1.180	186.5	146	86	1.70	2.16	100
HR50-2	41.34	∅6/130	1.180	220	176	92,5	1,90	2.38	83
HR100-2	39.75	∅6/130	1.180	189.5	165	86	1.92	2.20	86

 # Calculated using the real yielding stress of the stirrups
** Approximate cracking shear force*

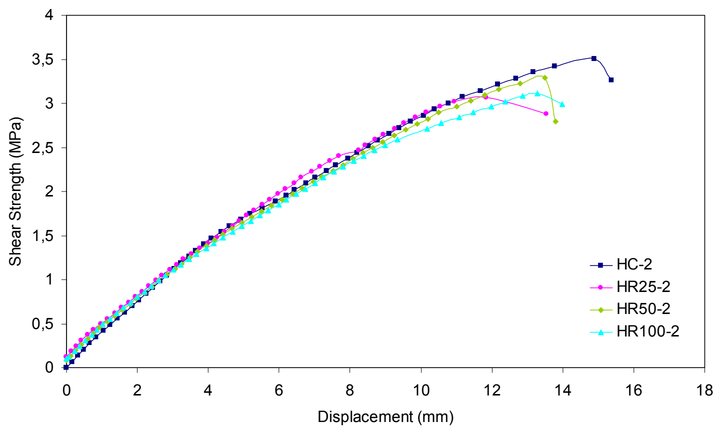


Fig.6.10: Shear Strength- Displacement of V2 beam specimens with different percentages of recycled aggregates in concrete

Figure 6.11 compares the shear strength ratio of beams made with recycled aggregate concrete to those made with conventional concrete. In the case of beam specimen HR25-2 employing 1.180 MPa of web reinforcement (a higher amount of web reinforcement than required by EHE) the ultimate shear force was 13% lower than that of conventional concrete, as was the yield load. The yield load was probably low due to the HR25 concrete producing more brittle failure.

For concrete beam specimens with 1.80 MPa of web and 50 and 100% of recycled aggregates (HR50-2 and HR100-2), the diagonal cracking load was lower than in conventional concrete (HC-2). In these concretes, probably the aggregate interlock and concrete-stirrups bond was effective. The V_{fail}/V_{cr} was higher in HR50-2 and HR100-2 beam specimens than in the conventional concrete beams (HC-2).

All of the stirrups in the HR100-2 beam specimen and three of stirrups in the HR25-2 beam specimen yielded. In this case the HR25-2 beam specimen had less strength (87%) with respect to conventional concrete.

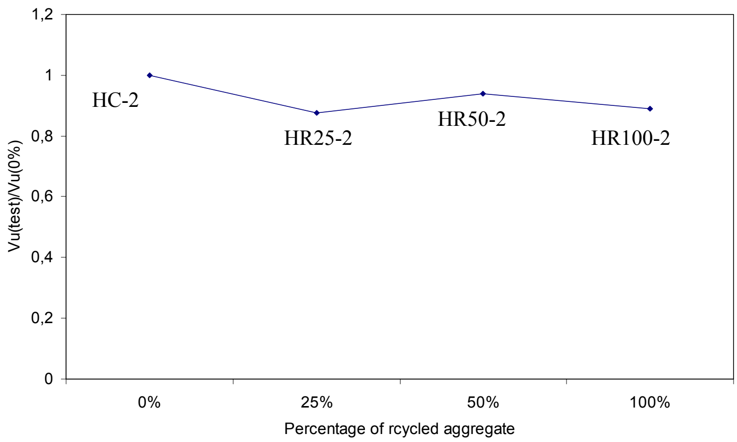


Fig.6.11: Beam specimens with more reinforcement (1.80MPa) than minimum required. Influence of the recycled aggregate percentage in shear strength

6.4.2 Influence of the amount of shear reinforcement

The amount of shear reinforcement was another variable during the test design. In this section this influence is going to be analysed separately for each different concrete mix.

Beam specimens HC-1, HC-3 and HC-2

Table 6.1 indicates that beam HC-1 did not contain stirrups. Beam specimens HC-3 and HC-2 had an amount of web reinforcement equal to 0,903 and 1,180 MPa, respectively. The shear force failure of series HC is represented in figure 6.12.

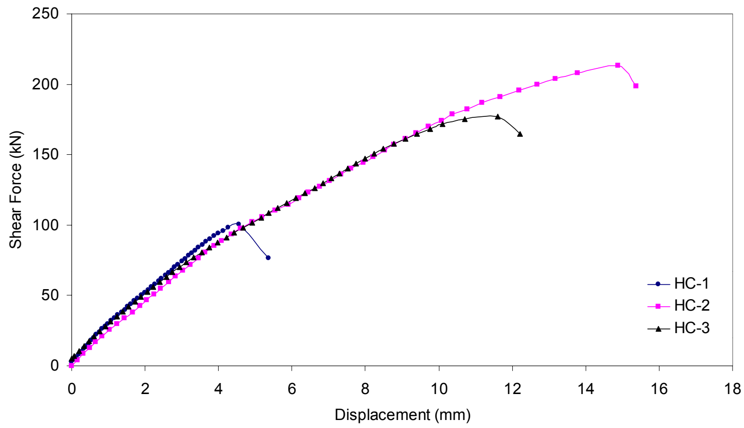


Fig.6.12: Influence of the amount of shear reinforcement. Displacement of beams HC-1, HC-2 and HC-3

The failure shear strength was 1,65, 2,91 and 3,50 MPa respectively. A trend line and its equation are represented in figure 6.13 by a dashed red line in the plot. This trend line will be compared with the other trend lines later detailed at the end of the chapter.

The web shear strain of the beams HC is plotted in figure 6.13. The addition of web reinforcement improved the shear response of the specimen by increasing the failure shear strength and ductility. The cracking pattern also changed. In beam HC-1 a single shear crack was noted, meanwhile two shear cracks were noticed in beam HC-3 and three shear cracks on beam specimen HC-2 (see figure 6.14).

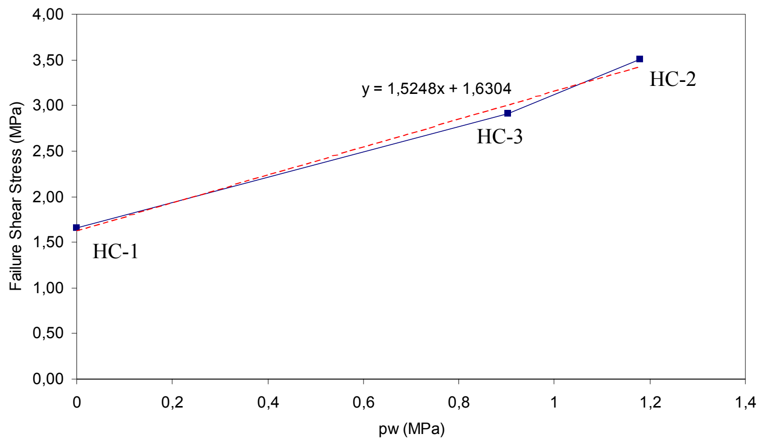


Fig.6.13: Influence of the amount of shear reinforcement. Failure shear stress of beams HC-1, HC-2 and HC-3

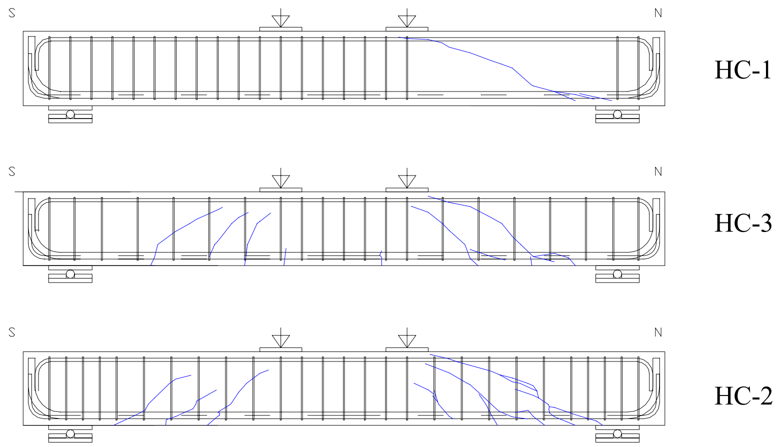


Fig.6.14: Influence of the amount of shear reinforcement. Crack pattern at failure for beams HC-1, HC-2 and HC-3.

An error occurred in the detection of strain in the transversal reinforcement in beam HC-2. Therefore it was not possible to compare their results with those of HC-3.

Beam specimens HR25-1, HR25-3 and HR25-2

Table 6.1 indicates that beam HR25-1 did not contain stirrups. Beam specimens HR25-3 and HR25-2 had an amount of web reinforcement equal to 0.903 and 1.180 MPa, respectively. The shear force failure of series HR25 is represented in Figure 6.15. The shear failure strength was 1.43, 2.78 and 3.07 MPa respectively.

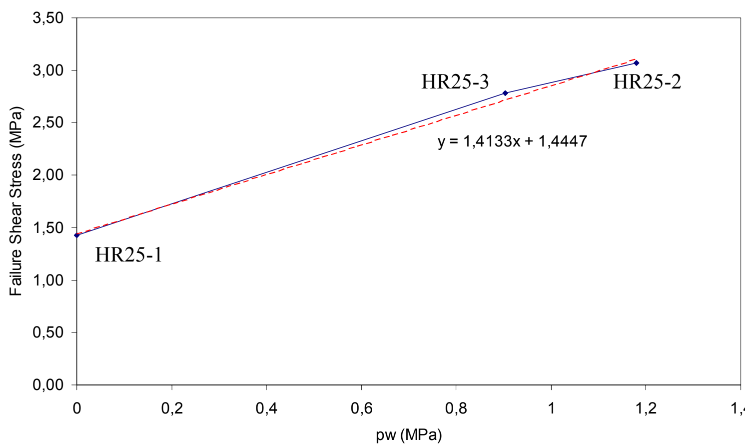


Fig.6.15: Influence of the amount of shear reinforcement. Shear Stress Failure of beams HR25-1, HR25-2 and HR25-3

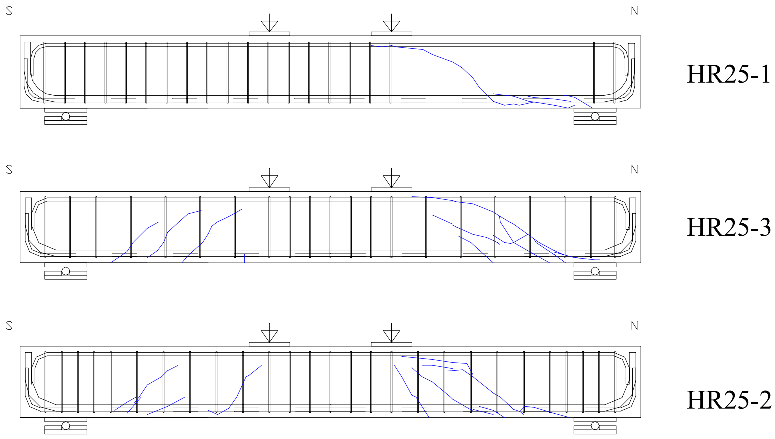


Fig.6.16. Influence of the amount of shear reinforcement. Crack pattern at failure for beams HR25-1, HR25-2 and HR25-3.

The trend line in figure 6.15 is flatter than the type HC beam specimens (fig. 6.13). The lines define that the shear reinforcement seemed less effective in the HR25 concrete than conventional concrete. Probably it was due to the fact that the 11,1% of the volume of the concrete was recycled aggregate. Three stirrups yielded in both beams.

Crack patterns at failure are presented in figure 6.16. The same performance as in HC beam specimens was reported.

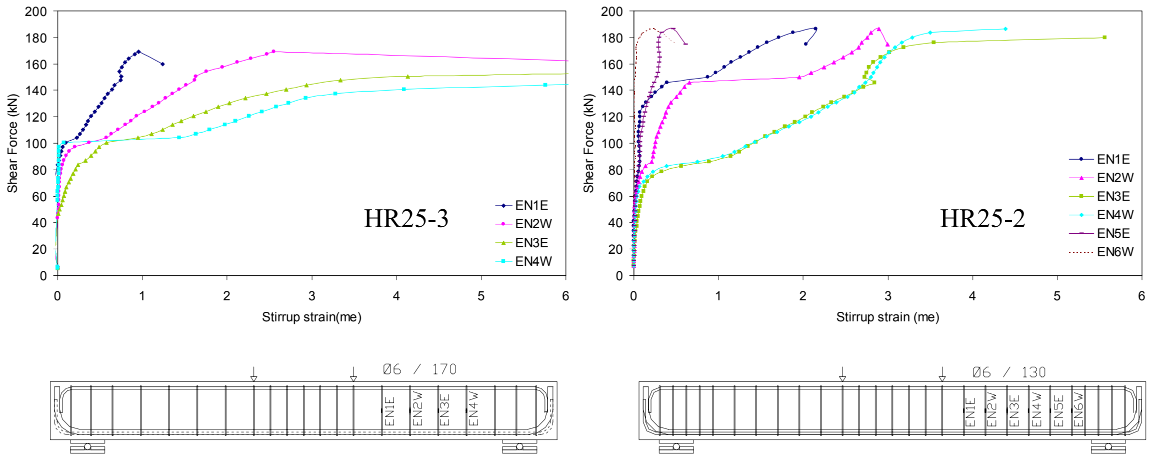


Fig.6.17. Influence of the amount of shear reinforcement. Stirrup strains for beam HR25-3 and HR25-2

Figure 6.17 compares the stirrup strains in beams HR25-3 and HR25-2. In both beams the first crack started in the middle of the span.

Beam specimens HR50-1, HR50-3 and HR50-2

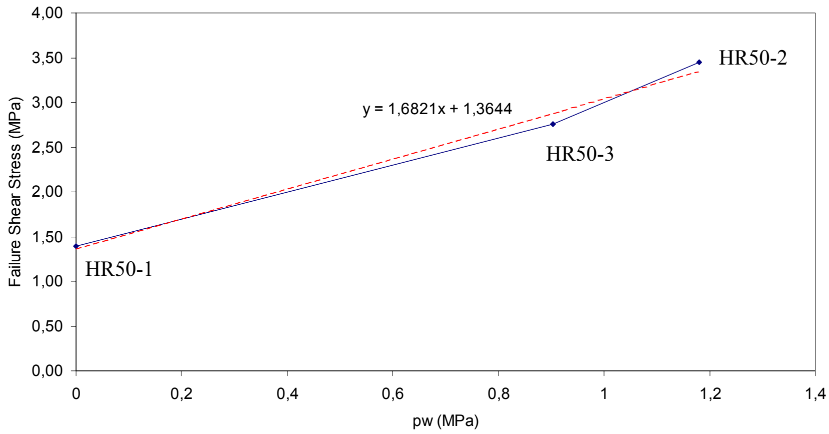


Fig.6.18: Influence of the amount of shear reinforcement. Shear stress failure of beams HR50-1, HR50-3 and HR50-2.

Beam specimens HR50-1, HR50-3 and HR50-2 had an amount of web reinforcement equal to that of 0, 0.903 and 1.180 MPa, respectively. The reported shear strength failure was 1.46, 2.89 and 3.62 MPa (see figure 6.18). Although the recycled aggregate percentage was higher, the cement paste was stronger producing a steeper trend line. The trend line was steeper than any other case previously studied.

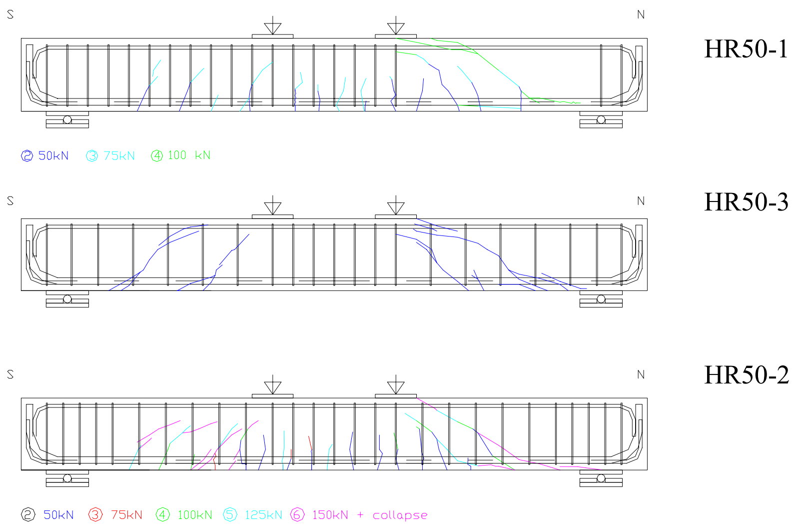


Fig.6.19: Influence of the amount of shear reinforcement. Crack pattern at failure for beams HR50-1, HR50-3 and HR50-2

When HR50 beam specimens were reinforced with stirrups the same performance as HC or HR25 occurred. However the cracking at the failure side was larger in beam specimen without stirrups, see figure 6.19. In this case the diagonal crack also started in the middle of the beam's span.

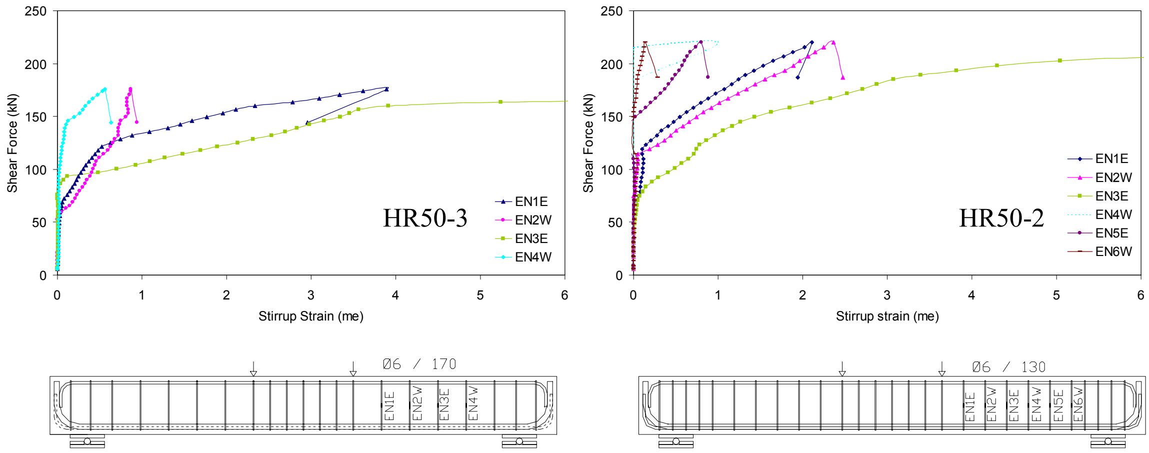


Fig.6.20: Influence of the amount of shear reinforcement. Stirrup strains for beam HR50-3 and HR50-2

Beam specimens HR100-1, HR100-3 and HR100-2

Beam specimens HR100-1, HR100-3 and HR100-2 had an amount of web reinforcement equal to that of 0, 0.903 and 1.180 MPa respectively. Figure 6.21 plots the shear strength failure versus the amount of web reinforcement. The shear strength at collapse was 1.36, 2.68 and 3.12 MPa, respectively. The trend line was flatter than HR50 and HC.

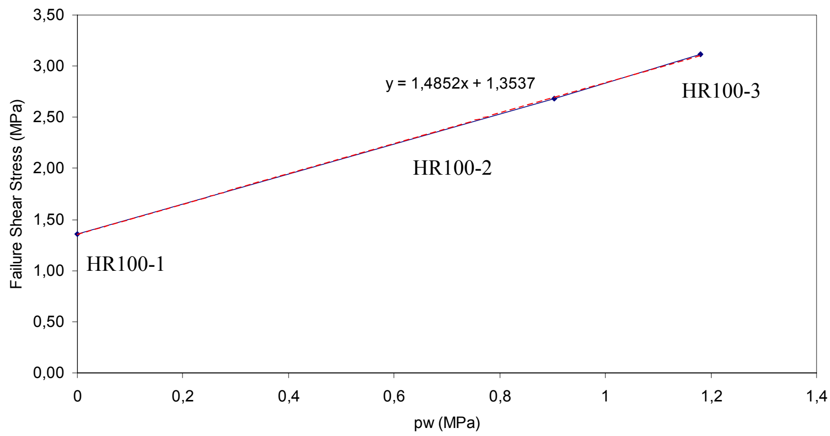


Fig.6.21: Influence of the amount of shear reinforcement. Shear stress failure of beams HR100-1, HR100-3 and HR100-2.

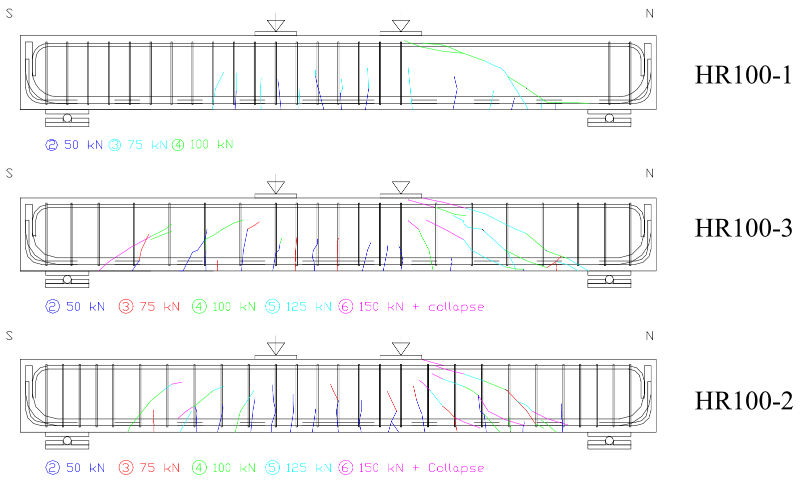


Fig.6.22: Influence of the amount of shear reinforcement. Crack pattern at failure for beams HR100-1, HR100-3 and HR100-2

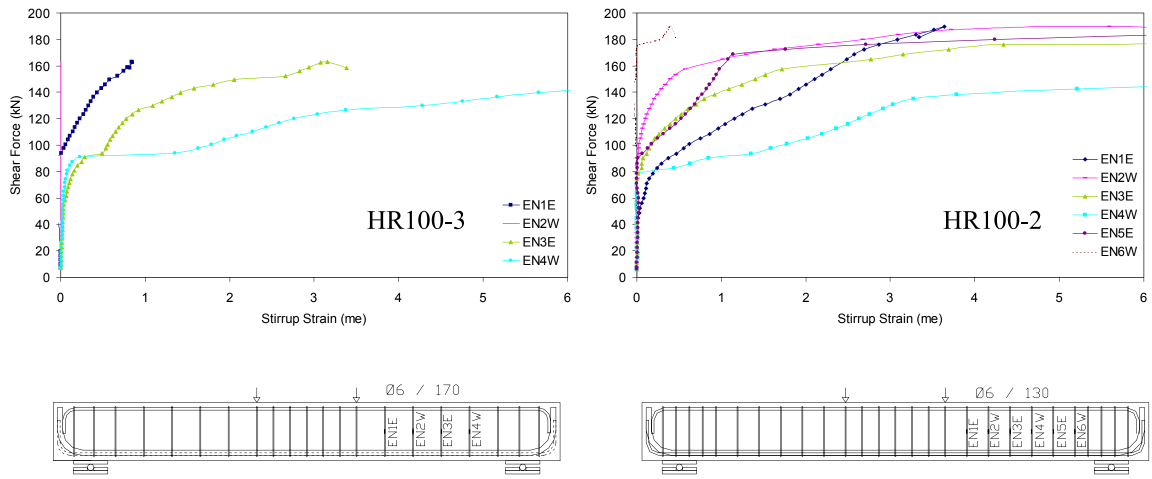


Fig.6.23: Influence of the amount of shear reinforcement. Stirrup strains for HR100-3 and HR100-2

In HR100-2 all the stirrups yielded to stress except the farthest stirrup from the load point (EN6W, see figure 6.23).

A summary of the influence of transversal reinforcement in all kinds of concretes is illustrated in figure 6.24. The influence of web reinforcement in all kind of concrete was very similar. Concrete HR25 was the less influenced.

In stirrups placed in the south side of the beam specimen, the side where the collapse did not occur, the strain of the transversal reinforcement was similar in all concretes. In annex A all the strains are presented.

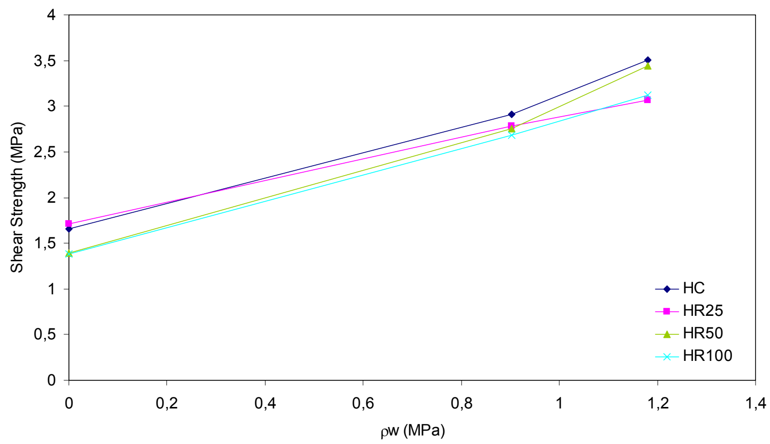


Fig.6.24: Influence of different amount of transversal reinforcement in all kind of concretes

6.5 COMPARISON OF TEST RESULTS WITH DIFFERENT APPROACHES

Tables 6.4 and 6.5, and figure 6.25 summarise the predictions according to the use of EHE Code, Eurocode-2 final draft 2002, AASHTO LRFD, CSA (2004), Response-2000 (Bentz, 2000) a computer program based on the modified compression field theory and the method proposed by Cladera. In Chapter 2, section 2.5.3 presents the code procedures for members without and with web reinforcement. The calculations by EC2 were considered cotθ equal to 2,5, so the Vu value was the maximum possible.

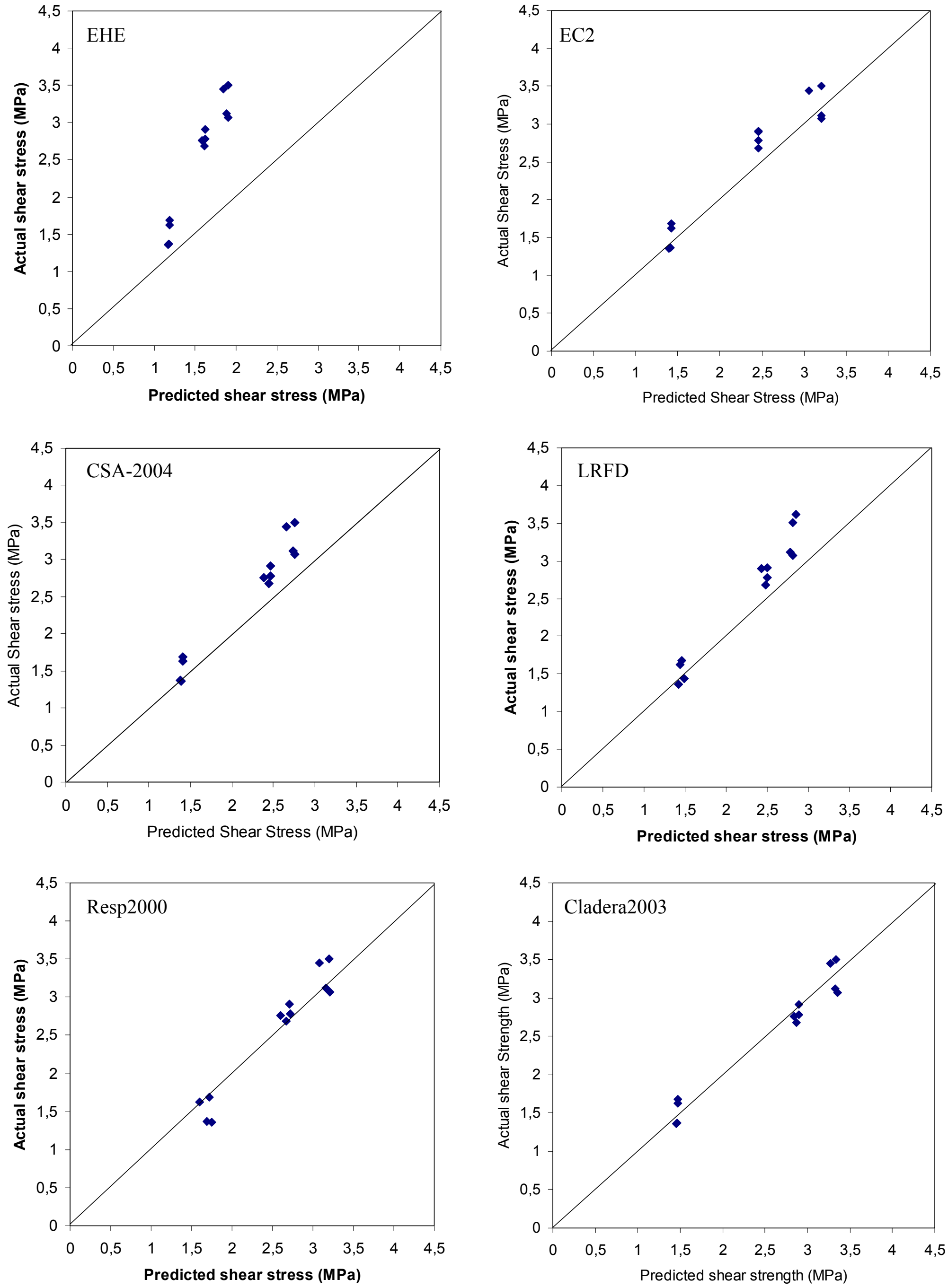


Fig.6.25: Summary of predictions by EHE-99, Eurocode 2, AASHTO LRFD, CSA (2004), Response-2000 program and proposed method by Cladera (2003)

Table 6.4: Summary of predictions according to concrete type by EHE-99, Eurocode 2, AASHTO LRFD, CSA (2004), Response-2000 program and proposed method by Cladera (march 2003)

Beam	fc (MPa)	b (mm)	d (mm)	A/d	ρw	ρ1	Vfal (kN)	Vpredicted							Vtest/Vpredicted						
								EHE	EC	LRFD	CSA	RESP	CLAD spl	CLAD	EHE	EC	LRFD	CSA	RESP	CLAD spl	CLAD
HC-1	41,9	200	350	3,23	0	2,92	100,5	73,2	87,84	89,0	87,1	98,6	90,8	90,8	1,37	1,14	1,13	1,15	1,02	1,11	1,11
HC-2	41,9	200	350	3,23	1,18	2,97	213,0	115,5	195,2	171,0	167,5	194,5	203,0	210,9	1,84	1,09	1,25	1,27	1,10	1,05	1,01
HC-3	41,9	200	350	3,23	0,903	2,97	177,0	98,7	149,3	152,0	150,1	164,8	176,0	185,4	1,79	1,19	1,16	1,18	1,07	1,01	0,95
Average HC															**1,67**	**1,14**	**1,18**	**1,20**	**1,06**	**1,05**	**1,02**
HR25-1	42,4	200	350	3,30	0	2,92	104,0	73,5	88,17	90,0	87,3	106,0	90,9	90,9	1,41	1,18	1,16	1,19	0,98	1,14	1,14
HR25-2	42,4	200	350	3,30	1,18	2,97	186,5	115,7	195,2	171,0	168,0	195,2	204,2	211,1	1,61	0,96	1,09	1,11	0,96	0,91	0,88
HR25-3	42,4	200	350	3,30	0,903	2,97	169,0	98,9	149,3	152,0	150,2	165,3	176,4	185,7	1,71	1,13	1,11	1,13	1,02	0,96	0,91
Average HR25															**1,58**	**1,09**	**1,12**	**1,14**	**0,99**	**1,01**	**0,98**
HR50-1	41,3	210	350	3,30	0	2,92	89,0	76,5	91,8	92,0	89,4	109,5	95,0	95,0	1,16	0,97	0,97	1,00	0,81	0,94	0,94
HR50-2	41,3	210	350	3,30	1,18	2,97	220,0	117,7	195,2	173,0	169,8	196,5	209,0	215,5	1,87	1,13	1,27	1,30	1,12	1,05	1,02
HR50-3	41,3	210	350	3,30	0,903	2,97	176,0	100,9	149,3	155,0	152,3	166,2	181,3	189,9	1,74	1,18	1,14	1,16	1,06	0,97	0,93
Average HR50															**1,59**	**1,09**	**1,12**	**1,15**	**1,00**	**0,99**	**0,96**
HR100-1	39,8	200	350	3,30	0	2,92	84,0	71,9	86,3	88,0	85,6	108,0	89,8	89,8	1,17	0,97	0,95	0,98	0,78	0,94	0,94
HR100-2	39,8	200	350	3,30	1,18	2,97	189,5	114,7	195,2	169,0	166,2	192,0	202,0	209,2	1,65	0,97	1,12	1,14	0,99	0,94	0,91
HR100-3	39,8	200	350	3,30	0,903	2,97	163,0	97,8	149,3	151,0	148,5	162,2	174,5	183,9	1,67	1,09	1,08	1,10	1,00	0,93	0,89
Average HR100															**1,50**	**1,01**	**1,05**	**1,07**	**0,92**	**0,94**	**0,91**
Average															***1,58***	***1,08***	***1,12***	***1,14***	***0,99***	***1,00***	***0,97***
Stand.Deviation															***0,25***	***0,09***	***0,09***	***0,09***	***0,10***	***0,08***	***0,09***
COV															***15,6***	***8,4***	***8,33***	***8,15***	***10,5***	***7,64***	***8,79***

Table 6.5: Summary of predictions according to web reinforcement by EHE-99, Eurocode 2, AASHTO LRFD, CSA (2004), Response-2000 program and proposed method by Cladera (march 2003)

Beam	fc (MPa)	b (mm)	d (mm)	a/d	ρw	ρ1	Vfal (kN)	Vpredicted							Vtest/Vpredicted						
								EHE	EC	LRFD	CSA	RESP	CLAD spl	CLAD	EHE	EC	LRFD	CSA	RESP	CLAD spl	CLAD
HC-1	41,9	200	350	3,23	0	2,92	100,5	73,2	87,84	89,0	87,1	98,6	90,8	90,8	1,37	1,14	1,13	1,15	1,02	1,11	1,11
HR25-1	42,4	200	350	3,30	0	2,92	104,0	73,5	88,17	90,0	87,3	106,0	90,9	90,9	1,41	1,18	1,16	1,19	0,98	1,14	1,14
HR50-1	41,3	210	350	3,30	0	2,92	89,0	76,5	91,8	92,0	89,4	109,5	95,0	95,0	1,16	0,97	0,97	1,00	0,81	0,94	0,94
HR100-1	39,8	200	350	3,30	0	2,92	84,0	71,9	86,3	88,0	85,6	108,0	89,8	89,8	1,17	0,97	0,95	0,98	0,78	0,94	0,94
														Average HC	***1,28***	***1,07***	***1,05***	***1,08***	***0,90***	***1,03***	***1,03***
HC-3	41,9	200	350	3,23	0,903	2,97	177,0	98,7	149,3	152,0	150,1	164,8	176,0	185,4	1,79	1,19	1,16	1,18	1,07	1,01	0,95
HR25-3	42,4	200	350	3,30	0,903	2,97	169,0	98,9	149,3	152,0	150,2	165,3	176,4	185,7	1,71	1,13	1,11	1,13	1,02	0,96	0,91
HR50-3	41,3	210	350	3,30	0,903	2,97	176,0	100,9	149,3	155,0	152,3	166,2	181,3	189,9	1,74	1,18	1,14	1,16	1,06	0,97	0,93
HR100-3	39,8	200	350	3,30	0,903	2,97	163,0	97,8	149,3	151,0	148,5	162,2	174,5	183,9	1,67	1,09	1,08	1,10	1,00	0,93	0,89
														Average HR25	***1,73***	***1,15***	***1,12***	***1,14***	***1,04***	***0,97***	***0,92***
HC-2	41,9	200	350	3,23	1,18	2,97	213,0	115,5	195,2	171,0	167,5	194,5	203,0	210,9	1,84	1,09	1,25	1,27	1,10	1,05	1,01
HR25-2	42,4	200	350	3,30	1,18	2,97	186,5	115,7	195,2	171,0	168,0	195,2	204,2	211,1	1,61	0,96	1,09	1,11	0,96	0,91	0,88
HR50-2	41,3	210	350	3,30	1,18	2,97	220,0	117,7	195,2	173,0	169,8	196,5	209,0	215,5	1,87	1,13	1,27	1,30	1,12	1,05	1,02
HR100-2	39,8	200	350	3,30	1,18	2,97	189,5	114,7	195,2	169,0	166,2	192,0	202,0	209,2	1,65	0,97	1,12	1,14	0,99	0,94	0,91
														Average HR50	***1,74***	***1,04***	***1,18***	***1,20***	***1,04***	***0,99***	***0,96***
														Average	***1,58***	***1,08***	***1,12***	***1,14***	***0,99***	***1,00***	***0,97***
														Stand.Deviation	***0,25***	***0,09***	***0,09***	***0,09***	***0,10***	***0,08***	***0,09***
														COV	***15,6***	***8,4***	***8,33***	***8,15***	***10,5***	***7,64***	***8,79***

6.6 CONCLUSIONS

In this chapter the structural behaviour of beam specimens having 0%, 25%, 50% and 100% of coarse recycled aggregate concretes are analysed and the behaviour of recycled aggregate concrete is compared with that of conventional concrete. All the concretes have the same compressive strength (around 40 MPa). The tensile strength of concretes made with recycled aggregates is higher than conventional concrete. The comparison is carried out based on the amount of material substitution and web reinforcement. The conclusions obtained are:

- The beam specimen without web reinforcement using a concrete with 25% recycled aggregate, HR25-1, achieved an ultimate shear load equal to that of conventional concrete. The cracking load was lower than that of conventional concrete, and the aggregate interlock action appeared to be as effective as conventional concretes.

- Beam specimens with web reinforcement using concrete with 25% coarse recycled aggregate, presented a yielding load of stirrups lower to that of conventional concretes. In beams specimens with an amount of web reinforcement required by EHE (HR25-3) the ductile behaviour was achieved and the ultimate shear load was approximate to conventional concrete. In the case of beam specimen with 0.217% of web reinforcement (a higher amount of web reinforcement than required by EHE, HR25-2), the ultimate shear force was 13% lower. In HR25-3 and HR25-2 beam specimens, the yield load was lower than conventional concrete, probably because HR25 concrete produces more brittle failure.

- For beam specimens without web reinforcement and concrete with 50 and 100% of coarse recycled aggregates (HR50-1 and HR100-1), the cracking load and ultimate shear strength were lower than for conventional concrete. The decrease in the ultimate shear strength was approximately 12% and 17%, respectively, with respect to conventional concrete.

- Beam specimens with web reinforcement and concrete with 50 and 100% of coarse recycled aggregate achieved approximately the same ultimate shear load as those of conventional concrete. However in beam specimens using 100% of recycled aggregate all the stirrups yielded whereas in conventional concrete only the stirrups placed at the middle of the span yielded. The first cracks occurred in the middle of the span of all the concrete beam specimens.

- For beam specimens with the minimum amount of reinforcement required by EHE, made with concrete using 50% and 100% of coarse recycled aggregate (HR50-3 and HR100-3), the diagonal cracking load was lower than in the conventional concrete beams (HC-3). However the yielded load was similar in beam specimens HR50-3, HR100-3 and HC-3. So, probably the aggregate interlock and concrete-stirrups bond was effective. The V_y/V_{cr} of HR50-3, HR100-3 was higher than HC-3. The yield load was considered when two stirrups yielded.

- For concrete beam specimens with 0.217% of web reinforcement (a higher amount of web reinforcement than required by EHE) using 50% and 100% of recycled aggregates (HR50-2 and HR100-2), the diagonal cracking load was lower than in conventional concrete (HC-2). In these concretes, probably the aggregate interlock and concrete-stirrups bond was effective. The V_{fail}/V_{cr} was higher in the HR50-2 and HR100-2 beam specimens than in the conventional concrete beam (HC-2). In the beam HR100-2 almost all the stirrups yielded.

- The displacement of beam specimens employing concrete with different percentage of recycled aggregates was similar to that of the conventional concrete beams. The displacement depended on the applied load.

- The strain of the longitudinal reinforcement of beam specimens of concrete with different percentage of recycled aggregates was similar to the conventional concrete beams. The strain of the longitudinal reinforcement depended on the applied load.

- EHE shear procedures offer better correlation in beams without web reinforcement when the concrete is made with either 50% or 100% of recycled coarse aggregates. These procedures are slightly conservative for conventional concrete (0% of coarse recycled aggregate) and for concrete made with 25% of coarse recycled aggregate. However, for beams with stirrups it is disproportionately conservative for all kind of concretes. The ratio V_{test}/V_{EHE} for the twelve beam specimens is 1,58 and the variation coefficient is equal to 15,57%.

- The Eurocode-2 (in its final Draft April 2003) is not conservative with respect to the values in non-reinforced beams using 50% and 100% of coarse recycled aggregates. However for conventional concrete and concrete with 25% of recycled aggregates the Eurocode-2 offers good correlation. In contrast, it is somewhat conservative regarding beams with shear reinforcement. The ratio V_{test}/V_{EC2} equals to 1,23 and the coefficient of variation is 12,94%.

- The AASHTO LRFD specifications and CSA (2004), based on the modified compression field theory, hold an unconservative view of beams without reinforcement employing 100% of coarse recycled aggregates. However they present a good correlation (similar to EC2). In contrast, they correlate with the empirical results compared to the other codes' correlations with respect to beam specimens with web reinforcement. The ratio V_{test}/V_{AASHTO} is 1,12 and the coefficient of variation is equal to 8,33. The ratio V_{test}/V_{CSA} is 1,14 and the coefficient of variation is 8,15%.

- The predicted shear strength failure by the computer program Response-2000, also based on the modified compression field theory, correlate satisfactorily for conventional concrete without and with web reinforcement, and for recycled aggregate concrete with web reinforcement. However it is slightly unconservative with respect to beams without reinforcement employing 50% and 100% of coarse recycled aggregate. The ratio $V_{test}/V_{Resp\text{-}2000}$ is equal to 0,99 and the coefficient of variation of 10,50%.

- The simplified method proposed by Cladera and Marí (March 2003), presented a good correlation for conventional concrete and concrete with 25% of coarse recycled aggregate, however it is unconservative with respect to concrete with 50% and 100% of coarse recycled aggregate. The ratio $V_{test}/V_{Caldera}$ is equal to 1,00 and the coefficient of variation of 7,64%.

- On the conventional concrete with high tensile strength, the diagonal cracking only appeared at high loads. On the other hand, recycled aggregate concrete with higher tensile strength than conventional concrete, showed diagonal cracking at a lower load.

Chapter 7
Environmental behaviour of recycled aggregate concrete

7.1 INTRODUCTION

Sustainable construction is regarded as an essential aspect of any developing society as sustainability itself is considered a measure of ecological quality based on our knowledge of the influences of waste materials on the environment and on health.
Recycling of any waste material produces a full cycle effect of "new-old-new" on one hand decreasing the waste material created by society and on the other diminishing the utilisation of depleting natural resources.

In this thesis the crushed waste concrete is used as aggregates for new concrete production. The material, through its recycling has once more a "new" renovated life as a raw material. The recycling of waste concrete and its consequent reduction as a waste quantity is a valid argument in itself for the employment of recycled aggregates. One only has to look at the detrimental affects of ploughing building waste back into the land without any prior consideration with respect to its “material make up”. Evidently there is a pressing need for the analysis of this waste material.

By their vary nature primary and secondary building materials contain organic and inorganic compounds, due to either contamination or to their production processes. When building materials are exposed to rain or groundwater, these compounds are often released through leaching (Figure 7.1), spreading into, streams, rivers, lakes and sea (surface water contamination) or the subsoil and watertable. The environmental burdening of the soil and surface water by leaching of compounds is called *immission* (figure 7.2) (Prof.Dr. Ir. Ch. F. Hendriks and Mrs J.S. Raad, 1997).

In the Netherlands, the Building Materials Decree set regulations under which building materials could be added to the soil in a safe and justified manner without detrimental affects on the soil’s nature or surface water. The concretes produced in this thesis are analysed under this decree. The Netherlands is considered a world leader with respect to the examination of the environmental impact on soil and water due to leached substances.

The *Building Materials Decree* is limited to *new* construction works such as road construction, civil engineering, hydraulic engineering works and building works.

In the *Building Materials Decree*, the classification of the building materials is based on both immission and composition values. In this case, the immission values determined in the mentioned Building Material Decree will be studied.

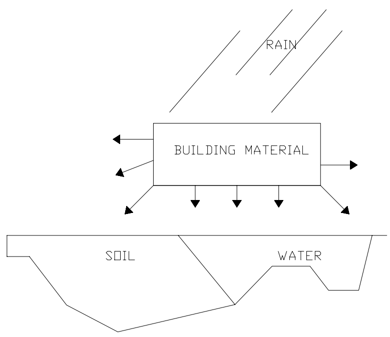


Fig.7.1: Leaching: organic and inorganic compounds which are released from building material after being exposed to rain or groundwater. (Prof. Hendriks, 1997)

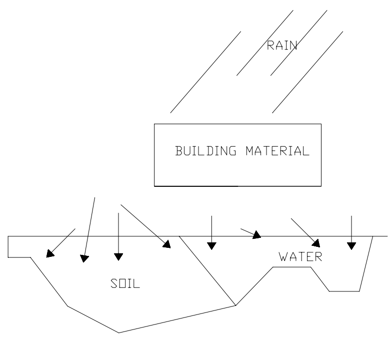


Fig.7.2: Immission: organic and inorganic compounds which disperse into the soil and surface water by leaching. (Prof. Hendriks, 1997)

7.2 IMMISSION AND COMPOSITION LIMIT VALUES ACCORDING TO THE DUTCH CONSTRUCTION MATERIALS DECREE

In order to determine the limit value for inorganic compound immission (I), it is necessary to establish as a starting point the maximum permissible quantity of inorganic matter that may be found in the soil or the surface water over a 100-year period.

This value is set at 1% of the target value for soil quality and is the same for both soil and surface water. This target value is derived from the negligible risk to ecosystems and the probability of soil contamination. The standards for soil and surface water are identical. Examples of the most important immission values are shown in table 7.1.

A construction material's immission value can be calculated by measuring the leaching of the construction material under laboratory conditions (according to normative NEN 7341 and NEN 7345) and by analysing the material's layer thickness and nature of its structure.

Recycled aggregate concrete is defined as *consolidated construction material;* construction material, in which the volume of the smaller unit is at least 50 cm^3, and which is of durable consistency under standard conditions.

For *consolidated construction material* the immission value is calculated on the basis of a 100-year period (1 year for bromide, chloride and sulphate):

$$I_{c.C} = E_{(64d)} \cdot f_v \cdot f_{temp} \tag{7.1}$$

$$I_{c.C} always \leq E_{ava} \cdot \rho \cdot d \tag{7.2}$$

where:

$I_{c.C}$ = immission of consolidated construction materials

E_{ava}= maximum leaching according to the availability test, NEN 7341

ρ= construction material density, kg/m^3

$E_{(64d)}$= Leaching according to the diffusion test (at 64 days), NEN 7345

Table 7.1: Examples of the most important immission and composition values of construction materials, and the constant needed for calculations
**To covert the composition values of a standard soil (C_{stnd}) to those soil to be assessed the other formulas (Prof. Dr.ir. Ch.F. Hendriks, 2000)*

Components		**Soil*(25% lutum and 10% humus)**		**Construction materials**
	I (mg/m^2, 100 yr)	**C_{s1} (mg/kg)**	**C_{s2} (mg/kg)**	**C(mg/kg)**
As	435	29	55	-
Ba	6300	200	625	-
Cd	12	0.8	12	-
Cr	1500	100	380	-
Co	300	100	380	-
Cu	540	36	190	-
Hg	4.5	0.3	10	-
Pb	1275	85	530	-
Mo	150	10	200	-
Ni	525	35	210	-
Sb	39	-	-	-
Se	15	-	-	-
Sn	300	20	-	-
V	2400	-	-	-
Zn	2100	140	720	-
Br	90	20	-	-
Cl	30000-87000	200	-	-
SO	45000-180000	200	-	-
CN	15	1	20	-
CN-complex	75	5	50-650	-
F	14000-56000	175+13 clay	-	-
SCN (sum)	-	-	20	-
S (total)	-	2	-	-
Aromatic substances	-	0.05+0.1(a)	125-100	1.25
PAC, separate	-	-	5-40	5-50
PAC, total	-	1	40	50-75
Chlorated	-	0.0001-0.01(a)	0.5-6	-
hydrocarbons	-	0.03	3	3
EOX	-	0.0005-0.01(a)	0.5	+
Pesticides	-	50	500	500
Mineral oil	-	0.1	0.4-270	-
Various other substances	-	0.02	0.5	0.5
PCB				

Maximum permissible *emission* in 64 days:

$$E_{\max(64d)} = I_{\max} \frac{1}{f_v f_{temp}} \tag{7.3}$$

I_{max}: maximum permissible immission in accordance with the building Materials Decree.

f_v: numerical factor

smallest value of: $f_v = 15\sqrt{f_{insul} \cdot f_{wet}}$ or $f_v = 2.5 \cdot 10^{-4} \cdot \frac{d}{\sqrt{D_e}}$

where:

f_{insul}: correction for insulation;

Insulated application f_{insul}=0.1 and f_{wet}=1

Non-insulated application:

a. structures above ground level: f_{insul}= 1 and f_{wet}=0.1
b. structures below ground level or under water: f_{insul}= 1 and f_{wet}=1

f_{wet}: correction of wetting

f_{temp}= 0.7 (Temperature correction for laboratory conditions)

d: design thickness. Minimum of 0.1 m for consolidated construction materials.

D_e: effective diffusion coefficient.

The following prevails for consolidated construction materials:

- Immission in vertical structures, e.g. walls, is calculated across the structure's thickness and not across the height.
- In the case of the results that vary greatly or of aberrant leaching behaviour of consolidated construction materials, the Dutch Code NEN 7345 should be consulted for calculating Ic,c.

7.3 AVAILABILITY OF INORGANIC COMPONENTS FOR LEACHING FROM BUILDING MATERIALS AND SOLID WASTES. AVAILABILITY TEST NEN-7341

NEN 7341 describes a method for determining the natural tendency or disposition (availability) of inorganic components in building materials and solid wastes that could be affected by leaching. The result of such leaching is the emission (cumulative) (in mg/kg) that may occur in practice over a long or very long period of time and under extreme conditions.

The availability for leaching is determined by the consecutive extracting (twice) of a partial sample of the finely ground material at a ratio between liquid and solid (L/S ratio) of 50 l/kg at a pH of 7 and 4 respectively.

In addition, the acid neutralising capacity (ANC) of the material can be calculated from the measurement data. The ANC specifies the degree to which a material may offer resistance to acid attack.

Calculation

Calculate the quantity of waste material that has a natural tendency for leaching (availability), using a completely dry sample of the material to be analysed. The disposition of the component for leaching can be calculated by employing the formula:

$$U_{avail,i} = \frac{C_i - C_{0,i}}{1000} * \frac{(2V_0 + V_1 + V_2)}{m_1 * (1 - g_{105} + g_{40})} \qquad (7.4)$$

Where:

$U_{avail,I}$	is the availability of component i, in mg/kg dry substance;
C_i	is the concentration of component i measured in the mixed extract, in µg/l
$C_{0,I}$	is the concentration of component I measured in the blank, in µg/l;
V_0	is the added volume of demineralised water in the first stage, in ml;
V_1	is the added volume of nitric acid in the first stage, in ml;
V_2	is the added volume of nitric acid in the second stage, in ml;
m_1	is the quantity of material to be extracted initially weighed, in g;
g_{105}	is the moisture content, determined by drying at 105ºC in g/g
g_{40}	is the moisture content, determined by drying at 40ºC in g/g.

The acid neutralising capacity of the material can be calculated from the measured data by the formula:

$$ANC = \frac{V_1 + V_2}{m_1 * (1 - g_{105} + g_{40})} * c(HNO_3) \qquad (7.5)$$

where:

ANC	is the acid neutralising of the material under investigation, in mol/kg
V_1	is the added volume of nitric acid in the first stage of the availability test, in ml;
V_2	is the added value of nitric acid in the second stage of the availability test, in ml;
M_1	is the quantity of material to be extracted initially weighed, in g;
G_{105}	is the humidity content, determined by drying at 105ºC, in g/g;

G_{40} is the humidity contest, determined by drying at 40ºC, in g/g;

$C(HNO_3)$ is the molarity of the added quantities of nitric acid (V_1 and V_2), in mol/l.

7.4 LEACHING BEHAVIOUR OVER A SET PERIOD OF TIME. DIFFUSION TEST NORMATIVE NEN 7345

The normative NEN 7345 describes the diffusion test which can be applied to determine the leaching behaviour of solid non-granular building and waste materials over a set period of time. The result of this test is the cumulative emission (in mg/m^2) as a function of time.

The releasing of components from solid products, monolithic and stabilised waste materials is obtained by placing a test piece under water and at fixed time intervals determining the concentration of the components in the water.

The test piece must be in complete contact with acidified water (all sides) and submerged to a depth of at least 5 cm ($V= 5*V_{piece}$).

The water is renewed 8 times (see figure 5.7), the last water sample is taken at 64 days.
Table 7.2 illustrates the leaching limits of substances employing the leaching diffusion test.

Calculations

The measured emission of each component per period:

$$E_i^* = \frac{(c_i - c_o)V}{1000A} \tag{7.6}$$

in which

E_i^* is the measured emission from each component in period I, in mg/m^2

c_i is the concentration of each component in period I, in µg/l

c_o is the concentration of the component measured in the control, in µg/l

V is the volume of the extraction liquid, in l;

A is the surface area of the test piece, in m^2.

The measured cumulative emission for all n periods;

$$\varepsilon_n^* = \sum_{i=1}^{n} (E_i^*) \quad \text{for n=1, 2, 3, …, N} \tag{7.7}$$

in which

ε^*_n is the measured cumulative emission, incluiding period I=n, in mg/m^2

E^*_1 is the measured emission in period I, in mg/m^2

N is the number of period, usually 8.

(In the measured cumulative emission (ε^*_n) the measured emission from a previous period is always taken into account, this means that any deviations in a period work follow through into the following periods).

Determine the calculated cumulative emission ε_n for each of the periods n=1 to N, according to the following formula:

$$\varepsilon_n = E_i^* \frac{\sqrt{t_i}}{\sqrt{t_i} - \sqrt{t_{i-1}}} \quad \text{for n=1, 2, …N, i=n}$$

in which

ε_n is the calculated cumulative emission of the component including the period i = n, in $mg/m^{2;}$

E^*_I is the measured emission of the component for the period i=n, in mg/m^2

t_i is the contact time after period i, in s

t_{i-1} is the contact time after period I-1, in s

For every period of water change the effective diffusion coefficient (D_e) is calculated for the relevant component from the measured emission per fraction (E^*_i) using the following formula:

$$D_{e,i} = \frac{\pi \cdot (E_i^*)^2}{(U_{bes} \cdot d)^2 \cdot (\sqrt{t_i} - \sqrt{t_{i-1}})^2} \tag{7.8}$$

where,

$D_{e,i}$ is the effective diffusion coefficient for a component calculated for data point i, in m^2/s;

(E^*_i) is the measured emission (data point i), in mg/m^2;

U_{bes} is the quantity available for leaching, in mg/kg;

d is the weight by volume of the sample, in kg/m^3

t_i is the contact time for data point i, in s;

t_{i-1} is the contact time for data point i-1, in s,

The percentage of the quantity which can be leached out from the test piece, is calculated according to the following formula:

$$U_p = U_t \cdot \frac{100}{U_{bes}} \tag{7.9}$$

where,

U_p is the leaching percentage in period t, in %;

U_t is the quantity leached out, in mg/kg;

U_{bes} is the maximum quantity which has a natural disposition for leaching, in mg/kg.

Table 7.2: Leaching limits of substances according to NEN 7345

The Netherlands Tank Leaching Test	**Leaching limits (mg/m^2)**	
Contaminants	**U_1**	**U_2**
As	40	300
Ba	600	4500
Cd	1,0	7,5
Co	25	200
Cr	150	950
Cu	50	350
Hg	0,4	3,0
Mo	15	95
Ni	50	350
Pb	100	800
Sb	3,5	25
Se	1,5	9,5
Sn	25	200
V	250	1500
Zn	200	1500
Br	25	200
Cl	20000	150000
CN- complex	6,5	50
CN-free	1,5	9,5
F	1500	9500
SO_4	25000	200000

Table 7.3: Maximum acceptable immissions into the ground or the groundwater a according to the definitions of margins burdening of the ground and groundwater

Substance	Ground	Ground water
	Max. Accept. Immission mg/m^2 per 100 year	Max. Accept. Immission mg/m^2 per 1 year
As	400	
Ba	3000	
Cd	10	
Co	300	
Cr	1500	
Cu	500	
Hg	4	
Mo	150	
Ni	500	
Pb	1000	
Sb	35	
Se	15	
Sn	300	
V	950	
Zn	2000	
Br	300	
Cl		30000
CN- tot	70	
CN-free	15	
F	7000	
SO_4		45000

7.5 CHARACTERISATION OF WASTE – LEACHING ACCORDING TO EUROPEAN STANDARD EN 12457-2

Many European countries, have developed test to determine the character and assess the constituents which can be leached from waste materials (earlier Dutch regulations have been illustrated). The release of soluble constituents upon contact with water is regarded as the main mechanism of causing a potential risk to the environment during the reuse or disposal of waste material. The intention of this test is to determine whether the waste from building materials complies with specific leaching reference values. The test consists of taking the sample material to be tested (which originally or after pre-treatment has a particle size inferior to 4 mm) and bringing it into contact with water under defined conditions. This European Standard is based on the assumption that an equilibrium or near-equilibrium is achieved between the liquid and solid phases during the duration of the test. The solid residue is separated by filtration. The properties of the eluate are measured using methods developed for water analysis adapted to meet criteria for such eluate analysis.

The 90±5 gr of waste material sample is placed in a bottle with a leachant (Distilled water, demineralised water, de-ionised water or water of equivalent purity) establishing a liquid to solid ratio (L/S)= 10 l/kg during the extraction. After which the bottle is agitated in an agitating device for 24 h.

The leachant is submitted to the whole procedure (blank test). In the eluate of the blank test, the concentration of each considered element shall be less than 20% of the concentration determined in the eluate of the tested waste. Or less than 20% of the concentration in the eluate with a limit value to which the measurement result will be compared.

Calculation

The analysis of the eluate produced by the leaching test provides the concentrations of the constituents in the eluate(s), expressed in mg/l.

The final results are expressed as the amount of constituent leached relative to the total mass of the sample, in mg/kg of dry matter.

Calculate the quantity of a constituent leached from the material, based on the dry mass of the original material from equation:

$$A = C \times [(L/M_D) + (MC/100)]$$

A: is the release of a constituent at a L/S = 10 (in mg/kg of dry matter);
C: is the concentration of a particular constituent in the eluate (in mg/l);
L is the volume of leachant used (in l);
MC: moisture material=0
M_D is the dry mass of the test portion (in kg)

The leached material is compared with the maximum required leached values for waste materials permitted in dumping sites (see table 7.4). The waste material is used as aggregates in concretes, therefore the leached components in this case are less than the waste material deposited in the dumping site and the values in this case are acceptable to check the utilization of the waste material.

Table 7.4: Maximum acceptable leached components according to European Standard EN 12457-2

Component	**L/S= 10 l/kg** **mg/kg of dry material**
As	0.5
Ba	20
Cd	0.04
Cr total	0.5
Cu	2
Hg	0.01
Mo	0.5
Ni	0.4
Pb	0.5
Sb	0.06
Se	0.1
Zn	4
Cl	800
SO4	1000

7.6 DIN 4226-100:2000

According to this draft code environmental impact, leached substances, have to be taken into account if they are to be used as recycled aggregates in concretes and in mortar production. The maximum acceptable values are shown in table 7.

Table 7.5: Acceptable maximum values. Environmental impact of recycled aggregates

Characteristic	Maximum Values	Process of analysis
Eluate		
PH Value	13,5 [a)]	DIN 38 404-5
Conductividad eléctrica	8000 [a)] µS/cm	DIN EN 27 888
Chloride	150 mg/l	DIN 38 405-1
Sulphate	600 mg/l	DIN 38 405-5
Arsenic	50 µg/l	DIN 38 406-22 DIN EN ISO 11969
Lead	100 µg/l	DIN 38 406-6 DIN 38 406-16 DIN 38 406-22
Cadmium	5 µg/l	DIN 38 406-16 DIN 38 406-22
Total Chrome	100 µg/l	DIN 38 406-16 DIN EN 1233
Copper	200 µg/l	DIN 38 406-7 DIN 38 406-16 DIN 38 406-22
Nickel	100 µg/l	DIN 38 406-11 DIN 38 406-16 DIN 38 406-22
Mercury	2 µg/l	DIN EN 1483
Zinc	400 µg/l	DIN 38 406-8 DIN 38 406-16 DIN 38 406-22
Index of Phenol	100 µg/l	DIN 38 409-16
Solid (Dry Extract)		
Hydrocarbon (H18)	1000 [b)] mg/kg	DIN 38 409-18
PAK In agreement with EPA	75 (100) [b)] mg/kg	Extración Soxhlet 3h con Cyclo-hexan, Analisis de los extractos análoga a U.S. EPA 610
EOX	10 mg/kg	DIN 38 414-17
PCB	1 mg/kg	DIN 38 414-20

a) There is no criterion of exclusion
b) Highest value that ensue of contents of asphalt they are not considered to be a criterion of exclusion.
c) *In individual cases the value can change up to the value indicated in brackets*

7.7 EXPERIMENTAL PHASE

The objective of the tests is to determine the total immission of substances by calculating the emission of substances from concretes made with different percentage of recycled aggregates. The emission of substances from recycled aggregate concrete is compared with the emission of substances from conventional concrete. The immission values are checked with the limits set by the Dutch Building Materials Decree.

The experimental phase began by analysing the quantity available for leaching of seven different materials in accordance with normative NEN 7341. The seven materials were: recycled aggregates (RA), natural raw aggregates (NA), natural sand (NS), conventional concrete (HC), concrete made with 25% of recycled aggregates (HR25), concrete made with 50% of recycled aggregates (HR50) and concrete made with 100% of recycled aggregates (HR100).

The following four concretes (HC, HR25, HR50 and HR100) were tested by employing diffusion tests according to normative NEN 7345. All concretes are 18 months old.

The results of the leaching experiments were used to calculate the immission and they were compared with the limits given by the *Building Materials Decree*.

7.7.1 Availability test according to NEN 7341

As previously defined this test was carried out on seven materials. All of the materials were crushed and passed through a 125 µm sieve. 16 kg of three different aggregates (NA, NS and RA) were crushed and broken until obtaining 200 gr. Concrete samples were taken from the beam specimens which were described in chapter 6. The concretes were crushed and broken until a representative samples of 200 gr was achieved. All the samples were in a dry (0% of moisture) condition before testing. The test was carried out twice for each kind of material.

All the samples were submitted to two stages of testing. In stage 1 the material was stirred (maintained) at pH 7 for 3 hours. In order to maintain the pH 7 over the 3 hour period mentioned a different volume of HNO3 (1M) had to be added to the water for each material tested (see table 7.6). In stage 2, the materials were stirred at pH 4 for 3 hours adding different quantities of HNO_3 (1M). One test is illustrated in figure 7.3.

Table 7.6: According to NEN 7341 the quantity of material, water and needed HNO3in each material to achieve pH-7 in stage 1 and pH 4 in stage 2.

		STAGE 1				STAGE 2			
Sample	**Material (gr)**	**Water V0_1(gr)**	**HNO3 V1 (ml)**	**Initial pH**	**Final pH**	**Water V0_2(gr)**	**HNO3 V2(ml)**	**Initial pH**	**Final PH**
HC_A	16,03	801,48	22,2	11,97	7,09	801,74	62,5	9,77	4,07
HC_B	16,00	800,00	29,0	12,10	7,07	800,59	77,5	9,35	4,04
HR25_A	16,01	800,50	41,7	12,34	7,07	800,52	107,2	9,40	4,05
HR25_B	16,01	800,48	31,2	12,18	7,01	800,48	104,8	9,65	4,05
HR50_A	16,01	800,54	30,3	12,13	6,96	800,57	98,0	10,36	4,05
HR50_B	16,04	802,09	35,4	11,95	7,00	802,34	135,2	9,24	4,03
HR100_A	16,00	800,00	43,2	12,18	7,01	800,01	143,4	9,59	4,07
HR100_B	16,02	801,05	44,7	11,80	7,01	801,30	178,3	9,14	4,09
RA_A	16,04	802,00	14,1	11,31	6,99	802,01	119,8	9,41	4,00
RA_B	16,03	801,49	14,0	10,92	7,00	801,49	140,4	9,35	4,07
NA_A	16,02	801,00	1,2	10,33	7,00	801,38	2,0	8,86	4,05
NA_B	16,02	801,01	1,3	9,93	7,00	801,05	2,2	8,66	4,08
NS_A	16,04	801,99	9,1	10,00	7,02	802,01	309,0	9,61	4,09
NS_B	16,03	801,52	9,3	9,97	7,01	801,54	317,2	9,55	4,08

Fig.7.3: Availability test according to NEN 7341

The total volume of mixed extract (water+HNO3) was analysed to determine the availability of inorganic components for leaching. The concentration of cations in the mixed extract (C_i) and in the black water ($C_{o,i}$) were obtained using a solution at pH-2 by ICP-OES (Inductively Coupled Plasma- Optical Emission Espectrometry) manufactured by Perkin Elmer model Optima 3200RL. The anions (SO_4^{-2}, Cl^-) were acquired by IC (Ion Exchange Cromatography) manufactured by Metrohm model IC760.

The availability of the components, in mg/kg dry substance is calculated by equation 7.4 and the acid neutralising capacity (ANC) of the material is also determined by equation 7.5. These values are shown in table 7.7 (they are average values of duplicated samples).

Table 7.7: Availability of components of all kinds of concretes and aggregates. Acid neutralising capacity of all materials according to NEN 7341.

Components	**Availability, Uavail (mg/kg)**						
	HC	**HR25**	**HR50**	**HR100**	**NA**	**NS**	**RA**
As	<0,1	<0,1	<0,1	<0,1	<0,1	<0,1	<0,1
Ba	*33,41*	*38,12*	*42,71*	*39,46*	*19,54*	*<0,01*	*52,88*
Cd	<0,01	<0,01	<0,01	<0,01	<0,01	<0,01	<0,01
Co	*4,80*	*25,50*	*13,83*	*10,05*	*38,08*	*<0,01*	*91,87*
Cr	<0,02	<0,02	<0,02	<0,02	<0,02	<0,02	<0,02
Cu	<0,01	<0,01	<0,01	<0,01	<0,01	<0,01	4,93
Mo	<0,01	<0,01	<0,01	<0,01	7,02	<0,01	3,25
Ni	*<0,02*	*24,99*	*20,34*	*19,69*	*17,54*	*6,59*	*54,16*
Pb	<0,1	<0,1	<0,1	<0,1	<0,1	<0,1	<0,1
Sb	<0,05	<0,05	<0,05	<0,05	<0,05	<0,05	<0,05
Se	*<0,2*	*102,91*	*<0,2*	*<0,2*	*<0,2*	*148,93*	*<0,2*
Sn	*53,03*	*85,47*	*76,09*	*107,06*	*5,51*	*157,34*	*76,87*
Sr	*84,32*	*129,06*	*120,41*	*193,34*	*<0,02*	*193,97*	*169,52*
V	<0,01	<0,01	<0,01	<0,01	<0,01	<0,01	<0,01
W	<0,1	<0,1	<0,1	<0,1	<0,1	<0,1	<0,1
Zn	*11,13*	*15,80*	*14,24*	*19,72*	*6,01*	*13,21*	*20,71*
Cl^-	*420,2*	*498,2*	*366,7*	*483,8*	*6,01*	*411,1*	*682,6*
SO_4^-	*3251,8*	*6712,8*	*5583,4*	*4920,1*	*150,9*	*533,9*	*2905,2*
Br	<1	<1	<1	<1	<1	<1	<1
	Acid Neutralising Capacity (ANC), mol/kg						
	HC	**HR25**	**HR50**	**HR100**	**NA**	**NS**	**RA**
	5,97	*8,90*	*9,32*	*12,79*	*0,21*	*20,10*	*8,99*

Concrete made with 100% of coarse recycled aggregates offers the highest resistance to acid attack. In fact any concrete made with recycled aggregates offers a high resistance to acid attack.

7.7.2 Diffusion test according to NEN 7345

The analysed construction material (CONCRETE) was tested in order to determine the amount of leaching by diffusion. This test is not necessary when the quantity of leached substances obtained by the availability test is lower than the maximum acceptable value of the Dutch Construction Materials Decree, as it was in this case. However, the diffusion test is closer to reality than the availability test and it was considered interesting to measure the amount of leached substances in real circumstances. Four concretes were tested, HC, HR25, HR50 and HR100. Two samples of each concrete were used. All the samples had similar geometry see figure 7.2 and table 7.8.

The samples were put into a tank with a volume of acidified water (pH-4) of $V=5*V_{sample}$. The water was changed 8 times in accordance with the NEN 7345 normative and it was analysed at specified periods (n=8) (see figure 5.7) determining the leaching of cations and anions. The cation and anoin concentrations were determined by the same method explained in section 7.5.1. The blank water is also analysed ($C_{o,i}$). The leaching by diffusion is calculated using the equation 7.6 and the cumulative emission is estimated by the equation 7.7. The cumulative emissions of the components at 64 days are considered important in order to measure the immissions at a 100 years. The results of the test for each type of concretes are illustrated in table 7.9 and 7.10.

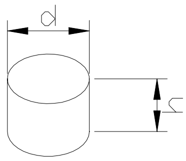


Figure 7.4: Geometry of samples, which are tested according to NEN 7345

Table 7.8: Geometry of pieces, which are tested according to NEN 7345.
** The volume is determined by the difference of mass and hydrostatic mass.*

Concrete	**Geometry**		**Vp (l)***	**A (cm^2)**	**m(gr)**	**m_H (gr)**
	d (cm)	**h (cm)**				
HC_1	9.3	4.13	277.3	256.52	677.8	400.5
HC_2	9.33	4.01	268.7	254.27	658.1	389.4
HR25_1	9.3	4.74	319.2	274.34	760.0	440.8
HR25_2	9.3	4.52	303.1	267.91	722.1	419
HR50_1	9.31	4.94	330.5	280.63	789.5	459.0
HR50_2	9.32	4.41	295.5	265.56	704.2	408.7
HR100_1	9.34	4.7	313.9	274.93	732.7	418.8
HR100_2	9.31	4.84	325	277.71	753.2	428.0

The leached component values must be less than the values described in table 7.2. The summary of the leached components and the leaching limits are shown in table 7.7. The leaching of the components are lower than the values required by the normative NEN 7345.

Table 7.9: Cumulative emission of components at 64 days for four types of concretes according to NEN 7345.

Components	**Cumulative emission (64 days) ε* (mg/m^2)**			
	HC	**HR25**	**HR50**	**HR100**
As	<0,10	<0,10	<0,10	<0,10
Ba	*2,03*	*0,31*	*1,15*	*1,23*
Cd	<0,01	<0,01	<0,01	<0,01
Co	<0,01	<0,01	<0,01	<0,01
Cr	<0,02	<0,02	<0,02	<0,02
Cu	<0,01	<0,01	<0,01	<0,01
Mo	<0,01	<0,01	<0,01	<0,01
Ni	<0,02	<0,02	<0,02	<0,02
Pb	<0,10	<0,10	<0,10	<0,10
Sb	<0,05	<0,05	<0,05	<0,05
Se	<0,20	<0,20	<0,20	<0,20
Sn	<0,01	*3,09*	*2,67*	*3,37*
Sr	*39,82*	*28,47*	*30,03*	*30,93*
V	<0,01	<0,01	<0,01	<0,01
W	<0,10	<0,10	<0,10	<0,10
Zn	<0,01	0,41	<0,01	<0,01
Cl	*63,91*	-	*111,39*	*158,8*
SO	*655,14*	*897,53*	*1033,8*	*1308,0*
Br	-	-	-	-

In the diffusion test (NEN 7345) the components leached by diffusion test are always lower than leached by availability test. The diffusion test is closer in its behaviour to reality than availability test. It can be seen in table 7.7 that using availability test Ba, Co, Ni, Se, Sn, Sr and Zn are leached, where as employing the diffusion test (see table 7.9 and 7.10) Ba, Sn, Sr and Zn are leached in much lower quantities.

Table 7.10: Comparison of the maximum cumulative emission of components in HC, HR25, HR50 and HR100 with limit of NEN 7345.

Components	**Cumulative emission (64 days) ε* (mg/m2)**				
	HC	**HR25**	**HR50**	**HR100**	**Limit by NEN 7345**
Ba	2,03	0,31	1,15	1,23	600
Sn	<0,01	3,09	2,67	3,37	25
Sr	39,82	28,47	30,03	30,93	-
Zn	<0,01	0,41	<0,01	<0,01	200
Cl	63,91	-	111,39	158,8	20000
SO_4	655,14	897,53	1033,8	1308,0	25000
Br	-	-	-	-	25

The percentage of the quantity available in the test samples, which is leached out is determined by the equation 7.9. The values are illustrated in table 7.11.

Table 7.11: Percentage of the quantity available in the HC, HR25, HR50 and HR100 test pieces which is leached out.

Components	Percentage of leached out (%)			
	HC	HR25	HR50	HR100
Ba	0,23	0,03	0,10	0,12
Sn	0,00	0,13	0,13	0,12
Sr	1,80	0,81	0,92	0,60
Zn	0,00	0,10	0,00	0,00
Cl	0,57	0,00	1,08	1,22
SO	0,77	0,49	0,67	0,98
Br	-	-	-	-

The effective diffusion coefficient of leached components in HC, HR25, HR50 and HR100 are represented in table 7.12. The effective coefficient diffusion of all materials is very low, however this is not the case when 100% of recycled coarse aggregates are used the results show it is higher.

Table 7.12: Effective diffusion coefficient of components in HC, HR25, HR50 and HR100

Components	Effective diffusion coefficient (De), m^2/s			
	HC	HR25	HR50	HR100
Ba	5,43E-22	4,97E-22	1,03E-21	2,58E-21
Sn	-	5,28E-21	4,76E-21	8,67E-17
Sr	8,34E-19	4,36E-20	1,18E-19	4,69E-16
Zn	-	3,38E-14	-	-
Cl	1,39E-13	-	1,8E-13	3,04E-12
SO	6,49E-15	1,41E-15	6,14E-15	6,1E-12
Br	-	-	-	-

In each period of the diffusion test, the conductivity and pH of the water is measured. These values for each concrete are illustrated in figures 7.5 and 7.6.

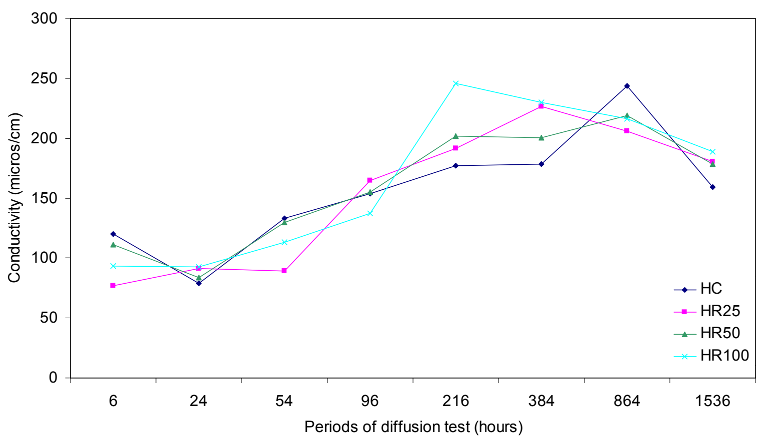


Fig. 7.5: Conductivity of water in each period of the diffusion test for concretes made with different percentage of recycled aggregates

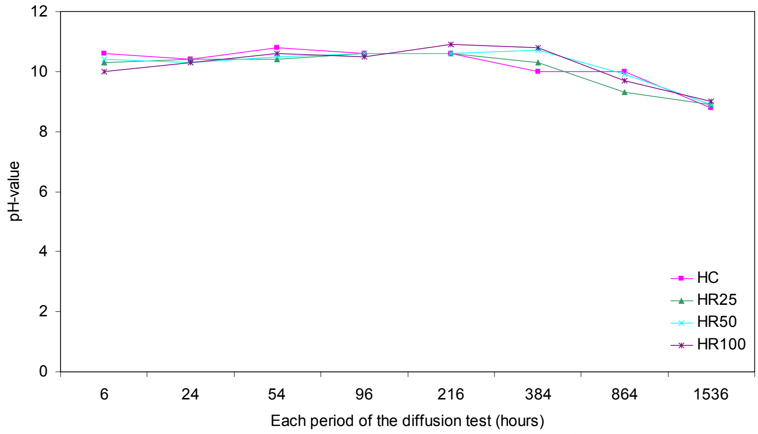


Fig. 7.6: Conductivity of water in each period of the diffusion test for concretes made with different percentage of recycled aggregates

The water extraction in each period has similar properties in all of concretes. In the last two periods the pH of the water was lower but this was expected due to the higher conductivity of the water in the last 5 periods: see figures 7.5 and 7.6. However, all the concretes studied had similar behaviour patterns and in the last stages the equilibrium between the water and the concrete samples revealed that, the leached components were lower in the lasts stages.

7.7.3 Immision values

Immision values for inorganic compounds are determined by the equations 7.1 and 7.2. Immission of consolidated construction materials is calculated by employing the equation 7.1, multiplying the cumulative emission at 64 days by f_v and f_{temp}.

Immission of concretes must not be higher than maximum quantity available for leaching multiplied by the density of concrete (ρ) and the thickness of the structural element (d). The thickness is considered the minimum permitted by the Construction Materials Decree (d=0,1 m). The real and limited immission values by the Construction Materials Decree (CMD) are shown in table 7.13 and 7.14 for Non-insulated application, structures above ground level and structures below ground level or under water, respectively.

Table 7.13: Non-insulated application. Structure above ground level (f_{insul}=1 and f_{wet}=0,1). Immision of leached components at 100 years from HC, HR25, HR50 and HR100 and comparison with the values of Construction Materials Decree. (Icc< $E_{ava}\cdot\rho\cdot d$)

Non-insulated application. Structures above ground level									
Components	**HC**		**HR25**		**HR50**		**HR100**		**I_{max} (CMD)**
	Ic,c	**$E_{ava}\cdot\rho\cdot d$**	**Ic,c**	**$E_{ava}\cdot\rho\cdot d$**	**Ic,c**	**$E_{ava}\cdot\rho\cdot d$**	**Ic,c**	**$E_{ava}\cdot\rho\cdot d$**	
Ba	6,74	8175	1,04	4537	3,81	10190	4,08	9174	3000
Sn	-	-	10,2	20356	8,85	18154	11,2	24892	300
Sr	132,2	20630	94,5	30737	99,7	28728	102,7	44953	-
Zn	-	-	1,35	1881	-	-	-	-	2000
Cl	212,2	68346,4	-	-	369,8	43795,1	527,2	112496,0	30000
SO	2175,3	795723,1	2980,1	1598686,3	3432,7	1332168	4343,0	1143981	45000
Br	-	-	-	-	-	-	-	-	300

Table 7.14: Non-insulated application. Structure below ground level or under water (f_{insul}=1 and f_{wet}=1). Immision of leached components at 100 years from HC, HR25, HR50 and HR100 and comparison with the values of Construction Materials Decree. (Icc< $E_{ava}\cdot\rho\cdot d$)

Non-insulated application. Structures below ground level or under water									
Components	**HC**		**HR25**		**HR50**		**HR100**		**I_{max} (CMD)**
	Ic,c	**$E_{ava}\cdot\rho\cdot d$**	**Ic,c**	**$E_{ava}\cdot\rho\cdot d$**	**Ic,c**	**$E_{ava}\cdot\rho\cdot d$**	**Ic,c**	**$E_{ava}\cdot\rho\cdot d$**	
Ba	21,30	8175	3,29	4537	12,03	10190	12,9	9174	6300
Sn	-	-	32,5	20356	27,9	18154	35,3	24892	300
Sr	418,1	20630	299	30737	315,3	28728	324,7	44953	-
Zn	-	-	4,26	1881	-	-			2000
Cl	671,1	68346,4	-	-	1169,6	43795,1	1510,9	112496,0	30000
SO	7286,1	795723,1	9424,07	1598686,3	10855,3	1332168	10233,4	1143981	45000
Br	-	-	-	-	-	-	-	-	300

Ic,c must be lower than $E_{ava}\cdot\rho\cdot d$, and lower than I_{max} limit of the Dutch Construction Materials Decree (CMD) in any kind of structures. The immissions are higher when the structure is in contact with water. However, the immision of any component in any kind of structure made with any type of concrete is much lower than the limits required by the Dutch Construction Materials Decree.

7.7.4 Recycled aggregates leaching according to European Standard EN 12457-2

Recycled aggregates were tested according European Standard EN 12457-2 in order to determine the leached components. The test was carried out with a material finer than 125 μm, and the European standard is defined for aggregates smaller than 4 mm. Consequently, in this case the leached component was much higher than in real case.

Table 7.15: Leached material according to European Standard EN 12457-2

Components	Leached material mg/kg	Maximum leached Material (mg/kg)
As	0,01	0,5
Ba	0,42	20
Cd	<0,0005	0,04
Cr total	*0,64*	*0,5*
cu	0,05	2
hg	<0,0005	0,01
Mo	0,06	0,5
Ni	<0,010	0,4
Pb	0,01	0,5
Sb	0,02	0,06
Se	<0,01	0,1
zn	<0,005	4
Cl	111,92	800
SO_4	992,82	1000

7.8 CONCLUSIONS

The utilization of waste material, completing a full cycle "new-old-new" is essential for sustainable development in construction sites. Organic and inorganic compounds can be released from waste materials through leaching and disperse into the soil and surface water. The quantity of leaching material for a 100-year period (1 year for bromide, chloride and sulphate) were measured in conventional concrete (HC), concrete made with 25%, 50% and 100% of recycled aggregates (HR25, HR50 and HR100, respectively). The tests were carried out following the Dutch Code and European standard. The Dutch Code is considered as being one of the most developed applications in this area.

From the results obtained, the following conclusions must be drawn:

- In all concretes the same components (Ba, Co, Ni, Sn, Sr, and Zn) are subject to the disposition of leaching over a long period under extreme conditions (NEN 7341). The leached components appear mainly from recycled aggregates and sand. Conventional aggregates (granites in this case) suffer low leached components.

- Concrete made with recycled aggregates has a larger capacity of acid neutralisation. Concrete made with 100% of recycled aggregates have more cement quantity than any other concrete, and a large part of recycled aggregates consist of adhered mortar, therefore the cement quantity in any recycled aggregate concrete is higher than that found in conventional concrete and the alkalinity is higher.

- Available components by leaching are more numerous and with higher values than leached components by diffusion over a 64 day period. Cumulative emission (64 days) in HC, HR25, HR50 and HR100 are very similar. Ba, Sn and Sr are leached in all concretes, but the cumulative emissions are always much lower than limited by NEN 7345.

- Diffusion test is not necessary when the quantity of leached substances obtained by availability test is lower than the maximum acceptable value of the Dutch Construction Materials Decree, as it was in this case. However, the diffusion test is closer to reality than availability test and it was considered interesting to measure the amount of leached substances in real circumstances.

- The percentage of consolidated material leached, HC, HR25, HR50 and HR100 is lower than 0,30 % in all cases. There is no difference between concretes made with conventional or coarse recycled aggregates.

- Concretes made with a higher percentage of coarse recycled aggregates have a larger effective diffusion coefficient. This is a logical result as recycled aggregates are far more porous than raw aggregates. The effective diffusion coefficient is very low in all concretes.

- During the duration of diffusion test the equilibrium of the components between the concrete samples and the water is achieved, all the concretes have approximately the same conductivity and the same pH after the tests and both values decrease.

- With respect to structures built above ground water using HC, HR25, HR50 or HR100 concretes, the possible immission to soil is 0,11% and 3,7% of Ba and

Sn respectively, with regard to the maximum value required by the Dutch Construction Materials Decree of leached components at a 100 years.

- In structures built of HC, HR25, HR50 or HR100 concrete below ground level or under water, although the immissions are higher than the previously above ground defined structures, in this case the immision values are also low. The immission of Ba and Sn are 0,34% and 11,8% respectively, with respect to maximum value required by the Dutch Construction Materials Decree of leached components at 100 years.

- The European Standard is shorter test (on time) than the Dutch standard. When the leached component is lower than the maximum acceptable leached values the test can be effective. However unlike the diffusion test (NEN 7345), The European Standard test does not produce results similar to the those obtained in real situations.

- The DIN 4226-100 normative gives maximum acceptable values for leaching. There are several Codes to measure leached components. However in reality it is not very useful. The European Standard or the Dutch standard are recommended for use.

- European Standard EN 12457-2 is an easier method than Dutch methods to determine the leached components. However, its maximum acceptable values are defined for material placed in dumpling site. The waste material used in concrete production is protected by cement paste and its leached component is lower. The Dutch methods are more elaborated methods, and NEN 7345 reproduces the reality case.

Chapter 8

General conclusions and future research

8.1 INTRODUCTION

Different percentages (0%, 25%, 50% and 100%) of recycled coarse aggregates obtained from crushed waste concrete were employed in the making of concrete structural elements. The structural behaviour of the mentioned recycled coarse aggregates concretes was later analysed. In order to understand this behaviour the properties of recycled aggregates and the mechanical properties of concrete employing them (recycled aggregates) were determined, as was the microstrusture of the concretes. The possible process of deterioration which would affect the useful life of the structural elements (cast with recycled aggregate concrete) was also defined.

8.2 GENERAL CONCLUSIONS

The following conclusions can be drawn from the different experimental works carried out in this thesis.

Recycled Aggregates' properties

Both the physical and chemical properties of the crushed recycled coarse aggregates employed in the concrete mixes described are adequate to use in concrete production. Concrete crushed by an impact crusher achieves a high percentage of recycled coarse aggregates without adhered mortar, their quality is acceptable for employing as secondary aggregates in concrete production. However the properties of the recycled aggregates are inferior to those of conventional aggregates.

According to RILEM recommendation and normative DIN 4226-100 the recycled coarse aggregates obtained from crushed concrete are classified as Type II and Type 1, respectively. In accordance with prEN-13242:2002 the recycled coarse aggregates can be classified as belonging to the best category.

The code DIN 4226-100 gives a more detailed description with respect to the properties of recycled aggregates than the recommendations set out in RILEM or Normative prEN-13242:2002. The European normative prEN-13242:2002, classified the recycled aggregates' properties into categories. However, the application of each category is open to interpretation and it should be defined by each country.

Recycled aggregates are formed by original coarse aggregates and adhered mortar. In addition, the adhered mortar consists of original cement paste and fine aggregates. These original coarse and fine aggregates can be Alkali-Aggregate Reactive. Consequently they should be tested separately in order to measure their reactivity. In this study, the sand present in adhered mortar is classified as a slow/late expanding reactive aggregate.

The structural elements studied, were not exposed to humid conditions, therefore these elements were not affected by any reaction caused by the aggregates.

The absorption capacity of the recycled aggregates is the property which should really be taken into account during concrete production.

Fresh Concrete

The humidity content in coarse recycled aggregates must be considered as high. Consequently they will be used in concrete production with little absorption capacity in order to produce controlled quality concrete (the effective w/c ratio and fresh concrete workability).

The production process has a great influence on the quality of concrete made with recycled aggregates. The adding of cement to the mix in the mixer machine after adding the humid but not saturated recycled aggregates produces an effective interface when the water is finally added, as it is absorbed due to the absorption capacity that recycled aggregates still maintain.

It must be stressed that concrete made with recycled aggregates needs more superplasticizer to achieve the same workability as conventional concrete.

Mechanical properties of recycled aggregate concrete

Concrete made with 100% of recycled coarse aggregates has 20-25% less compression strength than conventional concrete at 28 days, with the same effective w/c ratio (w/c=0,50) and cement quantity (325 kg of cement/m^3).

Concrete made with 100% of coarse recycled aggregate requires far too much cement to achieve a high compressive strength and consequently is not an economic proposition as it is not cost effective. These recycled aggregates should be used in concretes with low-medium compression strength. Moreover, the adhered mortar in recycled aggregates is lower in strength than conventional aggregates and the new paste. Consequently the weakest point in concretes made with coarse recycled aggregates employing a cement paste of a medium-high strength can be determined by the strength of the recycled aggregates or their adhered mortar.

Medium compression strength concrete made with 25% of recycled coarse aggregates achieves the same mechanical properties as that of conventional concrete employing the same quantity of cement and the equal effective w/c ratio.

Medium compressive strength concrete made with 50% or 100% of recycled coarse aggregates needs 4-10% lower effective w/c ratio and 5-10% more cement than conventional concrete to achieve the same compression strength at 28 days. The modulus elasticity is lower than that of conventional concrete. However, the tensile strength of recycled aggregate concrete can be higher than conventional concrete (concrete using raw aggregates).

Standard Deviation increases up to 50% employing a 100% recycled aggregate concrete mix.

Durability of recycled aggregate concrete

In Spain, crushed stone was not used as fine aggregate until approximately 30 years ago. Consequently, it should be taken into account the demolition of concrete structures dating before the 70s, where concrete was cast employing sand in which silica could be a main component.

Alkali Silica Gel can be produced as a result of the alkali contribution from new cement and the reactive fine aggregates present in adhered mortar (the case under study), given conditions in which the concrete is saturated or almost saturated.

- CEM I 52.5R (which is a very fine material), has a relatively high amount of alkalis (>0.6%). The cement is accumulated in the interface as a result of; a) The concrete production process. The order of adding materials to the concrete mix employed in the automatic mixer machine results in the cement being adhered to the recycled aggregates before the water is added. When the water is added the cement is absorbed into the recycled aggregates; b) The fineness of the cement employed and c) The recycled aggregates absorption capacity. This accumulation produces an effective interface between recycled aggregate and new cement paste however, it must be pointed out that alkali accumulation can also occur.

- The water which is in contact with this accumulated cement reaches a high pH value by dissolving the reactive silica on the aggregates surface to form an alkali silica gel.

- With potentially reactive or reactive original aggregates, it is recommended to use low alkali portland or blast furnace slag cements as they may increase the durability of the recycled concrete.

- The concrete beam specimens cast with the recycled coarse aggregate were dry, therefore they did not suffer any alkali silica reaction and their structural behaviour is possible to describe.

Macrostructure and microstructure of recycled aggregate concrete

Regarding macrostructure, the distribution of aggregate, is homogeneous in concrete made with recycled and raw coarse aggregates. When the percentage of recycled aggregate in the concrete is increased the heterogeneous nature of the aggregates also increases as does its colour.

The percentage of “aluminium aggregate” in concrete production must be nil (0%). Contaminants must be avoided in order to produce good quality concrete.

The Fluorescent thin-section (FTS) method of analysis by optical microscope is an excellent method to compare the w/c ratio of the paste, the interfacial transition zone and also the quality of the aggregates. In conclusion this studies findings are that:

- The adhered mortar of original concretes is not as dense as that of the new cement paste. The porosity is high as a result of the quality of the original mortar adhered to the recycled aggregates employed.

- The density of the cement paste is homogeneous in concrete made with recycled aggregates. The air void percentage increases significantly in concrete where 100% of recycled coarse aggregates are employed, due to the larger size of the pores. In addition when we take into consideration the air voids in adhered mortar, the percentage of porosity increase is approximately 0,4%. (It does not refer to voids of submicroscopical dimensions).

- The air voids are not interconnected, consequently the permeability of recycled aggregate concretes is low.

- In order to achieve the same compression strength the cement paste of concrete made with 50% and 100% of recycled aggregates is denser (with lower effective w/c ratio) than that of conventional concrete. The water contained in the aggregates does not react in the cement hydration.

- The concrete production process seriously influences the improvement of the new interface. As a result of the order in which the material was added to the mixing machine, an accumulation of cement occurred in the interface, producing an interfacial transition zone with a very low w/c ratio. Consequently the weakest point is the adhered mortar and it is the adhered mortar's strength that determines the material strength and behaviour. This is one of the largest differences with respect to conventional concrete.

- Although it is probable that a certain quantity of unhydrated cement can accumulate the hydration which will occur during the concrete's life guarantees an effective interfacial zone.

Structural behaviour of recycled aggregate concrete

The effect of the use of recycled aggregate on the beams' shear strength depends on the percentage of coarse aggregate substituted. For low values (less than 25%) it can be said that this influence is practically negligible for beam specimens without shear reinforcement and has a small influence in beams with stirrups.

The non-reinforced beam specimen using a concrete mix of 25% recycled aggregate achieves an ultimate shear load equal to that of conventional concrete, with lower cracking load. For beam specimens without web reinforcement and concrete with 50% and 100% of coarse recycled aggregates, the cracking load and ultimate shear strength is 10-20% lower than that of conventional concrete.

The beam specimens with web reinforcement using a concrete mix of 25% coarse recycled aggregate present a slightly lower ultimate shear capacity than that of conventional concrete. Being mechanical properties of concrete produced with 25% of recycled aggregates similar to conventional concrete.

Beam specimens with web reinforcement and concrete with 50 and 100% of coarse recycled aggregate achieved approximately the ultimate shear load of conventional concrete. However in beam specimens using 100% of recycled aggregate all the stirrups yielded whereas in conventional concrete only the stirrups placed at the middle of the span yielded.

Based on the tests carried out in this work it can be said that not only is the concrete-reinforcement bond effective but that the interlock mechanism in concrete employing a high percentage of recycled aggregate works correctly.

The standards employed in the study are based on the modified compression theories (AASHTO LRFD specifications and CSA) present a good correlation with respect to non-reinforced conventional concrete beam specimens. These standards hold a none conservative view of concrete made with more than 50% of recycled aggregates. As a consequence this theory considers the concrete's tensile strength as an important factor in its shear strength. In beam specimens with web reinforcement, the results correlate well with the empirical results compared to the other codes' correlation.

EHE shear procedures in beams without web reinforcement is somewhat conservative, offering better correlation in concretes made with 50% and 100% recycled coarse aggregate. For beams with stirrups it is disproportionately conservative, as it does not consider the influence of aggregate interlock on the ultimate shear force.

The final Draft of Eurocode-2 is conservative with respect to all types of concretes in beams with stirrups. However, it is not conservative with respect to beams without stirrups that employ a high percentage of recycled aggregates in their production.

The method proposed by Cladera and Marí (2003) is not conservative for concretes produced with a high percentage of recycled aggregates. It is also considered that shear strength depends on tensile strength, which is true in conventional concretes but it is not true in the case in recycled aggregate concretes.

The shear design of concrete beams with recycled aggregates (25%) must take into account the strength reduction observed experimentally. Thus, it is proposed to design the beams for a design shear force 10% higher than the one obtained in the structural analysis, both for beams with and without transverse reinforcement.

Environmental behaviour of recycled aggregate concrete

The utilization of waste material, completing a full cycle “new-old-new” is essential for sustainable development on construction sites. Organic and inorganic compounds are released from waste materials through leaching and dispersed into the soil and surface water. The leaching of these organic and inorganic materials can be measured employing different codes, examples being the two Dutch codes (NEN 7341 and 7345) and the European Normative (EN 12457-2) the three codes considered are very important in Europe.

The Netherlands is considered a world leader with respect to the examination of the environmental impact of leached substances on soil and water.

The amount of leached components in concrete made with recycled aggregates is always lower than the acceptable values required by the Dutch Construction Materials Decree and European standard.

The test method defined by both standards with respect to leaching are carried out in extreme conditions. In these conditions the amount of leached component is higher than it would be in normal circumstances, therefore the methods employed are valid. However, the maximum values required by European standard are too low, and at times they are not possible to measure.

The European Standard is an easier method to employ than the Dutch one, however the Dutch method (NEN 7345) is more representative of a real situation.

There is no defined European standard for measuring the leached components of recycled aggregates employed in concrete. It measures the leached components of waste material on dumping sites. In these conditions (when the concrete has been broken and the aggregates exposed) the leaching of the components in the waste material is more easily produced and consequently easier to define than when the aggregates are protected within the cement paste.

The leaching of consolidated materials in concretes made with different percentages of recycled aggregate is determined according to the NEN 7345 diffusion test. This method represents the natural or real situation and is valid to determine the immision in soil. The Immision in soil of any component from any recycled aggregate concrete (recycled aggregates obtained form crushed concrete) employed in structures built above ground level or below ground level or under water is much lower than the maximum values required by the Dutch Construction Materials Decree.

8.3 FUTURE RESEARCH

The study carried out in this work opens up many avenues of research (in structural behaviour and durability aspects) regarding recycled aggregate concretes.

Most probably concrete produced with 25% of recycled aggregates will appear in the next edition of the Spanish Structural Concrete Code (EHE). However, few experimental results are available. Therefore more experimental work is needed in order to verify the structural behaviour of RC using such amounts of recycled aggregate under different load conditions (for example, sustained loading, reversal loading shear-bending interaction, torsion). Of special interest are all those mechanisms that depend on the concrete’s tensile strength, such as bond, anchorage and shear friction. In this aspect, the MCFT (which is the most rigorous model for analyzing the shear behaviour on concrete structures) relies on the concrete tensile strength and on the experimental result on shear panels. New tests on shear panels with recycled concrete would be of great interest in order to extend the MCFT to recycled aggregate concrete.

In the concrete mixing processes using automatic machines (the largest and most used production mixing machines), the cement (CEM 52.5) is adhered to and absorbed into the border of the recycled aggregates, resulting in an effective interface. However, alkali accumulation also occurs producing an alkali silica reaction due to the reactive fine aggregates presence in the adhered mortar. The concrete structures cast earlier than the 80s have silica as a fine aggregate composition, consequently the recycled aggregates obtained from those crushed concretes can be reactive fine aggregates. The possible reaction of CEM I 42.5 and 32.5 to fine original aggregate should be analysed, because of its lower amount of alkalis. Its behaviour in the recycled aggregate’s interface must also be analysed.

As a consequence of the adhered mortar and its life span recycled aggregates are a much more porous material than raw aggregates. There is a greater possibility of Alkali silica gel expansion because of the pores, however as the pores are internal pores probably the real expansion could be lower than that measured by the accelerated mortar bar's test. Therefore, it will be necessary to continue the research in recycled aggregate concrete according to the ASR criteria.

However, because of its porosity, the permeability is much higher than in mixes employing new cement paste and raw aggregates. Consequently the behaviour of recycled aggregate concrete exposed to chloride, carbonate or sulphate attacks must be measured in order to determine its durability as a construction material.

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