UNIVERSITY OF STRATHCLYDE

RECYCLED AGGREGATE CONCRETE (RAC) FOR STRUCTURAL PURPOSES

BY
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A Thesis submitted to the Department of Civil Engineering for the Degree of Doctor of Philosophy

Glasgow, U.K.
1999

ACKNOWLEDGMENTS

The author would like to take this opportunity to express his deep appreciation and gratitude to **Mr Roy Cairns** and **Prof Ing Ezio Dolara** for their continuous encouragement, supervision and guidance throughout this research.

Thanks are also due to **Prof Howard Wright** Head of the Department of Civil Engineering at the University of Strathclyde.

The assistance given by **Mabo Prefabbricati S.p.A.** in Bibbiena (Arezzo), Italy and **Pescale S.p.A.** in Castellarano (Reggio Emilia), Italy in producing the prestressed beams made from RAC is gratefully acknowledged.

The technical staff in the structural Laboratories University of Strathclyde and of Mabo Prefabbricati gave unstinting support in the experimental work, especially **Mr John Harper** and **Mr Enzo Martini**.

Finally the author would like to express his greatest appreciation and gratitude to **his parents** and all **his family** for their patience and support throughout this work.

ABSTRACT

The possibility of using demolished concrete waste as aggregate in fresh concrete in the production of prestressed concrete beams is checked in this research.

As opposed to the use for road foundations or as fill-in material the use of the **Recycled Aggregate (RA)** for concrete structures requires more tests and processing of results. In fact to be able to use a material for construction it is essential to assess more than just its compressive strength.

After the physical and chemical characteristics of the **RA** and the properties of both the wet and hardened **Recycled Aggregate Concrete (RAC)** have been determined, it is important to check if the mathematical models and numerical correlation normally used for design of ordinary concrete (such as mix-design procedure, design codes, non-linear analysis) are suitable for **RAC**.

For this reason the main task of this investigations has been to ensure that **RAC** has satisfactory mechanical performance for structural use and later to guarantee a consistency of the results using methods checked for **RAC**.

A mix-design procedure suitable for **RAC** to attain the desired workability and the target strength was the first step.

Tests on durability of **RA** and **RAC** have been performed and the results reported.

Finally three 15.0 metres span prestressed beams cast with different percentages of **RA** (one with 100% of **RA,** one with 100% of Natural Aggregate **NA,** and one with 50% of **RA** and 50% of **NA**) have been tested.

The results show that it is practicable to make prestressed concrete elements using concrete made with Recycled Aggregate and that these elements can have satisfactory and predictable mechanical performance.

To my parents.

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Chapter One

Introduction

1.1 Introduction

Today the culture of recycling pervades each sector of our life.

Yet in the world of civil engineering there is no great evidence of the use of demolition waste.

Lots of international studies and research [1], [2], [3] have, however, reported an interest in recycling of demolition waste from concrete structures

The increasing demand for natural aggregate and the quantity of demolition waste in industrialised countries mean that it must now be seriously considered.

The term Recycled Aggregate Concrete (commonly abbreviated to RAC) is used to describe concrete in which the natural aggregate is partially or totally replaced by the aggregate coming from demolition of concrete structures.

A few countries already allow up to 20% of Recycled Aggregate in a normal concrete mix [4].

The Civil Engineering Industry faces two increasing problems.

The first one is availability. In many major urban areas natural aggregates are locally unavailable and it becomes necessary to transport them from increasingly longer distance with higher costs.

The second one concerns the difficulties of disposing of the demolition wastes.

In Italy alone the need for natural aggregates is 541 million tons / year.

In 1988 the production of natural aggregates was 250 million tons / year (this figure places Italy second behind Germany in the EEC) and shows the imbalance between supply and demand of natural aggregates.

Again in Italy, the demolition waste produced was 34.4 million tons. This value is comparable only with the United Kingdom in all the EEC countries.

Recent studies indicate that demolition waste in the EEC countries averages 200 million tons / year and, with a careful policy of recycling this could account for 10% of the world's requirements for construction aggregates.

Based on a RILEM (The International Union of Testing and Research Laboratories for Materials and Structures) study and on complementary international studies it has been established that most of the world's demolition waste could be processed and recycled.

The total amount of demolition waste in Europe is shown in Figure 1.1.

In 1970 a British society (Environmental Resources Limited) [3] estimated the amount of demolition wastes in the period 1970-2060.

This estimation, as it has been reported in the RILEM study, is shown in Figure 1.2.

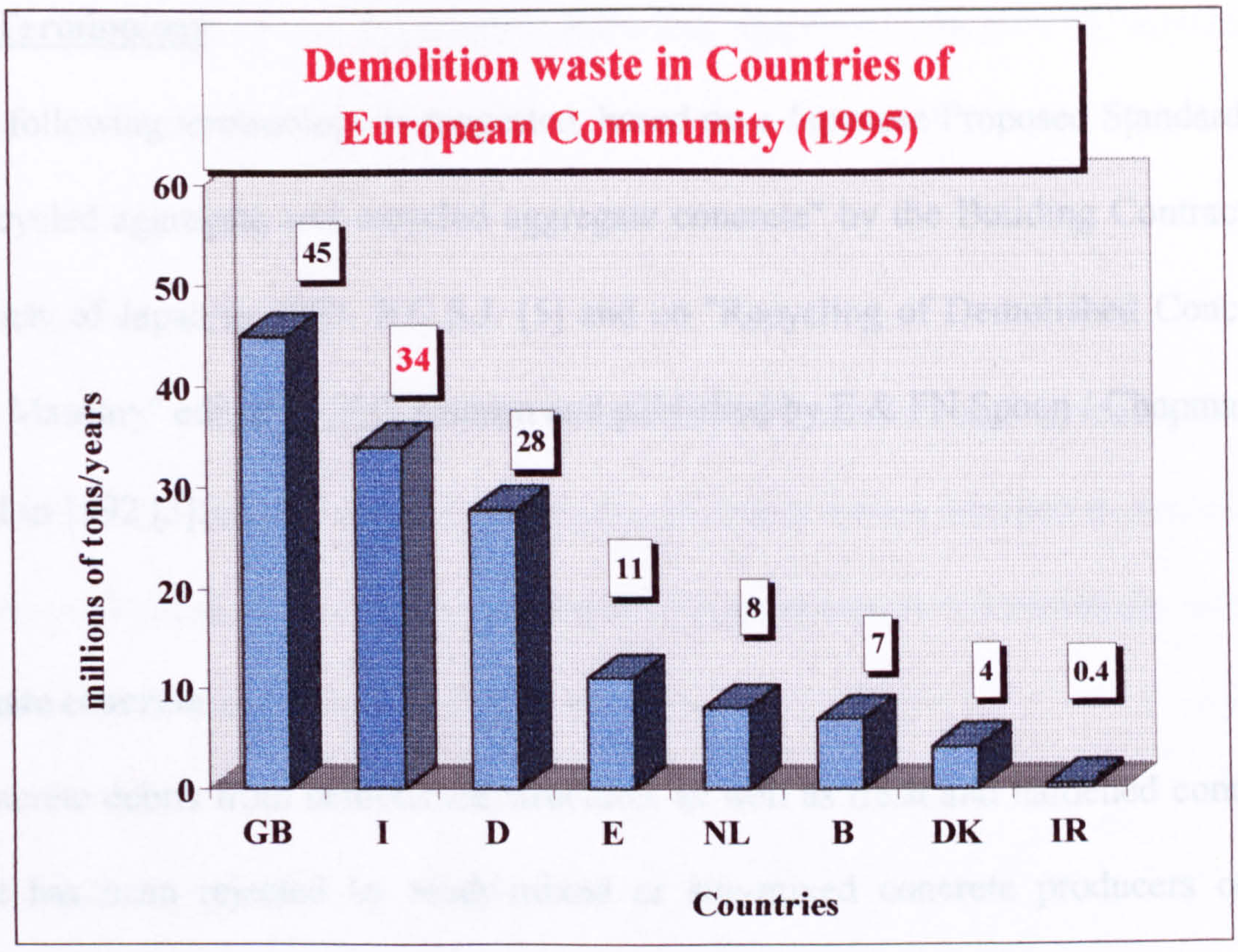


Fig. 1.1 Amount of demolition waste in Europe (RILEM).

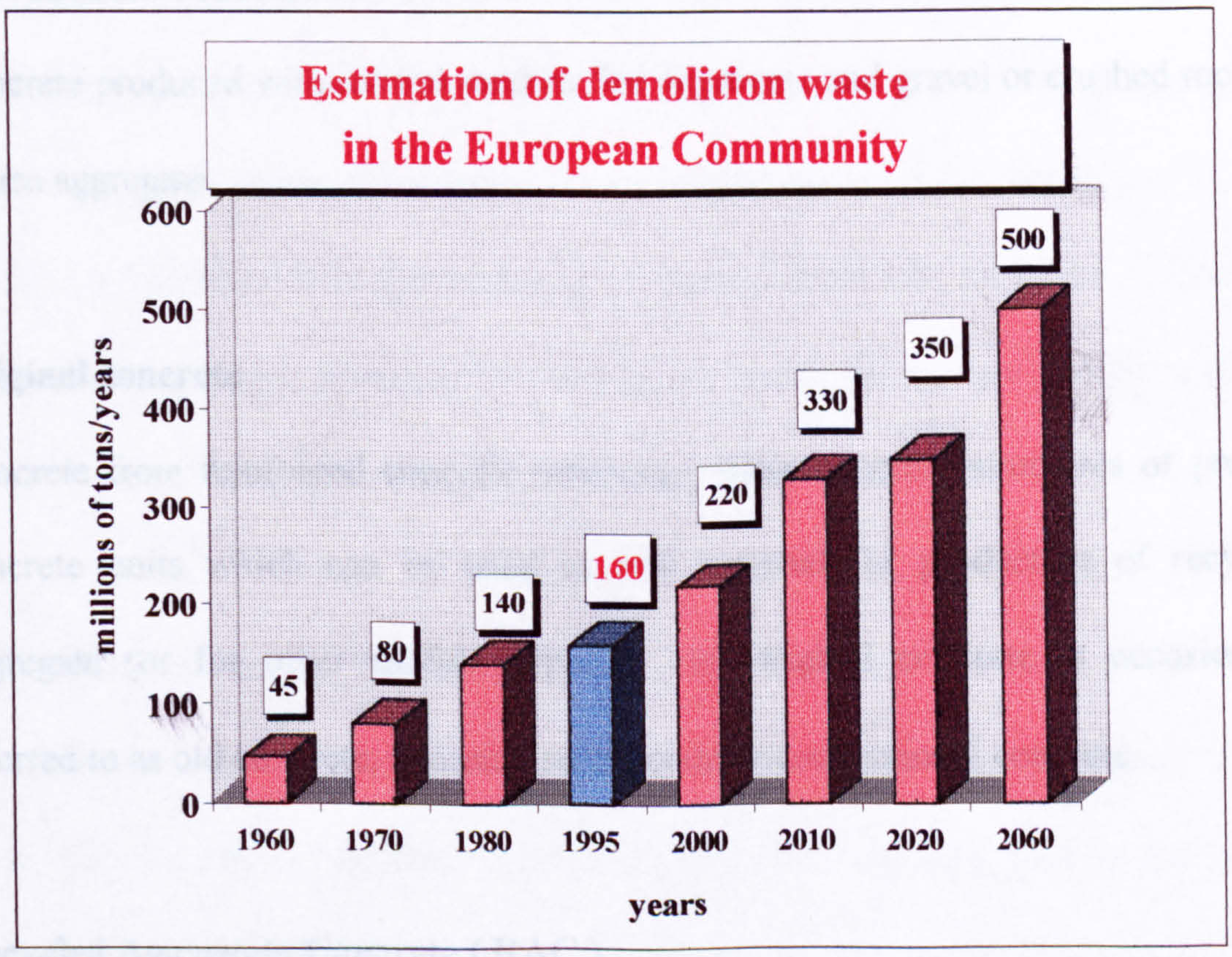


Fig. 1.2 Estimation of demolition waste in Europe (RILEM).

1.2 Terminology

The following terminology is suggested, based on a Japanese Proposed Standard on "Recycled aggregate and recycled aggregate concrete" by the Building Contractors Society of Japan in 1977, B.C.S.J. [5] and on "Recycling of Demolished Concrete and Masonry" edited by T.C. Hansen and published by E & FN Spoon / Chapman & Hall in 1992 [3]:

Waste concrete

Concrete debris from demolished structures as well as fresh and hardened concrete that has been rejected by ready-mixed or site-mixed concrete producers or by concrete product manufactures.

Conventional concrete

Concrete produced with natural sand as fine aggregate and gravel or crushed rock as coarse aggregate.

Original concrete

Concrete from reinforced concrete structures, plain concrete structures or precast concrete units which can be used as raw material for production of recycled aggregate (or for other useful purpose). The original concrete is occasionally referred to as old concrete, demolished concrete or conventional concrete.

Recycled Aggregate Concrete (RAC)

Concrete produced using recycled aggregates or combinations of recycled aggregate and other aggregates. Recycled aggregate concrete is sometimes referred to as new concrete.

Original mortar

Hardened mixture of cement, water, and conventional fine aggregate less than 4-5 mm in original concrete. Some original mortar is always attached to particles of original aggregate in recycled aggregates. Original mortar is occasionally referred to as old mortar, or conventional mortar.

Original aggregates

Conventional aggregate from which original concrete is produced. Original aggregates are natural or manufactured, coarse or fine aggregates commonly used for production of conventional concrete. When no misunderstanding is possible, original aggregates may also be referred to as virgin or conventional aggregates. It is suggested to use the notation Ns for natural sand, Ng for natural gravel, Ncs for sand produced by the crushing of natural materials, and Ncc for natural crushed aggregate. *N* stands for "natural", *g* stands for "gravel", while *cs* stands for "crushed sand" and *cc* for "crushed coarse aggregate".

Recycled concrete Aggregates (RA)

Aggregates produced by the crushing of original concrete; such aggregates can be fine or coarse recycled aggregates. Fine recycled aggregate is sometimes referred to as crushed concrete fines. When no misunderstanding is possible, recycled concrete aggregates may be referred to as recycled aggregates. It is suggested to use the notation Rs for recycled fine aggregate and Rc for recycled coarse aggregate. *R* stands for "recycled" and *s* stands for "sands", while *c* stands for "coarse aggregate".

1.3 Objective of this study

The main aims of this thesis are:

I) - To carry out an experimental investigation to improve the overall understanding of the behaviour of structures made with RAC and to verify if this material can be used to make pre-stressed concrete beams.

II) - To investigate the non-linear behaviour of pre-stressed beams made by RAC.

III) -To study the influence of some numerical parameters on the stability and efficiency of the LUSAS Concrete model in the analysis of structures made with RAC.

IV) –To extend to RAC the evaluation of the compressive strength of concrete using non-destructive methods (e.g. rebound hammer test, Pundit test, etc.).

V) -To study the surface between the RA and new cement paste using Scanning Electron Microscope (SEM) tests, trying to understand the mechanical failure of RAC.

The research in this study was mainly divided into two parts:

a) In the first part experimental work was carried out to establish the mechanical properties of the RAC. Preliminary tests on RA were performed.

Afterwards three pre-stressed beams made from RAC were cast and tested. The experimental results of the tests were compared with analytical results from the finite element package LUSAS.

Non-destructive tests were carried out on RAC to correlate with the results from other compressive strength tests.

SEM tests were also carried out to investigate the contact surface between the old RA and new cement paste.

b) In the second part a theoretical Non-linear analysis of the concrete structures was undertaken. The validity of the LUSAS concrete material modelling has been checked for structures made from RAC.

c) The relationship between the compressive strength of concrete and non-destructive tests on concrete (Rebound hammer test, Pundit test) was adapted for RAC.

d) Pulse ultrasonic velocity tests were used to evaluate the crack patterns of the pre-stressed beams during the failure test.

1.4 <u>Organisation of the remainder of the thesis</u>

In Chapter One the terminology adopted by the international literature on RAC is reported. The objective of this research is given.

Chapter Two consists of a review of previous investigations on RAC. Two previous investigations on the same Recycled Aggregate (from R.O.S.E., Recupero Omogeneizzato Scarti Edilizia that means Building Industry Waste Homogenised Recovery) used in this research are reported.

The site-plant for production of the R.O.S.E. Recycled Aggregate is described in Chapter Three. The physico-chemical characteristics of the Recycled Aggregate are also given.

The durability of both wet and hardened Recycled Aggregate Concrete is reported in Chapter Four.

In Chapter Five the possibility of using RAC for structural purposes is discussed. The design of pre-stressed beams made by RAC is given and the results of the tests shown in figures and tables.

The LUSAS finite element package is used to compare the experimental and theoretical results. The significant parameters affecting the numerical solution are discussed.

Non-destructive testing on a pre-stressed beam made from RAC and the evaluation of crack patterns using Pundit test is reported in Chapter Six.

Lastly in Chapter Seven conclusions derived from the study are reported. Economical and environmental aspects using RAC are discussed. Suggestions and recommended areas for futures study are given.

Chapter Two

Review of Previous Investigations

2.1 Introduction

In this chapter a review of previous investigations on RAC is given. Two previous investigations carried out in the University of Rome "La Sapienza" and in the University of Strathclyde on the same Recycled Aggregate used in this research are reported.

2.2 The state-of-the-art

Since 1945 extensive but fragmented research has been carried out on RA and RAC all over the world.

This thesis refers in particular to three state-of-the-art reports by P.J. Nixon [1] and T.C. Hansen [2], [3].

However during the course of the present study, new research has taken place in many countries and an important event was the International Symposium organised by the Concrete Technology Unit, University of Dundee and held at the Department of Trade and Industry Conference Centre in London on 11-12 November 1998 [4] which contains papers describing more recent research. These papers are not discussed here but they do confirm the most important properties and characteristics of RA and RAC as previously summarised by T.C. Hansen in [3]. During the description of the specific lab tests (see Chapters 3, 4 and 5) any relevant papers will be referred to.

2.2.1 Quality of recycled aggregates

Grading, particle shape, and surface texture of recycled aggregates

Generally the RA as it comes from the crusher is somewhat coarser than the lower limit of ASTM C-33 (see Fig.2.1) grading requirements [5].

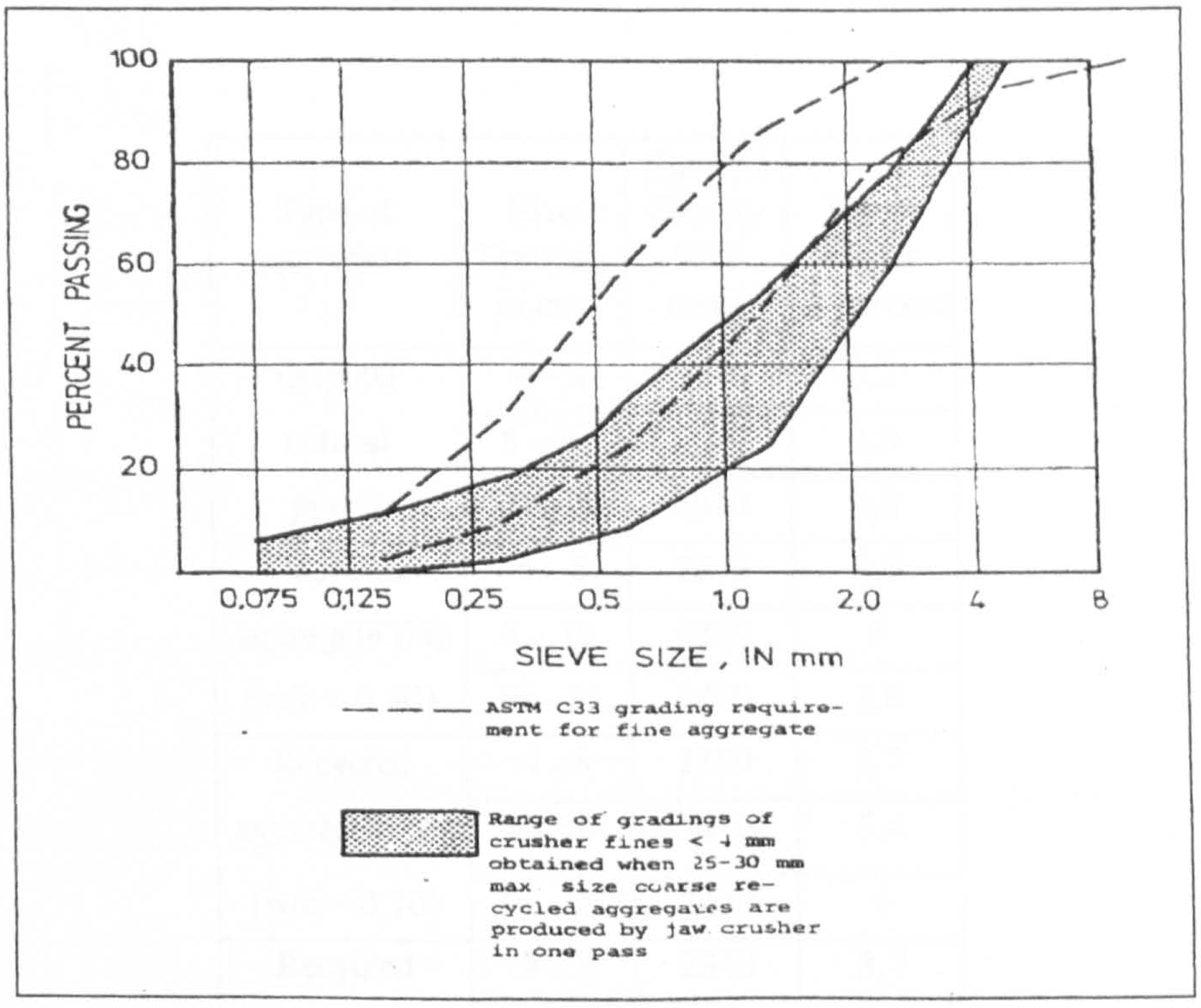


Fig. 2.1 Range of gradings of crusher fines < 4 mm (fine aggregate) obtained when 25-30 mm max. size coarse recycled aggregates are produced by jaw crusher in one pass.

They are also more angular than natural aggregate and for these two reasons the concretes which are produced exclusively with coarse and fine RA tend to be harsh and unworkable. This problem could be solved by adding some finer natural blending sand to bring the fine RA within the grading limits of ASTM C-33 and to improve concrete workability.

As regards the quantity of RA finer than 75 μm it depends on the quality of the original concrete but it may be concluded that RA in most cases can be used for production of concrete without being washed.

Density

The density of recycled aggregates is somewhat lower than the density of original aggregates due to a relatively low density of the old mortar which is attached to original aggregate particles (see Tab.2.1).

Type of aggregate	Size Fraction in mm	Specific Gravity SSD cond.	Water Absorb. in percent
Original natural gravel	4 - 8	2500	3,7
	8 - 16	2620	1,8
	16 - 32	2610	0,8
Recycled aggregate (H) (w/c = 0,40)	4 - 8	2340	8,5
	8 - 16	2450	5
	16 - 32	2490	3,8
Recycled aggregate (M) (w/c = 0,70)	4 - 8	2350	8,7
	8 - 16	2440	5,4
	16 - 32	2480	4
Recycled aggregate (L) (w/c = 1,20)	4 - 8	2340	8,7
	8 - 16	2420	5,7
	16 - 32	2490	3,7
Recycled aggregate (M) (w/c = 0,70)	< 5	2280	9,8

Tab. 2.1 Properties of natural gravel and recycled aggregates according to [6].

This can be explained by the fact that the recycling process causes lots of diffuse microcracks in the old cement paste.

Additionally the variation in w/c ratios of the original mortars give a different strength between the cement paste and the original aggregate.

Water absorption

The water absorption of coarse RA is much higher than the water absorption of original aggregates. This is due to the higher water absorption of old mortar attached to original aggregate particles (see Tab.2.2 and Tab.2.3).

Type of Aggregate	Density (SSD) Kg/m^3	Water Absorption %	B.S. Aggregate Crushing Value in percent	B.S. 10% Finess Value, kN	Sodium Sulphate Sondness % Loss	Content of Old Mortar Vol. %
15 mm max size natural gravel	2700	1.14	-	-	-	-
25 mm max size recycled w/c = 0.42	2430	6.76	23.0	133	23.9	38.4
25 mm max size recycled w/c = 0.53	2430	6.93	23.1	130	23.1	36.7
25 mm max size recycled w/c = 0.74	2430	7.02	24.6	113	28.6	35.5
Unspec. fine recycled aggregate < 5 mm	2310	10.9	-	-	-	-

Tab. 2.2 Properties of natural gravel and recycled aggregates, from [7].

Water/ Cement	Size of Fraction in mm	Density in kg/m^3	Water Absorption in Percent
0,4	4 - 8	2036	17
	8 - 16	2060	17
	16 -32	2148	15,6
0,7	4 - 8	2041	17
	8 - 16	2060	16,2
	16 -32	2091	15,8
1,2	4 - 8	2070	16,5
	8 - 16	2068	16,6
	16 -32	2081	16,5

Tab. 2.3 SSD-densities and water absorption of original mortars referring to recycled aggregates in Tab. 2.1[6].

The Building Contractors Society of Japan (BCSJ) [7] recommends that RA should not be used for concrete production when water absorption is more than 7% for coarse aggregate and more than 13% for fine aggregate. It suggests using pre-soaked aggregates for production of RAC in order to maintain uniform quality during concrete production (see Fig.2.2).

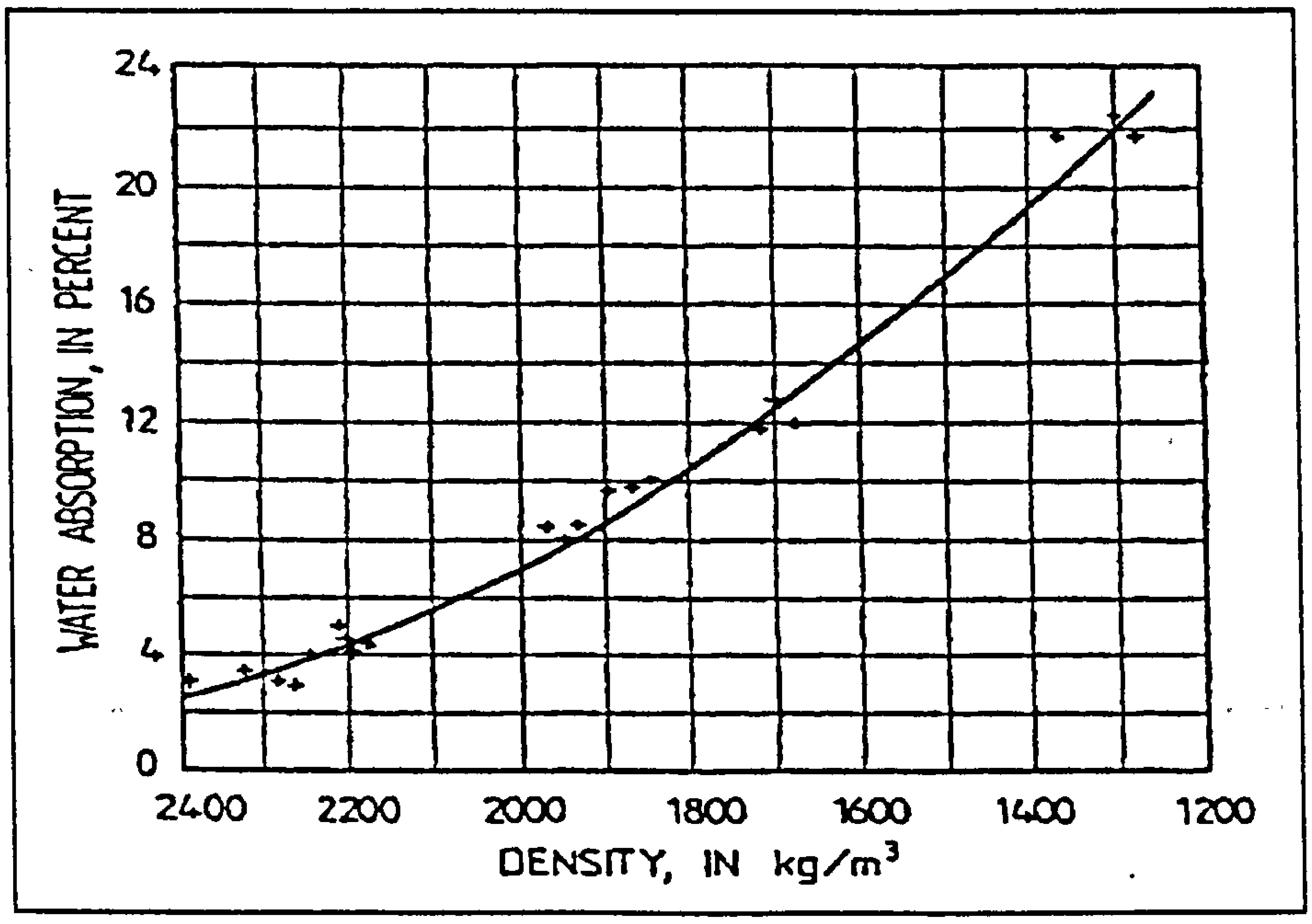


Fig. 2.2 Water absorption as a function of density of recycled concrete aggregate, from [8].

Regarding the density and water absorption of RA, it will be shown in the next Chapters that in order to attain a good mix design, it is very important to determine the correct *water absorption* value of the recycled aggregate.

This extra water can influence markedly the workability of the concrete. In fact it is very difficult to assess accurately the surface dry condition in an aggregate containing such fine material.

During this research trial mixes containing fine and coarse RA with water absorption higher than the previous limits given by B.C.S.J. [7] have been investigated. It is also important to obtain a correct value for the relative density of the aggregate since this has been shown to have a significant effect on the mix proportion and thus the compression strength.

It is apparent that the fine fraction absorbs appreciably more water than the coarse. Pre-soaking the recycled aggregate seems to have marked effect on workability during the time as reported in the test performed in Chapter Four.

Contaminants

Tab.2.4 reports the volume percentage of six contaminants which, when added to the aggregate, gave a 15% reduction of compressive strength compared to the control concrete.

Impurities	Lime Plaster	Soil	Wood Japanese cypress	Hydrated Gypsum	Asphalt	Paint Made of Vinyl Acetate.
Volume % of aggregate	7	5	4	3	2	0.2

Tab. 2.4 Volume percentages of impurities which gave 15% reduction of compressive strength compared control concretes, from [10].

Tab.2.5 shows the limiting values of injurious impurities contained in RA proposed by the B.C.S.J. [8].

Type of aggregate	Plasters, Clay Lumps and other Impurities of Density < 1950 kg/m^3	Asphalt,Plastics,Paints,Paper, Wood, and similar material particles retained on a 1.2 mm sieve. Also other Impurities of Density < 1200 kg/m^3
Recycled Coarse	10 kg/m^3	2 kg/m^3
Recycled Fine	10 kg/m^3	2 kg/m^3

Tab. 2.5 Maximum allowable amounts of injurious impurities, from [8].

2.2.2 Mechanical properties of recycled aggregate concrete

Compressive strength and rate of strength development

The compressive strength of RAC depends on the strength of the original concrete and is largely controlled by a combination of the water-cement ratio of the original concrete and the water-cement ratio of the recycled concrete other things being equal.

The literature suggests that the compressive strength of RAC is reduced by 5% compared with the compressive strength of the corresponding control conventional concretes.

The following Tables and Figures report the results of some of these studies showing the variation of compressive strength as a function of water-cement ratio, mix design of RA and natural sand etc.(see Tab.2.6a and Tab.2.6b; see Tab.2.7; Fig.2.3 and Fig.2.4).

	Compressive Strength of Original and Recycled Aggregate Concrete, in MPa											
Series	H	H/H	H/M	H/L	M	M/H	M/M	M/L	L	L/H	L/M	L/L
1	56.4	61.2	49.3	34.6	34.4	35.1	33.0	26.9	13.8	14.8	14.5	13.4
2	61.2	60.7			36.0		36.2		14.5			13.6
3	58.5	60.6			33.2		36.0		15.0			12.8

Tab. 2.6 a Compressive strength in MPa of original and recycled aggregate concrete made with natural sand and coarse recycled aggregate after 38 days of accelerated curing, from [6] as reported in [3].

Symbols H, M and L indicate original high-strength, medium strength and low strength concretes made with natural gravel.

Symbols H/M indicates a high-strength, recycled concrete made with coarse aggregate produced from medium strength concrete, etc.,

		Compressive Strength of Original and Recycled Aggregate Concrete, in MPa											
Series	Curing time	H	H/H	H/M	H/L	M	M/H	M/M	M/L	L	L/H	L/M	L/L
4	14 days in water at 20 C	49.5	37.3	33.6	33.7	23.9	16.1	17.2	19.1	9.7	5.5	4.5	6.8
5	204 days in water at 20 C	56.1	51.4	45.7	38.9	38.9	24.9	25.8	24.3	17.0	9.3	6.8	10.3

Tab. 2.6 b Compressive strength in MPa of original and recycled aggregate concretes made with both fine and coarse recycled aggregate, from [6] as reported in [3].

w/c	Compressive Strength of concretes (Mpa)			
	Natural coarse and fine aggregate (original concrete)	Recycled coarse aggregate and 100% natural sand	Recycled coarse aggregate, 50% recycled fine aggregate, and 50% natural sand	Recycled coarse aggregate and 100% recycled fine aggregate
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

Tab. 2.7 Compressive strength of original and recycled aggregate concretes made from the same original concretes using recycled coarse aggregate and various proportions of recycled fine aggregate and natural sand, from [11].

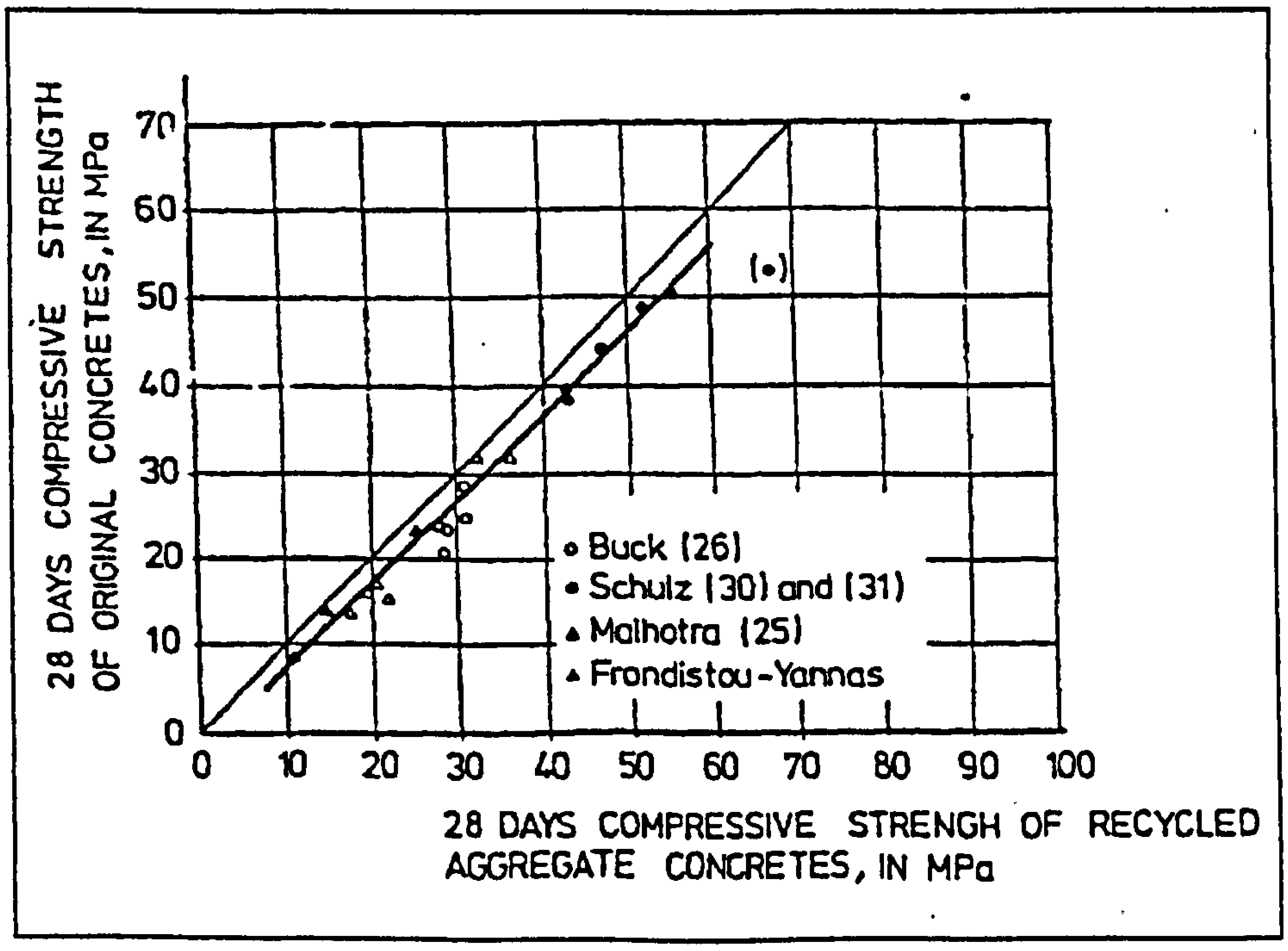


Fig. 2.3 Compressive strengths of recycled aggregate concretes as a function of the strength of original concretes, from [12].

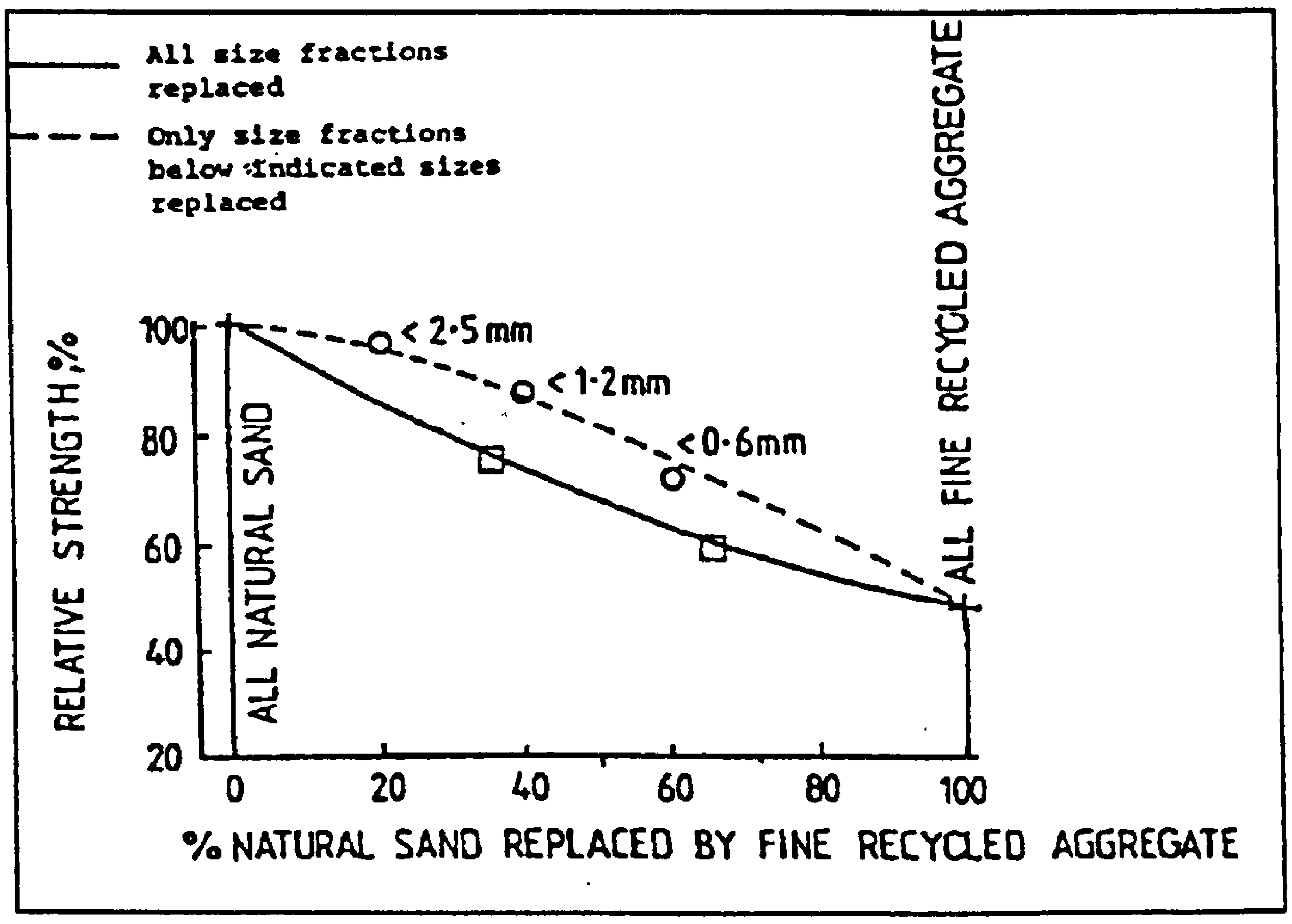


Fig. 2.4 Compressive strengths of recycled aggregate concretes made with a water-cement ratio of 0.65 where various volume percentages of natural sand were replaced by fine recycled aggregate, from [13].

However, we have to say, that Japanese researchers [14], [15], [16], [17] and [18] agree that up to 30 percent of natural aggregate can be replaced by recycled concrete aggregate without significantly changing the properties of new concrete compared to corresponding control concretes made with natural aggregates.

Already some countries allow the use of up to 20% of Recycling Aggregate in a normal concrete mix [19].

Modulus of elasticity

The modulus of elasticity of RAC is always lower than that of corresponding control concretes made with conventional aggregates. This is due to the large amount of old mortar with a comparatively low modulus of elasticity which is attached to original aggregate particles in RA (see Fig.2.5; see Tab.2.8).

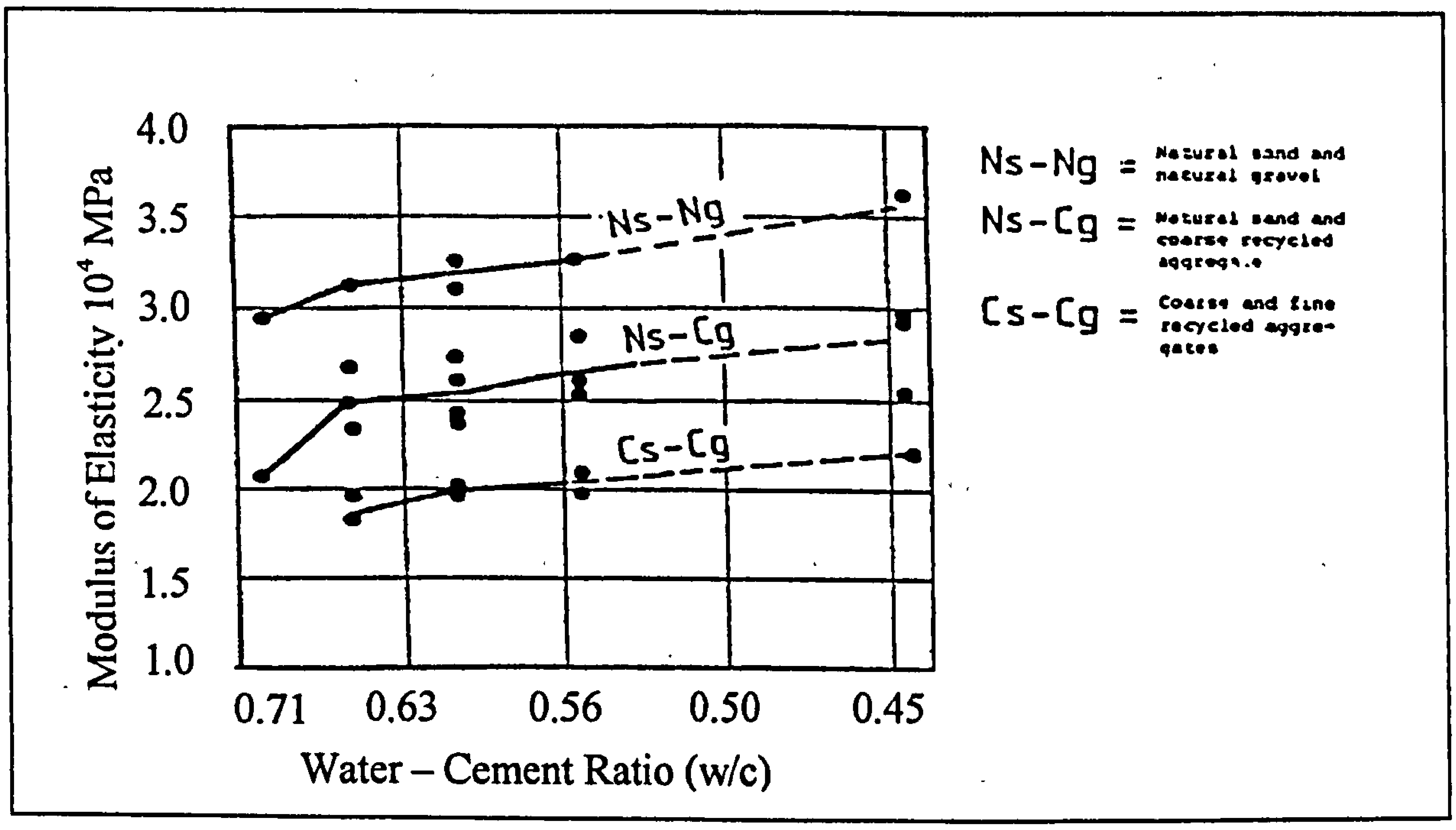


Fig. 2.5 Modulus of elasticity as a function of water-cement ratio of original and recycled aggregate concretes, from [7].

Type	Modulus of Elasticity of original and recycled aggregate concretes, 10^3 MPa											
	H	H/H	H/M	H/L	M	M/H	M/M	M/L	L	L/H	L/M	L/L
Dynamic Modulus	46,7	40,3	37,6	39,1	42,3	36,4	35,8	35	36,6	31	28,8	28
% Reduction below controls	0	13,7	19,5	16,3	0	13,9	15,4	17,2	0	15,3	21,3	23,4
Static Modulus	43,4	37	36,3	34,8	34,8	33	32	30	30,8	27,5	22,3	22,6
% Reduction below controls	0	14,7	16,4	19,8	19,8	14,3	16,9	22,1	0	10,7	27,6	26,6

Tab. 2.8 Static and dynamic modulus of elasticity of original and recycled aggregate concretes after 47 days of accelerated curing from [20].

Symbols H, M and L indicate original high-strength, medium-strength and low strength concretes made with natural gravel. Symbols H/M indicates a high strength recycled concrete made with coarse aggregate produced from medium-strength concrete, etc., A minimum value for the modulus of elasticity of RAC (Ec) to be used in the design of structures made from such concrete can be calculated from Eq. 1 [12] when the compressive strength of the RAC (fc) and the density γ of the concrete is known:

$$E_c = 2.1 \times 10^5 \times \left(\frac{\gamma}{2.3}\right)^{1.5} \times \sqrt{\frac{f_c}{200}} \qquad \text{Eq. 1 (metric units)}$$

Creep

Research shows that the creep of RAC, made with recycled coarse aggregate and natural sand, is from 20% to 60% higher than creep of corresponding control concretes. (see Fig.2.6).

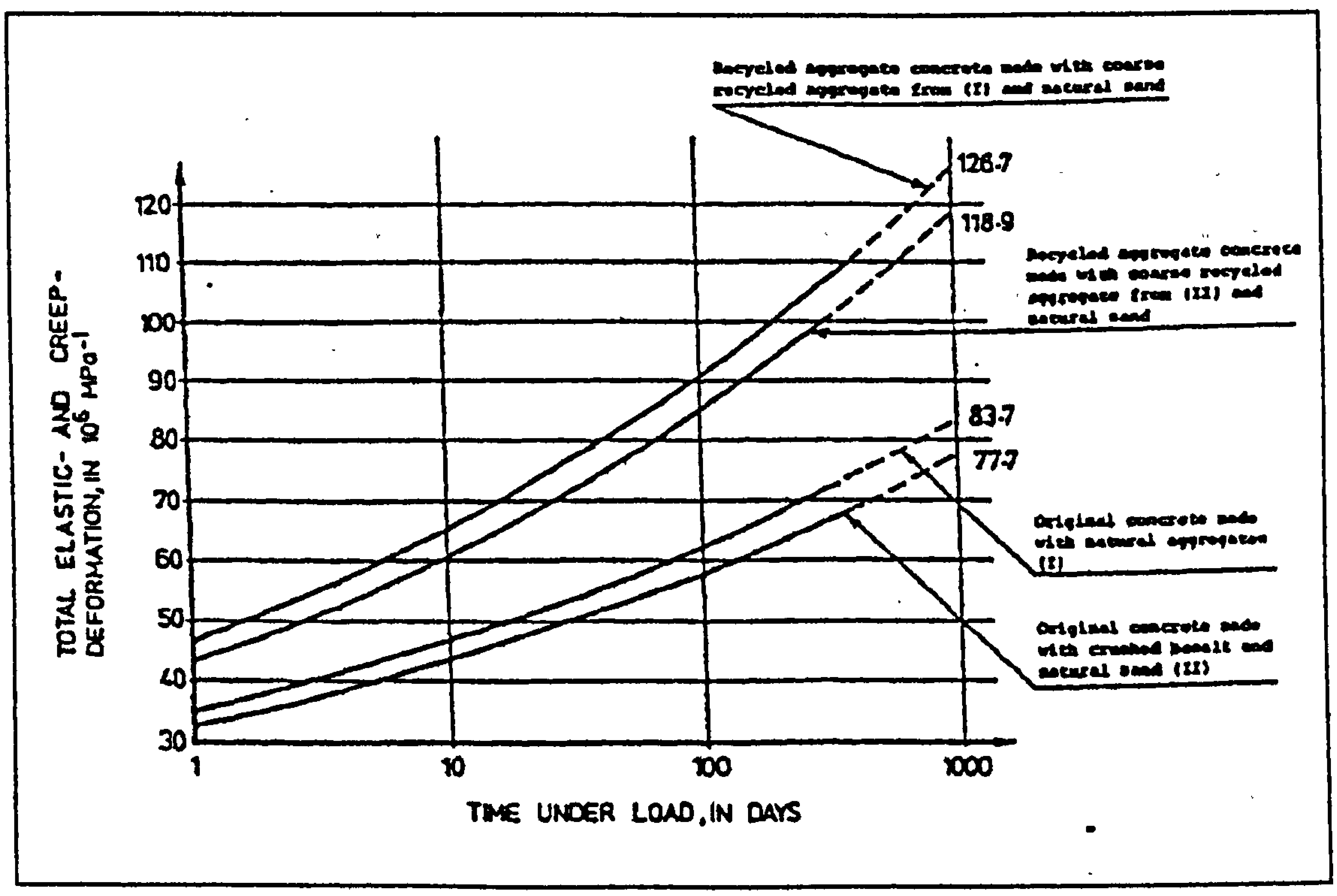


Fig. 2.6 Total deformation of original and recycled concretes (per Mpa) versus time under load, from [12] as reported in [3].

It is expected that the creep of recycled concrete will be much larger if such concretes are produced with both fine and coarse RA, but more research is needed to confirm this expectation.

Drying shrinkage

Drying shrinkage of RAC made with coarse RA and natural sand is approximately 50% higher than shrinkage of corresponding control concretes made with conventional aggregate.

When both coarse and fine aggregates are used, drying shrinkage of RAC is somewhat higher (perhaps 70%) [7], than shrinkage of corresponding control concretes made entirely with conventional aggregates (see Fig.2.7 and Tab.2.9).

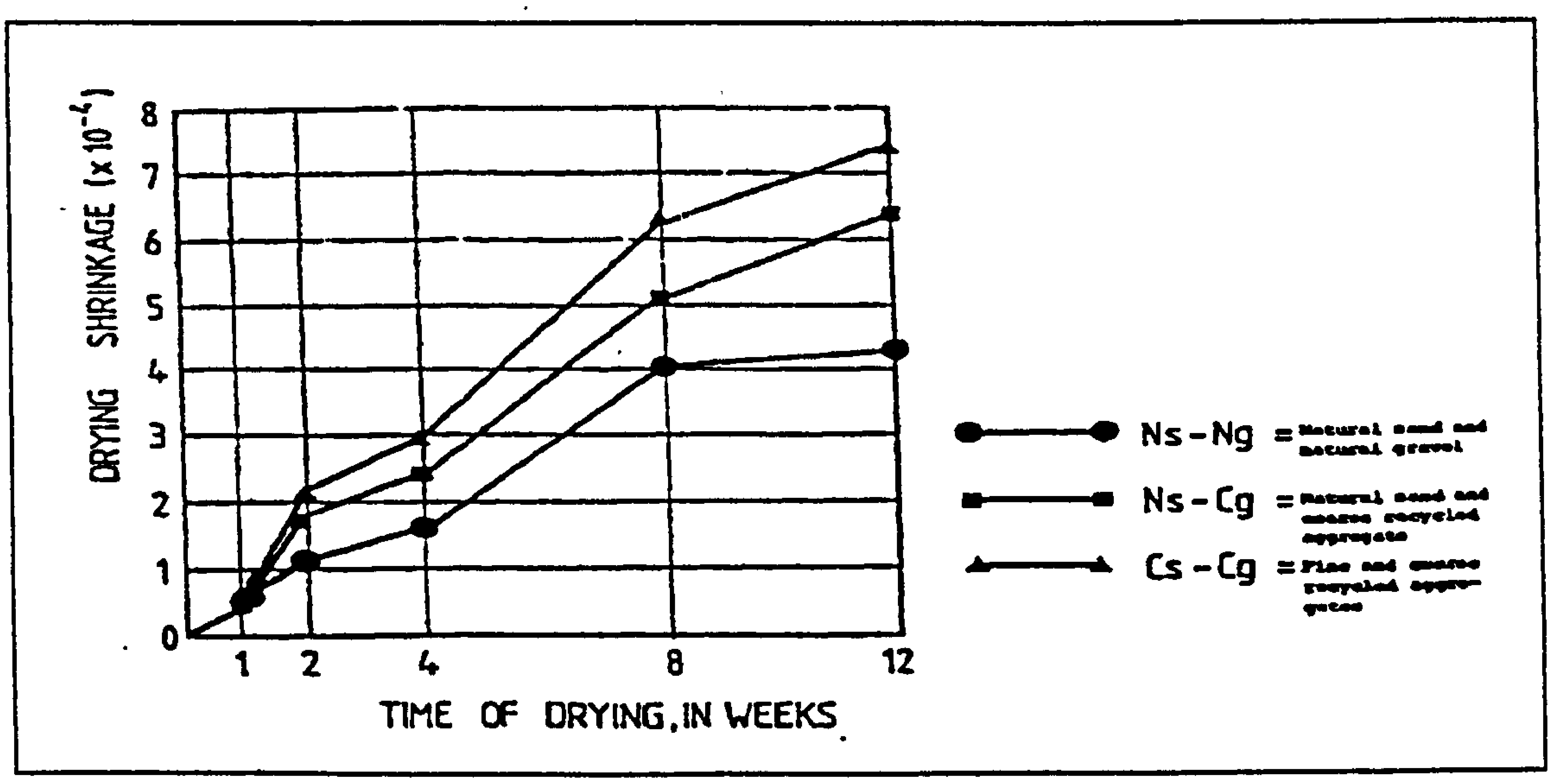


Fig. 2.7 Drying shrinkage of original and recycled aggregate concretes as a function of time of drying, from [7].

Item	Shrinkage after 13 weeks of drying at 40% RH and 25 °C of original and recycled aggregate concretes											
	H	H/H	H/M	H/L	M	M/H	M/M	M/L	L	L/H	L/M	L/L
Total shrinkage x 10^{-4}	3,4	5,1	4,9	5,3	3,5	4,9	5,3	5,2	4,5	6,8	5,7	6,8
% increase in shrinkage above controls	0	50	44	56	0	40	51	49	0	51	27	51

Tab. 2.9 Shrinkage after drying for 13 weeks at 40 per cent RH and 25 °C of original and recycled aggregate concretes from [20].

2.2.3 <u>Durability of recycled aggregate concrete</u>

Permeability and water absorption

It may be inferred from Fig.2.8 that there may be no significant difference between the water absorption (and thus presumably no significant difference in permeability) of RAC and corresponding control concretes when both concretes have water-cement ratios higher (and therefore lower compressive strengths) than those of the original concrete from which the RA is derived.

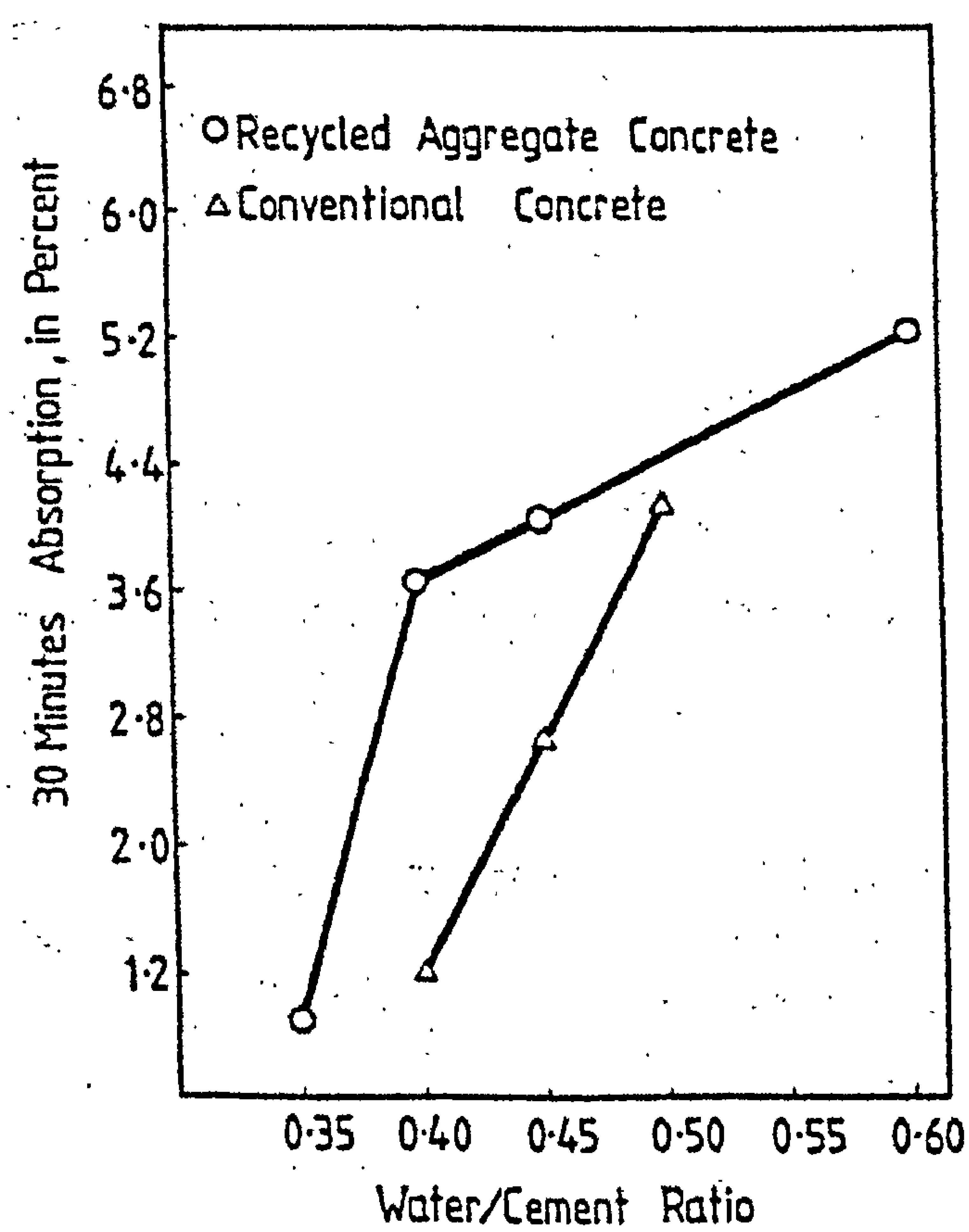


Fig. 2.8 30 minutes water absorption for recycled aggregate concretes and conventional concretes made with different water-cement ratios, from [21].

In Fig 2.8 all concretes were made with recycled aggregates from original concrete with a water-cement ratio of approximately 0.55.

However, the situation is different when both RAC and corresponding control concretes are produced with water-cement ratios lower (and therefore higher compressive strengths) than those of the original concrete from which the RA is derived.

In this case, water absorption (and thus presumably permeability) of RAC may be up to three times that of corresponding conventional concretes made with natural aggregate. This is not surprising when considering that such RAC contains a large volume fraction of more porous coarse RA which is distributed in a relatively dense matrix, while control concretes contain original coarse and comparatively dense natural aggregate in the same relatively dense matrix.

Frost resistance

Fig 2.9 shows the results of Hasaba *et al.* [7] on frost resistance.

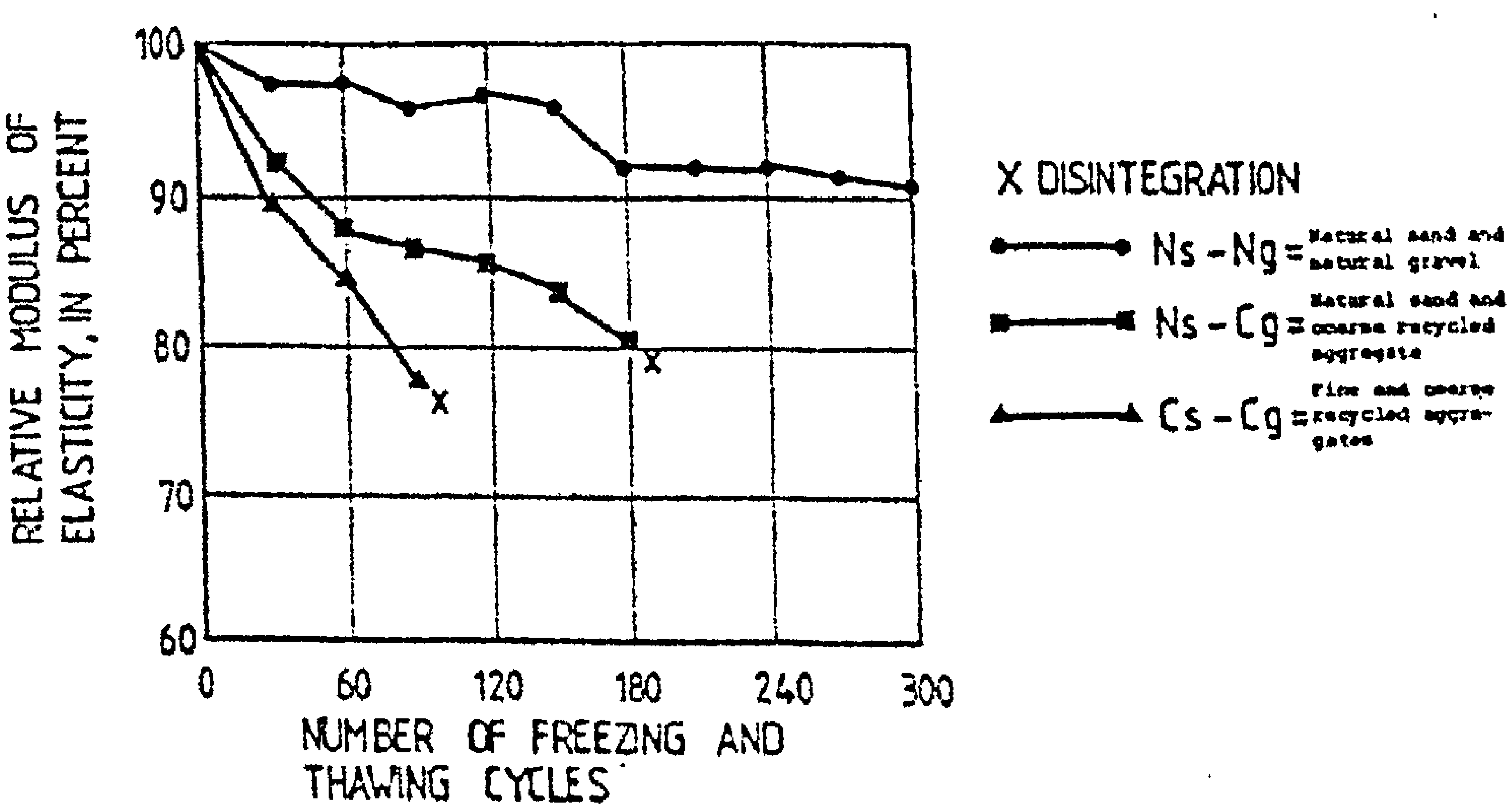


Fig. 2.9 Frost resistance of air-entrained concretes made with 30 MPa cement and natural as well as recycled aggregate, from [7].

They found the freeze-thaw resistance of air entrained concrete made with RA always to be inferior to that of control concrete made with natural sand and gravel. This was true whether RAC was made with coarse RA and natural sand or with both coarse and fine RA.

However the RAC made with both fine and coarse RA deteriorated much faster than RAC made with coarse RA and natural sand (Fig. 2.10).

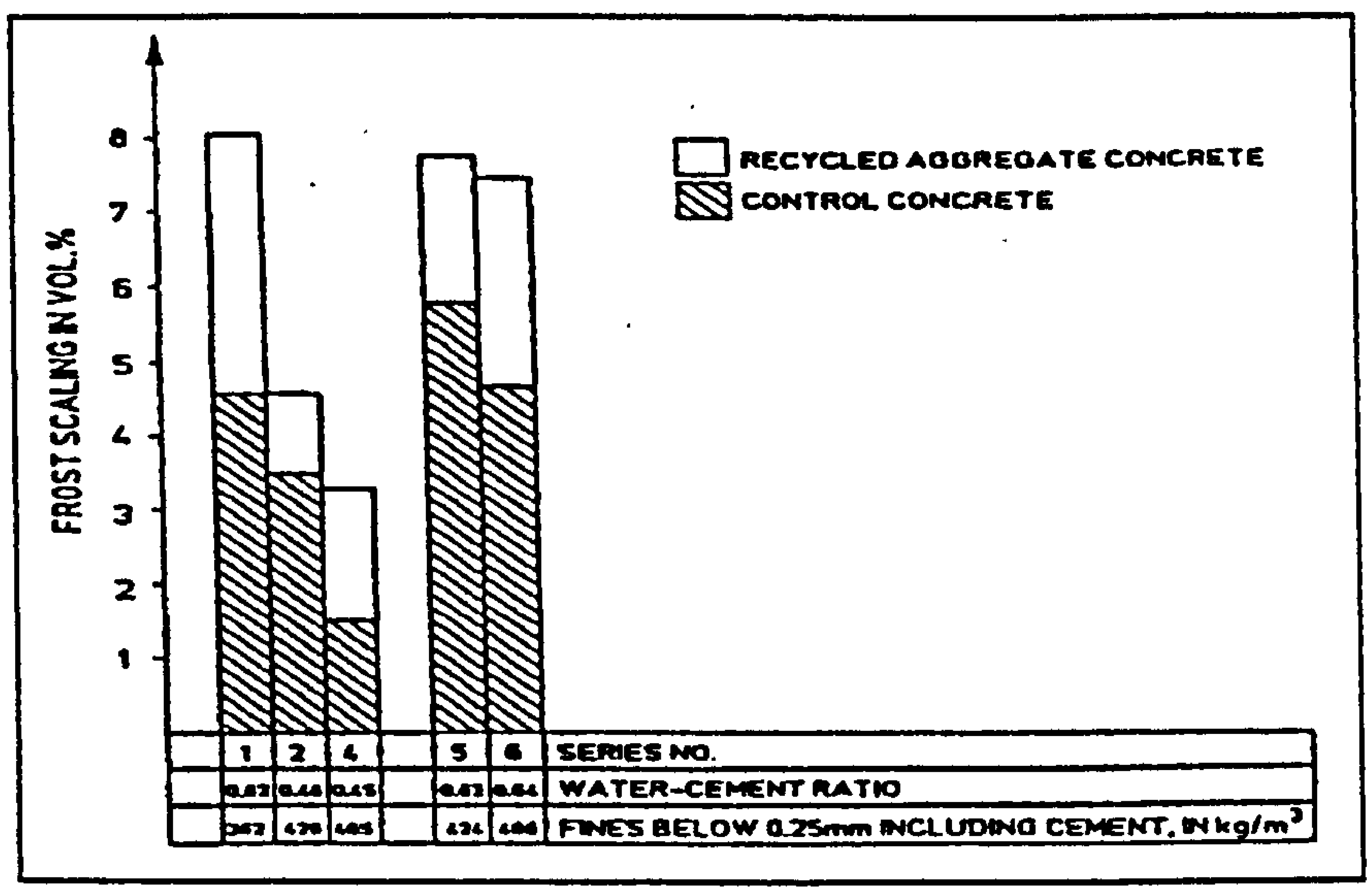


Fig. 2.10 Comparison between frost scaling of recycled concretes and control concretes when a mixture of crushed concrete and masonry was used as coarse aggregate while natural sand was used, from [22].

Carbonation and reinforcement corrosion

It will be seen from Fig. 2.11 that for the same water-cement ratio, reinforcement bars corroded slightly more in RAC than in corresponding control concretes.

However, it will also be seen from Fig.2.11 that such increased corrosion can be compensated for by producing RAC with slightly lower water-cement ratios than concretes made with conventional aggregates.

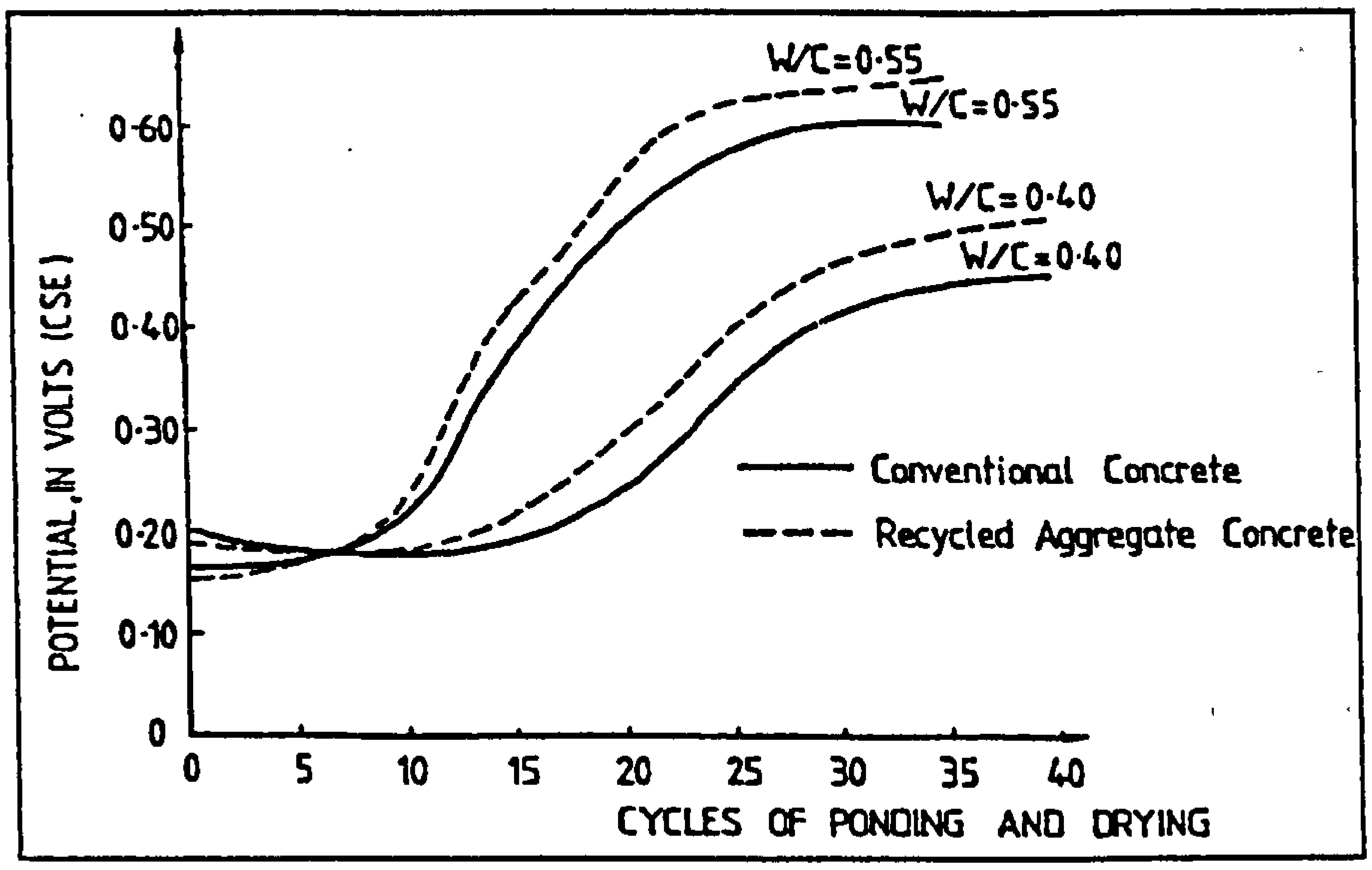


Fig. 2.11 Half cell potentials of steel bars embedded in specimens made from recycled aggregate concrete and concrete made from conventional aggregates, from [21].

Alkali-aggregate reactions

According to Foster [23] three things are necessary to cause damaging alkali-aggregate reactivity in concrete:

1. an aggregate with sufficient amounts of reactive constituents that are soluble in highly alkaline aqueous solutions;
2. enough water-soluble alkali from some source (usually the cement) to drive the pH-value of the pore liquid in the concrete up to 14-15 and hold it there so that swelling alkali-silica gel is produced;
3. sufficient water to maintain the solutions and provide moisture for the swelling of the gel.

The consequences of using recycled material, which has suffered from alkali-aggregate reactions, as an aggregate in a new concrete have not been thoroughly defined.

As there is as yet no recognised method for the testing of alkali reactivity of RA, the limitation on the total amount of alkali in concrete found in local specifications for production of concrete with reactive aggregates should be used.

2.2.4 <u>Properties and mix design of fresh recycled aggregate concrete</u>

Water requirement and workability

Many researchers confirm that recycled concretes produced with coarse RA and natural sand required approximately 10 l/m^3 or 5% more free water than control concretes produced with corresponding natural aggregate, in order to achieve the same slump, [10] as reported in [3].

Approximately 25 l/m^3 or 15% more free water was required when both fine and coarse RA were used.

Thus in actual concrete production it may be necessary to pre-soak RA to avoid rapid slump loss and early setting of the fresh RAC. This is conveniently done by immersing the aggregates in water for one hour prior to mixing. However, as far as the compressive strength of the hardened RAC is concerned, there is no significant difference whether the concrete is produced with the aggregate in air-dry or saturated surface dry condition, provided the two concretes are produced with the same free water-cement ratios allowing for full absorption of the recycled aggregates.

Air-dry coarse RAs apparently saturate themselves with mixing water from the fresh mix within the first 15 minutes. This is probably what causes rapid slump loss and early setting of the fresh concrete, but, in my opinion, it does not affect the compressive strength or the other properties of the hardened concrete.

Free water-cement ratio law

The basic water-cement ratio law, which is fundamental to all concrete mix design, applies without modification to all types of recycled concretes.

The strength may in some cases be lower for RAC than for conventional concrete.

It will be seen from Fig. 2.12, Mukai *et al.* [10] found an excellent straight-line relationship between the ratio of cement to free water and compressive and tensile strengths of RAC made with coarse RA and natural sand. The same is true for RAC made with coarse and fine RA.

Cement content

RAC, except for no-fines concrete made with RA, always requires more cement than conventional concrete for equivalent strength and slump.

It may also be concluded that it is uneconomical in terms of cement consumption to use fine RA in concrete production.

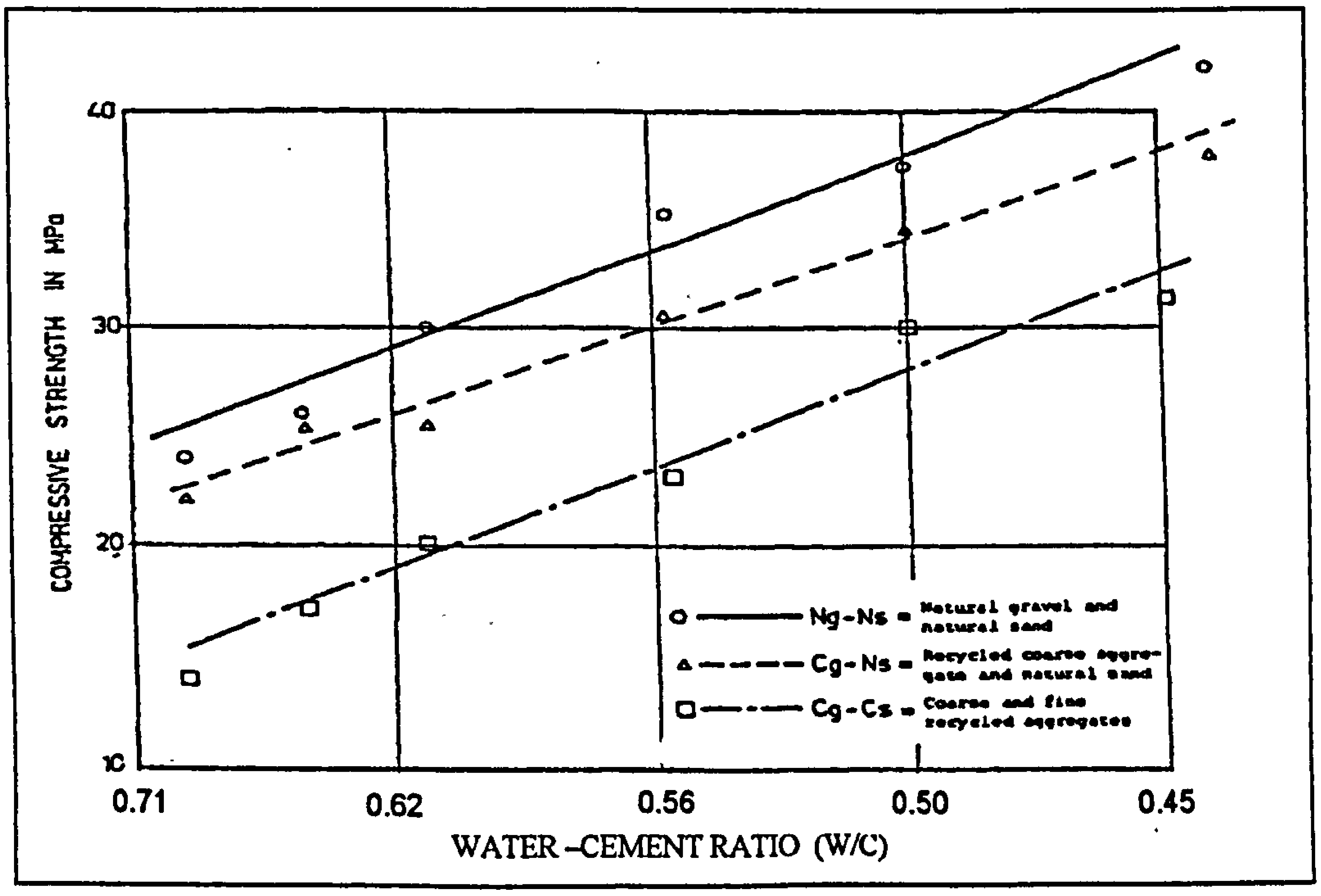


Fig. 2.12 Relationship between cement-water ratio and compressive strength of concretes made with natural and recycled aggregates, from [10].

Mix design of recycled aggregate concrete mixes

In principle, mix design of RAC is no different from mix design of conventional concrete, and the same mix design methods can be used. In practice slight modifications are required [3].

2.3 Previous investigations on RAC in the University of Strathclyde and in the University of Rome "La Sapienza"

2.3.1 The R.O.S.E. Recycled Aggregate

R.O.S.E. (Recupero Omogeneizzato Scarti Edilizia that means Building Industry Waste Homogenised Recovery) is a recycled aggregate produced in Italy by the company Pescale S.p.A.. This material has been obtained from the demolition of reinforced concrete structures and it has been used in all the laboratory work described in this thesis.

The properties of this material will be explained in Chapter 3 during the description of the specific tests. A summary of the results of a previous investigation on R.O.S.E. carried out in the University of Rome " La Sapienza " and in the University of Strathclyde follows.

2.3.2 Degree thesis on RAC in Rome in 1992

In this work [24] the behaviour of RAC made with R.O.S.E. and with the addition of some chemical additives was studied.

The conclusions of this study are reported here but in chapter four and five reference will be made to it for comparisons.

The purpose of the study was the attainment of the best compromise between workability and compressive strength of concrete made with RA using admixtures.

The use of admixtures in RA is not as straightforward as for natural aggregate concrete.

The contaminants in the RA can neutralise the admixtures and this could explain some contradictory results obtained during the investigation.

The research started with the determination of the R.O.S.E.'s chemical and physical characteristics (see Tab.2.10). To gauge the quantity of admixtures ten different concretes were produced. The results of this first experimentation on specimens are presented in Tab.2.11. The Admixtures codes are reported in Figure 2.13.

	Correct grading 1993	1993 Aggregate	1992 Aggregate	1991 Aggregate	UNI 8520 standard limits		
					A	B	C
Bulk density (gr/cm^3)	2.7	2.72	2.64	2.61	-	-	-
Apparent relative density (gr/cm^3)							
grading 0/5	2.57	2.64	-	-	-	-	-
grading 5/30	2.643	2.638	-	-	-	-	-
grading 0/30	2.61	2.639	2.56	2.53	> 2.4	> 2.2	-
Relative density on a saturated surface-dried basis (gr/cm^3)							
grading 0/5	2,254	2.292	-	-	-	-	-
grading 5/30	2.391	2.44	-	-	-	-	-
Relative density on an oven-dried basis (gr/cm^3)							
grading 0/5	2.053	2.083	-	-	-	-	-
grading 5/30	2.239	2.318	-	-	-	-	-
Water absorption (%)							
grading 0/5	9.81	10.06	-	-	< 5	< 10	-
grading 5/30	6.82	5.21	2.5	-	< 5	< 10	-
Porosity (%)	3.43	2.97	3.03	3.1	-	-	-
Passing to 0.075 μm (%)	9.4	4.71	10	6.8	< 4	< 6	< 6
Los Angeles Test (%)	32.6	32.4	26	23.2	< 30	< 40	-
Equivalent in sand (%) *	57.7	-	68.8	64	> 80	70÷80	70
Alkaline sulphate (%)	0.815	0.7	0.8	0.815	< 0.2	< 0.2	< 0.2

* This test will be described in detail in Chapter Three.

Tab. 2.10 Calcestruzzo R.O.S.E.. Grading 0/30. Geotechnical characteristics (chemical-physical), from [24].

Mix Code	Admixtures		Real w/c	Theoretical w/c	Mixing time	Slump			Concrete density	Compressive Strength		
						max	after t minutes					
	Type	Quantity				cm	cm	min	kg / m³	7 days Mpa	28 days Mpa	60 days Mpa
C1	-	-	0.7	0.324	3.5	dry	-	-	-	1.8	-	-
C2	-	-	0.88	-	14.5	-	0.5	-	-	14.5	-	-
C3	-	-	0.9	0.566	13	10	3.5	11.5	-	16.9	23.4	28.0
C4	E	1	0.85	0.507	27	1	1	26	2144	21.4	-	-
C5	E	0.6	0.9	0.566	34	15.1	2.5	32.5	2084	12.5	18.7	23.3
C6	E	2.5	0.85	0.507	26	15	3	24.5	2122	18.3	29.1	-
C7	-	-	1.51	1.08	32	15.5	15	30.5	1997	41.2	71.1	-
C8	-	-	0.9	0.566	67	-	dry	-	2135	19.6	27.2	-
C9	A, F	A=2 F=0.4	0.9	0.566	46	13.5	3	44.5	2111	17.7	25.5	-
C10	A, F	A=2 F=0.4	0.9	0.566	46.5	> 23	3	45	2121	18.1	24.3	-

Tab. 2.11 Calcestruzzo R.O.S.E.. Mix characteristics. Mixes C1 – C10, from [24].

A	Water reducing
B	Retarding
C	Accelerating
D	Water reducing and retarding
E	Water reducing and accelerating
F	High-range water reducing and superplasticizing
PTL 425	Ordinary Portland Cement with a cement strength class of 42.5 MPa
Pre-blending 52.5 Cement	Superplasticizing

Fig. 2.13 Admixtures and cement codes.

Tab.2.12 shows the results of the second part of the investigation carried out on specimens coming from eleven different concretes.

This has led to the following conclusions [24]:

1. Admixture type A exhibited strong flowing power but the concrete did not retain its workability for as long.
2. The admixture type E did not seem to influence the workability of the fresh concrete.

Mix Code	Real w/c	Admixtures 1		Admixtures 2		Cement type	Concrete density at 28 days	Compressive Strength	
		Type	Quantity l	Type	Quantity l		kg / m³	7 days Mpa	28 days Mpa
C11	0.5	A	2	F	0.4	PTL 425	2086	14.0	23.6
C12	0.42	A	3	F	0.5	PTL 425	2219	24.6	36.7
C13	0.46	A	3	F	0.5	PTL 425	2188	23.5	34.2
C14	0.5	A	3	F	0.5	PTL 425	2174	22.7	32.1
C15	0.5	F	0.5	-	-	525 Pre B.	2205	35.3	42.8
C16	0.7	F	0.5	-	-	525 Pre B.	2205	33.5	41.6
C17	0.56	-	-	-	-	PTL 425	2150	19.7	27.6
C18	0.56	E	0.8	-	-	PTL 425	2148	20.6	29.2
C19	0.51	B	1.2	-	-	PTL 425	2123	17.6	23.7
C19 bis	0.5	B	1.2	-	-	PTL 425	2131	22.4	29.6
C19 bis t.	0.5	B	1.2	-	-	PTL 425 test	2140	26.2	36.0
C20	0.56	C	10	-	-	PTL 425	2187	24.8	36.6
C21	0.65	F	0.5	-	-	525 Pre B.	2018	20.6	26.2
C22	0.5	-	-	-	-	525 Pre B.	2056	23.1	30.0

Tab. 2.12 Calcestruzzo R.O.S.E.. Mixtures' characteristics. Mixes C 11 – C 22, from [24].

3. Admixture type B exhibited higher flowing power than the additive type A, consequently it was decided to use Type B with half the dosage of type A (see Tab. 2.12).
4. Additive type C increased the initial workability and maintained it for a long time both in the fresh and hardened concrete (increasing the compressive strengths). Additive type C, includes amorphous silica with its high pozzolanic activity which neutralises the sulphate contaminants. This restores the RA to its full potential at least for negligible amounts of sulphate contaminants.

   These observations seem to confirm the postulations of Samarai [25] regarding the best behavior of the RA in concrete made with pozzolanic cement.
5. The best performances came from the use of the cement type 525 pre-blending.

The reasons for the success of this type of cement are clear. As well as its greater strength it gives a flow of the mixture in parallel with the hydration process of the cement granules, allowing the full flowing potentiality of the admixtures contained in the cement.

It is this gradual process of flowing that limits greatly the possibility of the admixture's absorption from its inert state and this makes more efficacious the whole process of the cement's dispersion in the water.

6. The values of the Brazilian traction strength, on cylindrical specimens (D=15 cm; H=30 cm) of the concrete type C3, C21, C22, compared to the relative compressive strength, have provided an average value of 1/11.61. This result is lower than the value for the conventional concrete which is 1/10, as reported in [24].
7. The flexure to compressive strength ratio at 28 days of the concrete type C21 was 1/6.86 which is higher than conventional concretes where the expected value lies between 1/7.5 and 1/9.4, as reported in [24].
8. The compressive strengths of different types of RAC, relative to conventional concretes with the same w/c ratio, were in the interval of 78% to 88%. The only exception was concrete type C20 (with additive type C) in which, thanks to the high pozzolanic activity of the admixture, the ratio has grown to 105%.
9. It is logical to expect that higher compressive strength could be achieved in a precast concrete factory because of improved vibration and compaction available when the slump range would be 40 to 50 mm. In fact lower w/c ratios and quantities of admixtures could be used with consequent benefits in strength and economy.
10. The researchers also asserted that other benefits could come from replacing the fine RA with natural sand.

In conclusion the present state of Italian recycling techniques gives rise to large variations in the mechanical properties of RA in comparison with the conventional concretes.

To attenuate the above problem a better organisation of the recycling process is necessary.

2.3.3 Degree thesis on RAC in Strathclyde 1994

This thesis [26] was written by the same author as the present thesis for his graduation at the University of Rome "La Sapienza".

In this research the characteristics of the Recycled Aggregate were determined including grading, particle shape and texture, density, water absorption and analysis for chemical contamination [26].

Mixes were produced using this recycled aggregate and also blends of recycled and natural aggregate. A normal natural aggregate was also used for comparison. The properties of both the wet and hardened concretes were measured [28].

Six reinforced concrete beams (100 mm x 200 mm cross section and 2130 mm long) were made using three different aggregate types, all recycled (AR), all natural (AN) and a blend of the two (RN). These were tested and it was demonstrated that it was possible to produce satisfactory structural elements using this recycled aggregate [29], [30].

The non-linear behaviour of the beams during testing was compared with a finite element analysis using ductile failure material modelling in the LUSAS finite element package.

All the results [31] of this experimentation on R.A.C. have been used to direct the present investigation.

The detailed results can be found elsewhere [26] and this paragraph reports just the conclusions of this research.

1. The physical and chemical examination of the aggregates indicates that their density is lower and their water absorption is higher than for normal aggregates. As received the R.O.S.E. aggregate can be used as a complete aggregate but suffers from workability problems.
2. The workability characteristics can be improved by partially substituting 20mm coarse aggregate. As little as 10%-15% substitution has significant affects on the workability and the technique can be used as an alternative to the use of admixtures.
3. Concretes made with the recycled aggregates or the blended aggregates show lower strengths than concretes made with normal aggregates. There is also a maximum strength which can be achieved with these aggregates and this appears to be around 35 MPa.
4. Despite this, it would appear practicable to make reinforced concrete elements using concretes made with recycled aggregate and these elements would have satisfactory mechanical performance.

Chapter Three

Quality of Recycled Aggregate

3.1 Introduction

The process used in demolition, treatment and re-use of building waste influences markedly the quality of the Recycled Aggregate.

Thus the use of Recycled Aggregate in the production of new concrete especially in a pre-cast plant involves preliminary discussions regarding:

- the recycling processing equipment;
- the amount of waste to be processed;
- the quality of the waste to be processed;
- the practical use of RAC relative to the different types of structural products.

In this Chapter the above subjects and the processing phases that could affect the final product are discussed. The information has been obtained from a mixture of company leaflets and interviews with key personnel. The site-plant for production of R.O.S.E. Recycled Aggregate used in this research is also described.

The physical and chemical properties of the Recycled Aggregate determined during this research are presented.

3.2 Production of Recycled Aggregate

3.2.1 Demolition and collection of building waste

The processing procedure for recycling of building and demolition wastes can be mainly divided into three steps:

- demolition and collection of the material;
- processing;
- re-use;

Three different techniques for demolition of concrete structures can be identified:

- techniques using mechanical action;
- techniques using chemical reaction;
- techniques using different physical principles;

The techniques using mechanical action are:

- bumping demolition, through hammering and bumping with the use of steel balls;
- impact demolition, with hydraulic hoist and hydraulic crushers;
- abrasion demolition, using drills, diamond perforator equipment or using squirts of abrasive water;
- cutting demolition, using hydraulic clippers;
- static bursting demolition, using hydraulic jacks that induce failure into the material;

The techniques using chemical reaction are:

- blasting demolition, using explosives and it could be static or dynamic;

The techniques using different physical principles are:

- dynamic explosion demolition, using the injection of high pressurised water into prearranged hollows;
- using fire lance or laser or plasma equipment;
- heating demolition, to separate the concrete from the reinforcements using electricity or microwaves.

The range of the techniques can affect all the demolition problems but presently their evaluation does not consider environmental safeguard problems.

- Problems like noise, dust and vibrations cannot be tolerated in particular environmental situations. Furthermore the speed of execution can influence markedly the validity of techniques and the safety protection is specific for each technique.

Finally the costs are one of the most important aspects for the evaluation of a demolition method. The demolition strategies adopted affect the demolition activities and the quality of the waste products.

One of the most used strategies is so called "throw down". This operative philosophy has some very important disadvantages. It causes heterogeneous materials that, even if processed in a recycling plant, can have only a superficial separation. The light fraction (plastic material, woods, and paper) and the metal can be easily separated but the bricks, concrete, ceramics, glass and bitumen present more of a problem and thus increase costs beyond reasonable limits. If one of the non-separable materials exceeds certain fixed limits it can influence negatively the mechanical properties and the behaviour of the recycled aggregate.

Consequently the use of recycled material coming from a "throw down" demolition can be adopted only in cases that involve secondary elements.

For example the waste materials coming from a "throw down" demolition of a building made with reinforced concrete and bricks panels can be sent to the waste area or used after treatment for filling operations, environmental arrangements, road sub-structures, but not for production of new structural concrete.

For this reason the "throw down" demolition even though it is low cost involves wastage of material that in countries with serious waste disposal problems could make this demolition technique totally uneconomic.

A different strategy that aims at a better separation process has been adopted in Switzerland. During the demolition phase pneumatic or hydraulic hammers and diamond saws are used. This type of demolition is carried out in different phases, e.g. starting from the demolition of brick walls or other type of materials and afterward with the demolition of reinforced concrete structures.

The result is a waste material already separated and ready for transport.

The demolition site area should provide separate containers for different materials:

- mono-material containers in which homogeneous materials are collected, like excavation materials, concrete, metals, plastic and wood ready for the recycling process;
- multi-material containers in which mixed building demolition wastes are collected, like bricks, roof elements, cement or concrete hand-made products; this material could be sent to a processing plant or to a landfill site;
- multi-material containers for combustible waste, like wood , paper that could be sent for burning;

- multi-material containers to be filled with large material that should be separated later, like non-combustible wood, plastic material, or insulating materials;
- multi-material containers for special wastes that need specific processing procedures like contaminated building wastes.

The advantages of this demolition strategy are:

- the elimination of the preliminary selection in the processing area;
- the reduction of the transport costs (each container is sent to the specific processing site);
- the possibility of disposing of small size homogeneous material without large size contaminants.

The disadvantages are:

- the high demolition costs coming mostly from the working time; from the necessity of using specific equipment and from the numbers of workers employed;
- the need to provide large area as site for the containers;
- longer working hours which give rise to problem in urban areas;
- long distances to the processing demolition place site.

All these problems sometimes make the recycling process impractical.

3.2.2 The processing procedure

The processing procedure regards all the operations involved in separation of the different components of the demolition waste and the consequent sorting shape and dimension of the Recycled Aggregate.

During building demolition, wastes with different composition are obtained. It depends mainly on the type of the demolished structures and on the demolition strategy used.

The waste should go through a preliminary separation phase in which the principal components are separated.

The metal wastes can be successfully and economically recycled because they have a recognised market value.

The remaining fraction of the demolition waste is made by concrete waste, bricks, and similar (stone).

This represents the biggest part of the demolition wastes (more than 80%) and it controls the recycling process.

Concrete waste comes mostly from the demolition of civil engineering structures like bridges, offshore viaducts, roads, airstrip, commercial and industrial buildings, partial or total demolition of building, and from restoration of residential buildings.

The masonry wastes come mainly from the demolition of walls, roofs and from ceramic coatings.

The stone wastes come from the demolition of very old buildings, walls and similar.

Demolished structures that produce mono-material are rare apart from roads and airstrips.

In most cases there are mixed wastes containing mortar, gypsum, ceramics, light concrete and some other contaminants that make the material more heterogeneous.

Mixed waste have to be crushed and sieved to a required grading for specific use as a recycled aggregate.

The recycling process of building demolition wastes can be divided into different phases on which the working systems of the recycling plants are based.

Recycling plant for demolition waste are not so different from the plant for production of crushed natural aggregate.

The plant can be fixed or mobile and the choice depends on what is wanted from the process.

Fig. 3.1 Mobile plant for production of recycled aggregate.

The main advantages of fixed site plant are:

- the possibility of obtaining a recycled product that is sufficiently diversified and with a better quality compared with that of a mobile plant;

- the possibility to use more powerful and bigger equipment that can give more valid crusher treatments;

On the other hand fixed site plants require big investment and a large area for all the processing involved in the plant. Normally the area should be almost 50.000 m^2.

The mobile plant usually consist of a crusher and metal catcher (magnetic catcher). These plant do not allow the possibility of introducing accessories. Using these plants processing of the wastes with only 40 mm – 300 mm grading can be realised. The plant has no sieves and the recycled material cannot be separated into different grading and therefore it cannot be used for the production of new concrete. It could be used for filling, roads, etc.

The main advantages of mobile plants are:

- lower costs, almost 5% less than fixed site plant;
- less time of installation;
- reduction of costs for transport of the demolished material to the recycling plant.

The most used plants at the moment are the mobile ones.

Small companies cannot raise the necessary finance and most operators are large national companies. Consequently there is a problem aligning the source of waste with the location of the plants.

3.2.3 <u>Processing and technologies of the R.O.S.E. site-plant</u>

The R.O.S.E. site-plant (R.O.S.E. is for Recupero Omogeneizzato Scarti dell'Edilizia that means Homogeneous Building Waste Recovery) has been designed and built in Castellarano (RE) in Italy. It is an exemplary plant for treatment of recycling material of stone origin.

The originality of the plant, the effective possibility of treatment and recycling of the building demolition waste have earned it the National Award "Industry-Environment" in the Material Recycling Field from the ENEA (an Italian Government Institution) in 1990.

As shown in Fig. 3.2 the plant consists of a first phase of quality control of the incoming wastes. In fact in this phase the demolition material should be still considered as proper "waste" with possible presence of polluted or toxic substances in it.

This first control consists of a closed-circuit TV system that allows the above observation of the incoming material during the weighing operations.

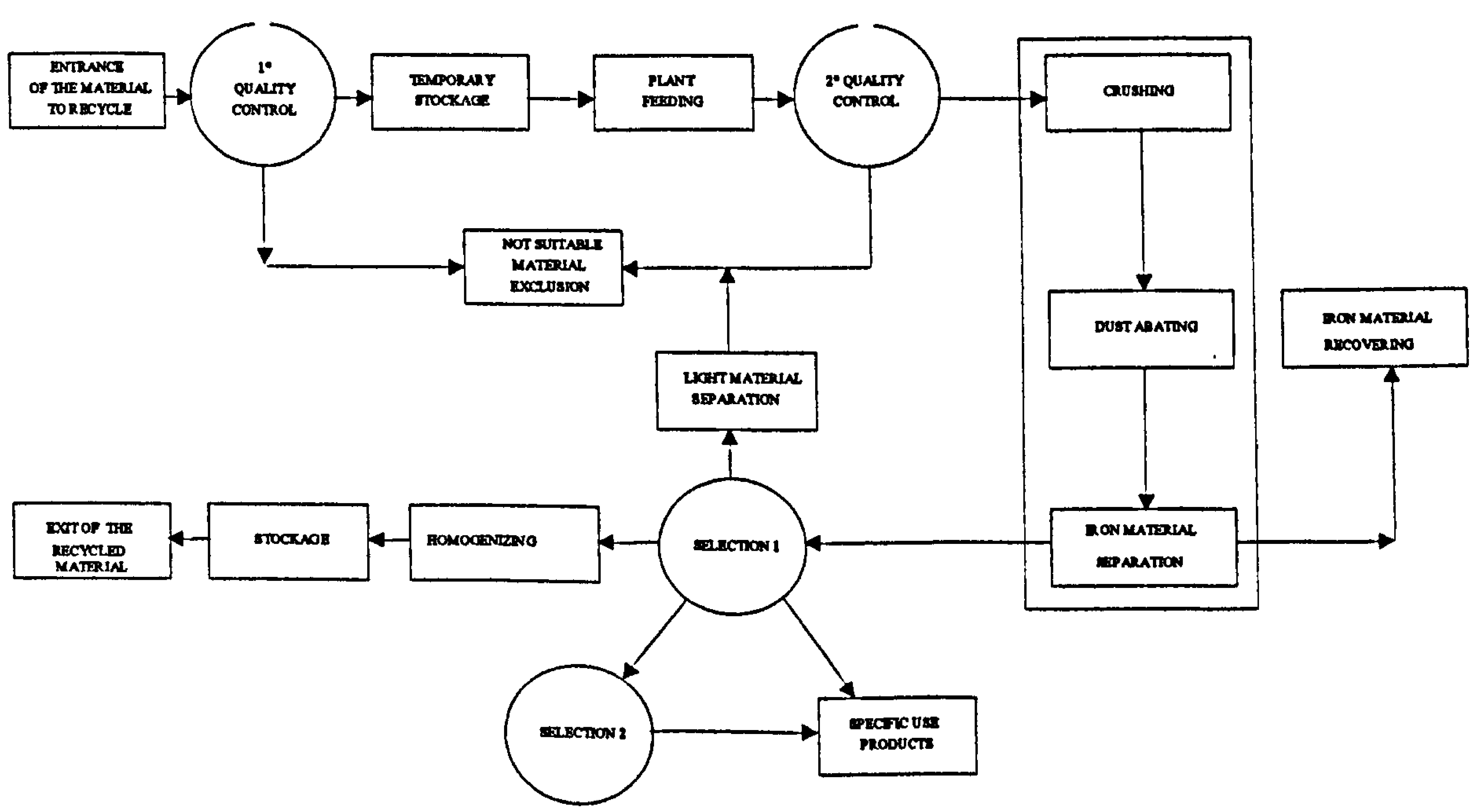


Fig. 3.2 Flow-chart of R.O.S.E. recycling plant.

After unloading in a specially equipped area, the processing plant is fed with an excavator. The loading hopper is of 20 m^3 capacity, 4.5 m top width with a feeder and an automatic variation loading capacity.

Fig. 3.3 Demolition waste from production of a pre-cast industry and stoke area of a recycling plant with hydraulic clipper for pre-treatment of the material to process.

The material is observed also in this starting phase of the treatment using a video system. During this phase the operator can stop the loading operations and see the material directly. He can also decide to use a by-pass and leave the material for a further analysis to check the quality.

The second phase is an initial sieving using a vibration-sieve that avoids the processing of the fine grading or clay fraction.

After this operation the material is sent to the mill mouth. The mill has been built to allow both the grading reduction of the waste material and the total separation of the steel reinforcement from the concrete. This is necessary to avoid damage to the mill itself.

Fig. 3.4 Loading hopper and crossing node for shunting of the material.

The pre-production test of the mill has been carried out using an HE 160 steel 800 mm long and steel bars 800 mm long and with a diameter of 90 mm. No damage has occurred during this test.

The total separation between steel reinforcement and concrete is very important because in other recycling plants only partial separation of the two materials has occurred.

Consequently a big quantity of little concrete blocks connected with steel bars is produced. This material is absolutely not usable.

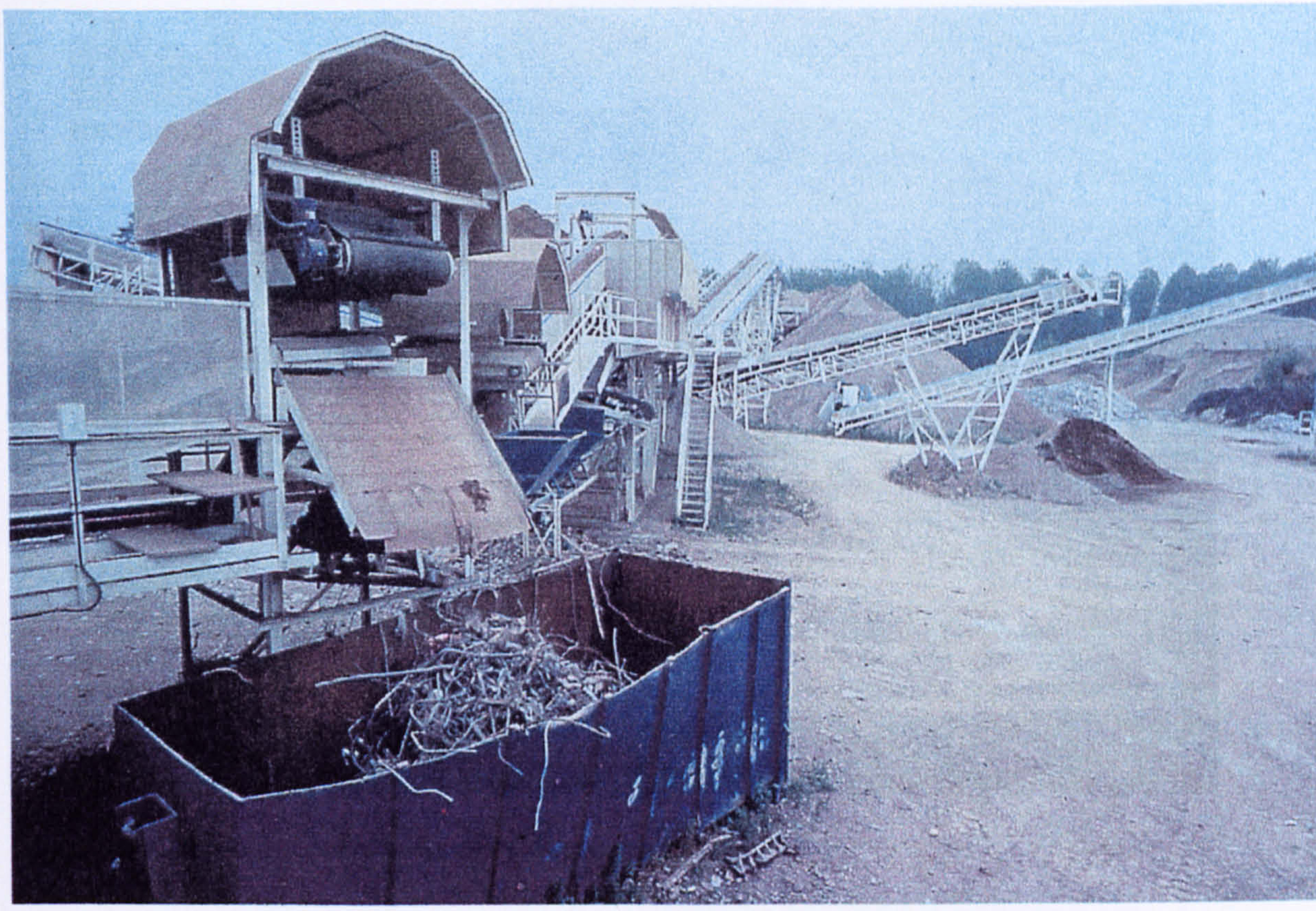


Fig. 3.5 Electric magnet for separation of steel.

Using an horizontal extractor the treated material is put on a conveyor belt that transfers the material to the first electric magnet to separate the steel and to put it in a special container.

The money that comes from selling the recycled steel is almost equal to the money spent for the electricity to run the plant.

After this operation, using another conveyor belt (on which is placed a second electric magnet), the material is sent to a two level vibro-sieve (multi-hole) that realises different grading (the standards are: 0/30, 0/70, 0/140, 30/70, 70/140, 30/140), with an optional recycling process (partial or total) for the grading over 30 mm.

The recycling process is automatic.

Fig. 3.6 Separation of paper, wood and plastic.

During this phase an automatic separation of the light fraction (paper, plastic, wood, etc...) is carried out. This is made possible by a patent system that uses the different specific weight of this material. This waste material is sent to a specific and authorised dump.

Using a by-pass conveyor belt (that is reversible) it is possible to make separate piles of recycled aggregate for special uses like for example using material from selected grading 0/6, 6/15, 15/30 for concrete production.

Using the same by-pass and another conveyor belt the recycled aggregate is sent to the stockage on a pile more then 12 metres tall placed on a prefabricated concrete reinforced tunnel.

The distribution in the tunnel is realised using a turning conveyor belt that can change the unloading height to avoid the dust production that could caused if the recycled aggregate falls from an appreciable height.

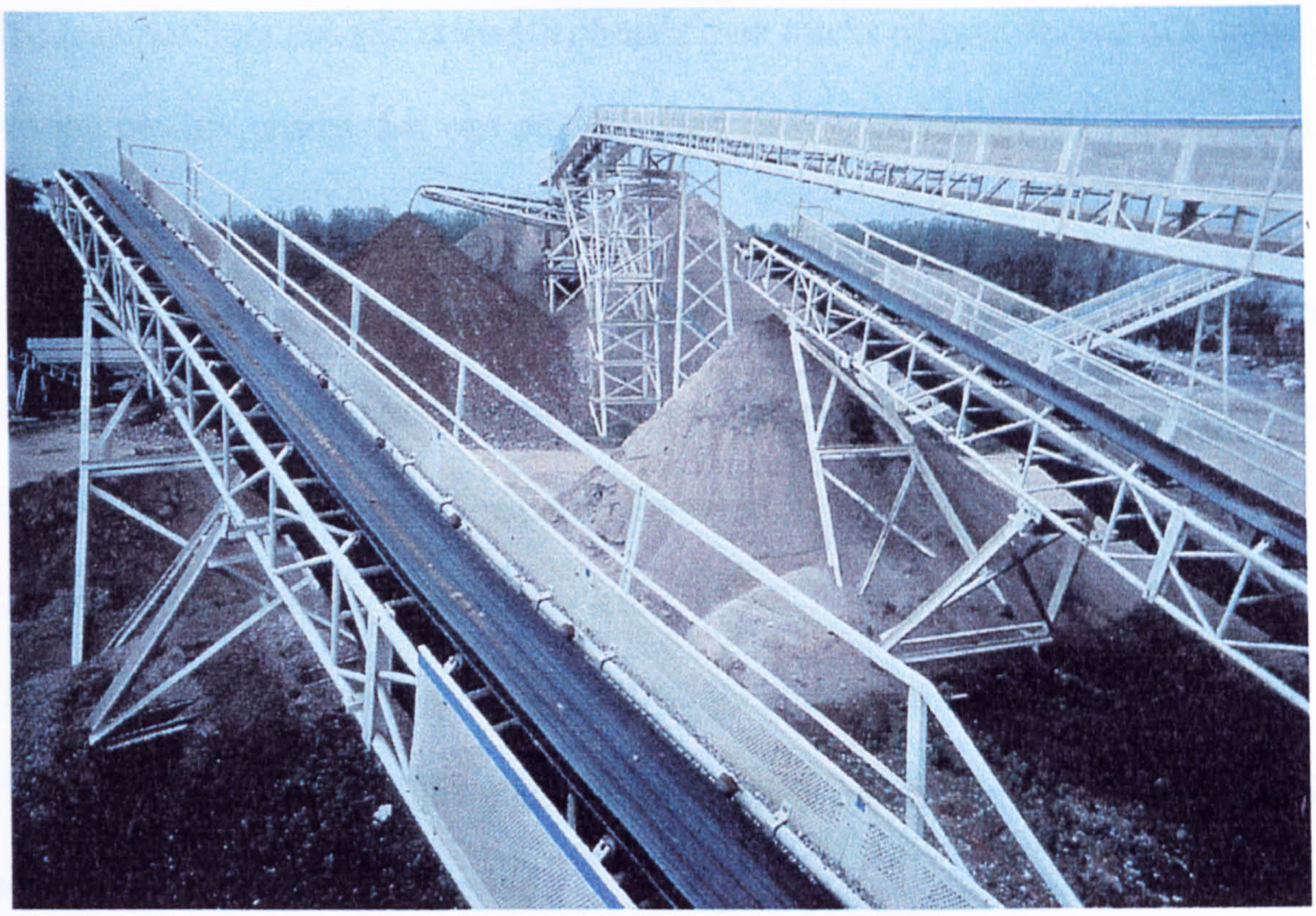


Fig. 3.7 Conveyor belt towards heaps of different grading fractions.

The tunnel is provided with five vibration suppliers that working together can guarantee the loading of the conveyor belt with a material that represents the whole grading assortment that is stoked in the tunnel.

This part of the plant represents one of the most interesting and innovative elements because it guarantees the homogeneity of the input material in the stoking silo. If the material had been simply heaped in the traditional manner, the larger particles would collect at the bottom leaving the finer particles on top. The resulting material would not be evenly graded and thus not so useful.

The consistency of the output material has been checked with the analysis of samples in different operating conditions of the plant.

The conveyor belt meets another conveyor belt that fills a storage bin with two compartments of 30 m^3 each.

The material from this silo is loaded directly onto trucks without the use of a digger.

All the process phases that can produce dust have been examined with particular attention to limit it.

In particular at the exit of the mill a patent system using water nebulization with volume expansion can eliminate the problem and recover the dust and increase the fine fraction of the material. The power involved is only 2 Hp.

This plant is completely noise controlled.

The total operation power is 90 kW (± 10%) for a production of 50 m^3/hour.

The management costs of the plant are particularly interesting because they can be amortised over a very short period.

The advanced automation of the different phases requires just two operators.

Using a digger the first one feeds the plant taking the material from the provisional stokage.

Fig. 3.8 Dust control system.

From the control room the second operator manages all the plant operations such as the filling of the mill, control of the conveyor belts, heaping of the material and the silo.

In the control room a second video is placed to check all the material in entrance in the mill.

3.3 Properties of Recycled Aggregate

In this plant there are two classes of Recycled Aggregate produced. One is called "R.O.S.E. macerie" ("macerie" means rubble) and contains all demolished waste such as bricks, gypsum, ceramics etc.

The second one is the previous mentioned "R.O.S.E. calcestruzzo" that contains only demolished concrete structures.

It is important to specify at this stage that all Recycled Aggregate used throughout this research has come from the Pescale site-plant and therefore from a plant that usually produces and sells Recycled Aggregate.

This means that the RA came from unknown sources of demolished concrete structures and for this reason the study mirrors a real situation and a practical possible application of this material with all its disadvantages and advantages.

It is clear that the most important factor is to produce a constant quality of aggregate from the plant.

All tests on aggregate have been carried out according to **UNI 8529, parts 1-22 [32]** and **BS 882 [33]** and **BS 812 [34]**. These recommendations have been used both for Recycled Aggregate and for Natural Aggregate.

The first part of the experimentation was carried out in 1995 at the University of Strathclyde on Recycled Aggregate and on fresh and hardened concrete made from RAC.

The results of this experimentation were used to direct the second part of the tests carried out in 1997.

For this reason in this Chapter the tests performed to investigate the physical and chemical characteristics of Recycled Aggregate have been mainly divided into two parts: 1995 and 1997 .

The aggregate sent to the Strathclyde Laboratories had a grading of 0-30 mm and they were sent in sacks of 60 kilos each.

In the 1997 tests the aggregate grading used was 0-16 mm, according to the grading used in the Mabo pre-cast plant for normal production. The aggregate was sent to the Mabo pre-cast plant in sacks of one ton each.

Tests for physico-chemical characteristics have been carried out after mixing samples from each sack in order to guarantee constant and homogenous characteristics of the aggregate.

The first tests established the properties of the material:

- sieve analysis and Fuller's grading insertion;
- equivalent in sand test;
- sand specific gravity;
- particle shape and flakiness index;
- bulk density.

This was followed by tests to establish the rheological characteristics of aggregate:

- moisture content;

- SSD;
- water absorption;
- Los Angeles test;
- freezing and thawing sensitiveness;

As we know the aggregate in concrete has different tasks:

- it is the skeleton of concrete and has considerable bearing on its mechanical characteristics;
- with its high modulus of elasticity it constrains the cement paste shrinkage (higher volume stability);
- by reducing the quantity bonding material (cement) it reduces the heat of hydration;
- reducing the quantity of cement reduces the total cost of the concrete.

The most important properties that make a material suitable as aggregate for concrete are:

- physical properties (porosity, permeability, particle shape and size, etc..);
- chemical properties (insolubility , chemical inertia with the bonding material, stability with the natural processes during its life);
- mechanical properties (compressive strength, tensile strength, resistance to wear by abrasion, hardness and toughness).

One of the most important characteristics of the aggregate is the compressive strength of the rock where the aggregate comes from. This value determines the maximum strength of the concrete. In fact if the strength of the aggregate is higher than the cement paste, the compressive strength of concrete could be improved by reducing the water/cement ratio. When the strength of the cement paste is higher than the strength of the aggregate the compressive strength of the concrete is independent of the cement paste quality. All the above considerations have been borne in mind in designing the test program.

Because the recycled aggregate has porosity and micro-cracking in old mortar it is very difficult to obtain satisfactory compressive strength for RAC.

This has been the main task of this research and the experimentation has proved that it is possible to produce concrete that attains the compressive target strength of 40 Mpa.

3.3.1 Grading,particle shape and texture

Representative samples of the 1995 aggregate were sieved in accordance with BS 812 part 103:1985.

The samples of the aggregate and the mixes are coded as follows: XXNNYY. Where XX is the target strength of the mix design, NN is the type of aggregate used i.e. AR signifies all recycled, AN signifies all natural and RN signifies a blend of the two. The last digit YY is the percentage of natural aggregate used in that mix.

Typical grading curves for the recycled aggregate, natural aggregate, and blends of the two (50% of RA and 50% of NA) are shown in Fig. 3.9. The grading curves of the Recycled Aggregate and Natural Aggregate mixed together in different percentages (e.g. 30% of RA and 70% of NA, 70% of RA and 30/ of NA) are reported in appendix A.

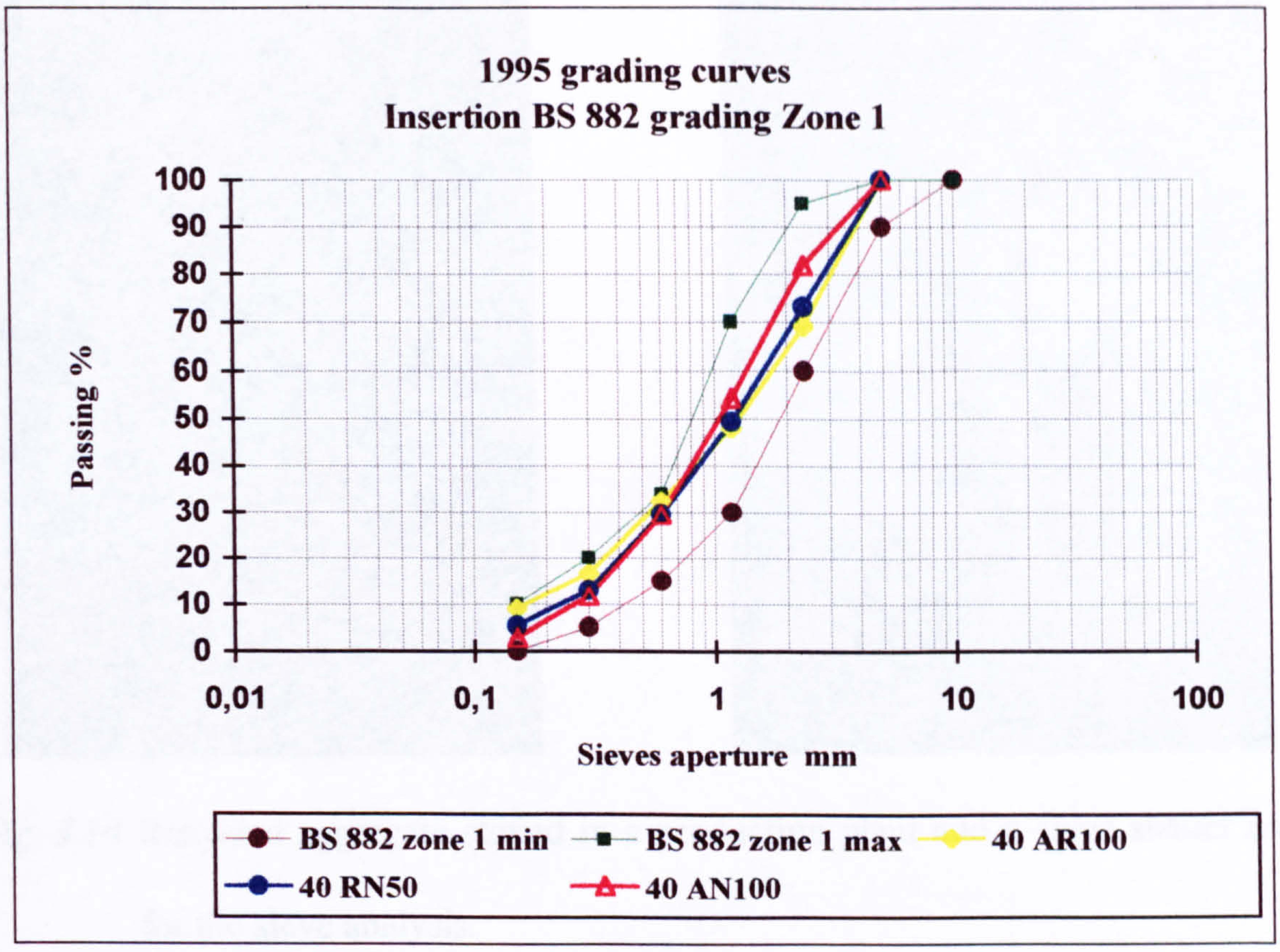


Fig. 3.9 Insertion grading zone 1995 aggregate.

In 1997 sieve analyses were carried out using samples of recycled aggregate taken by quartering. The recycled aggregates have been taken from sacks and homogenised by mixing and heaping into a cone.

This has been repeated twice and the final cone has been flattened and divided by quartering. Diagonally opposite quarters of almost 2 kilos each have been taken for testing. They have been oven dried for 24 hours at 110 °C.

As already said the grading distribution of the aggregate is determined by sieve analysis Fig. 3.10.

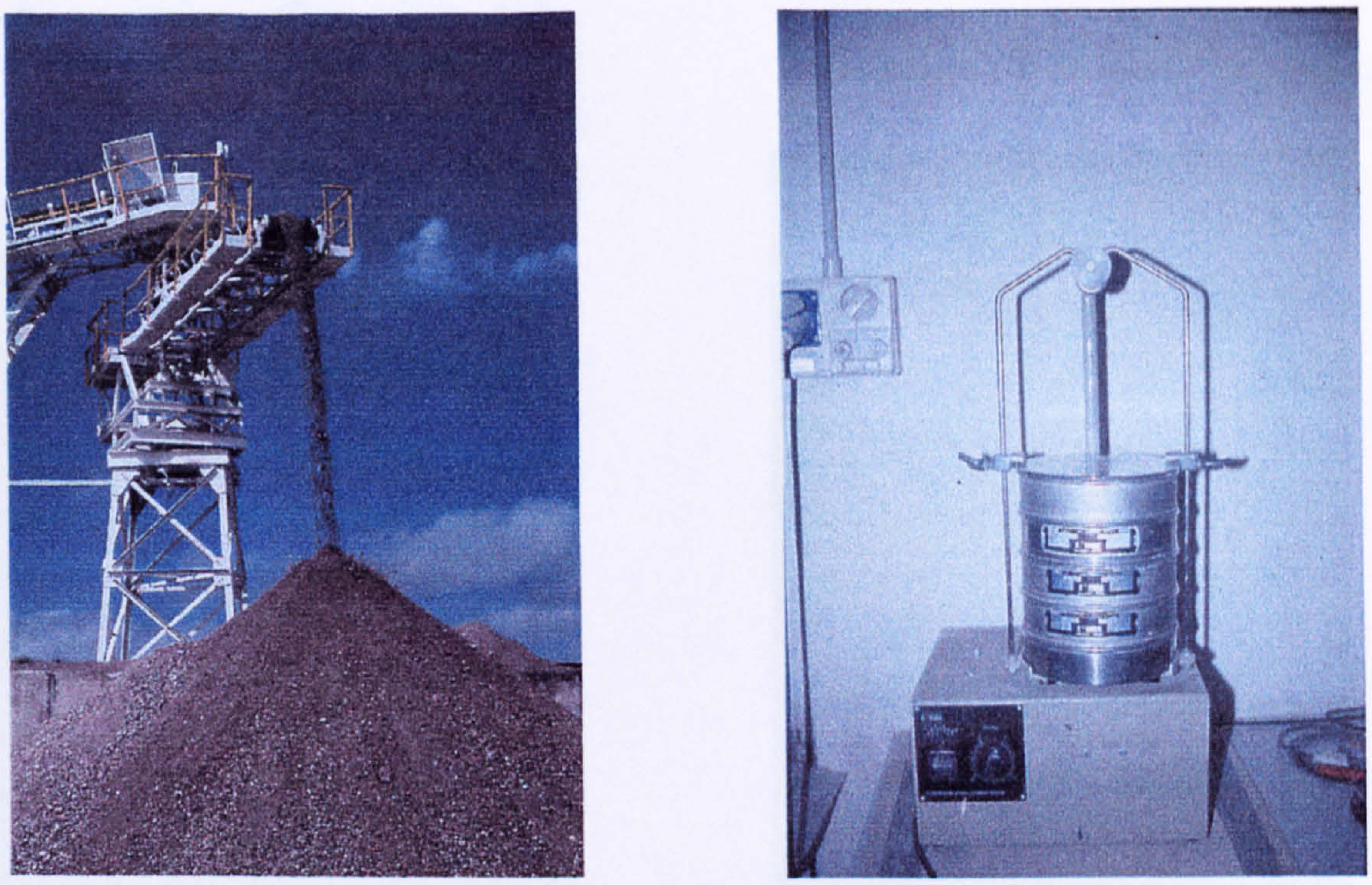


Fig. 3.10 Recycled aggregate stoked in a production plant and a sieve shaker used for the sieve analysis.

Fig. 3.11 Recycled Aggregate: portion retained at 12.5 mm and 10 mm sieve.

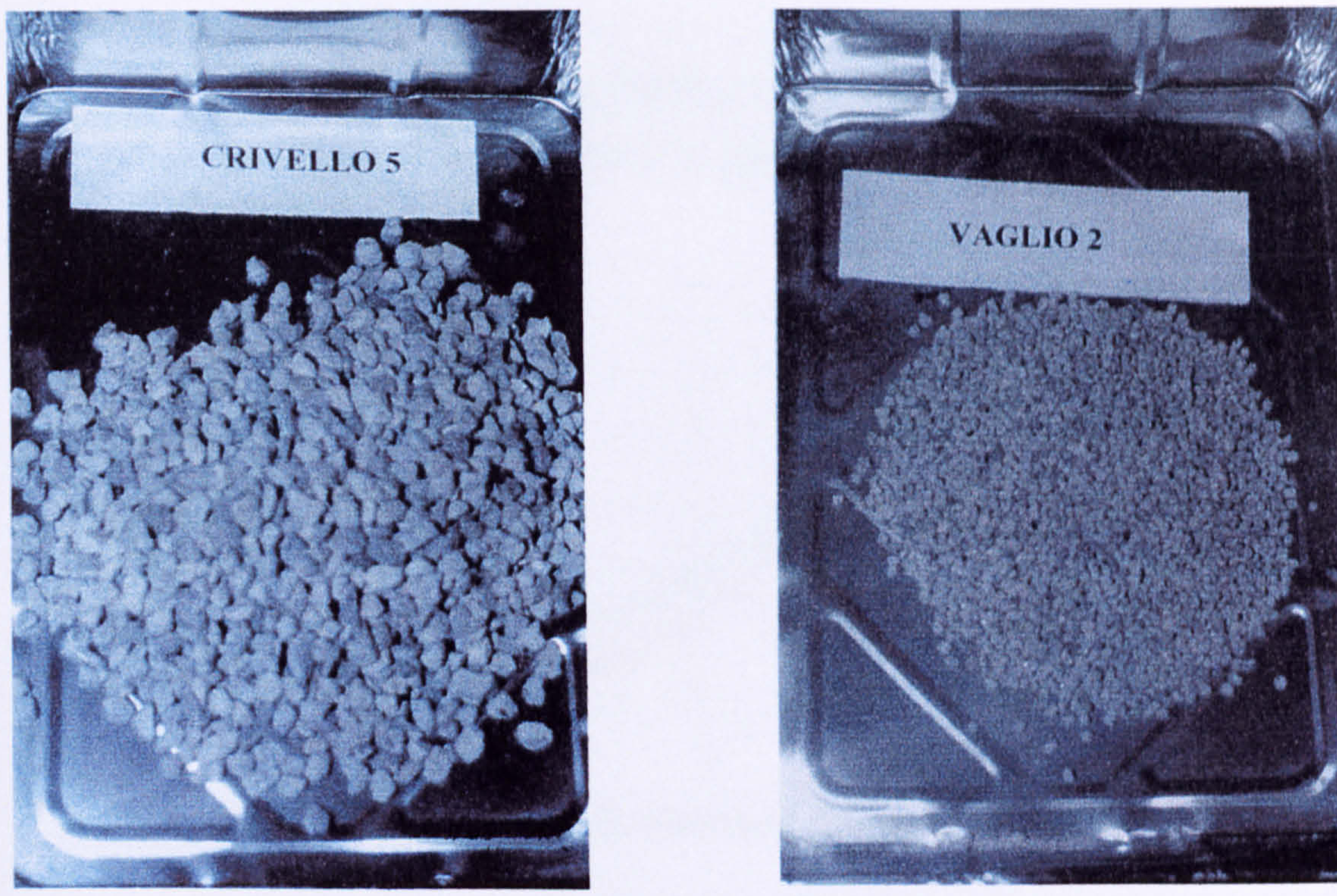


Fig. 3.12 Recycled Aggregate: portion retained at 5.0 mm sieve and at 2.0 mm sieve.

The grading curves (UNI 8520/ 5°) for a significant number of samples of the 1997 Recycled Aggregate are reported in the Appendix A. These samples are named CP1,CP2,CP3, CP4, CP MIX and are placed into Fuller and Bolomey zone.

In Fig. 3.13 the insertion Fuller's grading zone (min and max) for the sample CP 1 is reported.

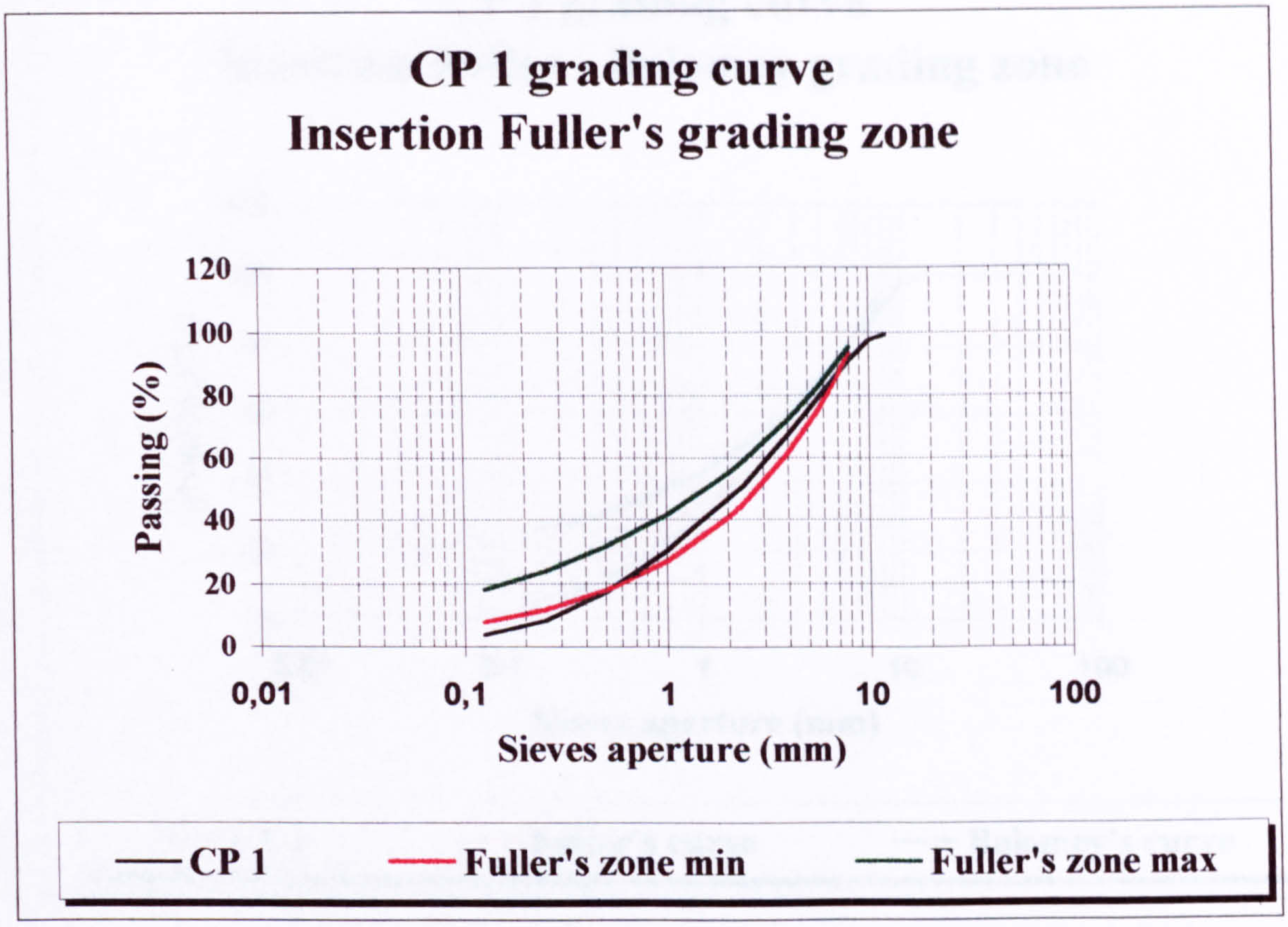


Fig. 3.13 Sample CP 1: insertion Fuller's grading zone.

From the sieve analysis carried out, the Recycled Aggregate used seems to lack fine material (Fig. 3.13).

In fact it can be noticed from the above curve that the grading distribution complies with Fuller's theoretical curve except with regard to the fine aggregate.

This scarcity is compensated with the trend to produce fine material for abrasion and crushing during the mixing operation inside the concrete mixer.

The insertion Fuller and Bolomey grading zone for the sample CP 1 is reported in Fig. 3.14.

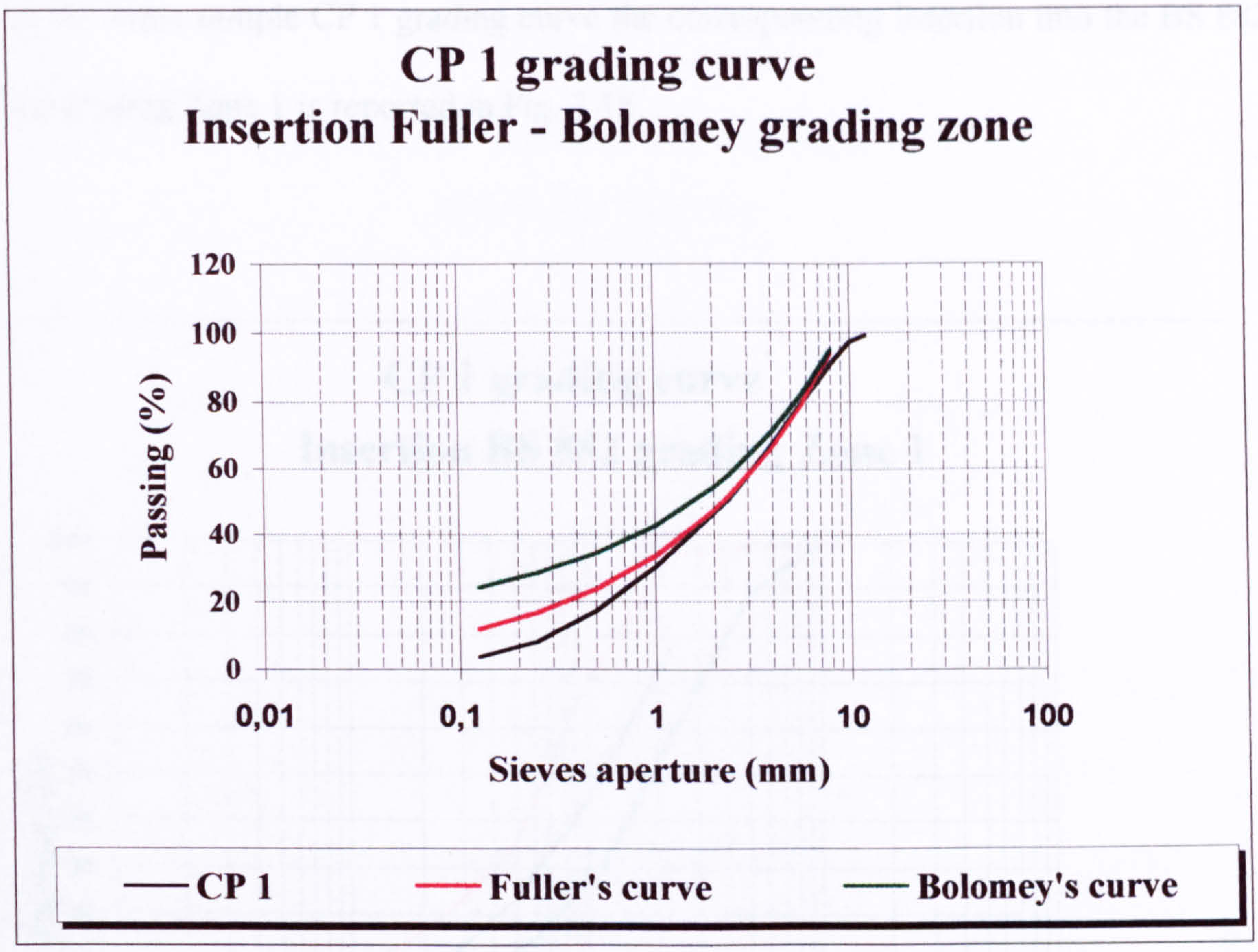


Fig. 3.14 Sample CP 1: insertion Fuller - Bolomey grading zone.

The recycled aggregate examined does not comply with the Fuller-Bolomey grading zone that guarantees the best compromise between the density requirement and the non-segregation of the aggregate.

The specific weight of the concrete is another property that depends on the grading distribution of the aggregate. Usually increasing the weight increases the mechanical strength performance of concrete.

An ideal grading distribution, that means the correct quantity of fine and coarse aggregate, can produce also a better filling of the interstices and consequently a higher specific weight of concrete.

For the same sample CP 1 grading curve the corresponding insertion into the BS 882 fine grading Zone 1 is reported in Fig. 3.15.

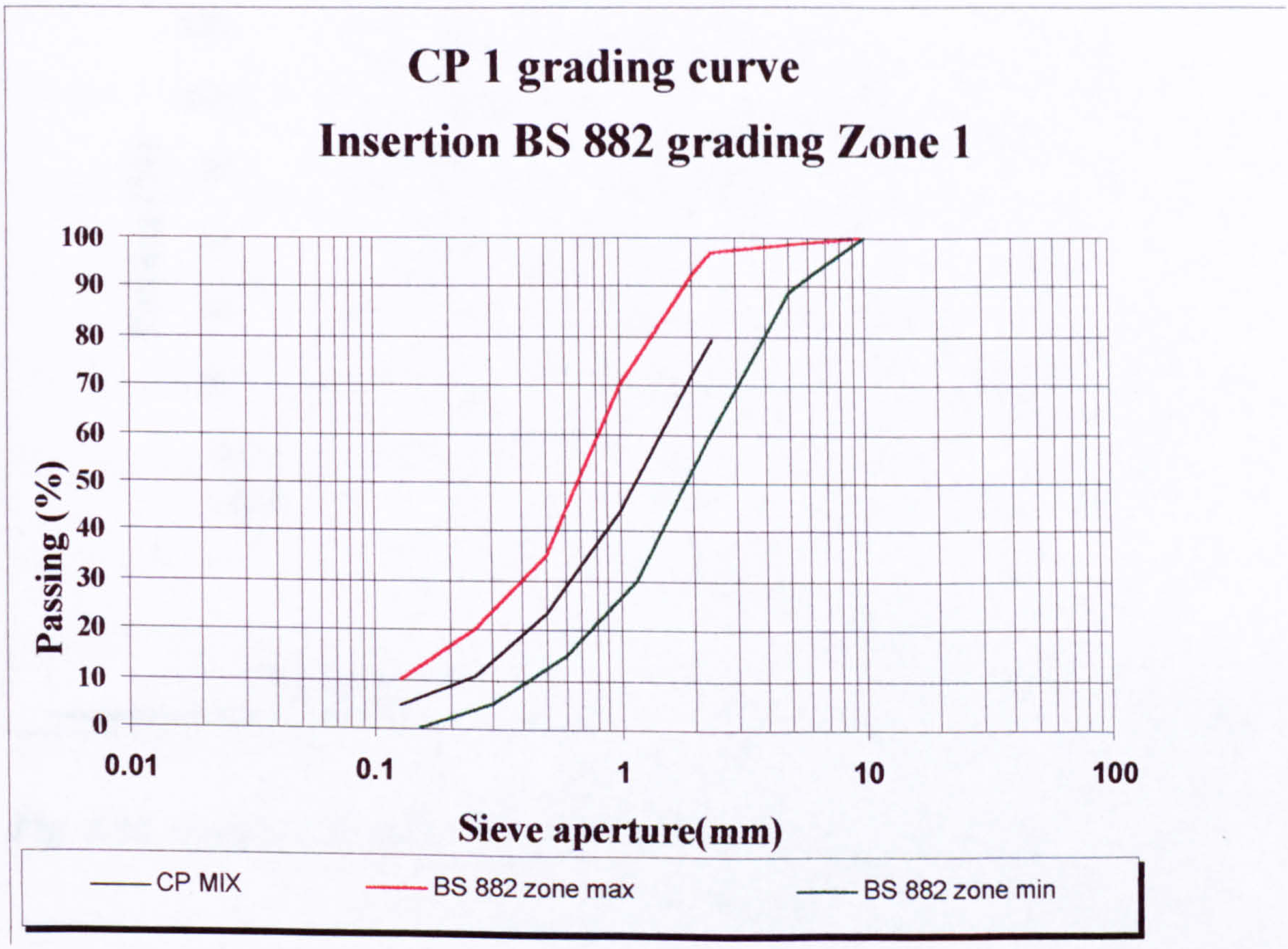


Fig. 3.15 Sample CP 1: insertion BS 882 grading Zone 1.

The same samples comply with the grading zone given from the British Standards 882: part 2: 1973 Grading Zone 1 for fine aggregate (Fig.3.15).

In order to check the consistency of production of the recycling plant the sample called CP 1, used in 1997, has been compared in *Fig.3.16* with a sample of recycled aggregate coming from the same plant but used in 1995.

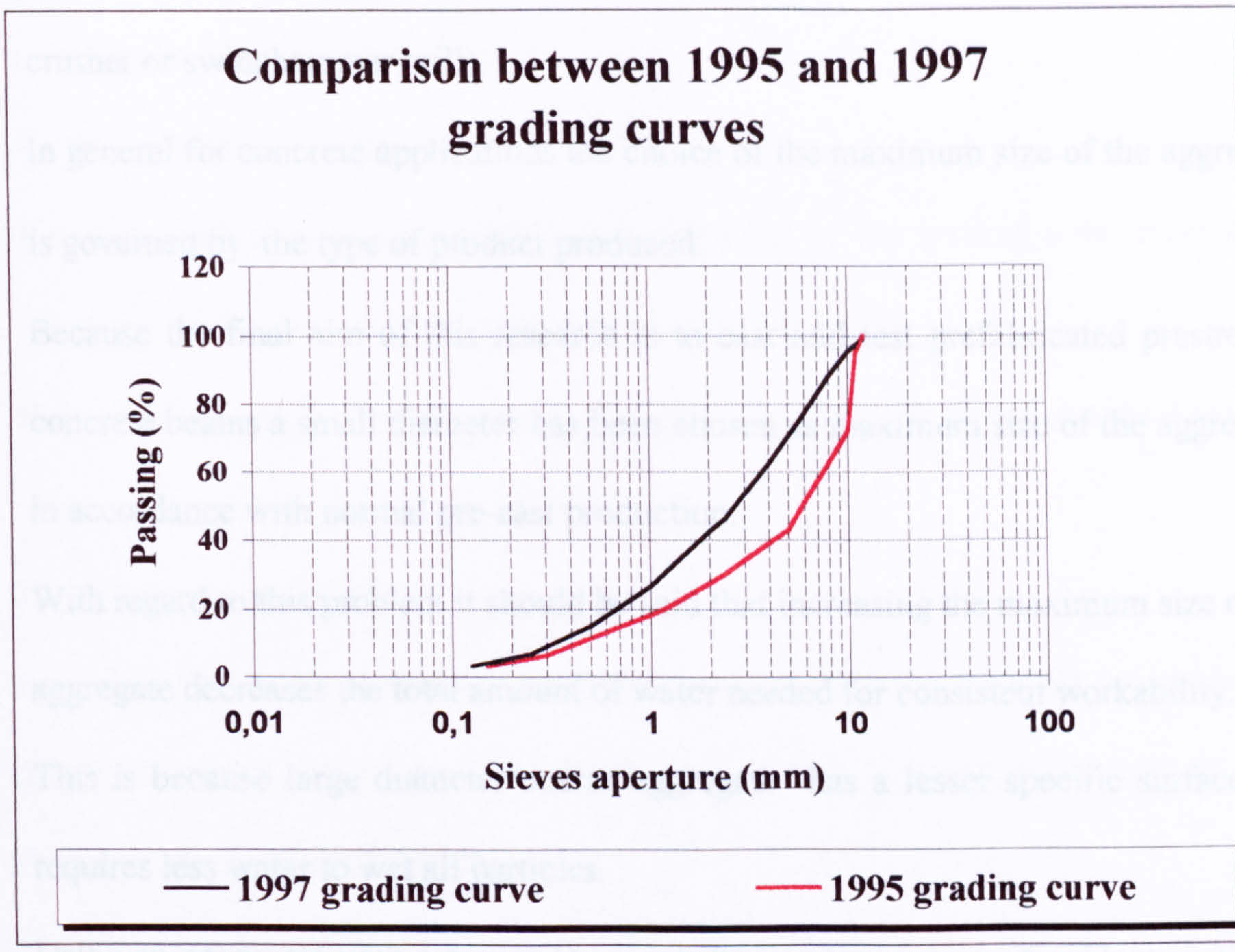


Fig. 3.16 Comparison between 1995 and 1997 grading curves.

There is a difference between the two grading curves. The reason for this non homogeneity of production during that time can be ascribed to the different original demolition concrete that arrives at the plant (in terms of quality of aggregate, cement and w/c ratio).

It is clear in fact that a concrete made with a high performance cement will not have after crushing the same grading curve of a concrete made with a low performance cement.

The same situation will pertain to different types of original natural aggregate.

It could be seen from the literature review that the particle size distribution is influenced also by the crusher characteristics (jaw crusher, cone crusher, impact crusher or swinghammer mill).

In general for concrete applications the choice of the maximum size of the aggregate is governed by the type of product produced.

Because the final aim of this research is to cast and test prefabricated prestressed concrete beams a small diameter has been chosen as maximum size of the aggregate, in accordance with normal pre-cast production.

With regard to this problem it should be said that increasing the maximum size of the aggregate decreases the total amount of water needed for consistent workability.

This is because large diameter coarse aggregate has a lesser specific surface and requires less water to wet all particles.

Using an aggregate with a bigger D_{max} results in a lower cement content with less shrinkage and less cost in order to achieve the same mechanical strength with the same wc ratio.

This encourages the use of mixes with large diameter aggregate but care has to be taken to avoid segregation.

It is usual to define the maximum diameter as the value equal to the sieve aperture of the total passing aggregate, starting the grading curve from this point. It should be said that this valuation is not representative of the real maximum diameter.

The maximum diameter D_{max} should satisfy the following formula:

$$D_{max} = d_1 + (d_1 - d_2) \cdot \frac{x}{y}$$

where:

d_1 is the first sieve on which the material is retained;

d_2 is the directly next sieve;

x is the percentage retained on d_1;

y is the percentage retained on d_1+d_2

With this method it is possible to avoid going out of the grading zone especially where it is particularly delicate as is the case with the sand.

The value found in 1997 is reported in *Tab. 3.1*. The average value is around 9 mm.

3.3.2 Fineness modulus

According to the international standards sand is considered as all the material passing the sieve n° 4 of the Tyler ASTM series with 4.76 mm of aperture or the sieve module 37 of the UNI series.

The fineness modulus MF is completely independent of D_{max} of the sand. It is calculated according to the percentage retained of the sieves that have the section of 1/4 of the net of the previous sieve with exclusion of the 0.075 mm sieve.

The MF of the sand is of fundamental importance in determining the workability of concretes. They should have MF between 2.3 and 3.3 according to the UNI 8520 parts 5°.

If a sand shows a fineness modulus MF not included in the previous values, a segregation of the coarse aggregate can occur during the mixing operations and consequently a decrease in workability.

The higher the value of MF the lower is the fineness in general.

This parameter on its own can define the grading distribution of an aggregate.

Tab. 3.1 reports the values found in 1997.

Sample	Date	Starting weight	Final weight	Difference %	D max	MF
CP1	18/02/97	1777.7	1781.9	0.236	9.024	4.361
CP2	18/02/97	1784.2	1786.2	0.112	9.145	4.390
CP3	18/02/97	1793.8	1794.1	0.017	9.323	4.582
CP4	19/02/97	1809.3	1805.1	0.232	9.122	4.544
CP Mix	19/02/97	1840.8	1839.2	0.087	9.123	4.561
CP Sabbia	03/03/97	1000.0	998.1	0.190	2.697	3.446

Tab. 3.1 Grading characteristics of samples of Recycled Aggregate.

3.3.2 **"Equivalent in sand" test**

The "equivalent in sand" ES is defined as the volume percentage of the clean sand to the total sediment aggregate. Because the ES test does not distinguish between the slimy-clayey material (that is noxious for concrete) and the finest material coming from the crushing of rocks, (that is useful for concrete) this test has been carried out (when required) with the value of the indicator *blue of methylene*. This has been done with the aim of qualifying the fine material passing the 0.075 mm UNI 2332 sieve.

The ES test is carried out on fine aggregate passing the 4 mm UNI 2332 sieve. A reasonable value for ES is 85.

In a graduated cylinder, using a washing tube, a washing away solution is inserted (until 100 mm) composed of anhydride glycerine, formalin and a watery solution of formaldehyde.

The vessel is filled up with a sample of fine aggregate paying attention that the aggregate is not compressed.

The cylinder is plugged and its bottom is beaten on a table. Using a mechanical agitator the cylinder is shaken (90 cycles of agitation) to dissociate the crumbly particles. The washing away solution is spilled into the cylinder and the height of the clean sediment sand is measured.

This is defined as percentage of the total height of the suspension after the sedimentation.

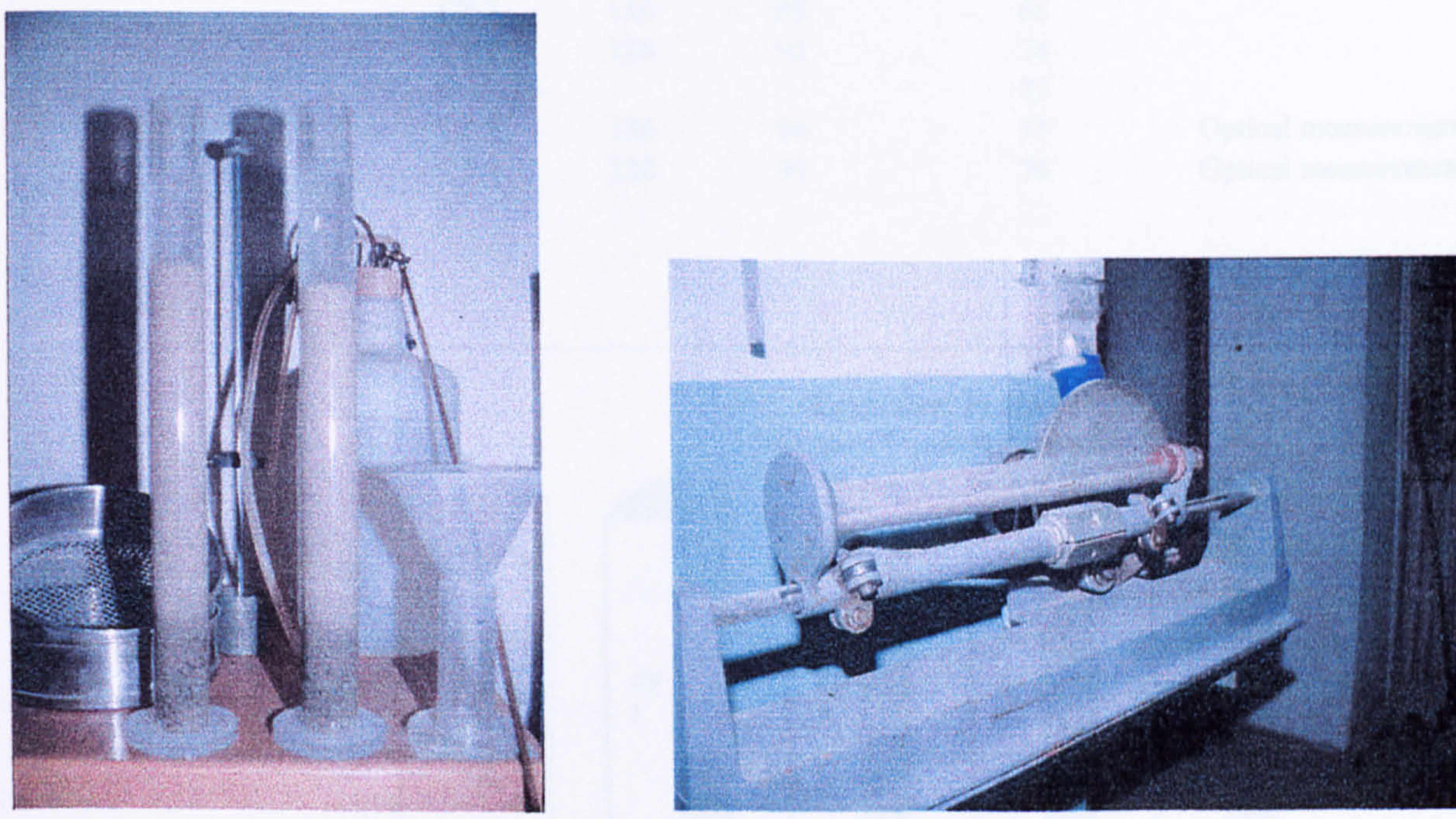


Fig. 3.17 Graduated cylinders with dosing and mechanical agitator for the determination of the "equivalent in sand" ES..

The results have been determined according with the *C.N.R. Anno VI, punto VI, n. 27, p. 1-20, 30 Marzo 1972 [35].*

Different tests have been carried out on the same sample for the statistic reliability.

The value of the ES has been determined also by optical measurement in accordance with the French standards.

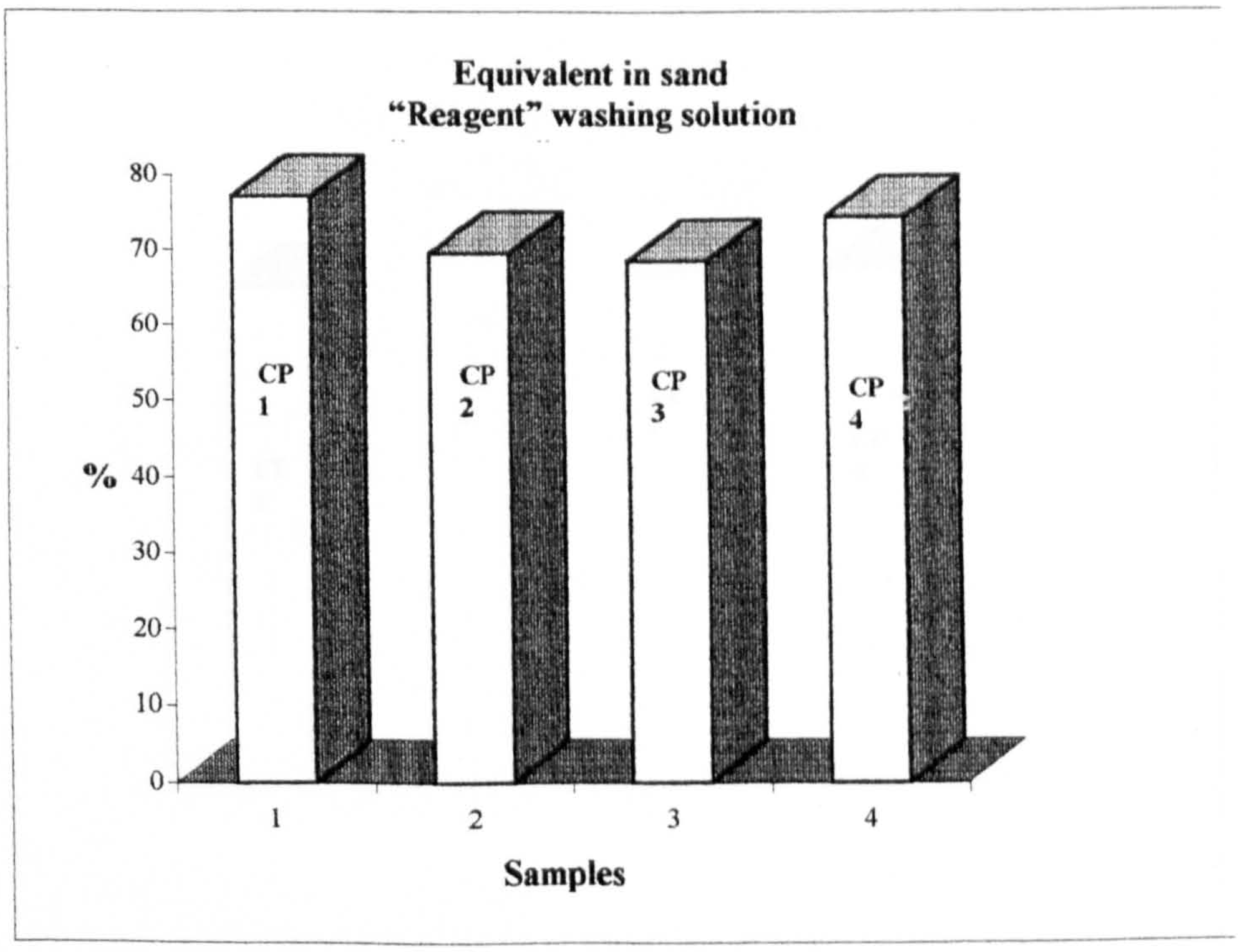
Equivalent in sand - E.S.
0 - 4.76 mm fraction - sieve n° 4 ASTM

Sample code **CP MIX** Date 05/03/97

Source **R.O.S.E. - Pescale S.p.a. - Site Plant**

Sample description **Recycled aggregate**

Sample	Height h mm	Height hs mm	Equivalent in sand %	
CP 1	122	94	77	
CP 2	141	98	70	
CP 3	136	93	68	
CP 4	128	95	74	
Average			72	
CP 3	136	96	71	Optical measurement
CP 4	128	97	76	Optical measurement



Notes: Tests carried out according with C.N.R standards
Anno VI pt. IV, n. 27, p.1-20 30 Marzo 1972
The washing away solution has been prepared according to the above standards.

Fig. 3.18 "Equivalent in sand" ES with washing away solution according with the standards.

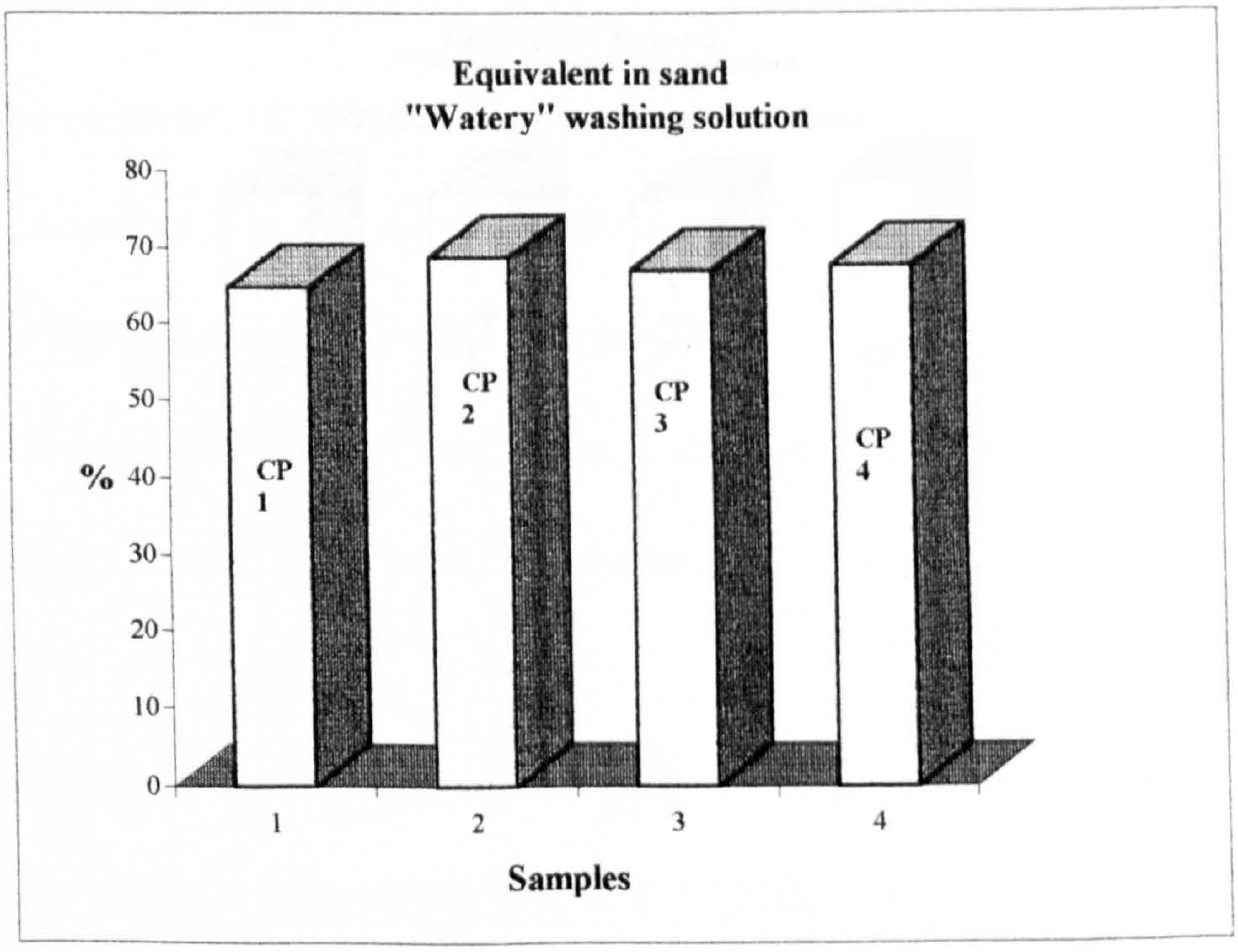
Equivalent in sand - E.S.
0 - 4.76 mm fraction - sieve n° 4 ASTM

Sample code **CP MIX** Date **05/03/97**

Source **R.O.S.E. - Pescale S.p.a. – Site Plant**

Sample description **Recycled aggregate**

Sample	Height h mm	Height hs mm	Equivalent in sand %	
CP 1	139	90	65	
CP 2	146	100	68	
CP 3	150	100	67	
CP 4	141	95	67	
Average			**67**	
CP 1	139	94	68	Optical measurement
CP 2	146	103	71	Optical measurement
CP 3	150	103	69	Optical measurement
CP 4	141	100	71	Optical measurement
Average			**69**	



Notes: Tests carried out according with C.N.R standards
Anno VI pt. IV, n. 27, p.1-20 30 Marzo 1972
The washing solution is natural water

Fig. 3.19 "Equivalent in sand" ES with watery solution.

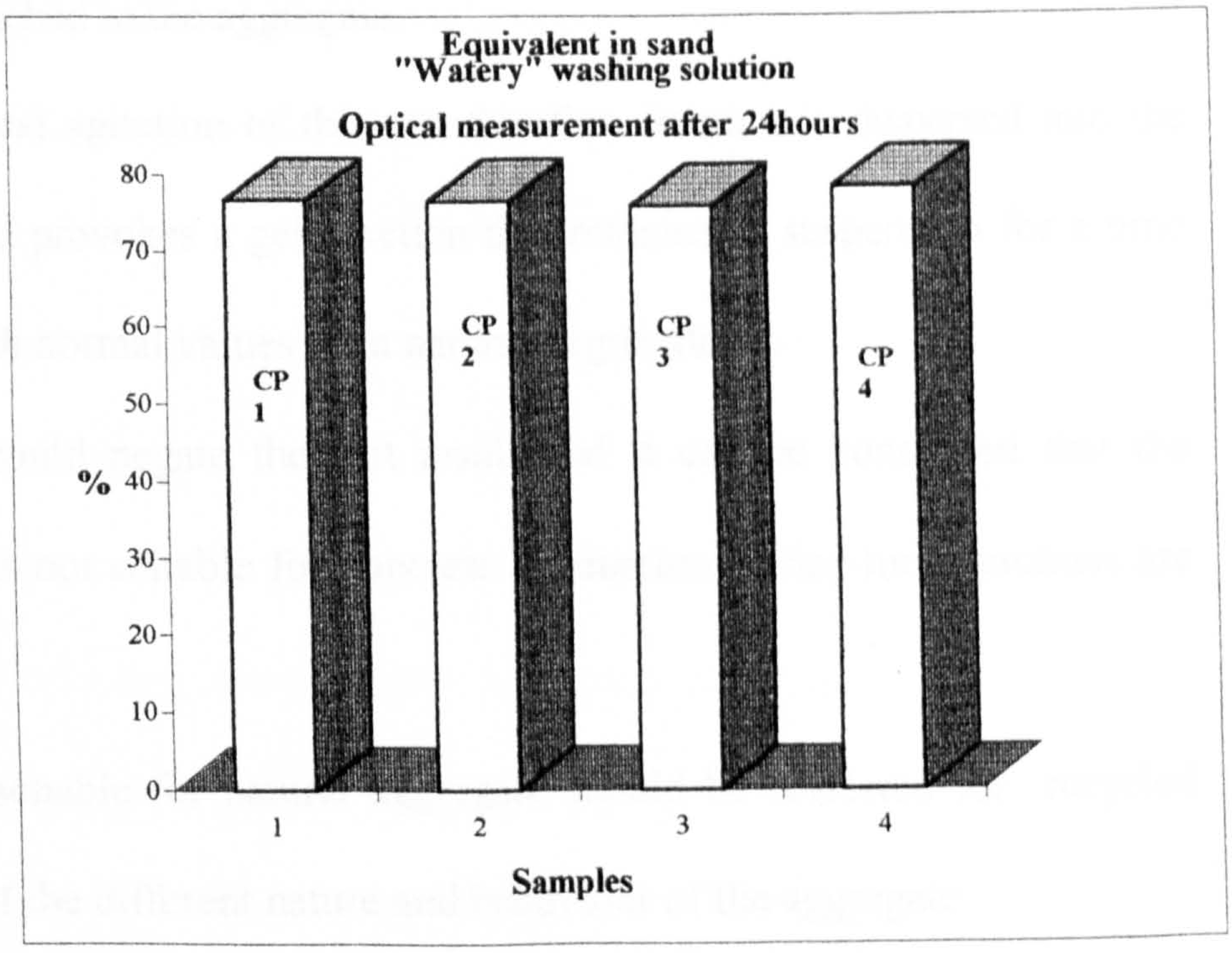
Equivalent in sand - E.S.
0 - 4.76 mm fraction- sieve n° 4 ASTM

Sample code CP MIX Date 05/03/97

Source R.O.S.E. - Pescale S.p.a. – Site Plant

Sample description Recycled aggregate

Sample	Height h mm	Height hs mm	Equivalent in sand %	
CP 1	124	95	77	Optical measurement
CP 2	131	100	76	Optical measurement
CP 3	135	102	76	Optical measurement
CP 4	128	100	78	Optical measurement
Average			77	



Notes: Tests carried out according with C.N.R standards
Anno VI pt. IV, n. 27, p.1-20 30 Marzo 1972
The washing solution is natural water
The optical measurement has been taken after 24 hours.

Fig. 3.20 "Equivalent in sand" ES with watery solution, after 24 hours rest.

This has been necessary because of the wide variability of values coming from measurements with the piston.

The results determined with this method are around 70%.

This is lower than the value suggested by the Italian standard for an aggregate complying with class C.

Further tests have been carried out to study the influence of the washing away solution used in this research.

From the above results it can be seen that because there are no clayey or slimy fractions in the recycled aggregate, the low value of the ES can be attributed to the old cement paste attached to the aggregate.

During the mechanical agitation of this test this fine fraction is dispersed into the washing solution and provokes a gel reaction that remains in suspension for a time longer compared with normal values for a natural aggregate.

This phenomenon could negate the test result and it can be concluded that the recycled aggregate is not suitable for concrete production if clay-lime fractions are present.

Values that are reasonable for natural aggregate should be reviewed for recycled aggregate because of the different nature and behaviour of the aggregate.

A reasonable value for this test seems to be 70%.

3.3.4 <u>Relative densities and water absorption</u>

As already mentioned in Chapter Two one of the most important physical characteristic of the Recycled Aggregate is the relative density of the aggregate.

Great care should be paid in the determination of the SSD (Saturated Surface Dried condition) density to ensure that the material is in a SSD conditions.

The low density and the high water absorption of recycled aggregate are two important factors that should be considered influential in the procedure for the mix design of RAC.

For this reason great care should be paid to the determination of the real water absorption value of this aggregate.

In fact it is very important to determine the extra water to add to the mix for good workability.

In the pre-cast plant it is very important to measure the moisture content of the aggregate continuously and to change the total amount of water in the mix.

In the Mabo site-plant this operation is performed automatically by a computer that controls the mixer machines.

It is also very important the sand dosage in a concrete mix should be based on the specific weight of the sand.

During this research work for the grading 0/5, 5/30 and 0/30 in 1995 and 0/4, 4/15 and 0/15 in 1997 the following characteristics have been determined:

- water absorption;
- relative density on an oven dried basis;
- relative density on a saturated surface dried basis;
- apparent relative density;
- moisture content (only in 1995);
- voids (only in 1995).

The tests have been performed on material dried in an oven at 105 °C for 24 hours and eliminating the fine material passing to the ASTM 0.075 mm.

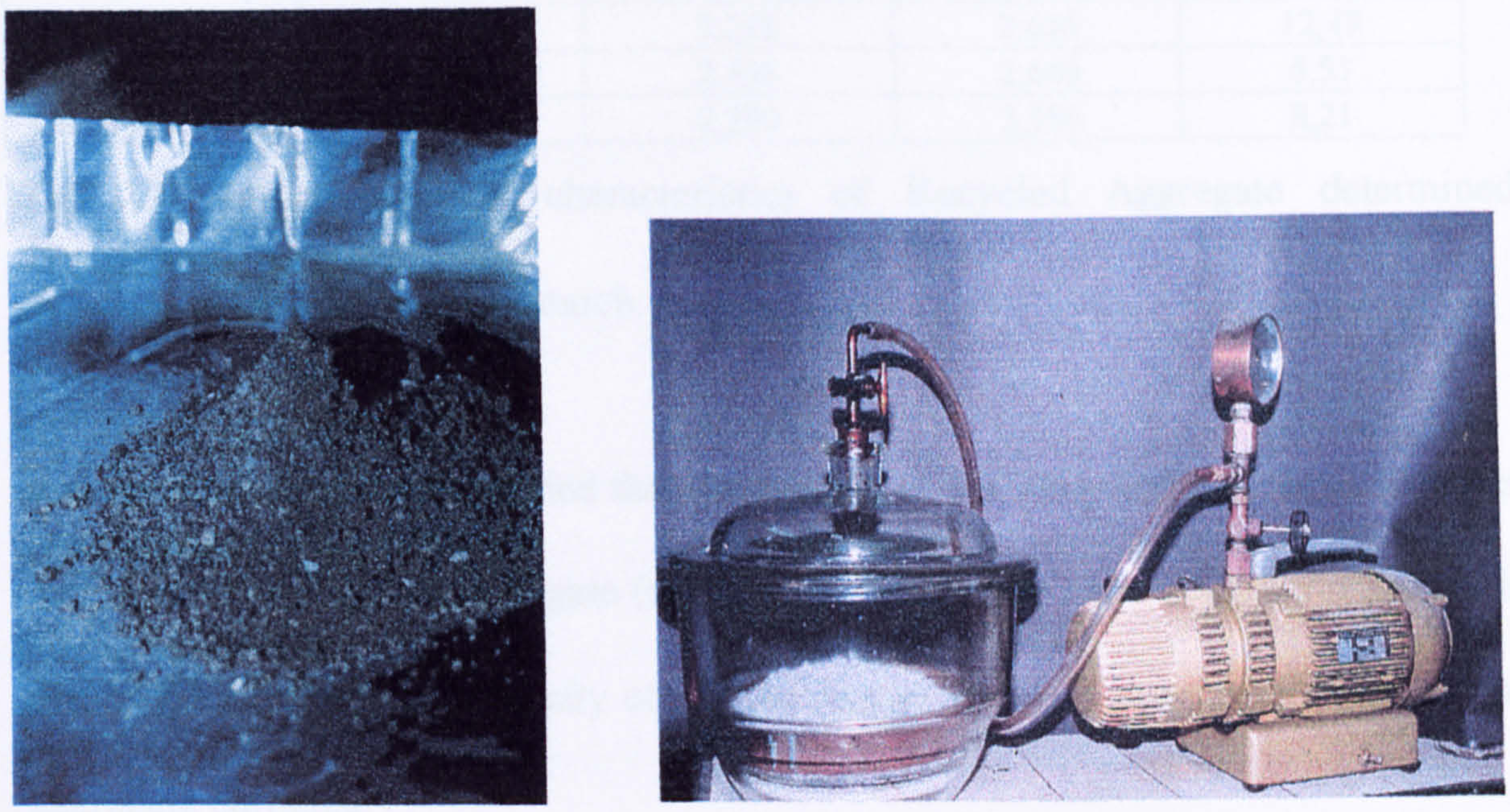


Fig. 3.21 Recycled Aggregate in a SSD conditions following the BS 812 procedure and air voids machine.

Tab. 3.2 shows the results from 1995 investigation.

Grading	Relative density on an oven-dried basis (kg/m^3)	Relative density on a saturated surface-dried basis (kg/m^3)	Apparent relative density (kg/m^3)	Water absorption (%)	Voids (%)	Moisture content (%)
Grading 0/5	2030	2255	2620	11,09	49	
Grading 5/30	2363	2474	2653	4,62	51	1,26
Grading 0/30	2153	2314	2567	7,50	43	

Tab. 3.2 Physico-chemical characteristics of Recycled Aggregate determined during the 1995 research.

Tab. 3.3 reports the test results from 1997 investigation.

Grading	Relative density on an oven-dried basis (g/cm^3)	Relative density on a saturated surface-dried basis (g/cm^3)	Apparent relative density (g/cm^3)	Water absorption (%)
Grading 0/4	1,972	2,218	2,616	12,49
Grading 4/15	2,311	2,438	2,649	5,53
Grading 0/15	2,109	2,280	2,550	8,21

Tab. 3.3 Physico-chemical characteristics of Recycled Aggregate determined during the 1997 research.

As expected it can be concluded that the density of the Recycled Aggregate is lower than that of the natural aggregate (normally between 2600 and 2700 kg/m^3). This is due to the relatively low density of the old mortar which is attached to the original aggregate particles.

The water absorption of the RA is much higher than the water absorption of the original aggregates. This is also due to the higher water absorption of the old mortar.

3.5 Contaminants

3.5.1 Chemical analysis by water extract of Recycled Aggregate

100 grams of Recycled Aggregate passing to the sieve 0.250 UNI series were placed in a flask with 300 cc of distilled H_2O and agitated for 24 hours.

The solution was filtered with Whatman 42 paper and 100 cc was used for tests. Test results from 1995 are reported in Tab. 3.4. Test results from 1997 are reported in Tab. 3.5.

Contaminants (by weight of both the fine and coarse aggregate fraction) (water extract)

Organic substances (%)	Sulphate $SO_4^{=}$ (%)	Chloride Cl^- (%)	Potassium K^+ (%)	Sodium Na^+ (%)	Calcium Ca^{++} (%)	PH of water extract $1/\log[H^+]$
1,97	0,001	0,005	0,0139	0,0155	0,0157	11,3

Sulphate (SO_3) by acid extract: 0,62 %

Tab. 3.4 Chemical analysis by water extract of RA (1995).

Contaminants (by weight of both the fine and coarse aggregate fraction) (water extract)						
Organic substances (%)	Sulphate SO_4^- (%)	Chloride Cl^- (%)	Potassium K^+ (%)	Sodium Na^+ (%)	Calcium Ca^{++} (%)	PH of water extract $1/\log[H^+]$
0,02	< 1ppm	0,022	0,0214	0,0128	0,093	11,9

Magnesium (Mg^{++}) < 1ppm

Tab. 3.5 Chemical analysis by water extract of RA (1997).

3.3.6 <u>Durability tests on Recycled Aggregate</u>

3.3.6.1 <u>Los Angeles test</u>

Conventionally this test measures the loss in weight for abrasion of the aggregate subjected to a roller and bump action with iron balls placed into a cylinder (Fig. 3.22). This test should be performed on the aggregate retained on the sieve of 2,38 mm of aperture. The material should after being washed be dried into an oven at 105°C until it reaches a constant weight.

After that the aggregate is placed into the Los Angeles machine together with the abrasion charge (iron balls) and subjected to 500 of rotation cycles into the cylinder. The material is then sieved to the 1,68 mm sieve. The retained material is washed and dried in an oven at 105°C until it reaches a constant weight.

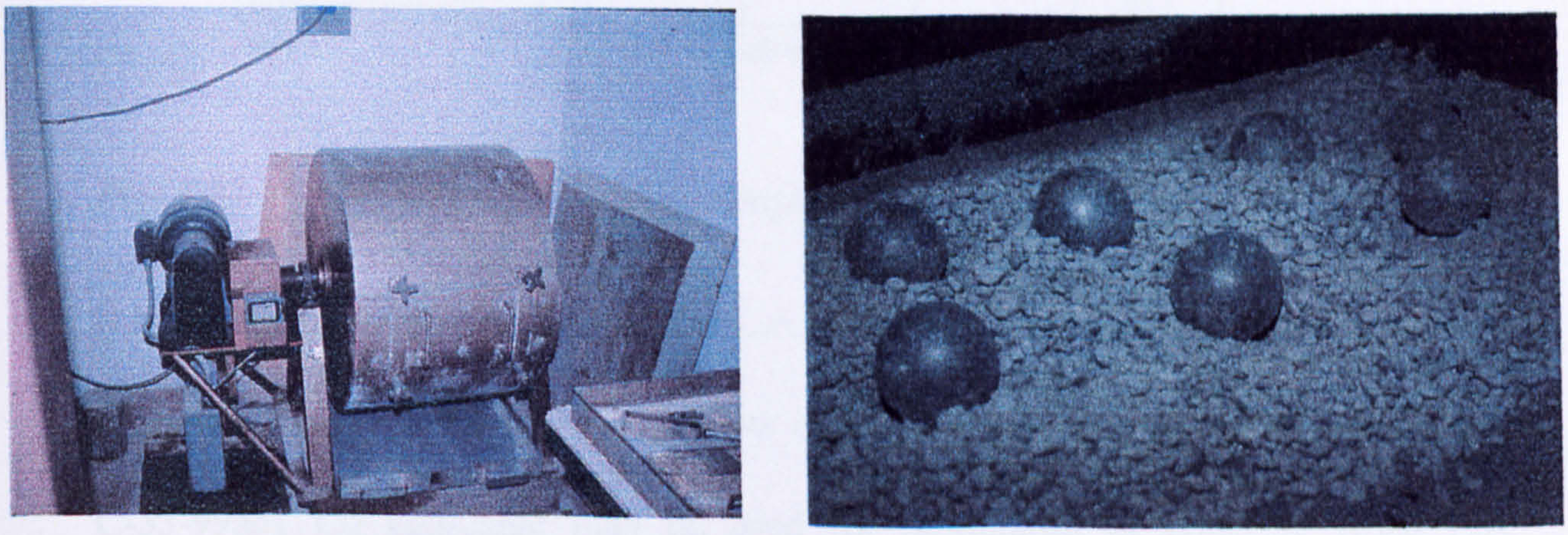


Fig. 3.22 Los Angeles test equipment and Recycled Aggregate after test together with the iron balls for abrasion.

The loss in weight due to abrasion is referred to as a percentage of the initial weight using the following relation:

$$L.A.\% = \frac{P_1 - P_2}{P_1} \cdot 100$$

where: P_1= initial sample weight, P_2= sample weight retained at the sieve 1,68 mm after Los Angeles test.

The tests have been performed following the C.N.R. anno VII, N°. 34 of 1992. The class of the sample is determined comparing the results with given values reported in tables in the C.N.R. standards. Test result are reported in *Fig. 3.23.*

Los Angeles test

Sample **CP MIX 2** Date **05/03/97**

Source **R.O.S.E. Pescale S.p.a. site plant**

Sample description **Recycled Aggregate**

Passing	Retained	A	B	C	D
37.5 mm	26.5 mm	(1250+/-25)	--------	--------	--------
26.5 mm	19.0 mm	(1250+/-25)	--------	--------	--------
19.0 mm	13.2 mm	(1250+/-25)	(2500+/-25)	--------	--------
13.2 mm	9.5 mm	(1250+/-25)	(2500+/-25)	--------	--------
9.5 mm	9.5 mm	--------	--------	**2500**	--------
6.7 mm	4.75 mm	--------	--------	**2500**	--------
4.75 mm	2.36 mm	--------	--------	--------	(5000+/-10)
Initial dry net weight Psi				**5000**	
Final dry net weight Psf				**3665**	
Psi-Psf				**1335**	
"Los Angeles" coefficient (percent)				**26.7**	

Notes: The Recycled Aggregate has a granulometry to be considered into the C class

Fig. 3.23 Los Angeles test on two samples of Recycled Aggregate.

It has been demonstrated [3] that the L.A. abrasion loss percentage for RA depends from the original concrete strength (low or high). According to ASTM Designation C-33-99ae1 [5] aggregate may be used for production of concrete when L.A. abrasion loss percentage does not exceed 50%. According to BS 882, 1201, Part 2, aggregates may be used for production of concrete wearing surface when the aggregate crushing value does not exceed 30%, or 45% for other concrete. It may be concluded that RA produced from all but the poorest quality concrete can be expected to pass ASTM and BS requirements to L.A. abrasion loss percentage, BS crushing value even for production of concrete wearing surfaces, but probably not for granolithic floor finishes.

3.3.6.2 **Frost sensitivity of Recycled Aggregate**

Concretes could be sensitive to freeze and thaw cycles. This phenomenon depends on the porosity of both the cement paste and the aggregate. The water absorbed when freezing increases its volume. If this volume exceeds 91% (critical saturation) there is no more space to allow the ice dilatation and the zone surrounding the cement pores are subjected to compressive stresses that can produce cracks and disaggregation of the cement paste. To avoid this phenomenon the aggregate is normally tested for frost sensitivity following the *Raccomandazioni C.N.R. anno XIV n. 80, 15 Novembre 1980*. The aggregate is completely saturated and subjected to 20 cycles of freeze and thaw from -20 °C (+/- 5 °C) to +20 °C (+/- 5 °C).

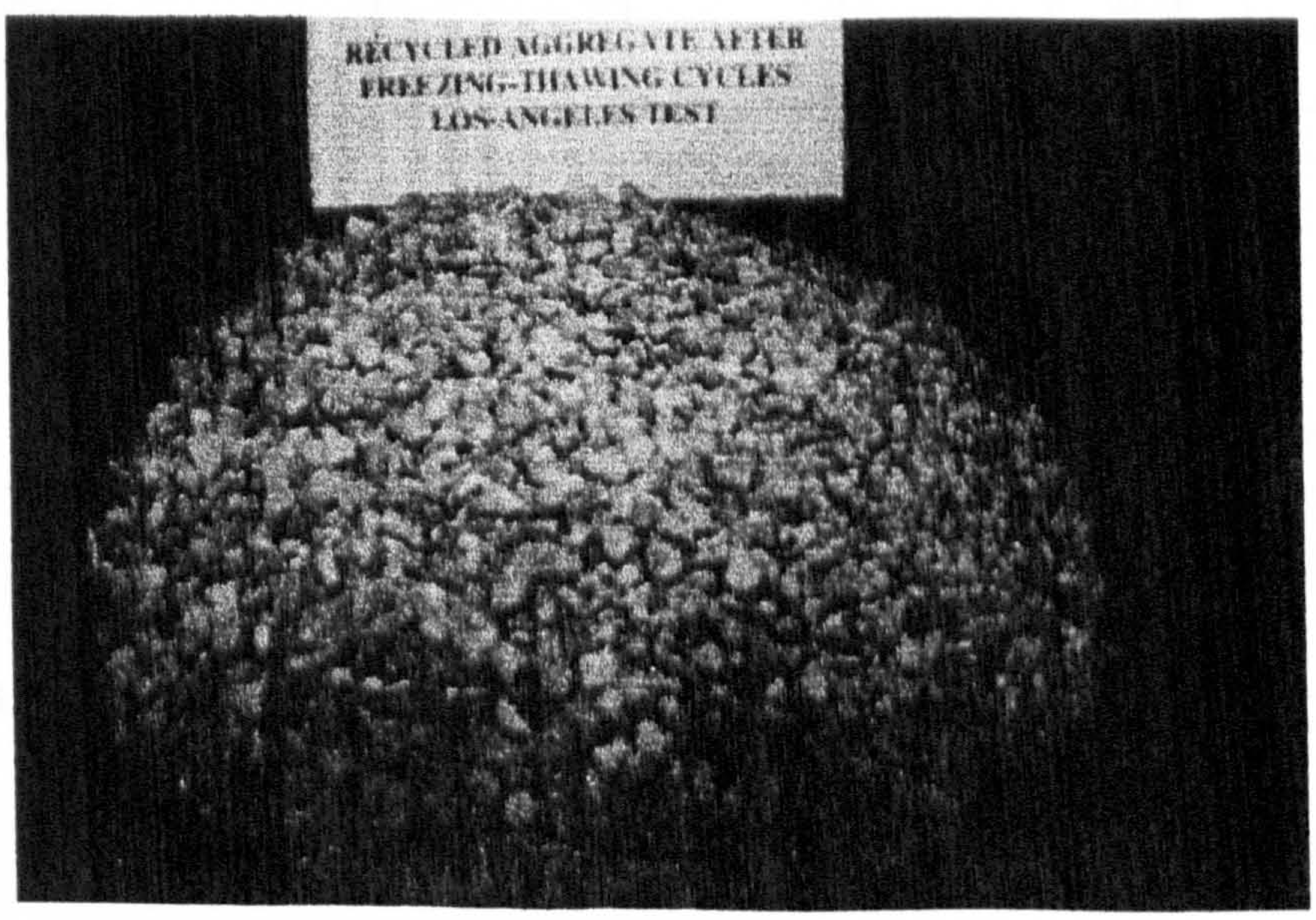


Fig. 3.24 Recycled aggregate after freeze and thaw.

After the freeze and thaw cycles (it takes twenty days) the Los Angeles test is performed on the aggregate and the factor of frost sensitivity is determined from the following relation: $G = 100 \cdot \frac{(LA_2 - LA_1)}{LA_1}$, where LA_1 is the Los Angeles test value before the freeze and thaw cycles and LA_2 is the Los Angeles test value after. In *Fig. 3.24* a photo of the recycled aggregate after the freeze and thaw cycles is reported. It could be seen that the material is completely cleaned from the old cement paste. Nevertheless the value found complies with the values given in the C.N.R. standards. Test results are reported in *Fig. 3.25.*

Frost sensitivity of Recycled Aggregate

Sample	CP MIX 2			Data 26/03/97	
Source	R.O.S.E. Pescale S.p.a. site plant				
Sample description	Recycled Aggregate				
		Class			
Passing	Retained	A	B	C	D
37.5 mm	26.5 mm	(1250+/-25)	-------	-------	-------
26.5 mm	19.0 mm	(1250+/-25)	-------	-------	-------
19.0 mm	13.2 mm	(1250+/-25)	(2500+/-25)	-------	-------
13.2 mm	9.5 mm	(1250+/-25)	(2500+/-25)	-------	-------
9.5 mm	9.5 mm	-------	-------	2500	-------
6.7 mm	4.75 mm	-------	-------	2500	-------
4.75 mm	2.36 mm	-------	-------	-------	(5000+/-10)
Initial dry net weight Psi				5000	
Final dry net weight Psf				3342	
Psi-Psf				1658	
Frost "Los Angeles" coefficient				33.16	
"Los Angeles" coefficient				26.7	
Frost sensitivity "G"				24.2	

Fig. 3.25 Los Angeles test on RA after cycles of freeze and thaw.

From Fig. 3.25 it can be seen that the frost L.A. abrasion loss percentage increased 20% compared to the corresponding normal L.A.. Of course the frost sensitivity depends also on the pore system and the strength of the old cement paste. For this reason it is advisable to use Recycled Aggregate Concrete designed with low water/cement ratio to reduce the pore system of the concrete.

Chapter Four

Properties of fresh and hardened RAC

4.1 Introduction

The use of RA for concrete structures requires more tests and processing of results than its use in road building .

In fact to be able to use a material for construction it is essential to assess more than just its compressive strength.

As well as determining the physico-chemical characteristics of the Recycled Aggregate (RA) and the properties of both the wet and hardened RAC, it is important to check that the mathematical models and numerical correlation normally used for the design of ordinary concrete (such as mix-design procedure, design codes, non-linear analysis) are suitable for RAC.

For this reason the main task of this investigation has been to ascertain whether RAC can have satisfactory mechanical performance for use as a structural material and later to guarantee consistency of results.

The first step was to formulate a mix-design procedure suitable for RAC to attain the desired workability and target strength.

This procedure is reported in this Chapter.

The properties of both the wet and hardened Recycled Aggregate Concrete are also reported in this Chapter.

4.2 Properties of fresh Recycled Aggregate Concrete

4.2.1 A mix-design procedure

It is important to establish a mix design procedure for RAC which will enable it to be used for structural use.

For this reason different modifications in the existing mix design procedure have been investigated by the author and a suitable one has been suggested for RAC.

The setting up of this procedure started in 1995 and was confirmed in 1997 using different types of cement and it has been used to obtain satisfactory mechanical properties to allow the casting of the three large pre-stressed beams.

In 1995 Ordinary Portland cement conforming to BS 12 was used for both recycled aggregate and natural concrete.

The natural aggregate was uncrushed zone 2 sand from Springbank Quarry. Uncrushed gravel of 10 mm and 20 mm. nominal size from the same source was also used.

As previously mentioned the recycled aggregate used in this investigation (named R.O.S.E.) was obtained from the crushing of reinforced concrete structural elements by a commercial organisation in Italy.

The mixes are coded as follows: XXNNYY. Where XX is the target strength of the mix design, NN is the type of aggregate used i.e. AR signifies all recycled, AN signifies all natural and RN signifies a blend of the two.

The last digit YY is the percentage of natural aggregate used in that mix.

During these investigations the grading 0/30 mm. has been adopted for the recycled aggregate concrete (all in).

Three different modifications of existing mix design procedures and trial mix have been investigated during 1995.

They are:

1) Concrete with pre-soaked recycled aggregate.

2) Recycled Aggregate Concrete with different percentages of 20 mm natural aggregate.

3) Recycled Aggregate Concrete with different percentages of 0 - 20 mm natural aggregate.

The third procedure is preferred for structural uses of RAC and is referred to in this Chapter. It was the mix-design procedure adopted for the 1997 experimentation. The first and second procedures are briefly discussed in this section.

The mix design procedure adopted in this research was as described in The Design of Normal Concrete Mixes BRE 1975 [3].

Two important factors should be considered as influencing this procedure for the mix design of RAC; the low density, and the high water absorption of the recycled aggregate.

Great care should be paid to the determination of the real water absorption value of this aggregate.

In fact at this stage it is very important to determine the extra water to add to the mix for good workability.

Both the coarse and fine recycled aggregate have been assumed to be crushed.

The maximum aggregate size has been 20 mm. The relative density on a saturated surface-dried basis (SSD) of the recycled aggregate has been assumed from tests reported in Chapter Three to be: 2400 kg/m^3. Concrete density: 2200 kg/m^3. Proportion of fine aggregate: 44%.

The same procedure has been followed for the concrete mix design with all natural aggregate. The proportion of fine natural aggregate was 34%.

1. Concrete with pre-soaked Recycled Aggregate

Following this procedure the recycled aggregates have been left in the horizontal pan mixer to pre-soak for 10 minutes with the extra water for water absorption.

The effect of 10 minutes of pre-soaking of the recycled aggregate can be seen by comparing the V-B test figures of mix 40AR100PS (the digit PS signifies Pre-Soaking) and 40AR100 in Table 4.2.

For the mix with the pre-soaked recycled aggregate the time (sec) for V-B test is shorter indicating a higher workability.

More significantly this workability is maintained for longer periods which is important on site (see Table 4.2, comparison of slump figures at 30 minutes).

However 10% reduction in the compressive strength of RAC has been noted from the report (see Table 4.4). This could be explained by the following considerations.

The crushing process for production of recycled aggregate produces aggregate with diffuse microcracking especially in the old mortar attached to the original aggregate particles [3].

This creates a sort of "sponge effect" when the water is added in the concrete mixer during the concrete production. It begin with the recycled aggregate absorbs all the water and then tends to expel the part not strictly stoichiometric with a consequent alteration of the w/c ratio.

2. *Recycled Aggregate Concrete with different percentages of 20 mm of Natural Aggregate*

In a previous research carried out by the same author [25], mixes with 100% of Recycled Aggregate (AR) were produced and it was found that even with a water/cement ratio of 0.35 which gave a strength of 60 MPa for the AN mixes, the AR mixes only achieved 35 Mpa [1], [4], [5].

It is important to underline that the recycled aggregates used in all the investigations by the author are aggregates that came from *unknown sources* of demolished concrete structures, with *different strengths* of the original concrete. Furthermore the recycling process involves the possibility of contamination by weak aggregate like refractory brick, gypsum, clay balls, etc.

For these reasons during this investigation different percentages (from 10% to 30%) of 20 mm natural aggregate have been added to the mix to improve the mechanical characteristics of the RAC. There was no significant improvement in cube strength with replacement by natural aggregate.

The greatest improvement measured was less than 10%.

This was probably due also to the fact that the 20 mm. natural aggregate was not statistically well distriibuted in the concrete mix.

In any case the investigation demonstrated that the workability characteristics can be improved by partially substituting 20 mm coarse aggregate. As little as 10%-15% substitution has important effects on both the workability and the loss of workability with time.

This observation is important since previous workers [23] have usually found it necessary to include workability aids (admixtures) in the mix in order to obtain satisfactory behaviour of the fresh recycled aggregate concrete.

Using admixtures in RAC is not as straightforward as for the ordinary concrete.

In fact the contaminants present in RA may annul the actions of the admixture.

So the use of partial substitution of the aggregate by natural coarse aggregate seems to be an alternative method to achieve a good workability

All these previous results were processed and used to direct the third part of the investigation referred to below.

3. *Recycled Aggregate Concrete with different percentages of 20 mm of Natural Aggregate*

During 1995 it seemed logical to the author that to attain a satisfactory strength for *structural uses* of RAC it would be better to blend different proportion of the two base mixes made by recycled aggregate concrete and natural aggregate concrete (eg. the mix 40 RN30 is a mix in which the 70% is recycled aggregate (all in) and the 30% is natural aggregate (sand, 10 mm, 20 mm).

This also improved the workability characteristics as referred to above.

On the other hand it seems also easier for the production of RAC to add natural aggregate (sand, 10 mm, 20 mm) to the mix instead of cutting the 0/5 grading of the recycled aggregate and to replace it with natural sand.

The exact proportions of the mix components per 1 m^3 of RAC are reported in Tab. 4.1.

no.	Mix code	w/c ratio	Cement kg	Free Water l	Extra water for water abs. l	Recycled Aggregate All in kg	Natural Aggregate Fine Sand kg	Natural Aggregate Coarse 10 mm kg	Natural Aggregate Coarse 20 mm kg
1	40AR100PS	0.45	420	190	79.5	1590	0.0	0.0	0.0
2	40AR100	0.45	420	190	79.5	1590	0.0	0.0	0.0
3	40RN30	0.45	400.7	181	61.4	1113	194.7	126.0	252.0
4	40RN50	0.45	388	175	49.3	795	324.5	210.0	420.0
5	40RN70	0.45	375.3	169	37.2	477	454.3	294.0	588.0
6	40AN100	0.45	356	160	19.1	0	649.0	420.0	840.0

Tab. 4.1. Mix proportions.

The free water reported in Tab. 1 is the stoichieometric one. Extra water for water absorption must be added to the mix to attain the exact water demand.

In this case a value of 5% for the water absorption for the grading 0/20 mm of RA has been adopted.

4.2.2 <u>Workability tests</u>

In 1995, Slump, V-B and Compacting Factor tests were performed on six different mixes of RAC (Fig. 4.1). The results are reported in Table 4.2.

The effect of partial substitution of natural aggregates on the workability of the fresh concrete can be noted.

In particular there is an optimum percentage of natural aggregate to get a significant improvement on workability.

This value is close to 20 to 30%, as confirmed in previous investigations [1], [4].

no.	Mix Code	V-B Consistometer Readings (seconds)			Slump (mm)			Compacting factor
		Time 0	15 minutes	30 minutes	Time 0	15 minutes	30 minutes	
1	40AR100PS	3	4	5	62	47	45	0,98
2	40AR100	5	7	10	46	36	8	0,89
3	40RN30	4	6	6	54	39	36	0,95
4	40RN50	5	6	7	50	38	17	0,91
5	40RN70	6	7	11	28	11	0	0,89
6	40AN100	7	11	14	0	0	0	0,8

Tab. 4.2. Workability of the fresh concrete with different percentage of recycled and natural aggregate: V-B test; slump and compacting factor tests. 1995.

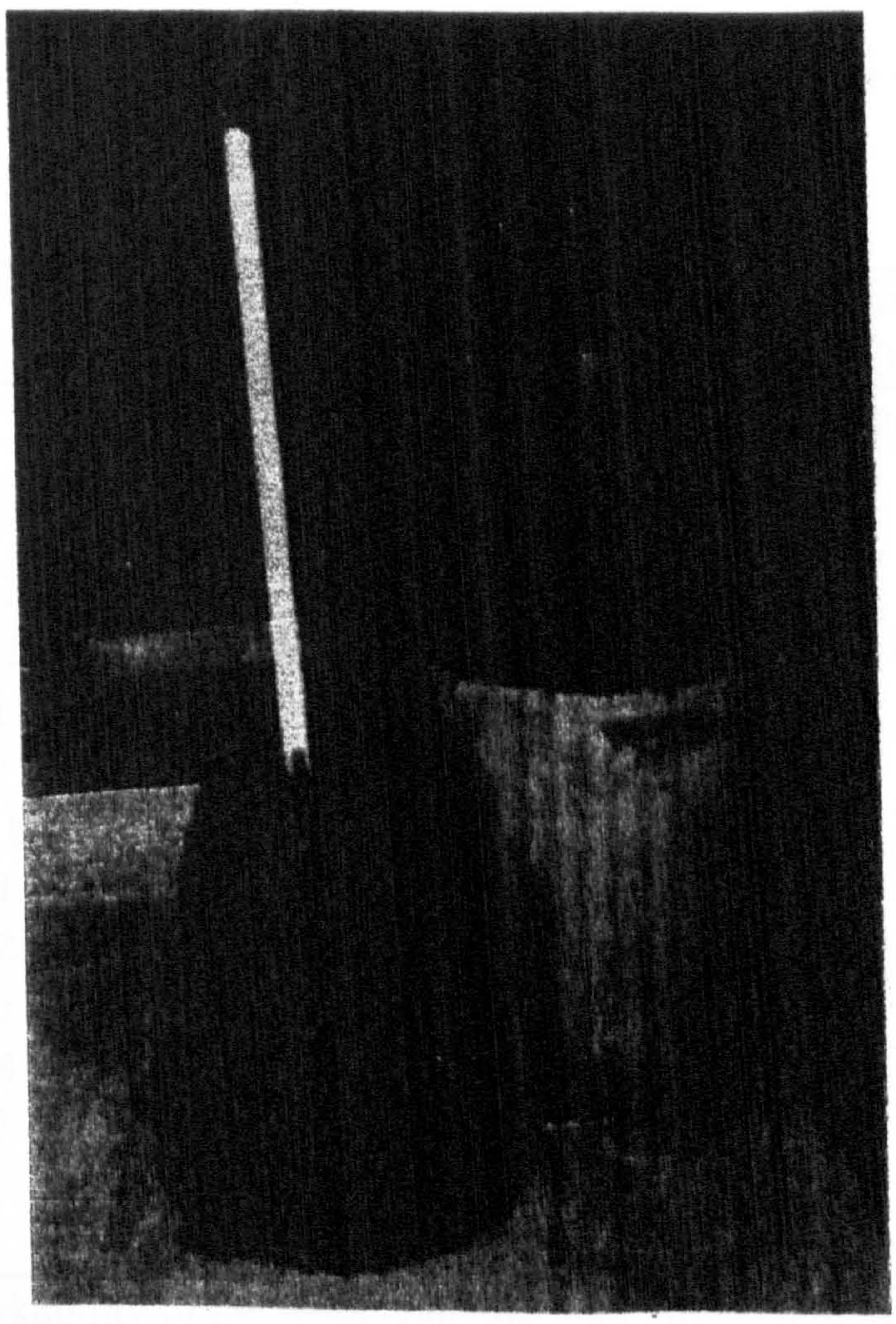


Fig. 4.1 Slump test during 1997 experimentation.

In 1997 the experimentation started by casting specimens with RAC following the mix-design procedure set up during 1995. This was performed in order to check any particular variation in the properties of fresh and hardened RAC by changing the source of the Natural Aggregate.

After this first stage the type of the cement has been changed but the same mix-design procedure has been followed. Using type 52.5 cement and the mix-design procedure described in the above paragraph, the compressive strength needed to sustain the precompression in a real prestressed beam application was reached.

Fourteen different batches have been cast and tested. Slump tests have been performed on each of them. Tests results are reported in Table 4.3 except for the specimens cast together with the prestressed beam for which the results are reported in Chapter Five.

no.	Mix Code	w/c design	Slump design	Cement Type	Slump (mm)		
			(mm)		Time 0	5 minutes	15 minutes
1	CP1AR	0,45	10-30	32.5 Pozzolanic	100	0	0
2	CP2AR-F	0,45	10-30	32.5 Pozzolanic	105	60	40
3	CP325AR1	0,45	10-30	32.5 Pozzolanic	160	80	80
4	CP325AR2	0,45	10-30	32.5 Pozzolanic	110	100	90
5	325CAR1	0,45	60-180	32.5 II b Portland	105	85	65
6	325CAR2	0,45	60-180	32.5 II b Portland	105	85	65
7	CP525AR1	0,45	60-180	52.5 Portland	190	165	150
8	525MAR1	0,45	10-30	52.5 Portland	30	0	0
9	525MAR2	0,45	60-180	52.5 Portland	105	80	60
10	525MAR3	0,45	10-30	52.5 Portland	25	0	0
11	525MAR4	0,45	60-180	52.5 Portland	100	90	80
12	525AR50	0,45	60-180	52.5 Portland	70	60	40
13	40AN1	0,45	10-30	52.5 Portland	20	0	0
14	40AN2	0,45	60-180	52.5 Portland	150	130	90

Tab. 4.3. Workability of the fresh concrete with different percentages of recycled and natural aggregate and different cement type: slump test. 1997.

The mixes are coded as follows: ZZZ KKK J. Where ZZZ is the type of cement used (e.g. 52.5, 42.5, 32.5 Portland cement); KKK is the type of aggregate used i.e. AR signifies all recycled, AN signifies all natural and RN signifies a blend of the two, CP / C signifies sample, M signifies mix and F signifies a type of admixture (superplasticizer) (see Mix Code in Table 4.3). The last digit J is just the chronological order of casting of the samples. For samples casted together with the prestressed beams the same notation of the 1995 investigation has been adopted.

4.3 Properties of hardened RAC

The properties of hardened Recycled Aggregate Concrete for use in structural work are of great interest.

These become very important for use in prestressing where a minimum strength is required to sustain the precompression.

For these reasons a large number of mixes (more than twenty) have been investigated during this research. Compressive strength, tensile strength, flexural strength and modulus of elasticity have been measured at various ages.The influence of different types of cement together with different w/c ratios has been studied. Mixes suitable for use in pre-cast use have been cast and tested. Following particular indications it has been possible to reach 40 MPa and 50 MPa compressive strength for RAC.

4.3.1 Compressive Strength

All test geometries for the 1995 investigation are shown in Figure 4.2.

The test geometry for the 1997 experimentation are shown only if they are different from 1995.

The compressive strength has been estimated on cubes of 100 mm x 100 mm x 100 mm and of 150 mm x 150 mm x 150 mm using a servo control press with 200 tons maximum load, Figure 4.3.

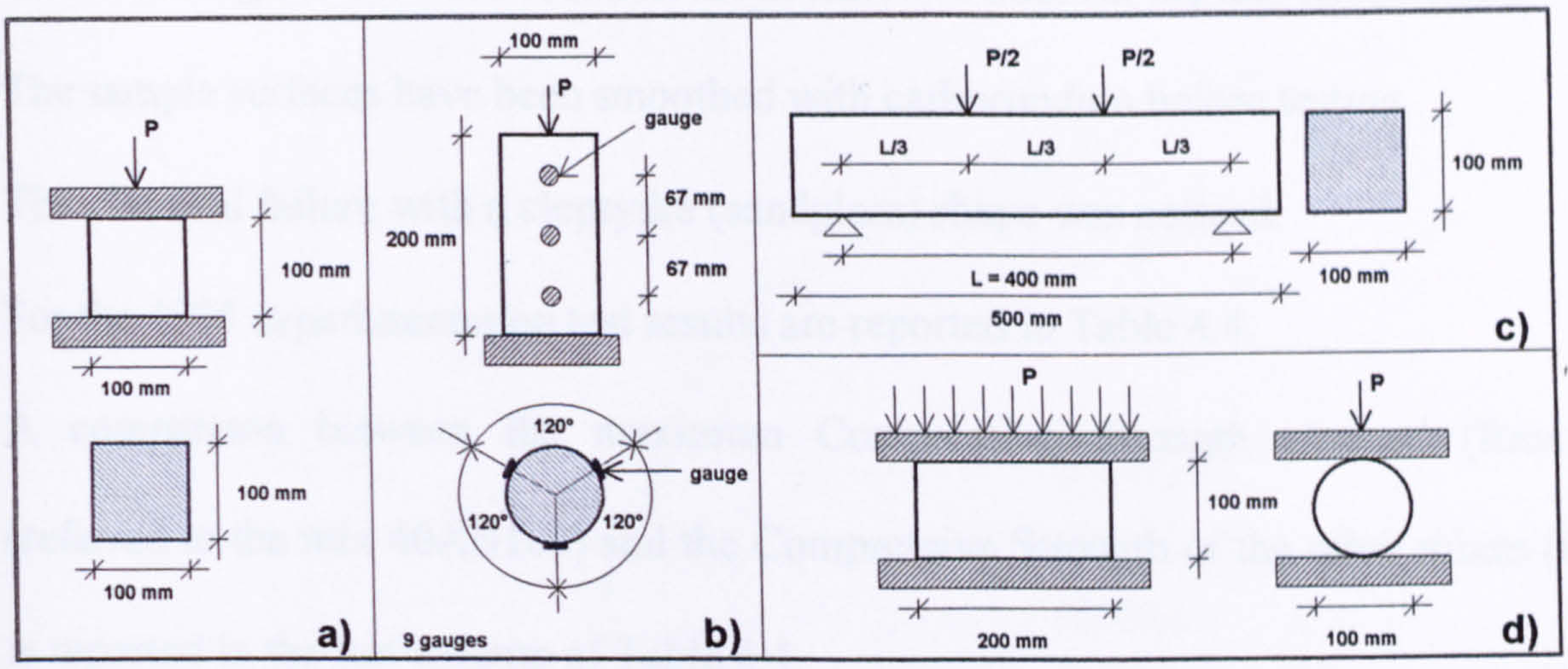


Fig. 4.2 Test geometry specimens. Compressive test (a), E-value (b), flexural test (c), splitting test (d).

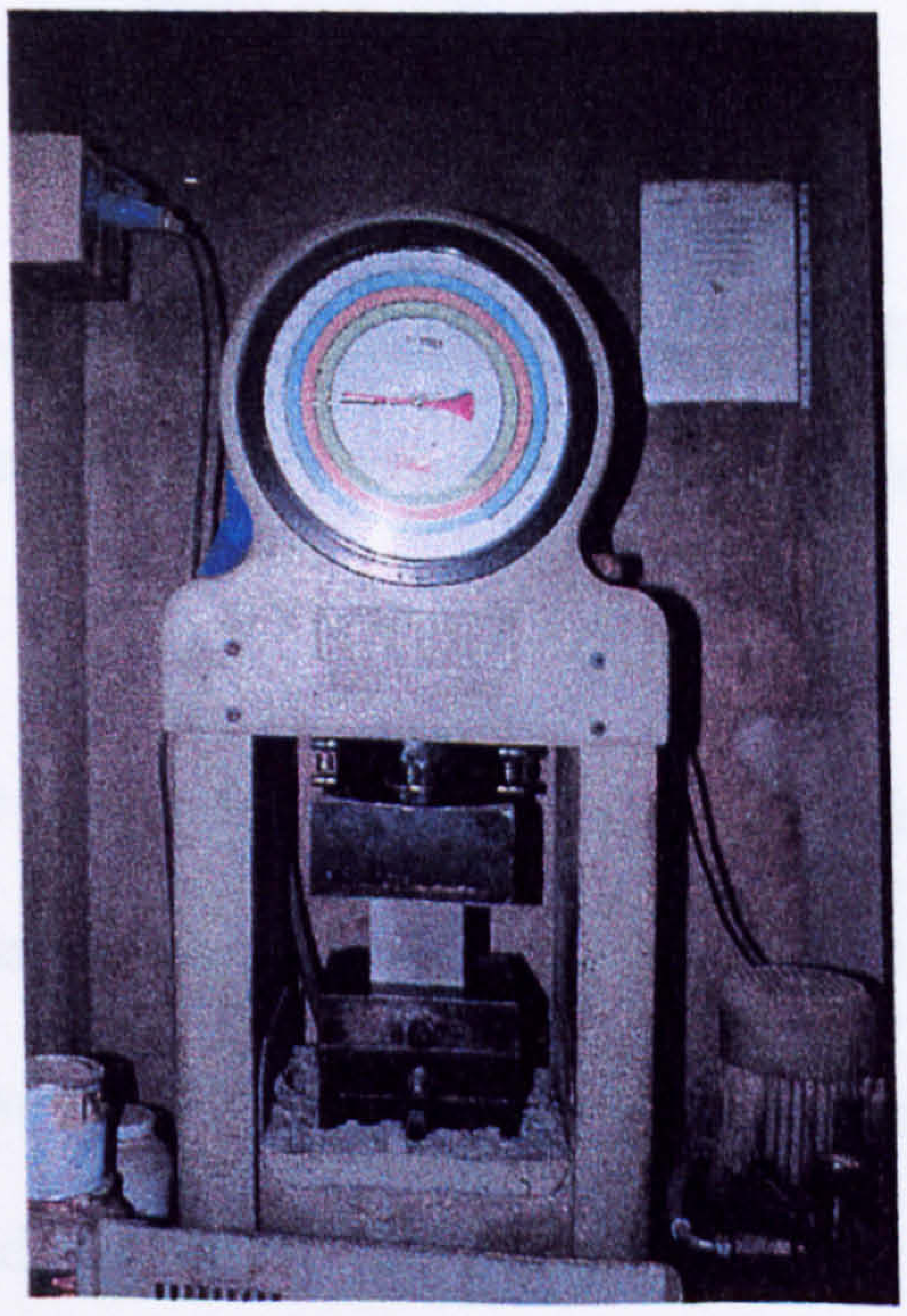


Fig. 4.3 Compressive strength equipment for the 1997 investigation.

The samples have been water cured.

Before testing the cube has been left for at least two hours at the Lab temperature.

The sample surfaces have been smoothed with carborundum before testing.

The classical failure with a clepsydra (sandglass) shape was noticed.

For the 1995 experimentation test results are reported in Table 4.4.

A comparison between the maximum Compressive Strength attained (Rmax) (referred to the mix 40AN100) and the Compressive Strength of the other mixes (R) is reported in the last column of Table 4.4.

Mix code	w/c ratio	Compressive strength at 7 days (Mpa)	Compressive strength at 28 days (Mpa)	R / R max at 28 days %
40AR100PS	0,45	27,8	32,1	71,3
40AR100	0,45	27,9	36,3	80,1
40RN30	0,45	27,3	36,6	81,3
40RN50	0,45	28,8	38,0	84,4
40RN70	0,45	29,9	43,0	95,6
40AN100	0,45	30,3	44,95 (R max)	100,0

Tab. 4.4 Compressive strength tests results for 1995 investigation on RAC.

Figure 4.4 is a plot of the compressive strength versus time for the above mixes with different percentages of natural and recycled aggregate.

It can been seen that there is a 20% difference in compressive strength at 28 days between the 40AR100 and 40AN100.

This difference is reduced to 15% for the mix 40RN50 and to only 4% for the 40RN70.

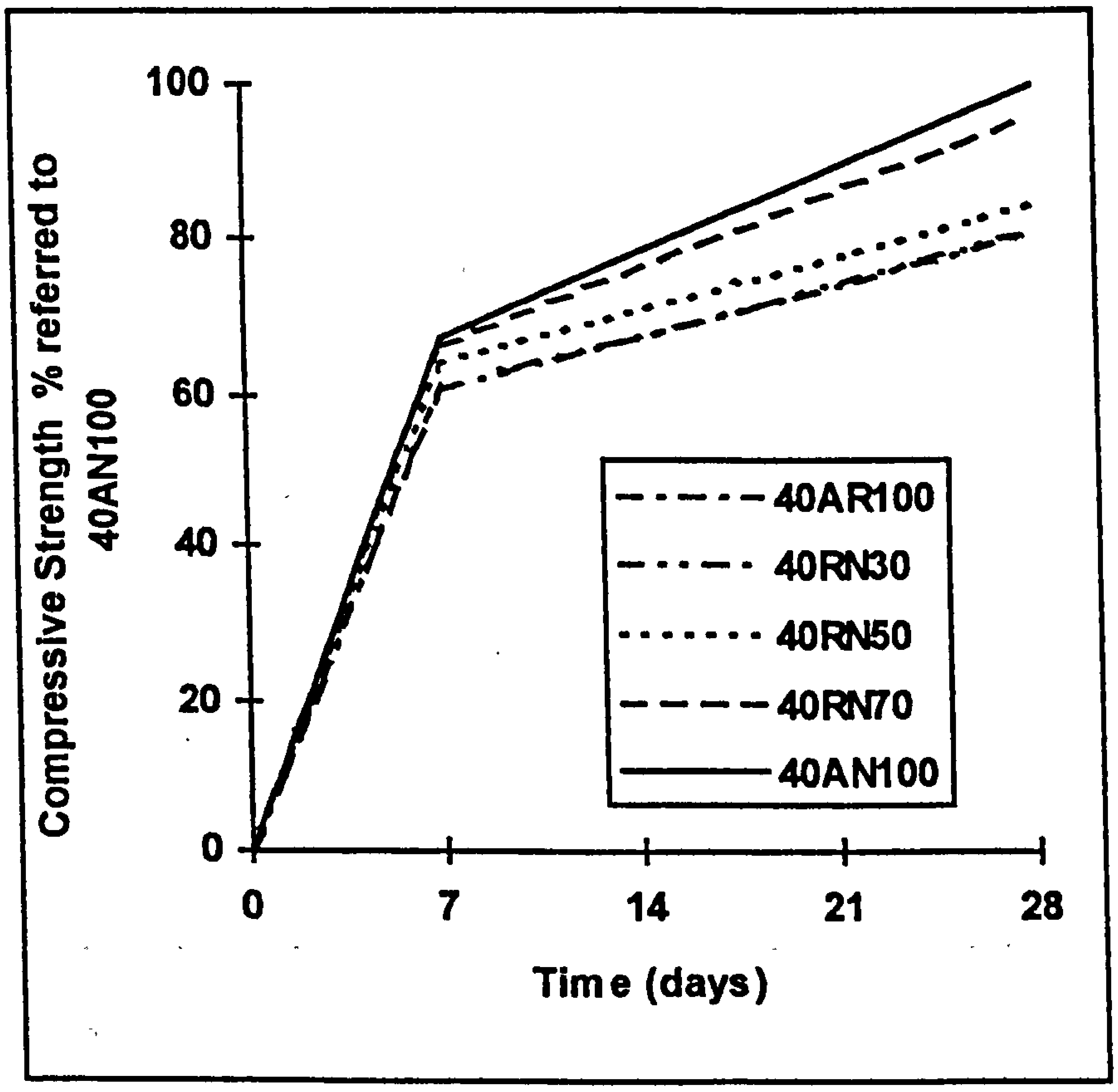


Fig.4.4 Compressive strength versus time for mixes with different percentages of natural and recycled aggregate.

The 40RN70 is the only mix that reaches the target strength of 40 Mpa.

From this first part of the experimentation it is concluded that to attain the design target strength it is necessary to blend the natural and the recycled aggregates.

In Table 4.5 test results for the 1997 experimentation are reported.

Mix code	Compressive strength (Mpa)				
	24 hours	3 days	7 days	28 days	90 days
CP1AR	16.63	29.38	28.50	36.38	-
CP2AR-F	19.38	33.80	33.25	40.50	-
CP325AR1	6.63	15.25	24.75	33.75	-
CP325AR2	3.13	9.75	18.15	26.25	-
325CAR1	3.63	12.05	14.03	19.25	26.90
325CAR2	4.63	15.00	21.13	26.55	36.20
CP525AR1	18.26	26.00	35.25	41.25	-
525MAR1	17.00	28.00	34.63	39.00	-
525MAR2	21.25	33.35	38.88	41.85	50.55
525MAR3	15.75	30.63	37.88	46.25	-
525MAR4	18.75	35.50	40.38	47.85	55.00
525AR50	22.88	37.75	47.00	53.00	-
40AN1	28.75	46.38	55.45	62.26	-
40AN2	-	45.50	55.25	62.75	-

Tab.4.5 Compressive strength tests results for 1997 investigation on RAC.

The differences in the compressive strength between the different mixes are due to the different cement types used during this investigation.

Samples with Portland 32.5 pozzolanic cement, 32.5 composite type IIB cement and 52.5 ordinary Portland pre-blending cement have been cast and tested.

The last one showed the best mechanical performances.

It should be noticed that the mix 525AR50 has been designed for a target strength of 50 MPa and it reached 53 MPa at 28 days.

The mixes 525MAR2/3/4 were designed for a target strength of 40 MPa and they reached this strength only with the use of 52.5 Portland cement.

4.3.2 Tensile Strength

The tensile strength of RAC has been evaluated using the indirect tensile test (Brazilian test) or splitting test.

A compressive load has been applied on a cylinder sample (H = 200 mm, D = 100 mm for the 1995 investigation and H = 300 mm, D= 150 mm for the 1997 investigation) placed in a horizontal position between plates of a press.

The load P is applied through the cylinder generatrix and it provokes on the sample a vertical compression σ_v given by:

$$\sigma_v = \frac{2P}{\pi \cdot LD}\left[\frac{D^2}{r(D-r)}\right]-1 \qquad (1)$$

where:

D = sample diameter

L = sample length

r = element distance from the generatrix on which the load is; and an horizontal tensile σ_o stress given by:

$$\sigma_o = \frac{2P}{\pi \cdot LD} = \frac{2P}{\pi \cdot A} = f_t \qquad (2)$$

where:

A = LD = area of the conventional vertical surface on which the failure occurs.

In the following figure the test geometry for samples used during the 1997 investigation is reported.

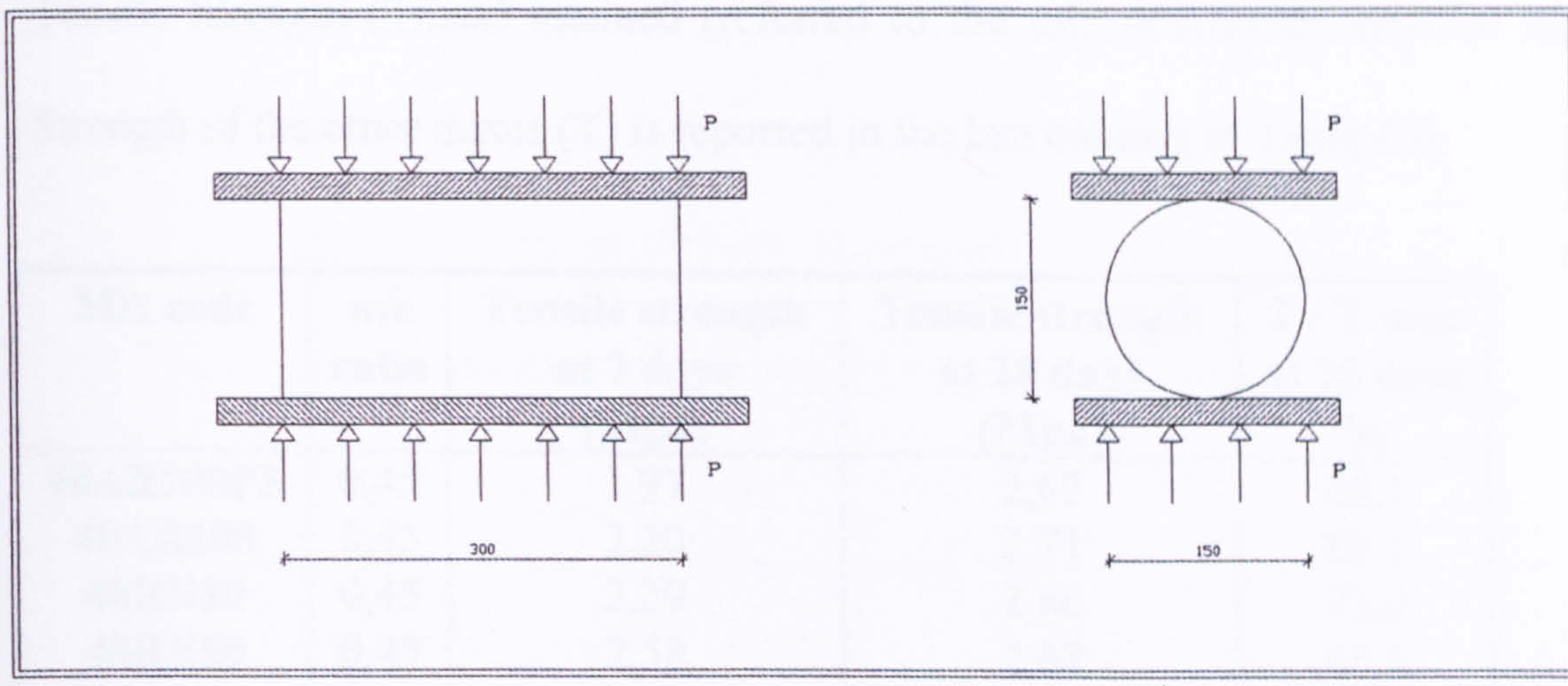


Fig. 4.5 Test geometry for tensile test in the 1997 investigation.

Figure 4.6 shows the sample during the tensile test of 1997.

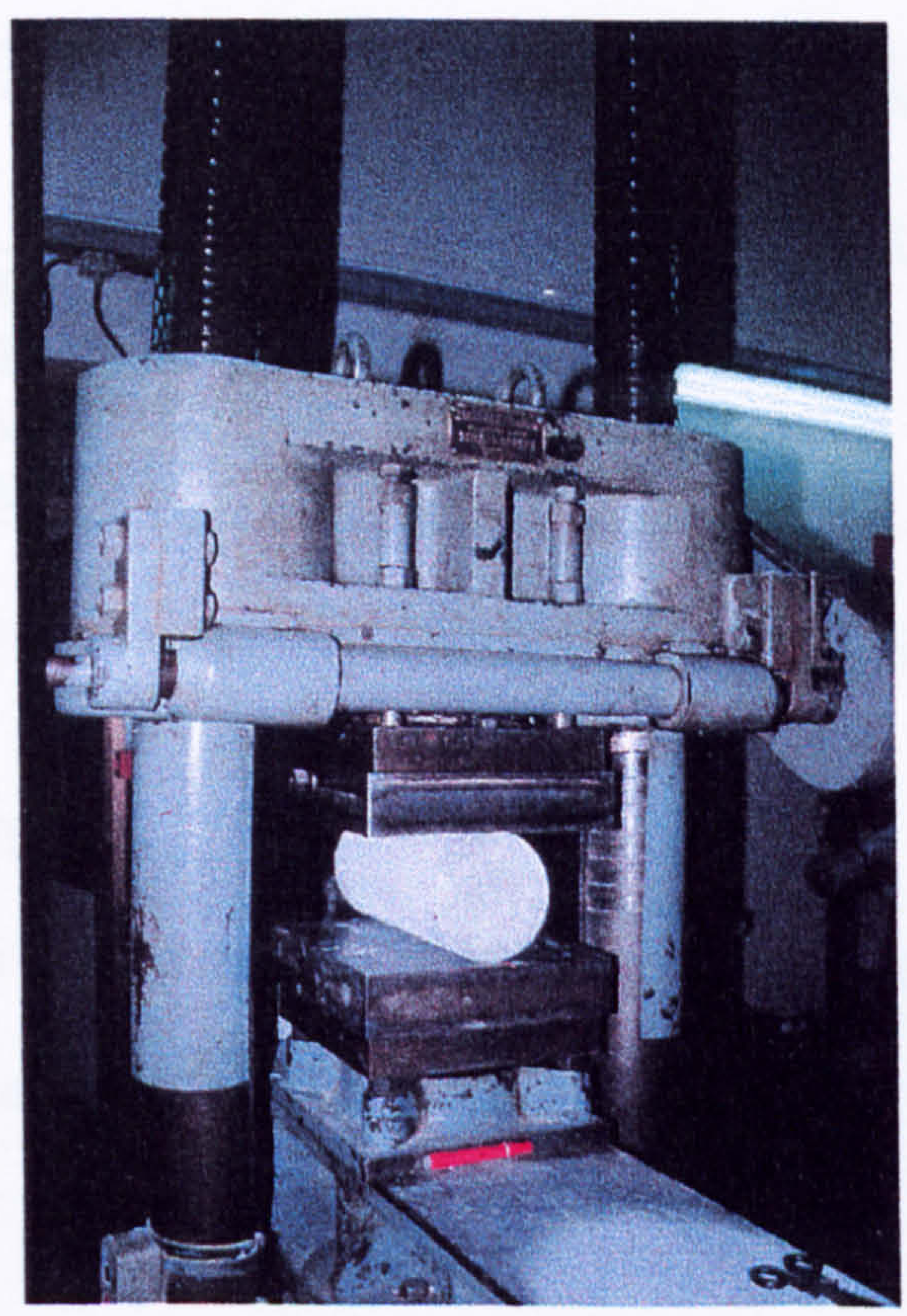


Fig. 4.6 Tensile strength equipment for the 1997 investigation.

Test results for 1995 are given in Table 4.6. A comparison between the maximum Tensile Strength (Tmax) attained (referred to the mix 40AN100) and the Tensile Strength of the other mixes (T) is reported in the last column of Table 4.6.

Mix code	w/c ratio	Tensile strength at 7 days (Mpa)	Tensile strength at 28 days (Mpa)	T / T max at 28 days %
40AR100PS	0,45	1,97	2,67	68,1
40AR100	0,45	2,20	2,71	69,1
40RN30	0,45	2,29	2,86	73,0
40RN50	0,45	2,58	2,67	68,1
40RN70	0,45	2,83	3,79	96,7
40AN100	0,45	3,06	3,92 (T max)	100,0

Tab. 4.6 Tensile strength tests results for 1995 investigation on RAC.

In Table 4.7 test results for 1997 are reported.

Mix code	Tensile strength at 7 days (Mpa)	Tensile strength at 28 days (Mpa)
525MAR3	2,21	2,79
525MAR4	1,94	2,60
40AN1	4,27	4,24
40AN2	3,85	3,68

Tab. 4.7 Tensile strength tests results for 1997 investigation on RAC.

4.3.3 **Flexural Strength**

The test has been carried out using a press with a maximum load of 10 tons on samples of 100 mm x 100 mm x 500 mm (Figure 4.7). Third point loading was adopted. .

The flexural strength f_h is given by:

$$f_h = \frac{lP}{b \cdot d^2} \qquad (3)$$

where:

P failure load (N);

l span between the support (400 mm);

b and d sample transversal dimension (100x100 mm).

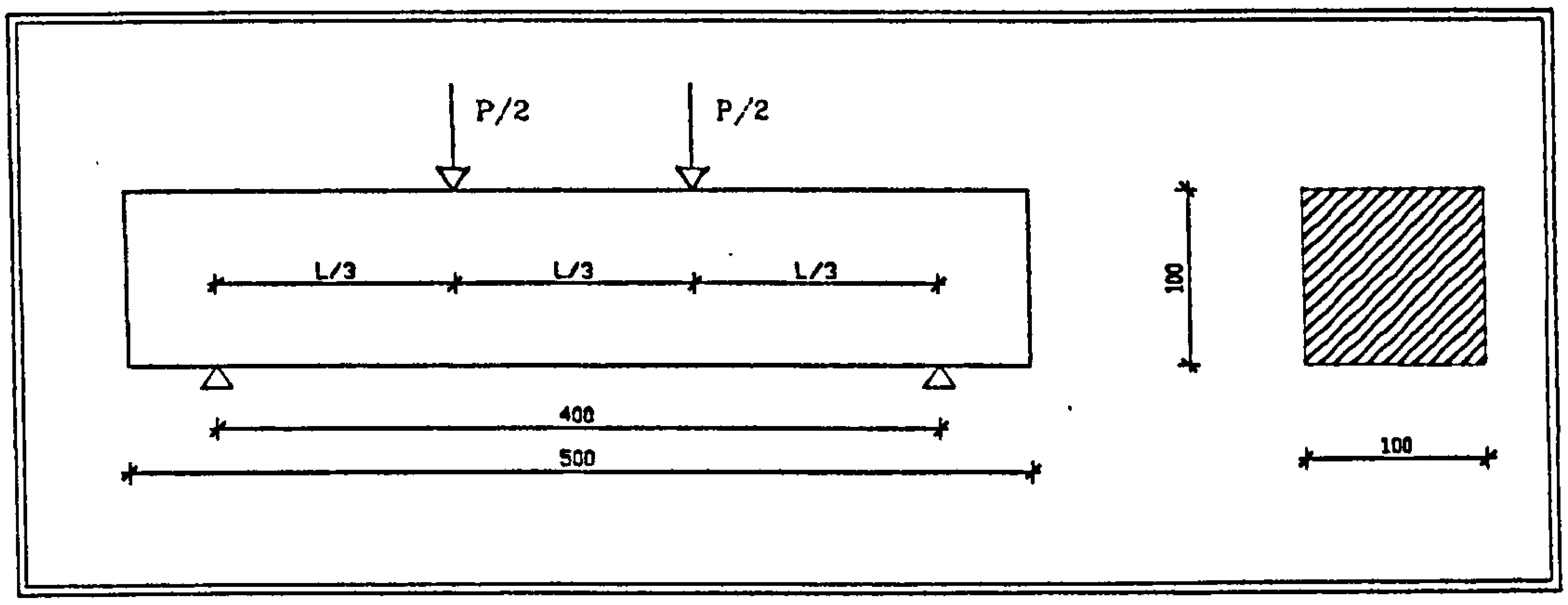


Fig.4.7 Flexural strength test geometry.

For the 1995 investigation test results are reported in Table 4.6 and for the 1997 investigation results are showed in Table 4.7.

Mix code	w/c ratio	Flexural strength at 7 days (Mpa)	Flexural strength at 28 days (Mpa)
40AR100PS	0,45	3.48	4.56
40AR100	0,45	3.12	3.38
40RN30	0,45	3.36	4.65
40RN50	0,45	4.03	4.51
40RN70	0,45	4.20	5.28
40AN100	0,45	3.79	5.48

Tab. 4.6 Tensile strength tests results for 1995 investigation on RAC.

Mix code	Flexural strength at 3 days (Mpa)	Flexural strength at 7 days (Mpa)	Flexural strength at 28 days (Mpa)
525MAR3	2,51	3,28	4,20
525MAR4	2,31	4,00	4,40
40AN1	8,80	6,44	8,40
40AN2	6,20	6,00	6,00

Tab. 4.7 Flexural strength tests results for 1997 investigation on RAC.

4.3.4 Modulus of Elasticity

Test geometry for determination of the Modulus of Elasticity in 1995 experimentation is shown in Fig. 4.2 b). It was measured on cylinder samples of 200 mm height and 100 mm diameter.

Test results are reported in Table 4.8.

Mix code	Modulus of Elasticity at 28 days (Mpa)
40AR100PS	19 345
40AR100	16 642
40RN30	17 249
40RN50	17 480
40RN70	18 149
40AN100	24 927

Tab. 4.8 **Modulus of Elasticity test results for 1995 investigation on RAC.**

During 1997 the modulus of elasticity was determined on cylinder samples (h=300 mm and diameter d=150 mm).

Deformations have been measured with three "strain gauge PL–30-11" with 120 ±0.3 Ω and length of 60 mm, gauge factor 2.10, placed at 120° between them as reported in Figure 4.8.

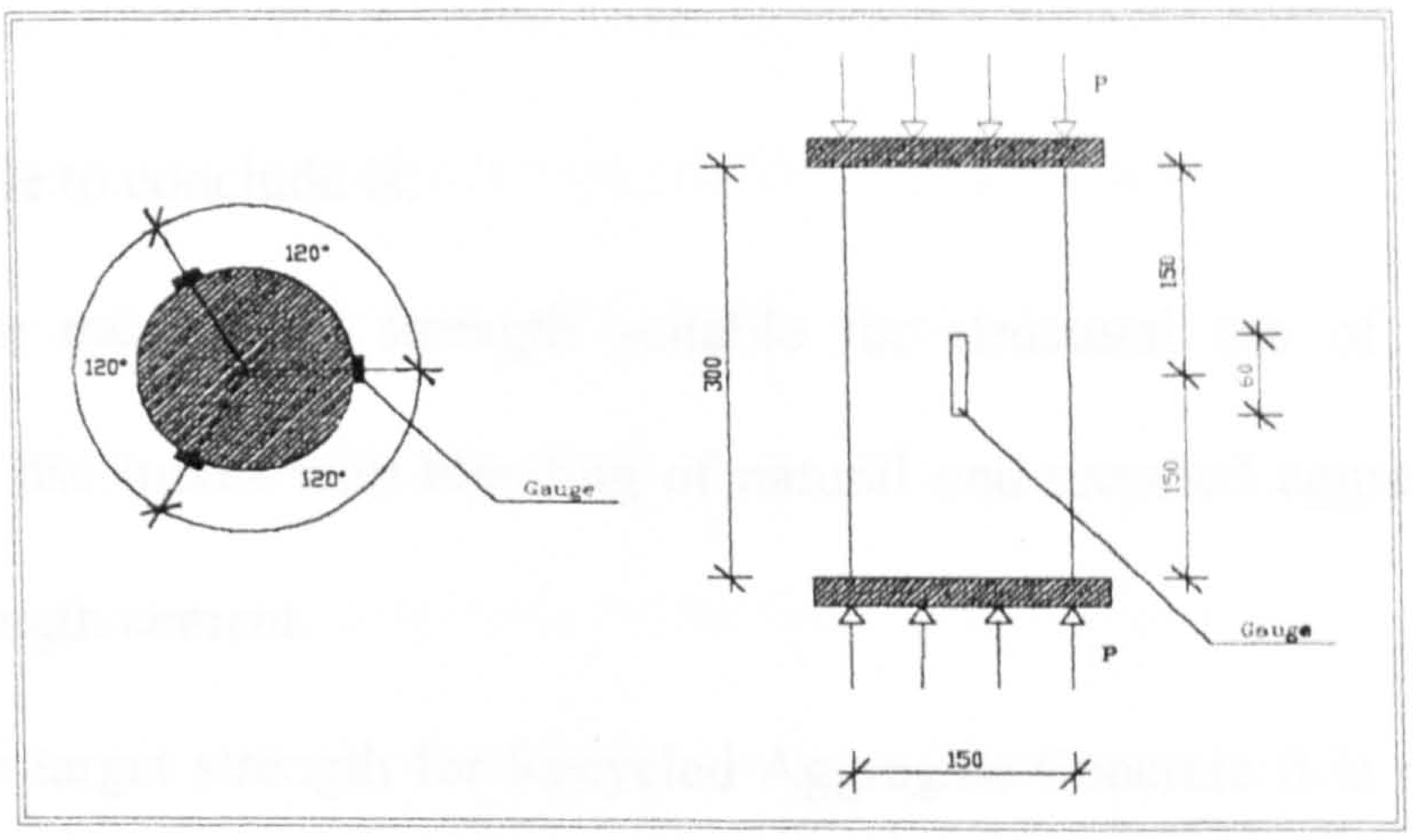


Fig. 4.8 Modulus of Elasticity test geometry for 1997 investigation.

The sample surfaces have been smoothed with carborundum before testing. The load rate has been 1/10 of the failure load and deformation was recorded for each rate. 1/3 of the failure load was reached. Three load cycles were carried out.

Test results are shown in Table 4.9.

Mix code	Modulus of Elasticity at 28 days (Mpa)
525MAR3	23 535
525MAR4	25 437
40AN1	26 765
40AN2	28 008

Tab.4.9 Modulus of Elasticity tests results for 1997 investigation on RAC.

The previous investigations on mechanical properties of hardened RAC show that it is possible to conclude that RAC can be considered as a construction material.

Further study should be carried out to check its time dependant behaviour (creep and durability).

What it is possible to conclude is:

1. To reach the mechanical strength suitable for structural use of RAC, it is necessary to use mixes with blending of natural and recycled aggregate and to use high strength cement.
2. To attain the target strength for Recycled Aggregate Concrete it is necessary to follow mix-design procedures specially modified for RAC.
3. It has been demonstrated that using a 52.5 Portland cement it is possible to reach a designed target strength of 50 MPa with 100% of Recycled Aggregate. This confirms the possibility of using RAC for structural purposes at least from the compressive strength point of view.
4. If it is considered that to produce prestressed elements is necessary to have a concrete with a minimum Compressive Strength of 40 Mpa to sustain precompression, Table 4.5 shows that it has been demonstrated that it is possible to produce a Recycled Aggregate Concrete suitable for prestressed elements.
5. The use of RAC in pre-cast work can guarantee a better quality of the Recycled Aggregate Concrete than in-situ and it can reduce the risk of errors in mix proportioning.

4.3.5 Bonding surface between Recycled Aggregate and new cement paste: petrografic analysis of RAC

In Recycled Aggregate Concrete (RAC) the aggregate is not a natural aggregate from the same rock fragment. It can be a mix of different rocks with different rheological properties bonding together with the old cement paste [39].

Normally the waste elements are stacked for long periods outside before being processed and recycled. It is therefore evident this material could have a progressive alteration that could influence its behaviour and each rock inside the recycled granule could have a different alteration depending on its mineral sources.

The aim of this research is to understand what level of alteration is reached in the recycled aggregate and how it could influence the behaviour of the new concrete.

Failure mechanics is one of the aspects involved in this process and this is briefly discussed.

A petrographic analysis has been carried out on samples of Recycled Aggregate Concrete.

The microscopic analysis was performed using a ZEISS optical polarizer microscope on six thin slides made with different orientation from different parts of the same sample of Recycled Aggregate Concrete.

The objective being to study the petrographic and geological characteristics of the aggregate and the nature of the bonding between the aggregate and the new cement paste.

The petrographic analysis can be used to determine the composition of the aggregate.

In this case it was found to be composed of:

* rock fragments;
* mono-mineral granules;
* concrete fragments of different shape and dimensions.

In the concrete fragments, granulated elements were found which had the same petrographic and/or mineralogical nature of the natural aggregate that has been found isolated in the new cement paste.

In general, for all three, good adhesion to the new cement paste was observed and only in few points of the contour of the concrete fragments was there separation between the aggregate and the old cement paste.

In some thin sections micro cracking was observed in the aggregate/cement paste interface. The pattern was short and irregular. This could be due to the shrinkage of concrete.

In Figure 4.9, 4.11, 4.13 sections are reported, 25 magnifications, made with a microscope with interference with crossed ray.

In Figure 4.10, 4.12, 4.14 the same sections are reported but made with a microscope with single ray.

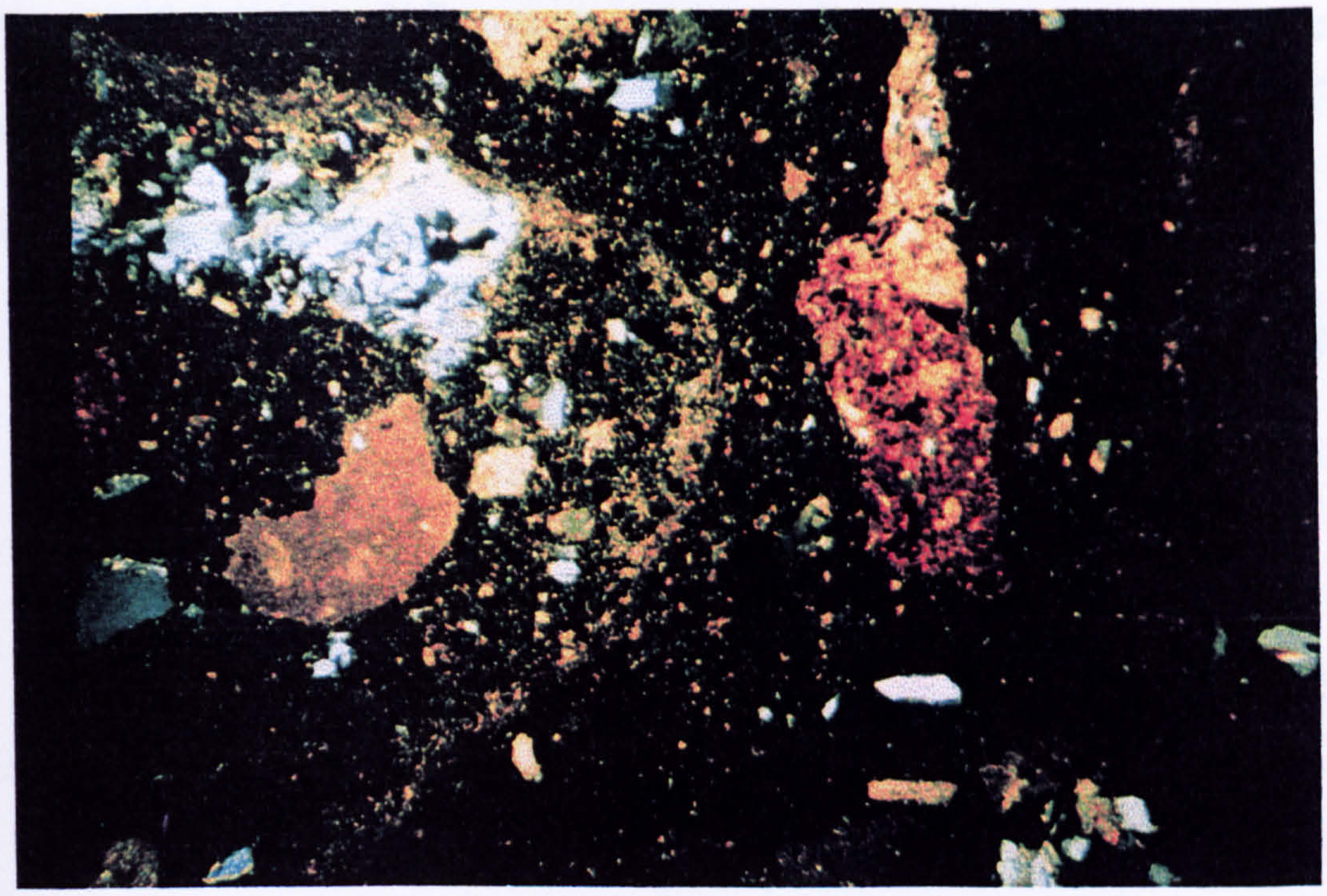


Fig. 4.9 Thin section of a RAC sample using a polarizer microscope with interference with crossed ray.

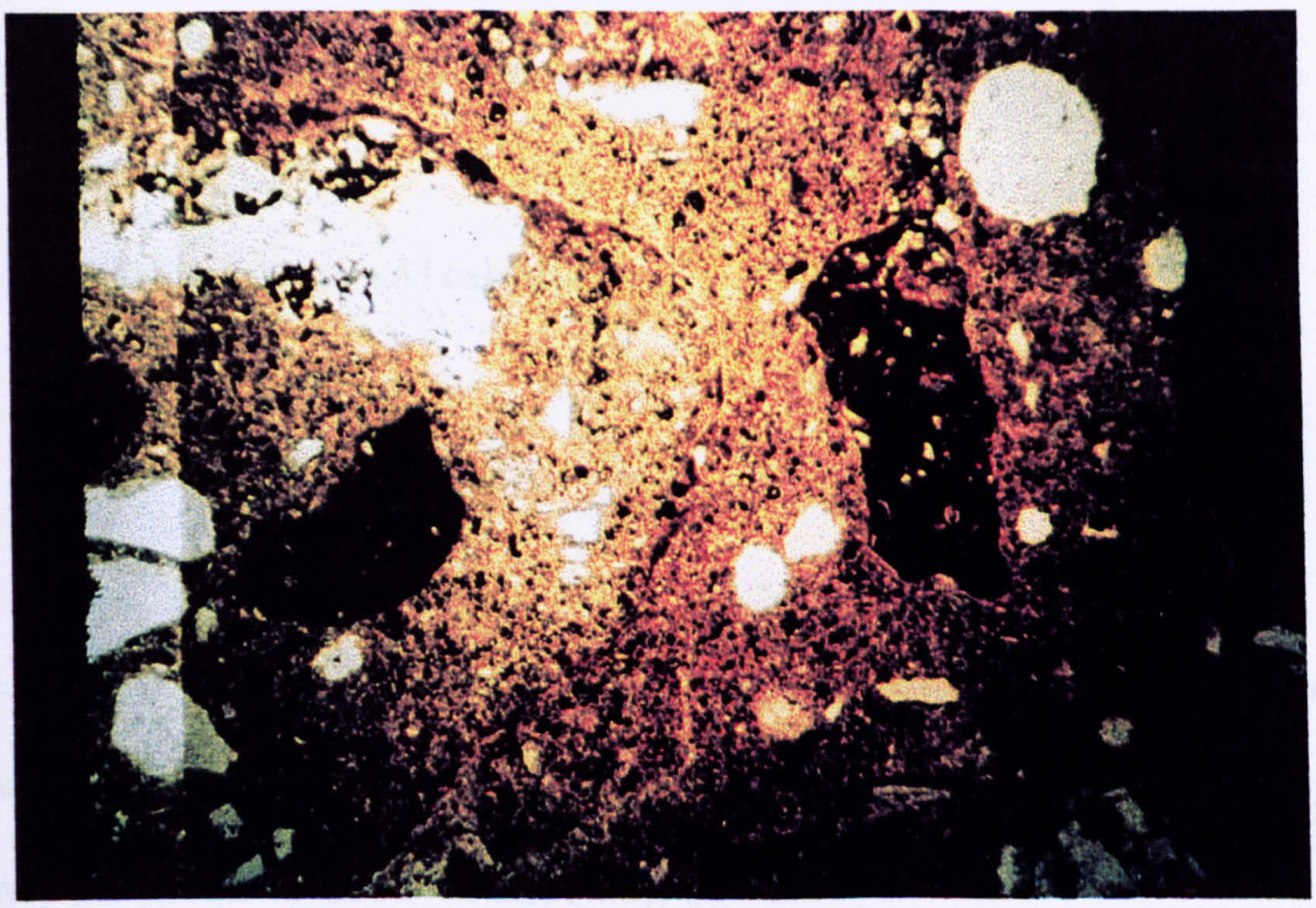


Fig. 4.10 Thin section of a RAC sample using a polarizer microscope with interference with single ray.

It was noticed that the rock fragments in the sample have a diversified petrographic nature.

In fact they have been identified as fragments of:

- limestone of different texture and composition: mica, biomica, biospariti, sand limestone;
- marble with medíum and coarse grain;
- quartzer feldspatic sandstone and mica-shist with calcium cement;
- siltiti;
- flintstone;
- quartzite and mica-quartzite;
- serpentine.

Among the mono-mineral granules, they have been identified as:

- quartz;
- alkaline feldspar and calcosodici;
- calcite;
- chlorite.

The different minerals appear mostly unbroken and not altered but the feldspars present in the sandstone in the old cement paste show evident signs of alteration (clayey phenomenon).

The micro photos performed on the thin section of RAC show the texture of the material and in particular emphasise the bonding surface between the granule of the old cement paste and the new cement paste.

These granules appear on the whole compacted. They are distinguished by a more diffused carbonation of the cement paste.

It seems useful to remember that the carbonation phenomenon occurs due to the penetration of the CO_2 (contained in air) into the superficial surface of concrete. CO_2 reacts with the alkaline hydroxide making the corresponding carbonates and consequently with the calcium hydroxide dissolved into the water solution provokes a progressive degradation of concrete.

Anyway it should be noted that during the preparation of the thin sections of RAC the abrasive process required to reduce the thickness of the material has provoked the disaggregation of some of these composite granules. It is probable that these were granules with a lower level of cementation.

The recycled aggregate comes from unknown sources of demolition waste and consequently from concretes with different w/c ratio.

The correlation between the pore system in hardened cement paste, the w/c ratio and the carbonation of concrete is well known.

It is therefore logical that concretes with high w/c ratio could be less durable compared with concrete with low w/c ratio.

In any case this consideration should not influence the recycling process in a site-plant because poorer concretes cannot be discarded.

For this reason particular attention should be paid to the design of RAC to reduce the risk of diffused carbonation.

A solution could be to use a low w/c ratio or a particular admixture that can prevent the carbonation phenomenon.

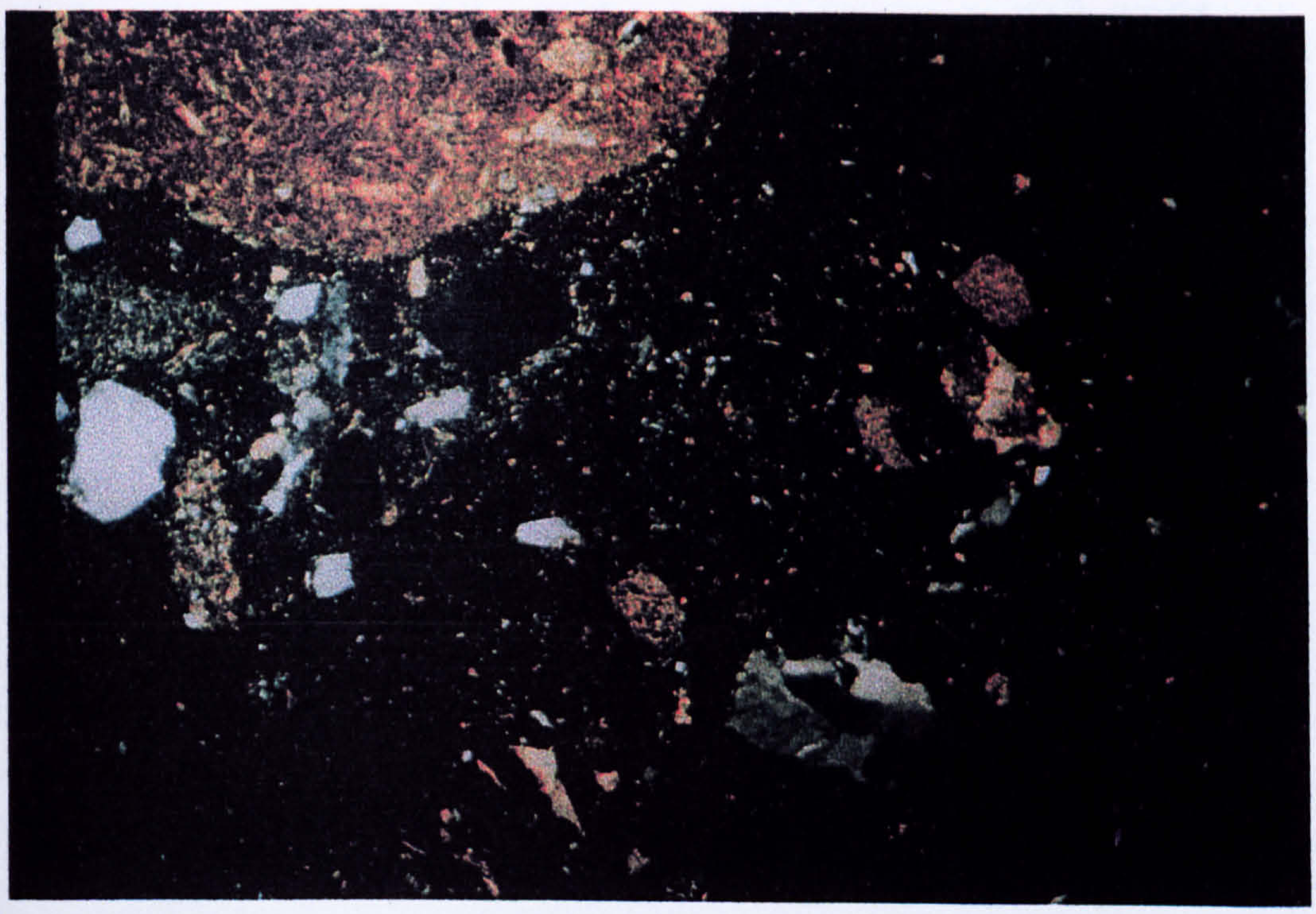


Fig. 4.11 Thin section of a RAC sample using a polarizer microscope with interference with crossed ray.

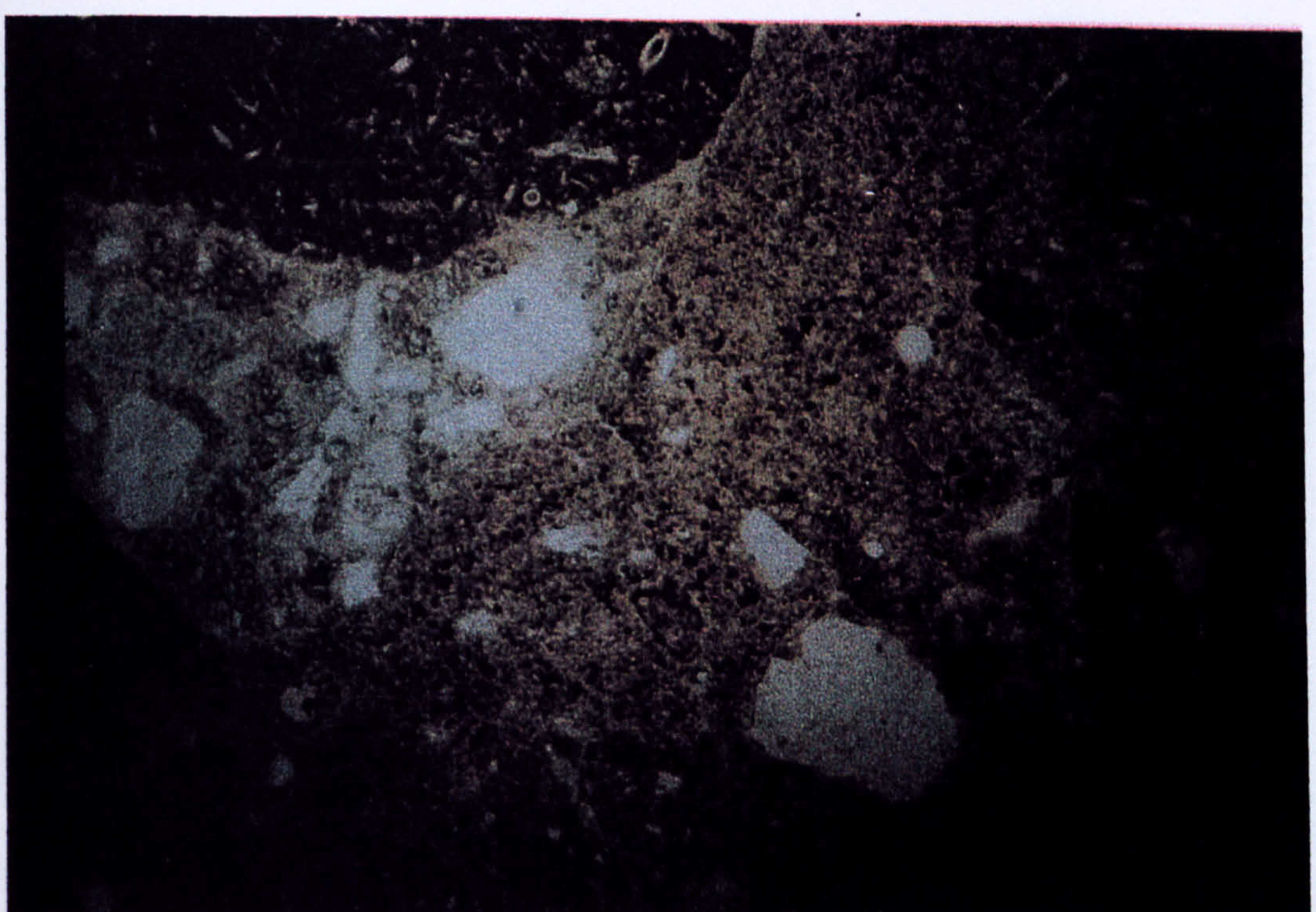


Fig. 4.12 Thin section of a RAC sample using a polarizer microscope with interference with single ray.

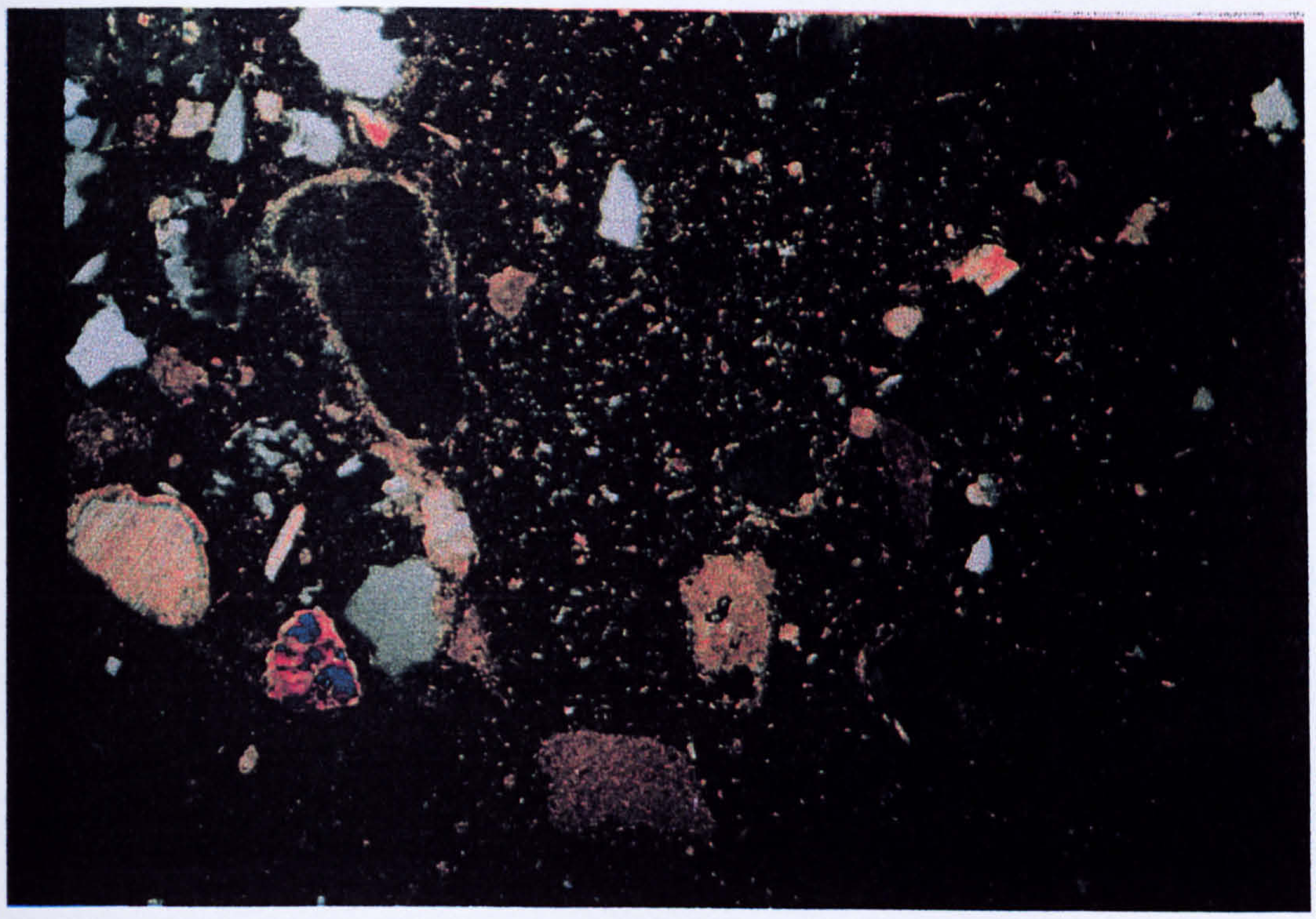


Fig. 4.13 Thin section of a RAC sample using a polarizer microscope with interference with crossed ray.

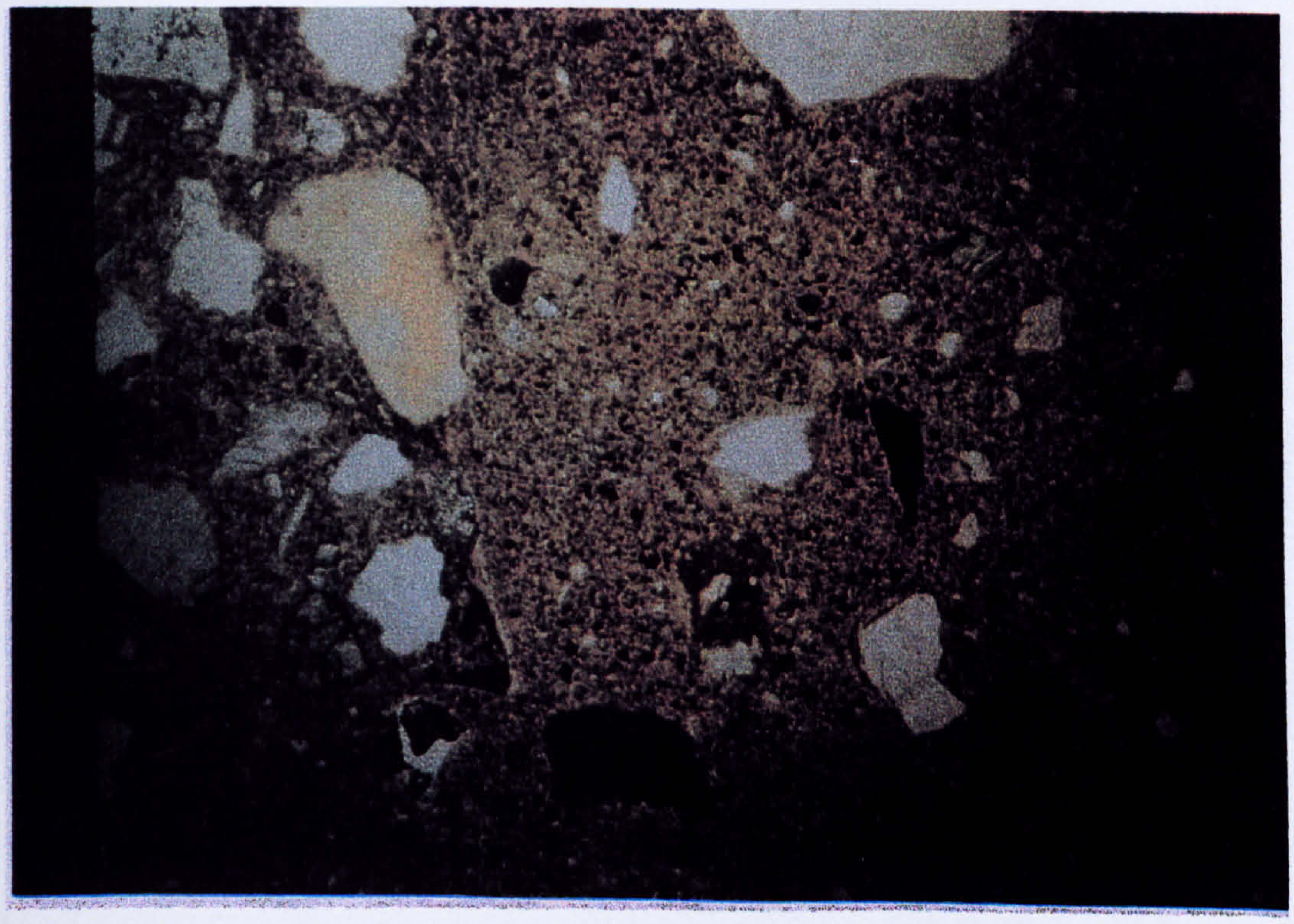


Fig. 4.14 Thin section of a RAC sample using a polarizer microscope with interference with single ray.

The use of admixtures is recommended so that the benefits of using RAC are not lost due to the presence of contaminants [2], [3], [4], [6].

Normally the failure of concrete could occur through the cement paste, through the aggregate or along the bonding surface between the aggregate and cement paste.

In Recycled Aggregate Concrete the aggregate is its own concrete and so the failure mechanism could occur with the same process.

If the bonding surface is altered, sufficiently to be seen in the thin section of concrete, this phenomenon could be accelerated.

From this analysis it could be concluded that:

1. Because carbonation of the old cement paste is one of the most common events that could occur in Recycled Aggregate it is advisable to use Recycled Aggregate Concrete designed with low water/cement ratio to reduce the pore system of the concrete [7], [8].
2. Wet curing (high humidity period) is recommended for the Recycled Aggregate Concrete so that hydration of cement continues and reduces the depth of carbonation.
3. The type of cement used to make the Recycled Aggregate Concrete is important (such as blended cement: silica and fly ash cement). The blended cement leads to a lower $Ca(OH)_2$ content in the hardened cement paste and a smaller amount of CO_2 is required to remove $Ca(OH)_2$ by producing $CaCO_3$ [43].
4. RAC with higher strength was less vulnerable to carbonation.

5. Prefabrication can encourage the use of RAC for its wet curing (e.g. steam curing) and for its high compressive strength in the production of prestressed elements [41], [42].

4.4 Durability of Recycled Aggregate Concrete

Durability is a very important aspect for the structural use of RAC (Recycled Aggregate Concrete) especially considering the unknown source of the RA (Recycled Aggregate) and the contaminants in it that could create problems with the reinforcement inside the structural elements.

For this reason the durability of the RA and RAC have been investigated during this research project.

Los Angeles test and chemical analysis for contaminant have been carried out on samples of RA (as reported in Chapter Three).

Freezing and thawing tests have been performed on cubes of RAC. The results have been processed and the loss in compressive strength and the loss in weight of cubes have been evaluated.

The water absorption of hardened RAC was recorded and considerations on durability of hardened RAC have been suggested for a structural use of RAC.

Finally tests for chemical attack of RAC have been performed.

All tests procedures and results on hardened RAC are reported in this section.

4.4.1 Initial surface absorption

This test was performed on cubes of 100 mm x 100 mm x 100 mm not oven dried but in laboratory conditions.

Test results are reported in Table 4.10

Mix code	Initial Surface Absorption (ml / m^2 per second)			
	10 minutes	30 minutes	1 hour	2 hours
40AR100	3.9	1.4	1.0	0.35
40RN50	3.3	1.5	1.3	0.2
40AN100	0.5	0.35	0.2	0.08

Tab. 4.10 Initial Surface Absorption test on RAC.

It can be noticed from Table 4.10 that the Initial Surface Absorption is higher for the mix with 100% of Recycled Aggregate (40AR100) than for the mix with 100% of Natural Aggregate (40AN100).

In fact in the first ten minutes the mix 40AR100 shows an Initial Surface Absorption of almost 7.8 times the value of the mix 40AN100. This value is reduced to 4.35 times after two hours.

Surprising is the value of the mix 40RN50 that shows values close to the mix 40AR100.

4.4.2 **Frost resistance**

In order to investigate the durability of RAC freezing and thawing tests have been carried out.

This test has been performed at 256 days. During this time all cubes were water cured at 20 °C.

No cubes were tested before the freezing and thawing test. For this reason the compressive strength of concretes at this age have been determined by the maturity equation suggested by Plowman, as reported by Neville in [43]. The compressive strengths at 256 days are reported in Table 4.11.

Mix code	Compressive strength at 28 days (Mpa)	Compressive strength at 256 days by [38] (Mpa)
40AR100	36.3	39.1
40RN30	36.6	39.4
40RN50	38.0	40.9
40RN70	43.0	46.3
40AN100	45.0	48.4

Tab. 4.12 Compressive strength test results for 1995 investigation on RAC.

The cubes were weighed before being placed in the cabinet for the freezing and thawing test. 100, 200, 300 cycles of freezing and thawing were performed. One cycle (six hours) was of two hours of freezing (until -20°C) and four hours of thawing (until +20°C).

Figure 4.16 reports the diagram of the compressive strength versus the number of cycles of freezing and thawing. Fig. 4.16 shows a reduction of 10% in the compressive strength of RAC after 300 cycles of freezing and thawing (-20°C / 20°C in six hours).

The loss of weight is almost the same for all the different mixes and it is equal to 1 % by weight of concrete.

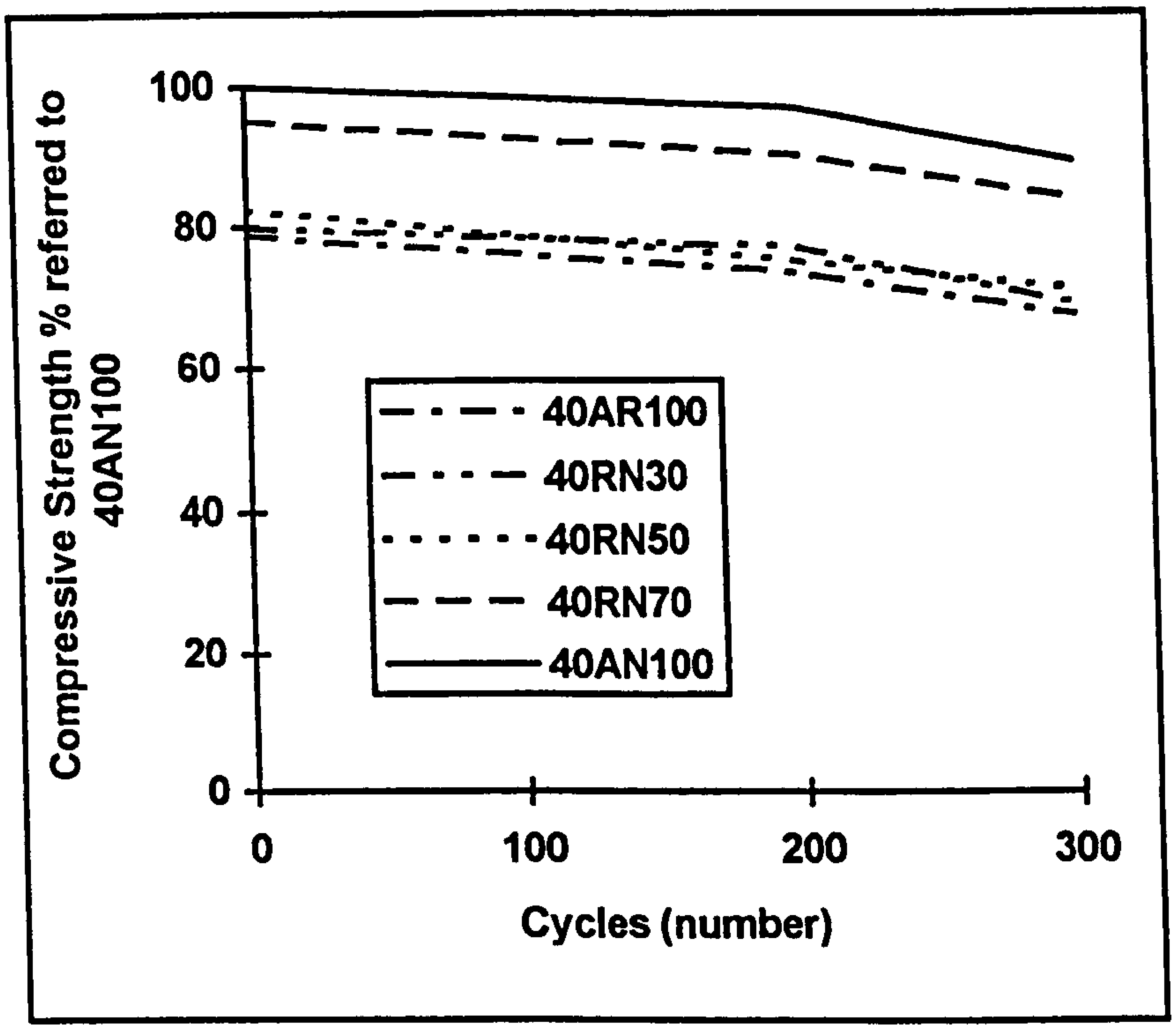


Fig. 4.16 Compressive strength (%) of RAC versus number of cycles of freezing and thawing.

Also it can be concluded that the RAC shows good behaviour considering the losses of strength and the losses of weight after 300 cycles of freezing and thawing.

The reduction in the freezing and thawing resistance is independent of the proportion of replacement of natural aggregate. The freezing and thawing tests indicate that RAC is as durable as concrete made with natural aggregates.

4.4.3 <u>Tests on sulphate resistance of RAC</u>

Cubes of 100 mm x 100 mm x 100 mm were placed into a solution of 5% of Sodium and Magnesium Sulphate.

Alternate wetting and drying cycles of one week each have been performed on samples in order to accelerate the damage due to the crystallization of salts in the pores of concrete. The effects of exposure have been estimated by the loss in compressive strength of cubes and by their loss of mass. Tests results are reported in Table 4.13.

Mix code	Time	Weight (gr)	Compressive strength (Mpa)
40AR100	0 week (0 days)	2070	44.4
	1 week (7 days) - wet -	2168	-
	2 weeks (14 days) - dry -	2132	-
	3 weeks (21 days) - wet -	2170	-
	4 weeks (28 days) - dry -	2137	39.5
	5 weeks (35 days) - wet -	2160	-
	6 weeks (42 days) - dry -	2135	-
	7 weeks (49 days) - wet -	2135	-
	8 weeks (56 days) - dry -	2165	35.0
40RN50	0 week (0 days)	2210	52.5
	1 week (7 days) - wet -	2244	-
	2 weeks (14 days) - dry -	2220	-
	3 weeks (21 days) - wet -	2243	-
	4 weeks (28 days) - dry -	2223	50.0
	5 weeks (35 days) - wet -	2240	-
	6 weeks (42 days) - dry -	2225	-
	7 weeks (49 days) - wet -	2225	-
	8 weeks (56 days) - dry -	2245	46.0
40AN100	0 week (0 days)	2325	67.0
	1 week (7 days) - wet -	2358	-
	2 weeks (14 days) - dry -	2344	-
	3 weeks (21 days) - wet -	2358	-
	4 weeks (28 days) - dry -	2345	58.0
	5 weeks (35 days) - wet -	2350	-
	6 weeks (42 days) - dry -	2340	-
	7 weeks (49 days) - wet -	2340	-
	8 weeks (56 days) - dry -	2355	54.0

Tab. 4.13 Sulphate resistance of RAC by evaluating loss in weight and loss in compressive strength.

This test has shown that there was no loss in weight of the cubes, but there was an evident loss in compressive strength. It was around 20% less than the original strength before the cycles of drying and wetting.

Chapter Five

Mechanical performances of RAC pre-stressed beams

5.1 Introduction

The main aim of this investigation is to check the possibility of introducing a recycling process in a pre-cast site plant for the production of pre-stressed structures.

It would reuse the concrete waste of the plant and thus be an environmentally friendly system.

After tests to determine the physical and chemical characteristics of RAC, the attention has been concentrated on the mechanical performances of RAC and in particular on the use of RAC in pre-stressing.

As reported in the previous Chapter more than twenty different mixes have been studied and tested in order to choose a mix for production of RAC pre-stressed beams.

Compressive strength of RAC has not been the only criteria for choosing the mix. Workability requirements have to be satisfied to be compatible with the plant production.

A mix-design procedure that can guarantee mechanical strength and acceptable slumps has been determined in this case.

Three 15.0 metres span pre-stressed concrete beams have been cast using 100% of Recycled Aggregate (RA) for the beam named 40AR100, 50% of RA and 50% of Natural Aggregate (NA) for the beam named 40RN50 and 100% of NA for the beam 40AN100.

The beams were loaded at third points and measurements were taken of load, mid-span deflection, settling at the support and stresses in the wires.

The experimental results of the tests provide the basis for comparing the non-linear analysis made with the LUSAS package.

5.2 Test of a prestressed beam made from RAC

5.2.1 Casting

16 mm maximum size Recycled Aggregate was used for the mixes with 100% of RA and 50% of RA.

River Sand and 16 mm maximum size gravel coarse Natural Aggregate was used for the mixes with 100% of NA and 50% of NA.

525 Portland Cement corresponding to the I A class of UNI-ENV 197/1 has been used for all the mixes.

The mix-design procedure adopted is reported in Chapter Four. The design target strength for the three mixes was 40 Mpa.

The mixes are coded as follows: XXNNYY. Where XX is the target strength of the mix design, NN is the type of aggregate used i.e. AR signifies all recycled, AN signifies all natural and RN signifies a blend of the two.

The last digit YY is the percentage of natural aggregate used in that mix. (eg. the mix 40 RN50 is a mix in which the 50% is recycled aggregate (all in) and the 50% is natural aggregate (sand, 16 mm).

The exact mix proportions for the mix 40AR100, 40RN50 and 40AN100 are reported in Table 5.1.

Mix code	w/c ratio	Cement kg	Free water l	Recycled Aggragate All in kg	Natural Fine Sand 0 - 4 mm kg	Aggregate Coarse 4 - 12 mm kg
40AR100	0,45	500	225	1415	-	-
40RN50	0,45	528	238	707	531	226
40AN100	0,45	556	259	-	757	757

Tab. 5.1 Mix proportions (1 m^3).

Slump tests have been carried out before casting each beam. The designed slump was 60-180 mm. This test has been necessary to ensure consistent workability in each batch. Three batches were required for each beam cast.

Test results are reported below for each mix used to cast the three beams.

Mix code	Slump (mm)		
Batch	1	2	3
40AR100	90	80	85
40RN50	60	75	80
40AN100	55	75	80

Tab. 5.2 Slump test at 5 minutes.

Together with the pre-stressed beams a series of cubes, cylinders and little beams have been cast with the corresponding dimensions: 100 mm x 100 mm x 100 mm, 150 mm x 300 mm and 100 mm x 100 mm x 500 mm.

These samples have been air cured at 25 °C like the pre-stressed beams.

Another series of the same samples have been water cured at the temperature of + 20 °C.

As can be noticed in the following Table 5.3 at 28 days those samples showed a compressive strength almost 10% higher than the corresponding samples air cured.

This is confirmation that an RAC mix can reach the designed target strength only if particular conditions such as accelerated curing are considered.

This aspect is one of a series of comments coming out from this research that may solve some of the actual limits on the use of this material.

Mix code	w/c ratio	Compressive Strength at 24 hours MPa	Compressive Strength at 3 days MPa	Compressive Strength at 7 days MPa	Compressive Strength at 28 days MPa
40AR100 air c.	0,45	29,6	32,8	35,5	38,4
40AR100 water c.	0,45	-	33,9	36,6	44,3
40RN50 air c.	0,45	30,1	36,3	42,7	43,2
40RN50 water c.	0,45	-	34,0	42,4	45,8
40AN100 air c.	0,45	33,4	44,0	49,0	51,5
40AN100 water c.	0,45	-	43,3	47,8	59,4

Tab. 5.3 Compressive strength of the three mixes used to produce the pre-stressed beams (air and water cured).

The splitting tests and flexural tests were carried out at 7 and 28 days with the same procedure followed in Chapter Four. The specimens were air cured. Test results are reported in Table 5.4 and 5.5.

Mix code	w/c ratio	Flexural Strength at 7 days MPa	Flexural Strength at 28 days MPa	Tensile Strength at 7 days MPa	Tensile Strength at 28 days MPa
40AR100 air c.	0,45	3,24	3,94	2,32	2,49
40RN50 air c.	0,45	3,60	4,56	2,54	2,62
40AN100 air c.	0,45	4,10	5,30	2,69	2,80

Tab. 5.4 Flexural and Tensile strength of the three mixes used to produce the pre-stressed beams (air cured).

The modulus of Elasticity values at 7 and 28 days are reported in the following Table 5.5 for the three mixes.

Mix code	w/c ratio	Modulus of Elasticity at 7 days Mpa	Modulus of Elasticity at 28 days MPa
40AR100 air c.	0,45	15 240	23 280
40RN50 air c.	0,45	20 760	26 520
40AN100 air c.	0,45	23 690	33 922

Tab. 5.5 Modulus of Elasticity of the three mixes used to produce the pre-stressed beams (air cured).

The variation of these Modulus of Elasticity is reported in the $\sigma-\varepsilon$ diagram (Figure 5.1) for the mixes 40AR100 and 40AN100 (Figure 5.2).

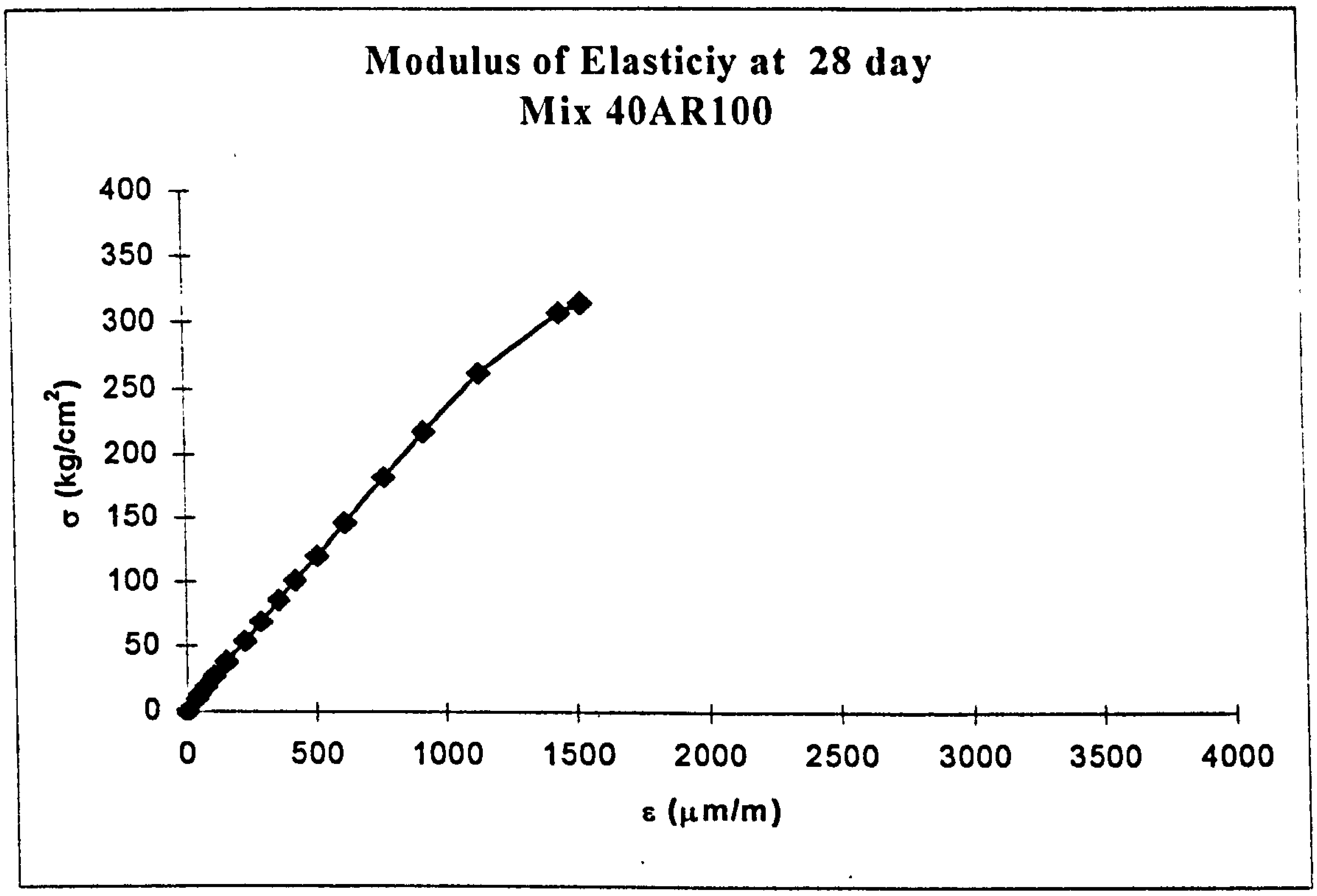


Fig. 5.1 Modulus of Elasticity for the mix 40AR100 (air cured).

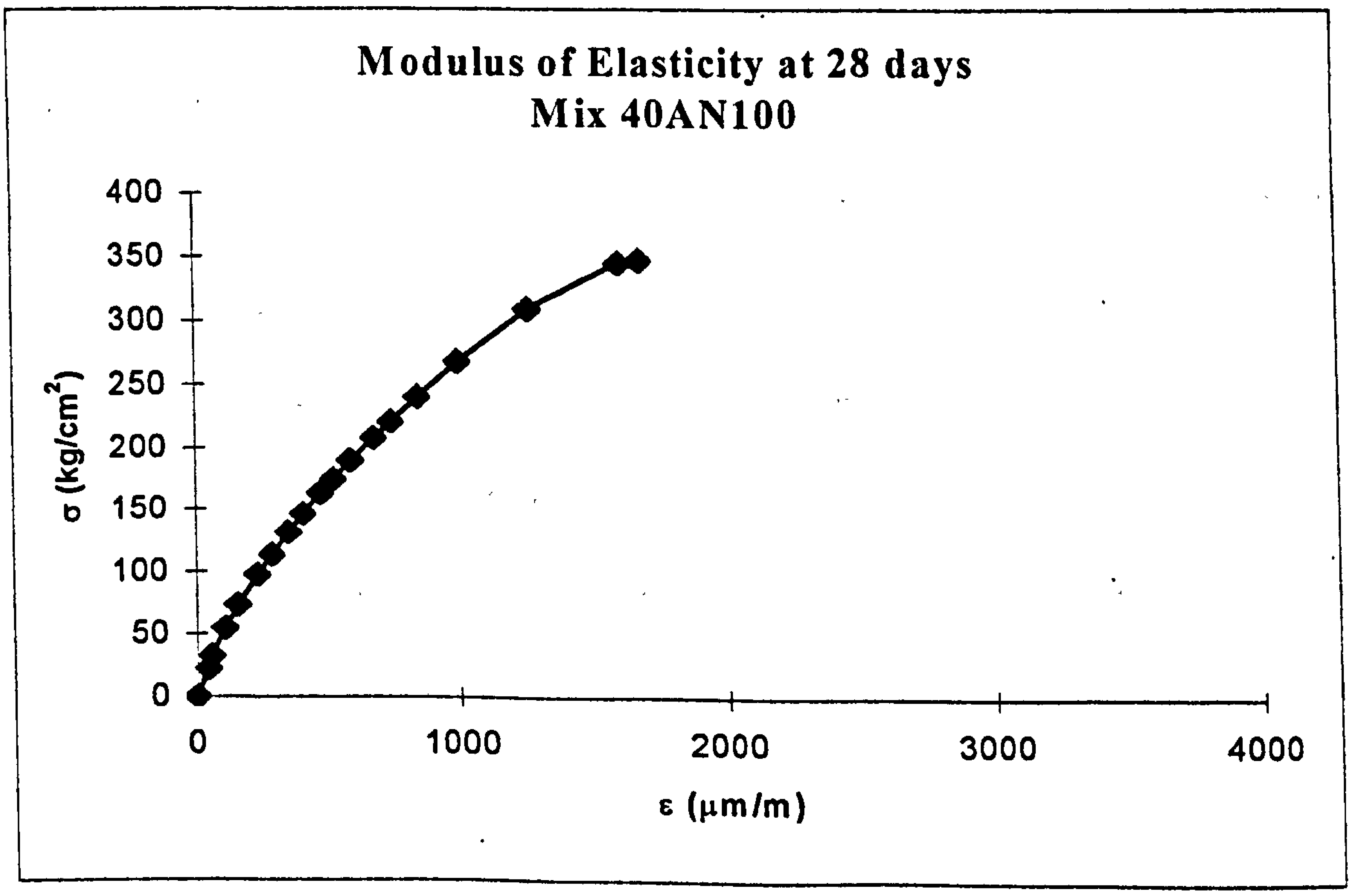


Fig. 5.2 Modulus of Elasticity for the mix 40AN100 (air cured).

Comparing the above variation of the Modulus of Elasticity of the mix with 100% of RA (40AR100) and the mix with 100% of NA (40AN100) it can be seen that the mix 40AR100 is more linear than the 40AN100 mix.

This aspect together with the lower value of the Modulus of Elasticity at 28 days, make the beam cast with the 40AR100 mix a more ductile material compared with the corresponding conventional concrete (40AN100).

The three pre-stressed beams made from RAC have been cast in a metal mould 130 metres long together with other beams during an ordinary production cycle in the plant.

This means that no significant modifications have been performed to the production procedure in order to facilitate the RAC beams casting.

The wires were pre-tensioned. The pre-stressing reinforcement was 10 tendons of 93 mm^2 of area each, with a pre-stressing stress of 142.5 kg/mm^2 and a total prestressing force of 132.5 tons for each beam.

The design of all beams was in accordance with the requirements of the Italian Standard for ordinary pre-stressed concrete [44].

Accelerated curing was adopted for all beams as is the case in normal production in the plant and the thermal cycle inside the beam has been recorded.

The beam 40AR100 cast with a concrete made with 100% of RA performed very well at the release of the tendons and it had a pre-camber almost double compared with the beam 40AN100% cast with a concrete made with 100% of NA.

This is due to the low E-value of the 40AR100 mix.

Slump tests have been carried out before casting each beam as shown in the previous section.

The batches have been prepared in automatic controlled mixers (see *Fig. 5.3*).

Fig. 5.3 Automatic controlled mixer.

The concrete vibration has been performed by immersing internal vibrators (poker) for a period of almost 30 seconds (see *Fig . 5.4*) and with the use of external vibrators rigidly clamped to the framework.

Fig. 5.4 Casting and vibration phases.

After casting, the top concrete surface of the beams was covered with overlapping polyethylene sheeting.

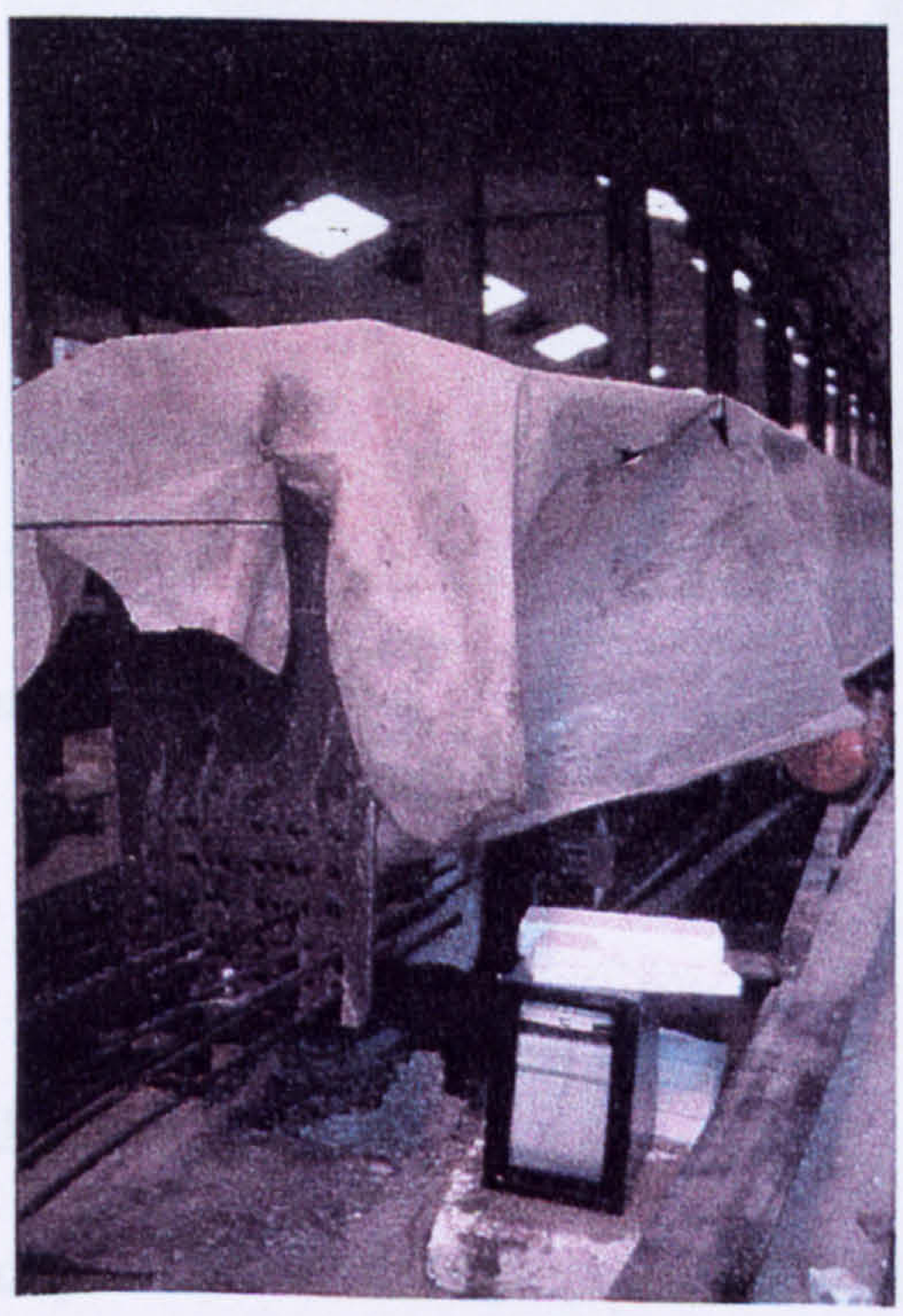


Fig. 5.5 Accelerating curing of the pre-stressed beams.

After almost 18 hours of accelerated curing (Figure 5.5) with the use of electric resistors applied to the framework, the pre-stressed beams have been demoulded (Figure 5.6).

The pre-stressing tendons used during this experimentation were produced by

Siderurgica Latina Martin S.p.a and they have the characteristics reported in the Appendix A.

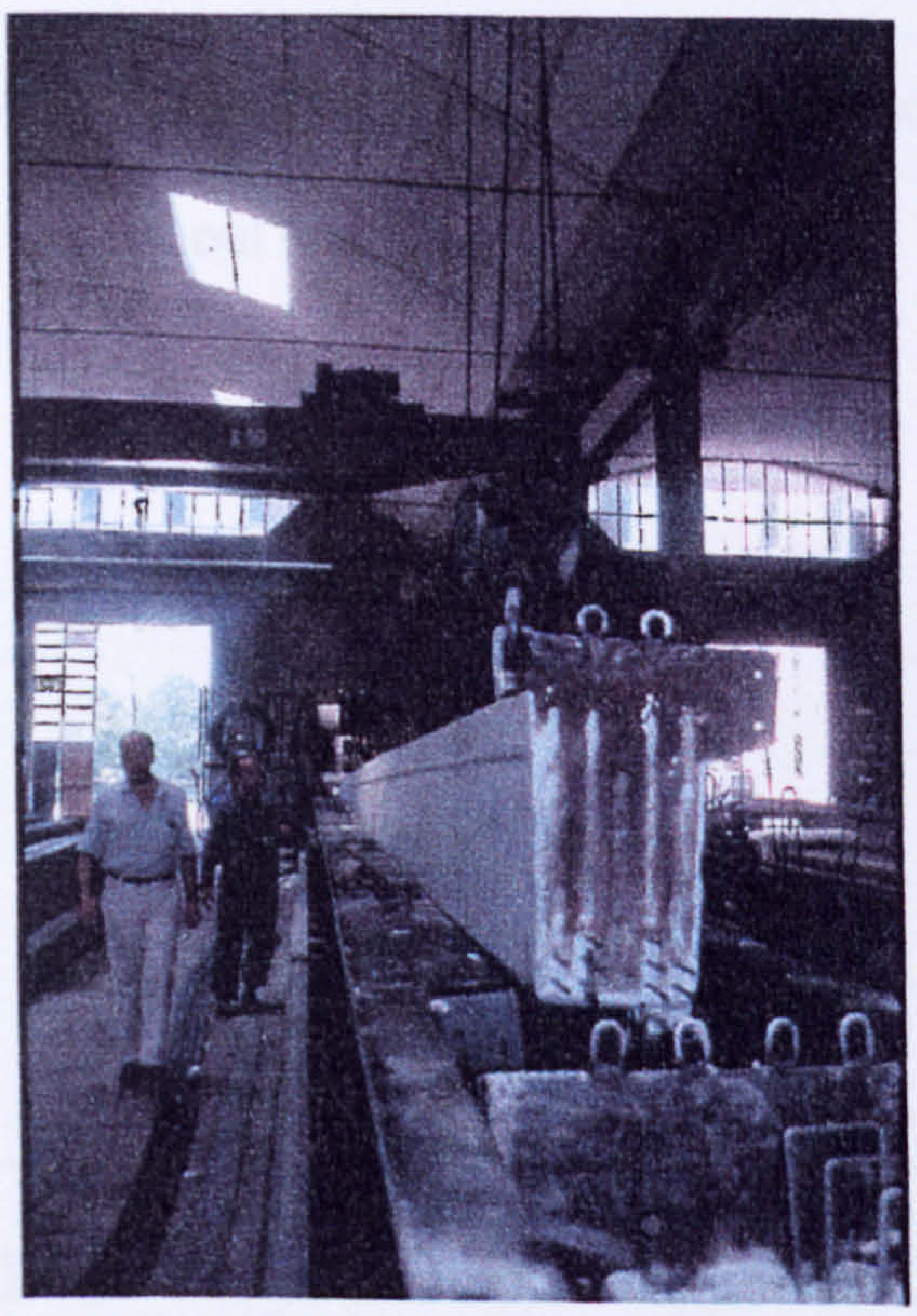


Fig. 5.6 Demoulding phases.

5.2.2 **Test arrangement and procedure**

After air curing for 28 days at 25 °C the three pre-stressed beams named *40AN100*, *40RN50* and *40AR100* have been tested to failure.

The simply supported beams were loaded at third points and measurements were taken of load, mid-span deflection, settling at the support and stresses in the wires.

The strain level in the top part of the beam was also measured at discrete points with the use of strain gauges.

The test set-up adopted for all three beams is shown in Figure 5.7.

The load rate during the test was chosen to be 500 kg / 3 min.

All crack patterns have been followed and recorded during the test.

A supplementary investigation on the beams made with the use of ultrasonic pulse velocity tests has been carried out (see Chapter 6).

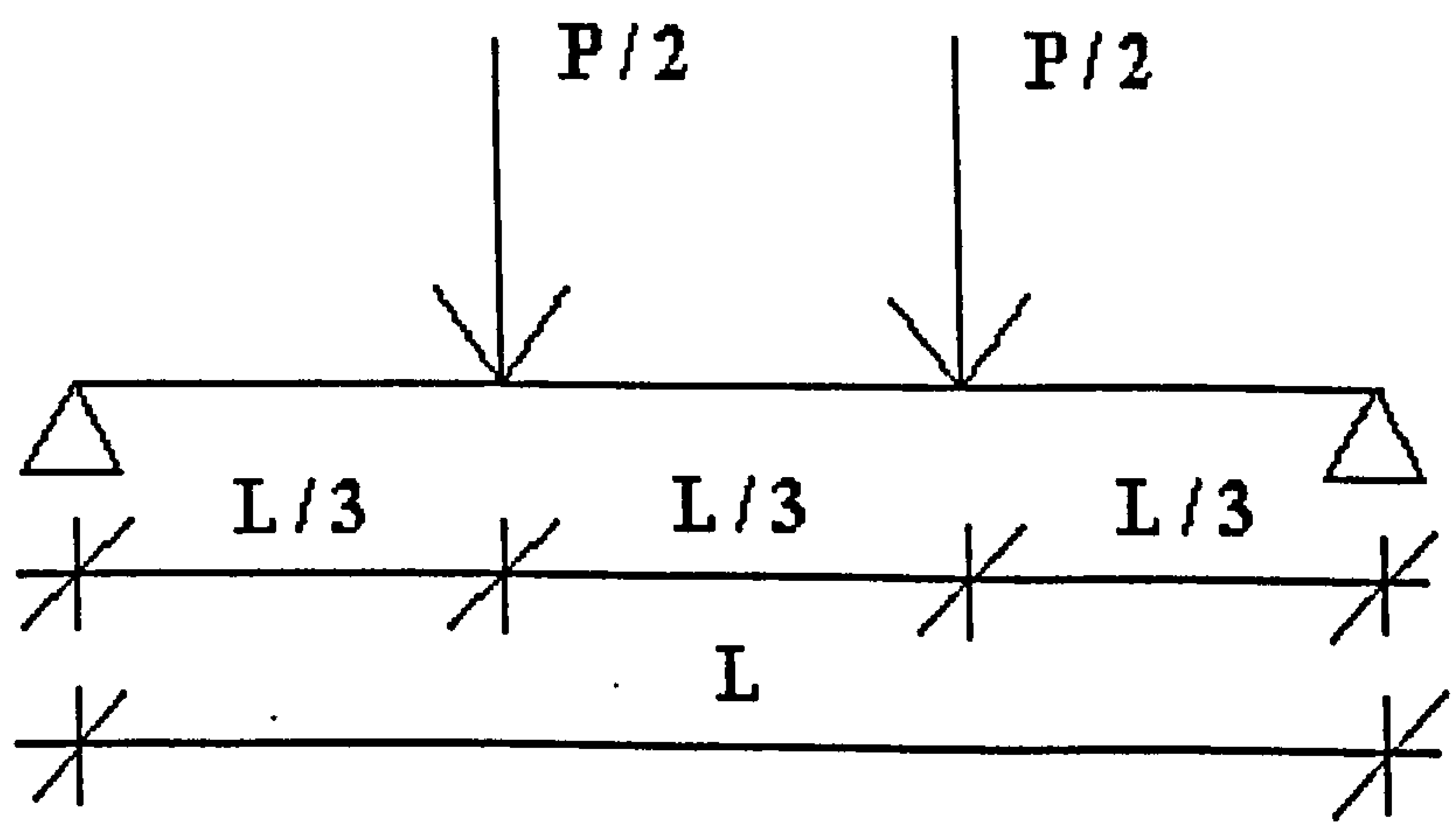


Fig. 5.7 Load arrangements.

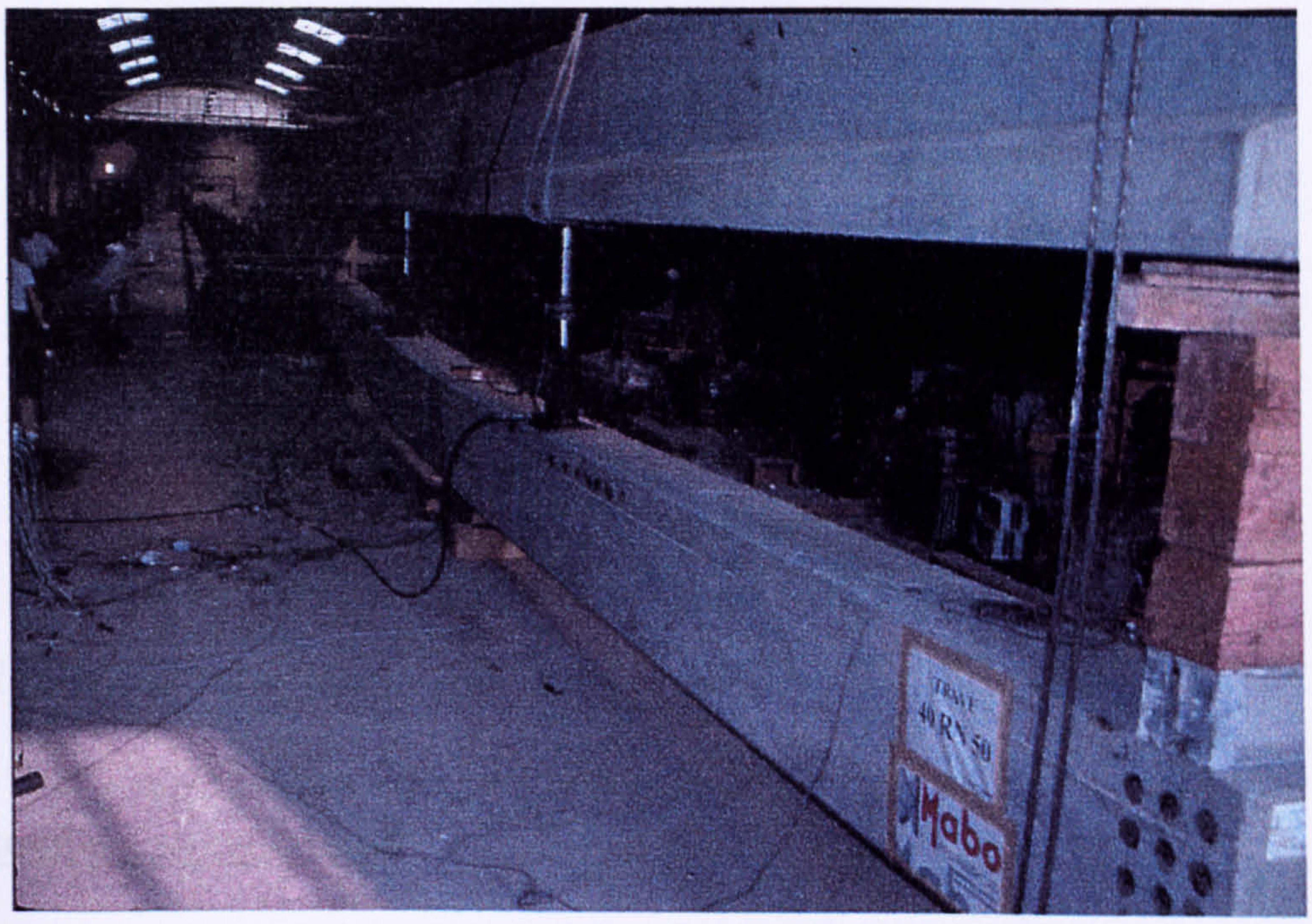


Fig. 5.8 Test equipment.

Fig. 5.9 Strain gauges inside the beam.

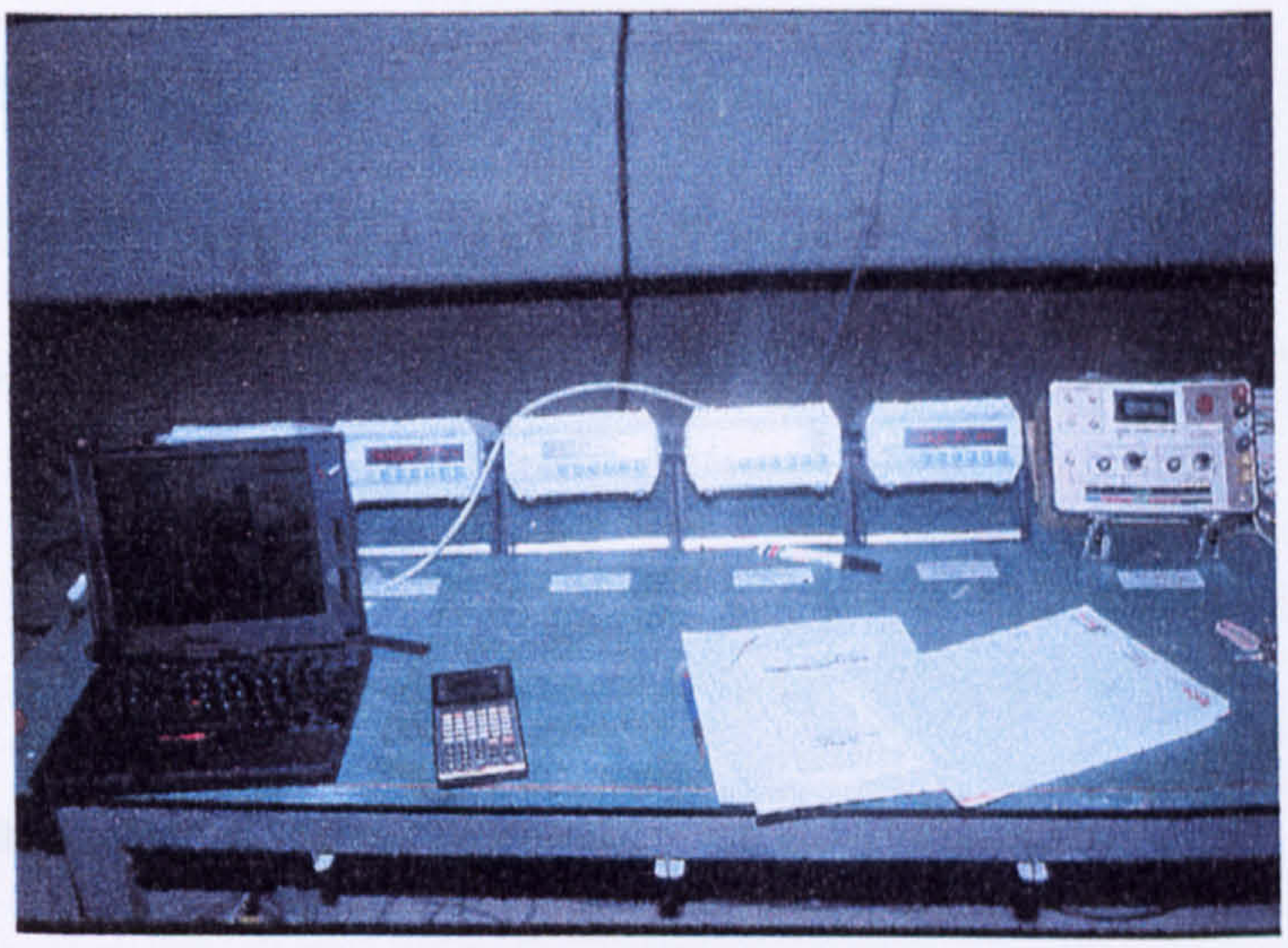


Fig. 5.10 Control data system.

5.2.3 <u>Results and comments</u>

The behaviour of the pre-stressed beams, under increasing loads is shown with the *P*, δ curves (applied loads versus mid span deflection) in the following Figure 5.11.

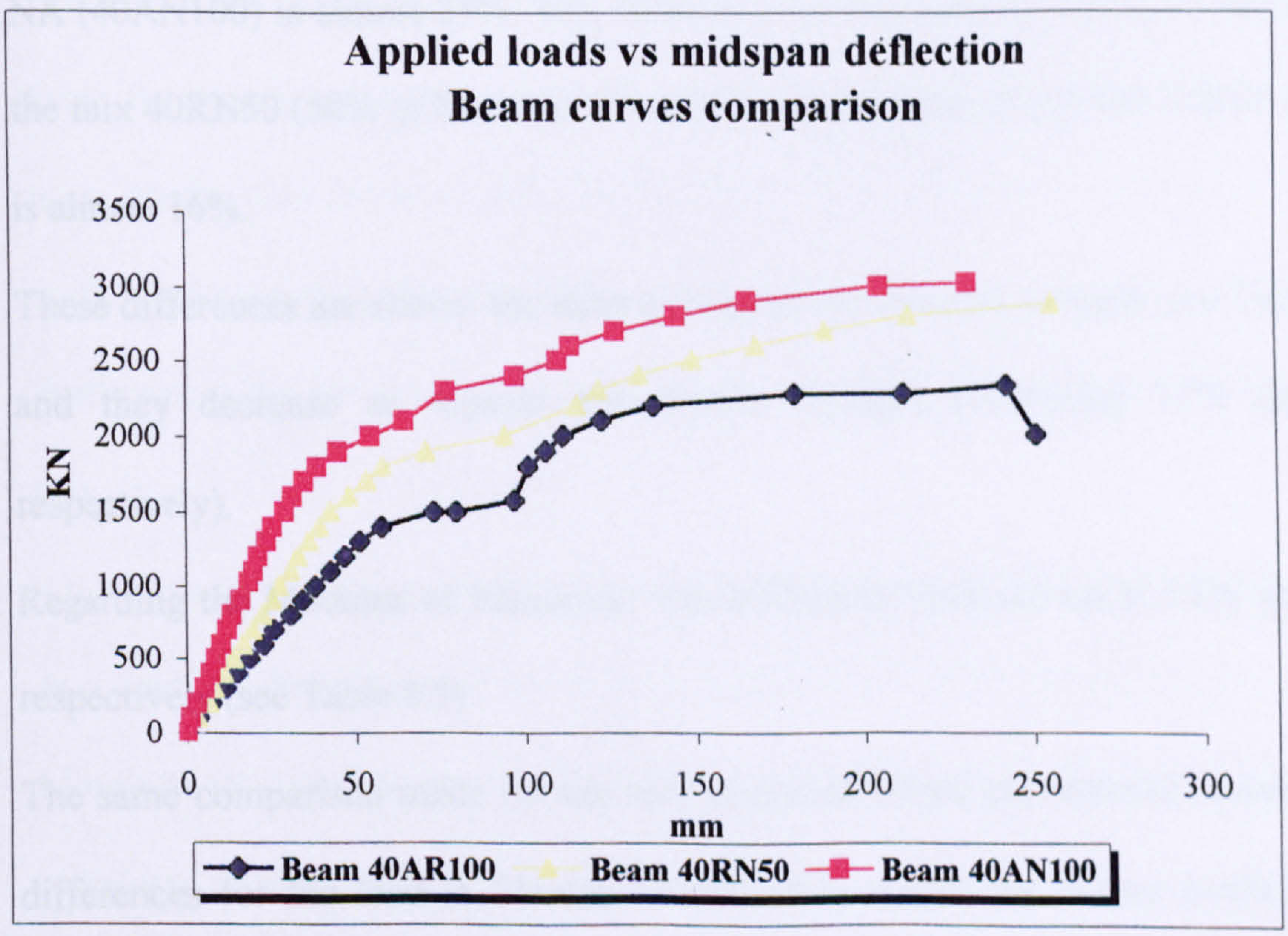


Fig. 5.11 Comparison between the P - δ (mid-span deflection) curves of the three pre-stressed beams.

It can be seen from the three curves shown in Figure 5.11 how the performance of the beam 40RN50 (cast with 50% of RA and 50% of NA) is almost in the middle of the two curves 40AR100 (with 100% of RA) and 40AN100 (100% of NA).

It seemed logical to deduce that the behaviour of beams cast with different percentages of RA and NA can be predicted by interpolation of the above curves.

Of course care should be taken in doing so.

In fact at this stage it is important to make comments on the differences between the mechanical performances of specimens and the real behaviour of structures (like pre-stressed beams).

From Table 5.3 it can be seen that the difference of the compressive strength (air curing) between the mix with 100% of RA (40AR100) and the mix with 100% of NA (40AN100) is almost 25%. The difference between the compressive strength of the mix 40RN50 (50% of RA and 50% of NA) and the mix 40AN100 (100% of NA) is almost 16%.

These differences are almost the same as regards the flexural strength (see Table 5.4) and they decrease as regards the tensile strength (becoming 11% and 6% respectively).

Regarding the Modulus of Elasticity the differences increase up to 31% and 22% respectively (see Table 5.5).

The same comparison made for the real structures (three pre-stressed beams) gives differences for the load at 35 mm of deflection where the beams could still be considered elastic of 47% comparing 40AR100 and 40AN100 and 24% comparing 40RN50 and 40AN100.

But at the failure load the differences become 23% and 5% respectively.

On the contrary, comparing the deflections of the three beams at the same value of load (1400 kN) where they are still elastic, the differences become 127% and 54%. In this case it seems obvious that no specific correlation can be made between the mechanical strength of specimens and the mechanical performances of the respective beams.

But an important aspect should be underlined.

The beam cast with 100% of Recycled Aggregate has a lower Modulus of Elasticity compared with the beam cast with 100% of Natural Aggregate, as previously commented at page. 7 and as reported in Table 5.5.

Moreover comparing the behaviour of the beams during the test, as shown in Figure 5.11, at the same value of load (e.g. 1400 kN) the beam 40AR100 has a bigger deflection (59 mm) than beam 40AN100 (29 mm).

From the above comments it can be concluded that the beam with 100% of RA is more ductile than the beam with 100% of NA and it has shown an elastic behaviour markedly longer than the beam with conventional concrete.

Furthermore the beam 40AR100 has shown a better deformation recovery after the test.

Of course this deduction could involve other application of this new type of concrete as it could be for the production of New Jersey elements for the motorway guard rail. Moreover tests for fatigue of concrete should be performed in order to investigate the behaviour of these structures under cyclical loads.

Suggestions for further areas of research are given in Chapter Seven on Conclusions.

The following diagrams (Figure 5.12 and 5.13) represent the beam behaviour at the support as a function of the applied load.

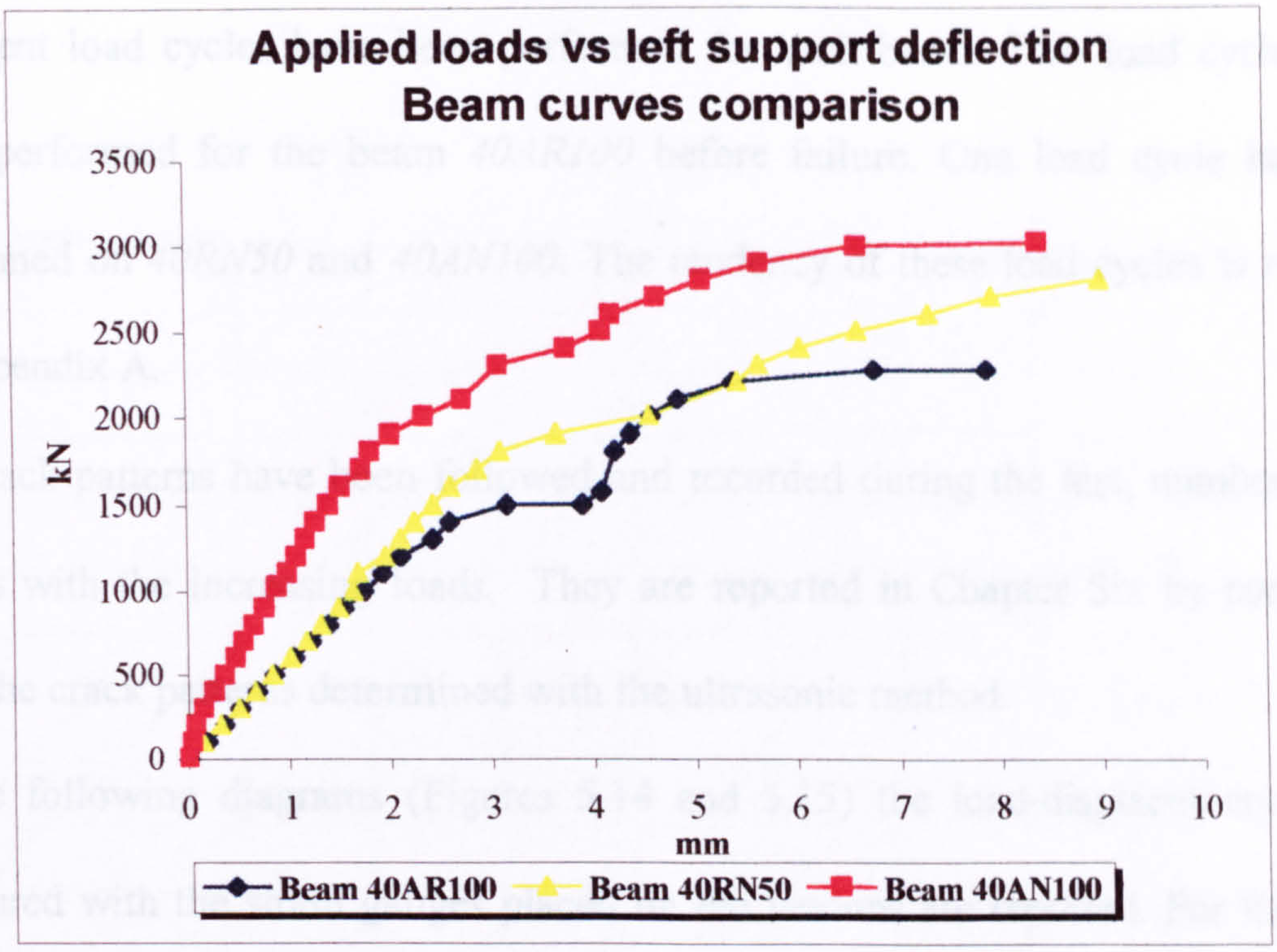


Fig. 5.12 Comparison between the P - δ (left support deflection) curves of the three pre-stressed beams.

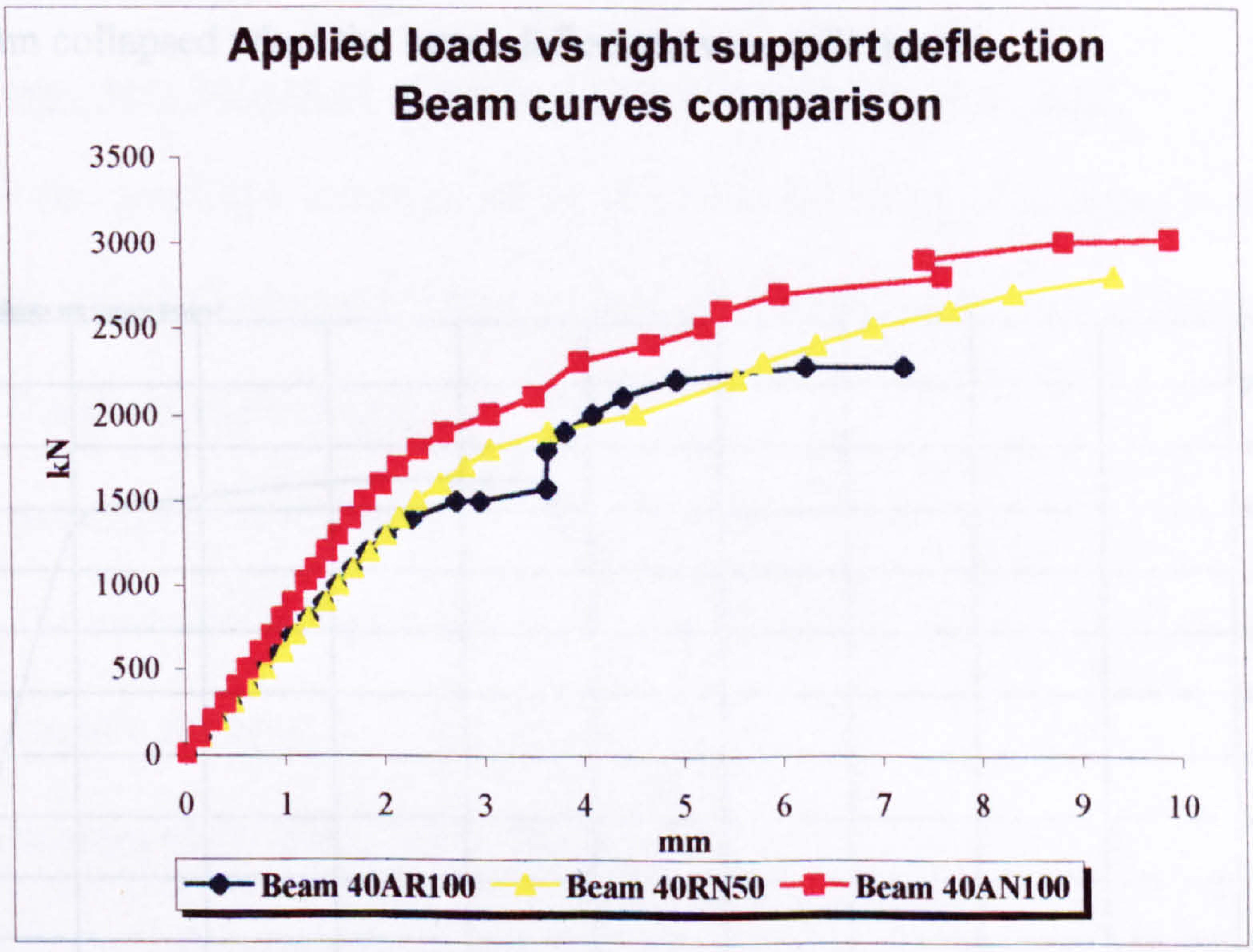


Fig. 5.13 Comparison between the P - δ (right support deflection) curves of the three pre-stressed beams.

Different load cycles have been performed for each beam. Two load cycles have been performed for the beam *40AR100* before failure. One load cycle has been performed on *40RN50* and *40AN100*. The tendency of these load cycles is reported in Appendix A.

All crack patterns have been followed and recorded during the test, numbering the cracks with the increasing loads. They are reported in Chapter Six by comparing with the crack patterns determined with the ultrasonic method.

In the following diagrams (Figures 5.14 and 5.15) the load-displacement curves measured with the strain gauges placed on the tendons are reported. For their own production range of measurement the strain gauges were not able to measure all the extension of the steel of the tendons used for the precompression. In fact it can be noticed, comparing for example Fig. 5.14 and Fig. 5.11, that the strain gauges inside the beam collapsed when the beam deflection were still linear.

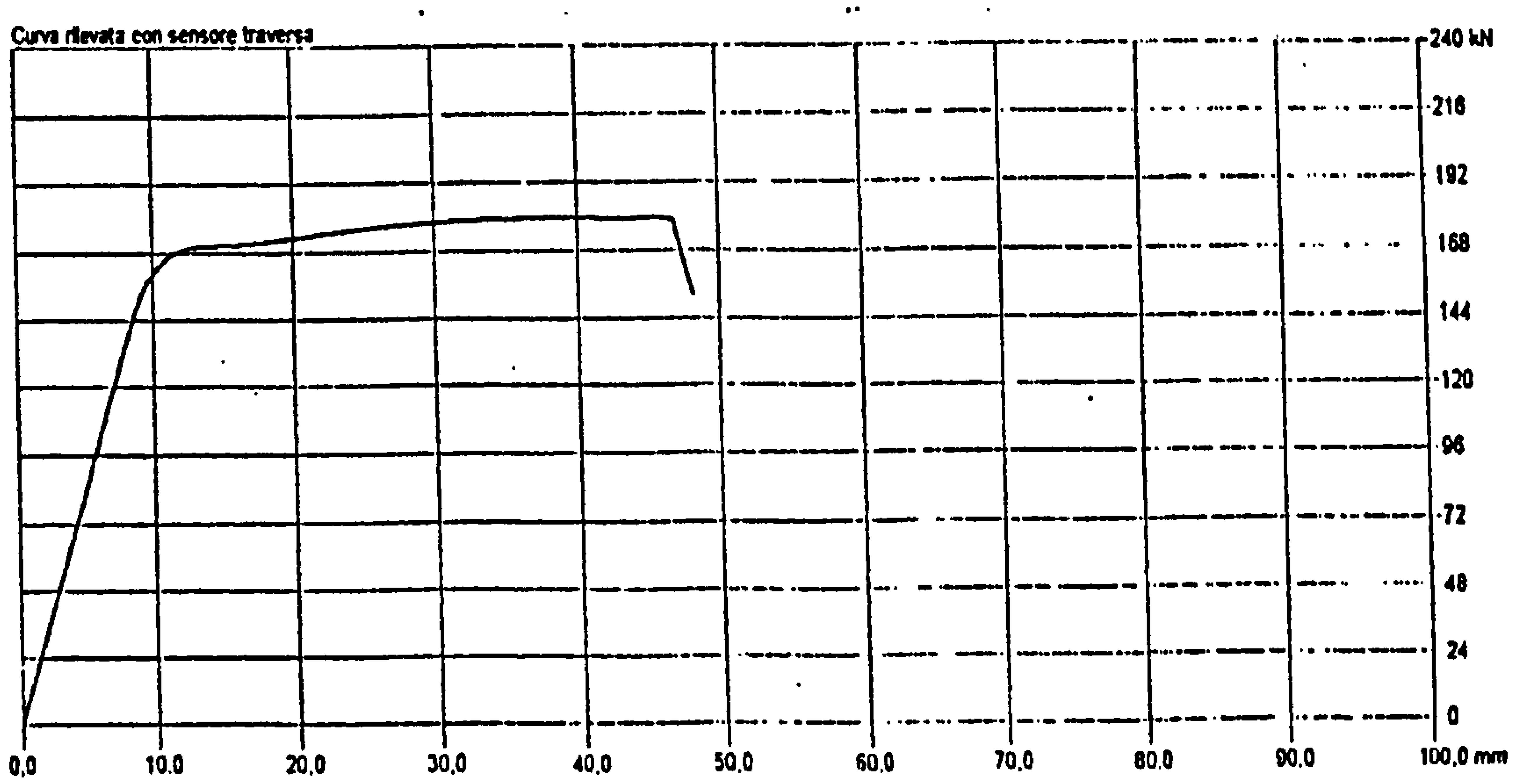


Fig. 5.14 Load displacement curve into the pre-compression tendons.

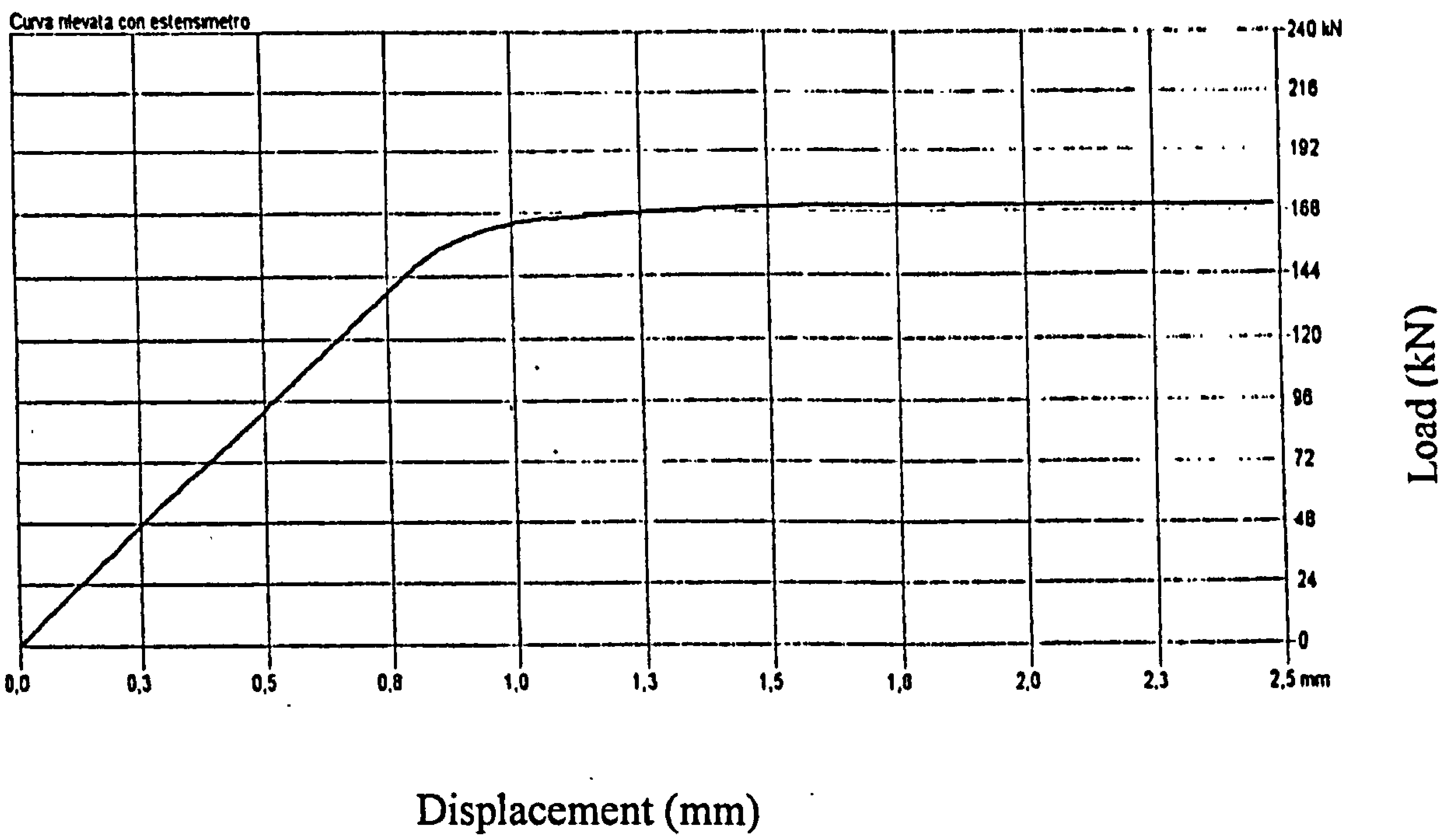


Fig. 5.15 Load displacement curve into the pre-compression tendons.

5.3 Non-linear behaviour of a pre-stressed beam made from RAC

In this paragraph the influence of the material modelling parameters is described.

This numerical simulation has been carried out leaving the following figures fixed:

- E (Modulus of Elasticity)
- ρ (concrete relative density)
- f'_c (compressive strength)
- f'_t (tensile strength).

The variation of the following parameters:

- ν Poisson's coefficient
- K Shear retention parameter
- β Shear retention factor

- α Thermal expansion factor

- α_s Softening factor

that are involved in the calculation have been investigated changing only one at time. The mesh used in this simulation was the 9 x 85 that gave satisfactory displacement values.

During this research five different types of meshes have been investigated.

In Table 5.6 the mid-span deflections for each different type of mesh are reported. They are compared with the corresponding deflections calculated with the classical formula for the load arrangement of Figure 5.7:

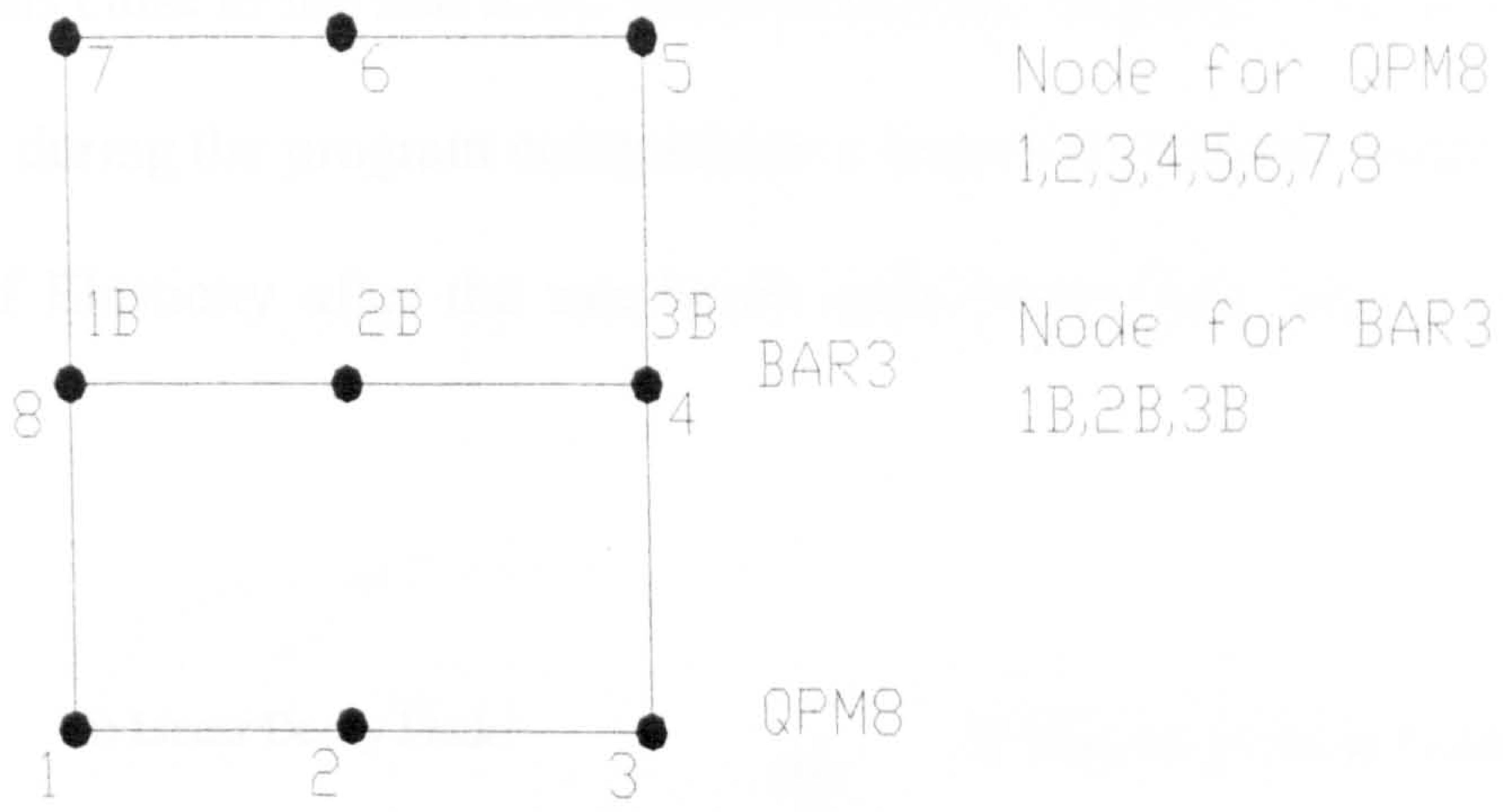
$$w_{midspan} = \frac{23}{648} \frac{Pl^3}{EI} \qquad [5.1]$$

Mesh	Midspan displacement (mm)
Numerical solution by [5.1]	134.8
7 x 150	266.7
7 x 75	193.1
8 x 75	136.9
9 x 75	136.7
9 x 85	136.6

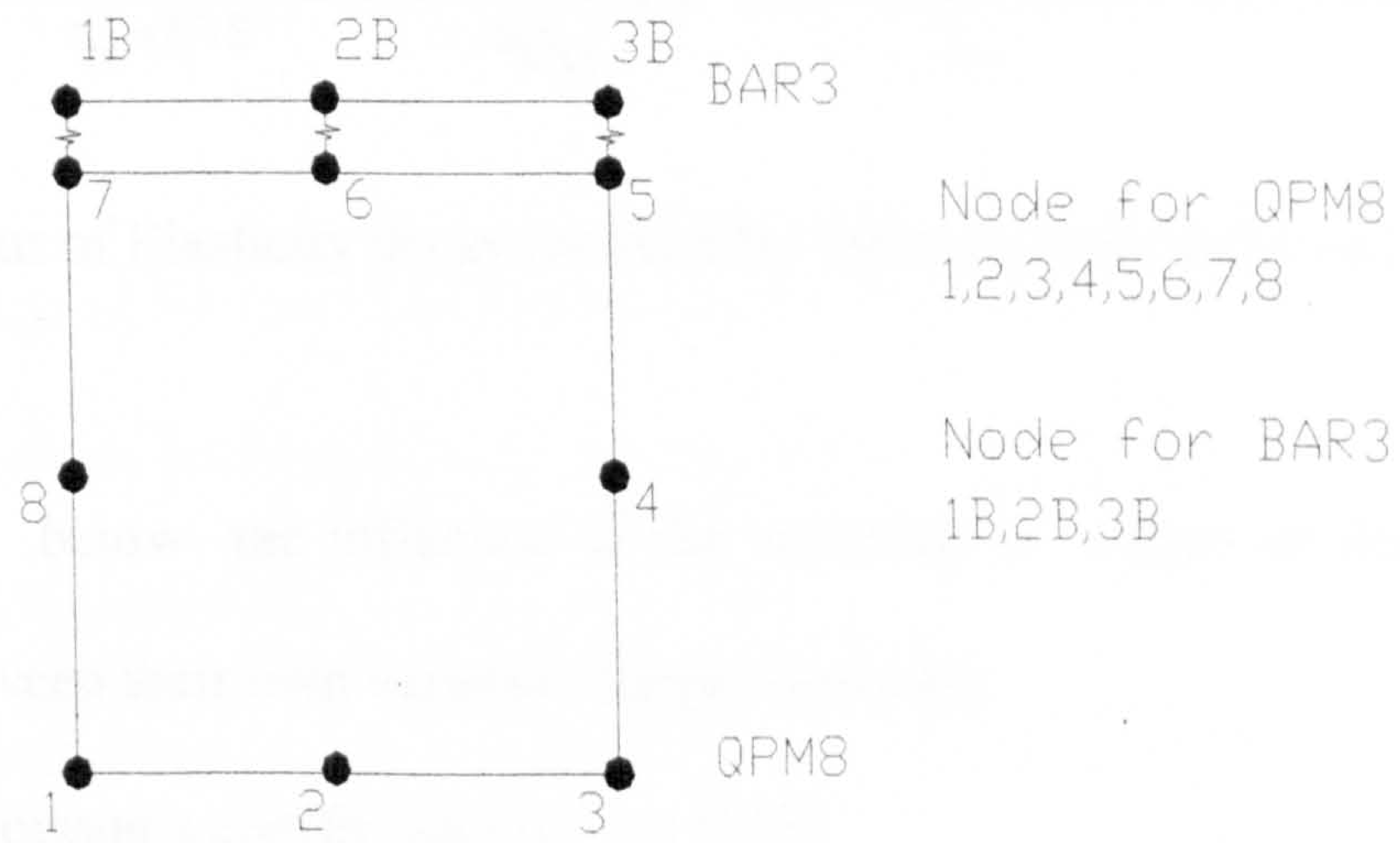
Tab. 5.6 Mesh investigation.

The excessive displacement of the meshes 7 x 150 and 7 x 75 is due to the way of bonding the joint of the elements BAR3 and QPM8.

In fact if there is not an biunivocal correspondence between the joints of the two elements, the displacement given by the program is that one of the free joints and not the one of the whole structure (see Figure 5.16).



1. wrong disposition



2. correct disposition

Fig. 5.16 Joint connection between element QPM8 and BAR3.

If a type 1 disposition of Figure 5.16 is adopted the displacement calculated by the program could be up to 50% bigger than the deflection calculated with the type 2 disposition.

The first parameter that has been necessary to decide on to have a computer structural analysis close to the real RAC beam behaviour, was the LUSAS *option 22.* This introduces during the program computation a exponential decay model curve of the Modulus of Elasticity after the maximum deformation has been reached (see Figure 5.17).

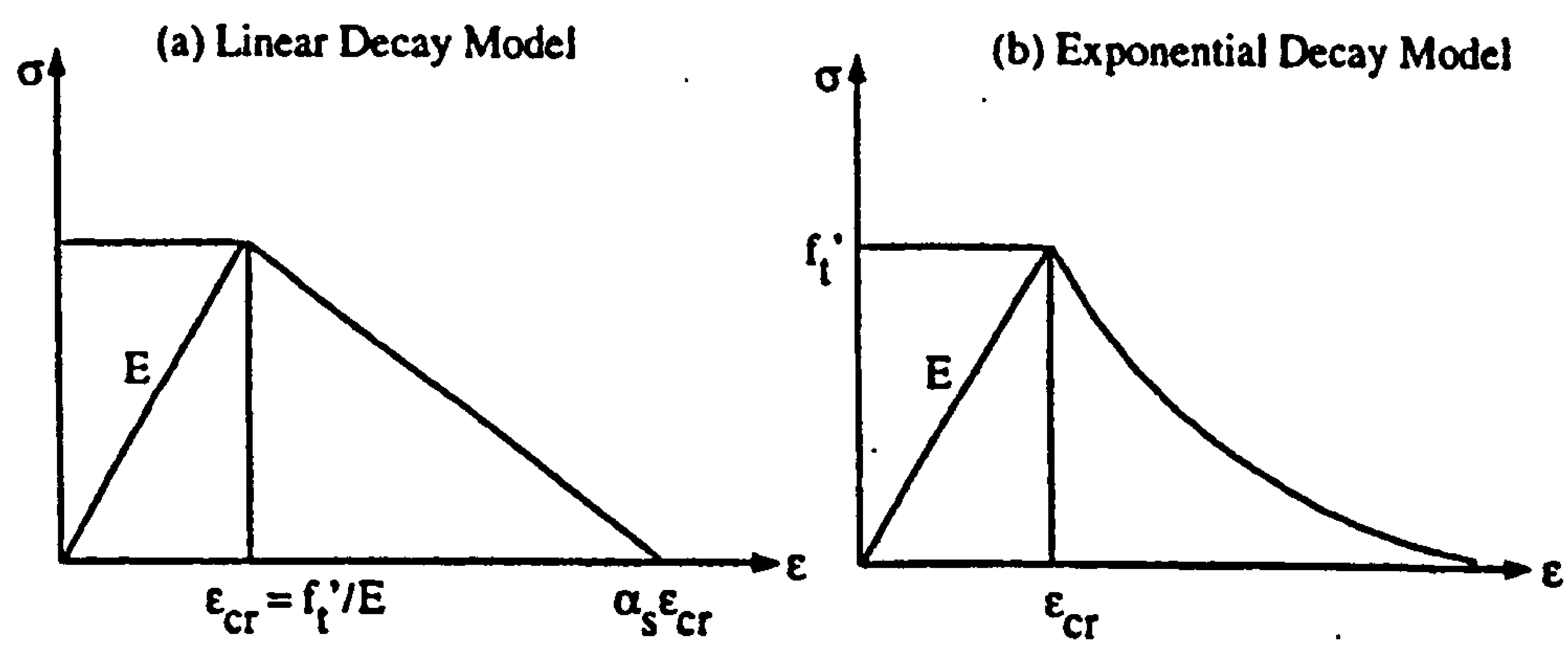


Fig. 5.17 Modulus of Elasticity decay model after the maximum deformation.

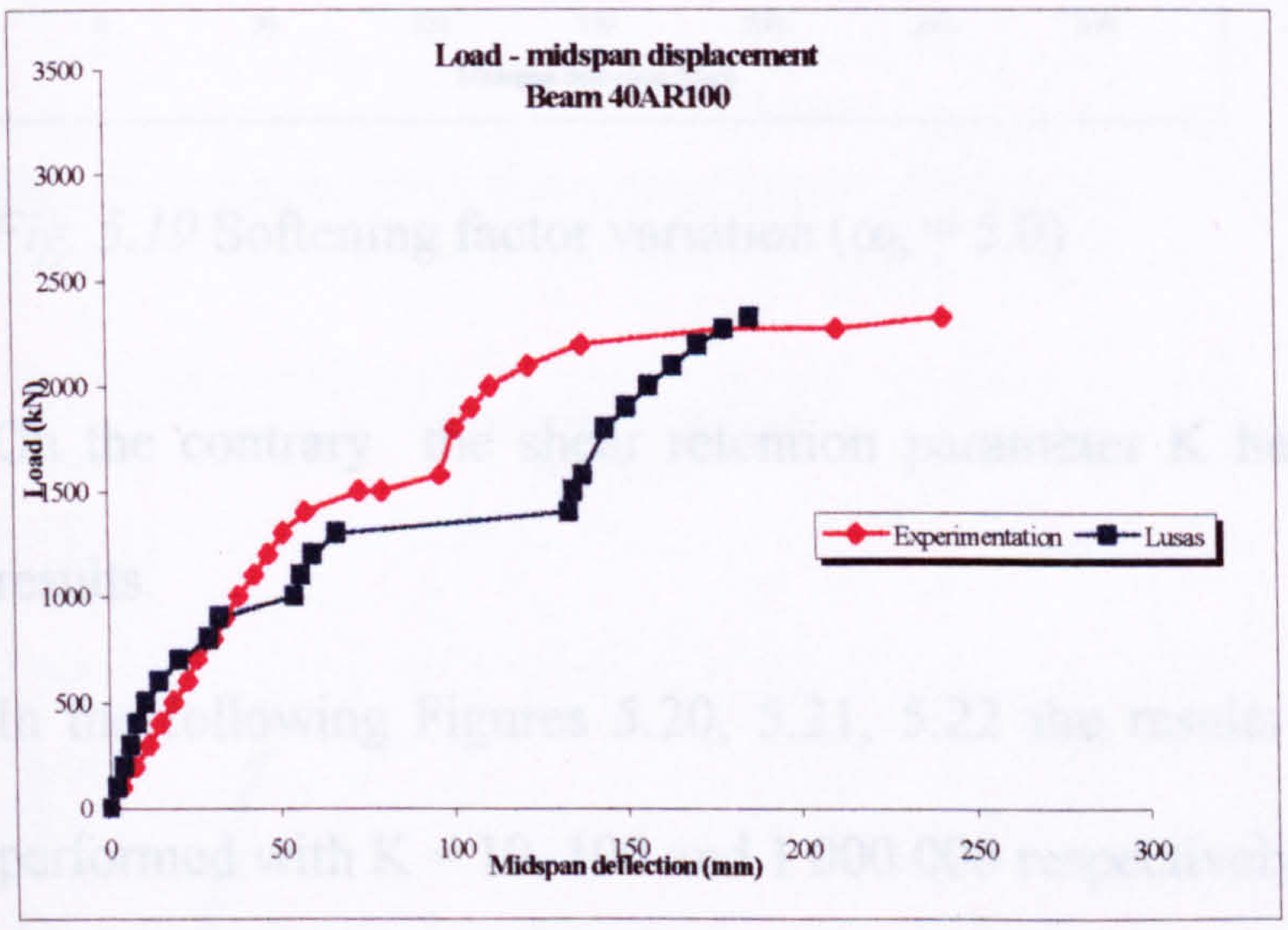
In the diagrams below the influence of the variation of values of the following parameters (between their own variation range) is shown:

- ν Poisson's coefficient ($0 < \nu < 0.25$)
- *K* Shear retention parameter ($0.0 < k < \infty$)
- β Shear retention factor ($0.0 < \beta < 1.0$)
- α Thermal expansion factor ($0 < \alpha < 0.0000012$)
- α_s Softening factor (tipic 5-50)

by comparing the computer structural non-linear analysis (blue curve) with the real beam test behaviour (experimental: red curve).

The curve of the computer analysis is almost the same for the three values of the Poisson's coefficient adopted ($\nu = 0, 0.15, 0.25$) during the calculus.

In Figure 5.18 the curve of the non-linear analysis performed with $\nu = 0.25$ is reported.



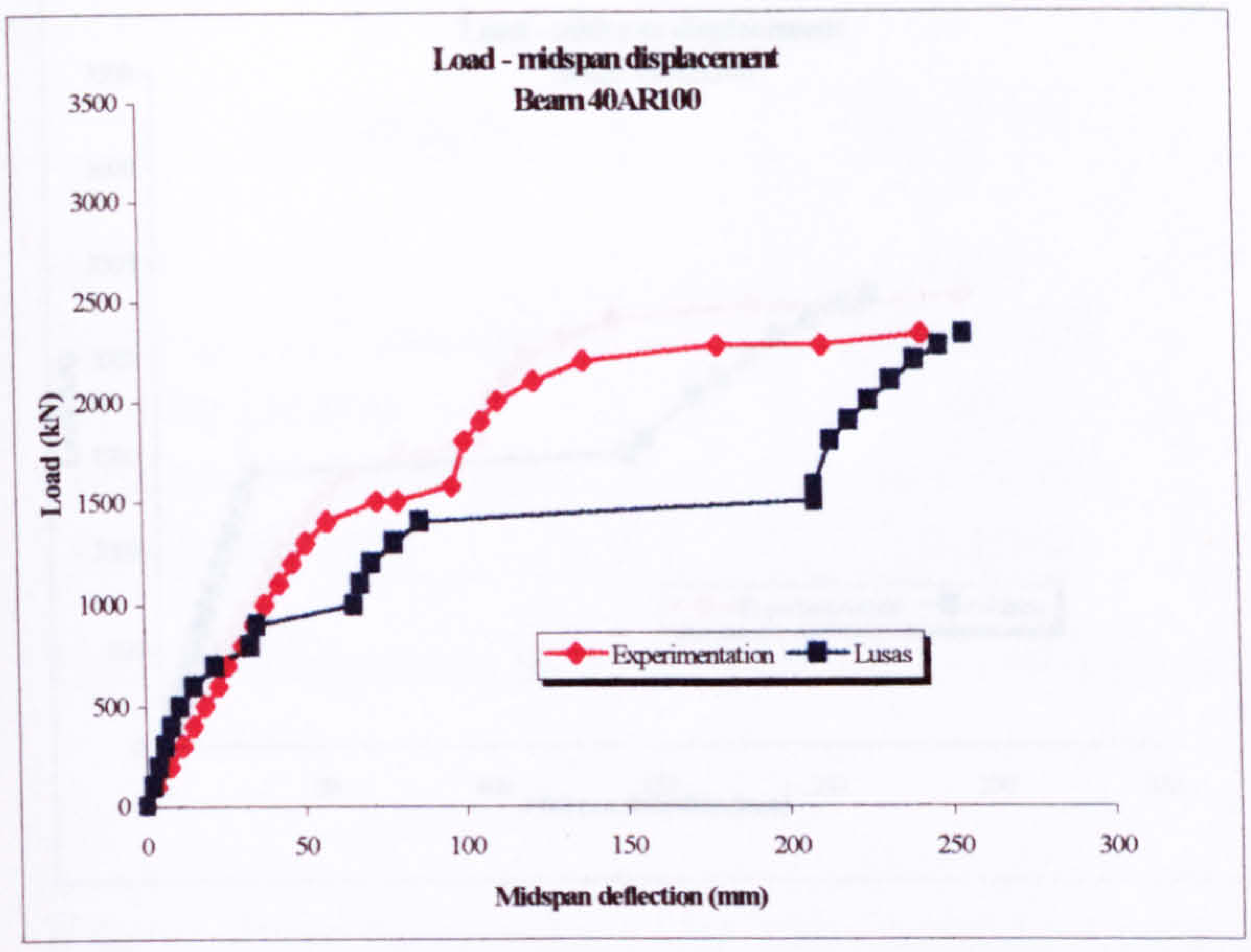
Concrete material characteristics

E	23000	Mpa
ν	0,25	
ρ	0,0023	
α	0	
a_r	0	
t_r	0	
T	0	
Rck	39	Mpa
β	0	
K	------	
f_t	2,5	MPa
εp	0	
αs	35	
Gf	option 22	

Fig. 5.18 Poisson's coefficient variation ($\nu = 0.25$).

The same tendency of the computer analysis curve reported in Figure 5.18 has been found for the analysis performed with the variation of the shear retention factor β and the thermal expansion factor α (see Appendix A).

The non-linear analysis performed with the softening factor $\alpha_s = 5$ has given a curve tendency (see Figure 5.19) with a part more linear than that one found with $\alpha_s = 50$ that is similar to the tendency of Figure 5.18. It has been also possible to evaluate the last step of the failure load.



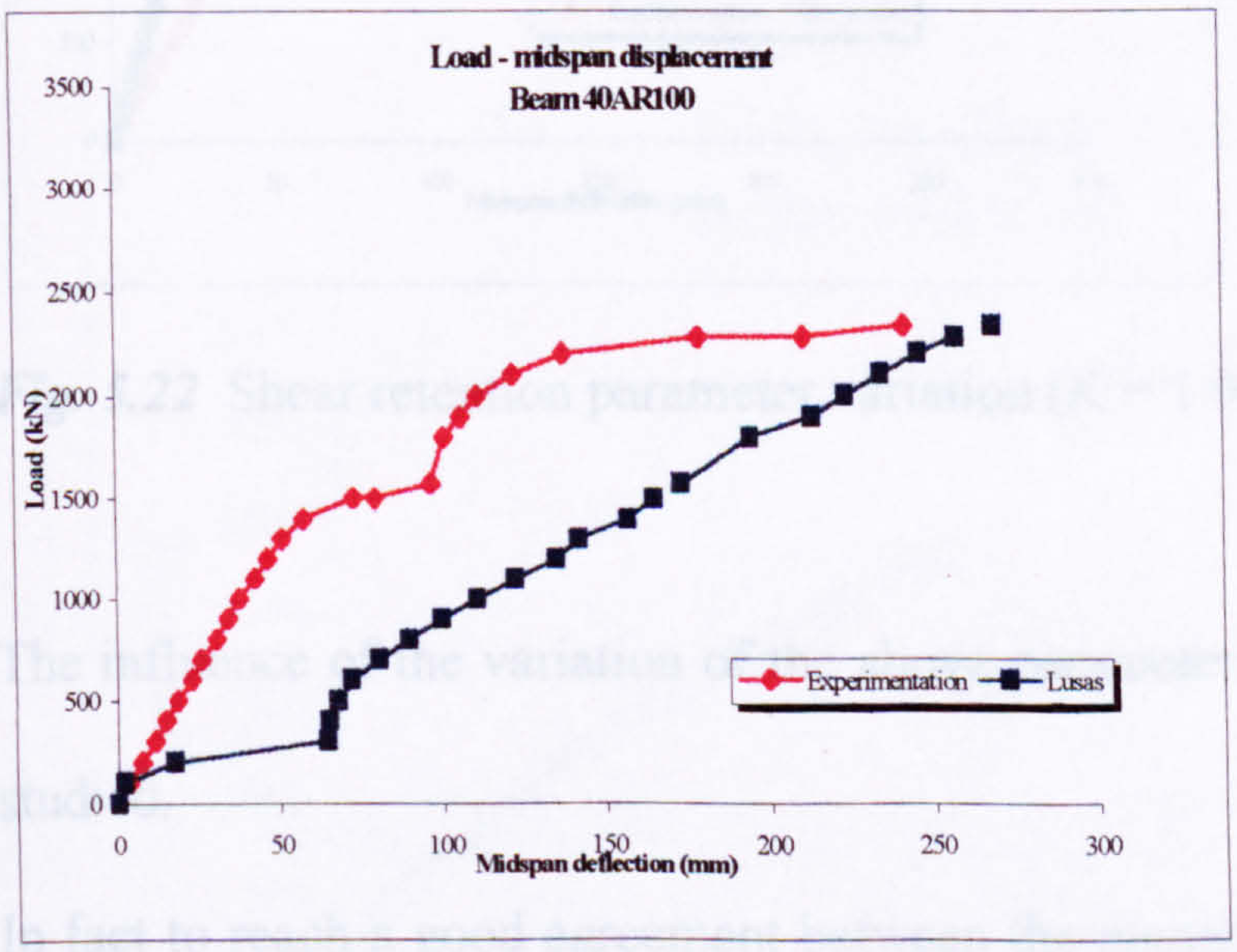
Concrete material characteristics

E	23000	MPa
ν	0	
ρ	0,0023	
α	0	
a_r	0	
br	0	
T	0	
Rck	39	MPa
β	0	
K	-------	
f_t	2,5	MPa
εp	0	
αs	5	
Gf	option 22	

Fig. 5.19 Softening factor variation ($\alpha_S = 5.0$)

On the contrary the shear retention parameter K has a definite influence on the results.

In the following Figures 5.20, 5.21, 5.22 the results from the non-linear analysis performed with K = 10, 100 and 1 000 000 respectively are reported.



Concrete material characteristics

E	23000	Mpa
ν	0	
ρ	0,0023	
α	0	
a_r	0	
br	0	
T	0	
Rck	39	Mpa
β	0	
K	10	option 23
f_t	2.05	Mpa
εp	0	
αs	35	
Gf	option 22	

Fig. 5.20 Shear retention parameter variation (K = 10)

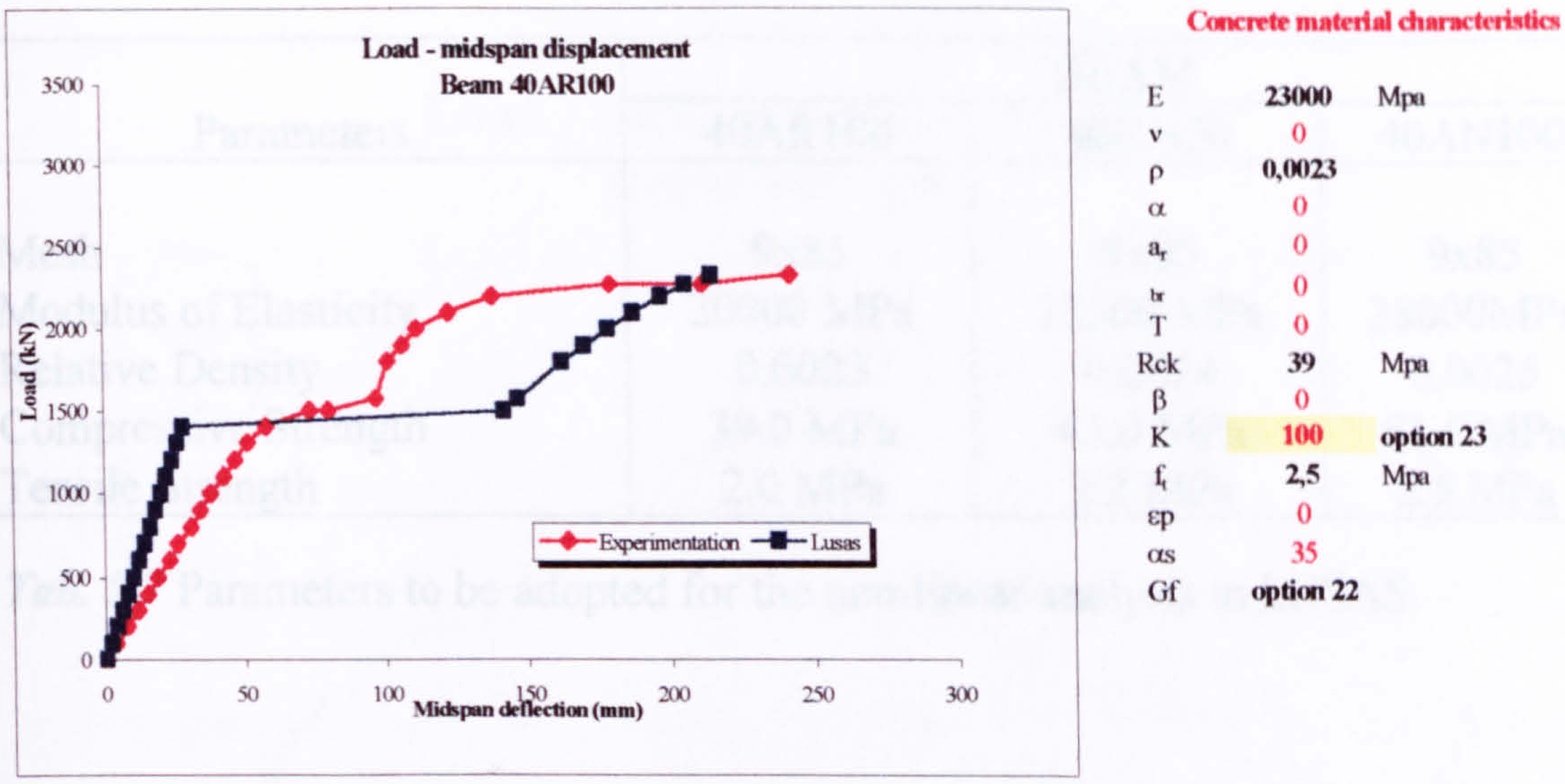


Fig. 5.21 Shear retention parameter variation (K = 100)

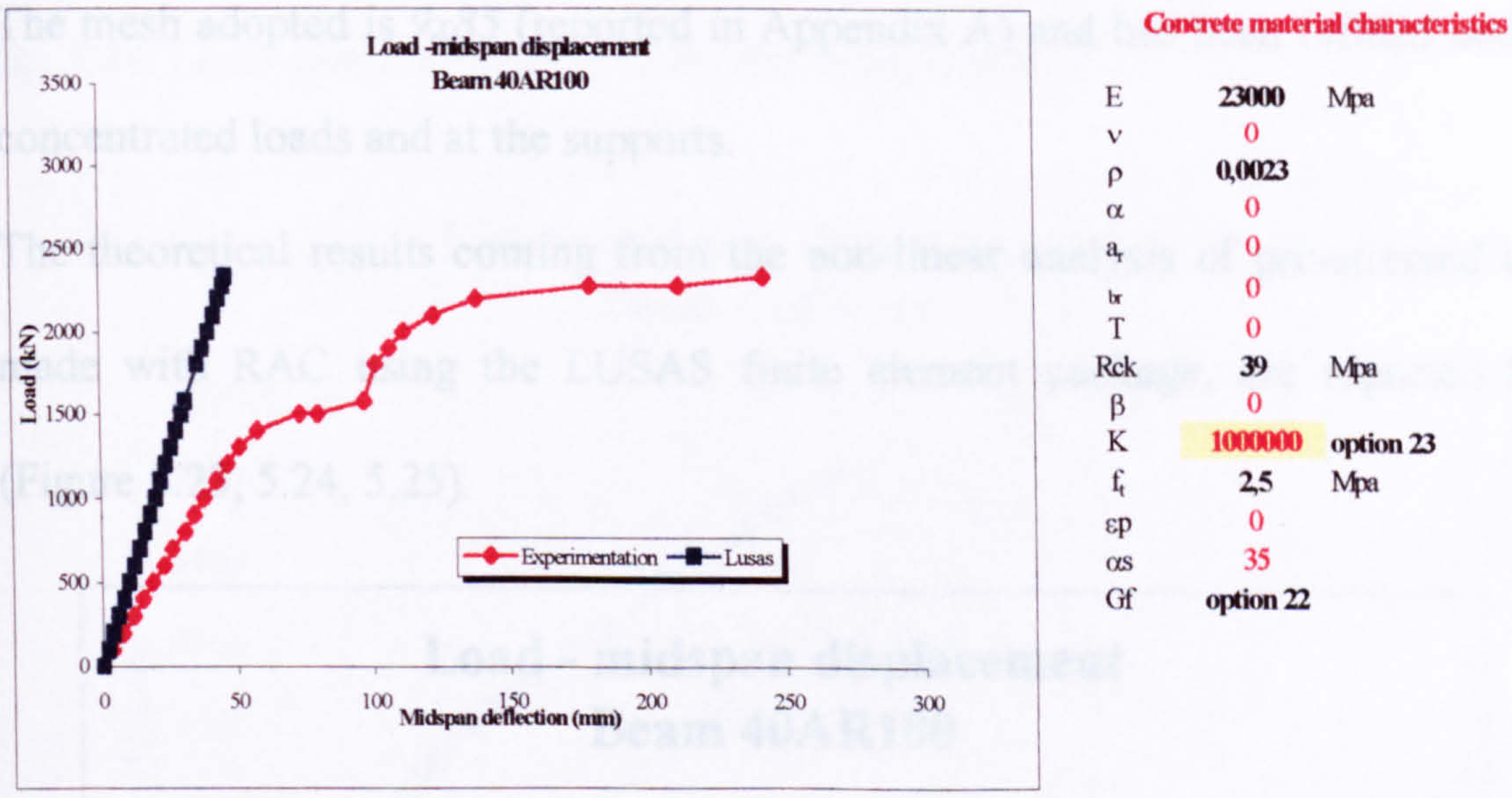


Fig. 5.22 Shear retention parameter variation (K = 1 000 000)

The influence of the variation of the above parameters has not been the only aspect studied.

In fact to reach a good agreement between the numerical simulation curves and the experimental ones, it is necessary that all beams (produced with RA and NA) should be modelled in LUSAS with the following parameters:

Parameters	BEAM		
	40AR100	40RN50	40AN100
Mesh	9x85	9x85	9x85
Modulus of Elasticity	20000 MPa	22500 MPa	28000MPa
Relative Density	0.0023	0.0024	0.0025
Compressive Strength	39.0 MPa	43.0 MPa	51.5 MPa
Tensile Strength	2.0 MPa	2.2 MPa	2.5 MPa

Tab. 5.7 Parameters to be adopted for the non-linear analysis in LUSAS.

E and f'_t have been reduced by 15% compared with the experimental results obtained from the specimens of the corresponding non reinforced concrete.

The mesh adopted is *9x85* (reported in Appendix A) and has been refined under the concentrated loads and at the supports.

The theoretical results coming from the non-linear analysis of pre-stressed beams made with RAC using the LUSAS finite element package, are reported below (Figure 5.23, 5.24, 5.25).

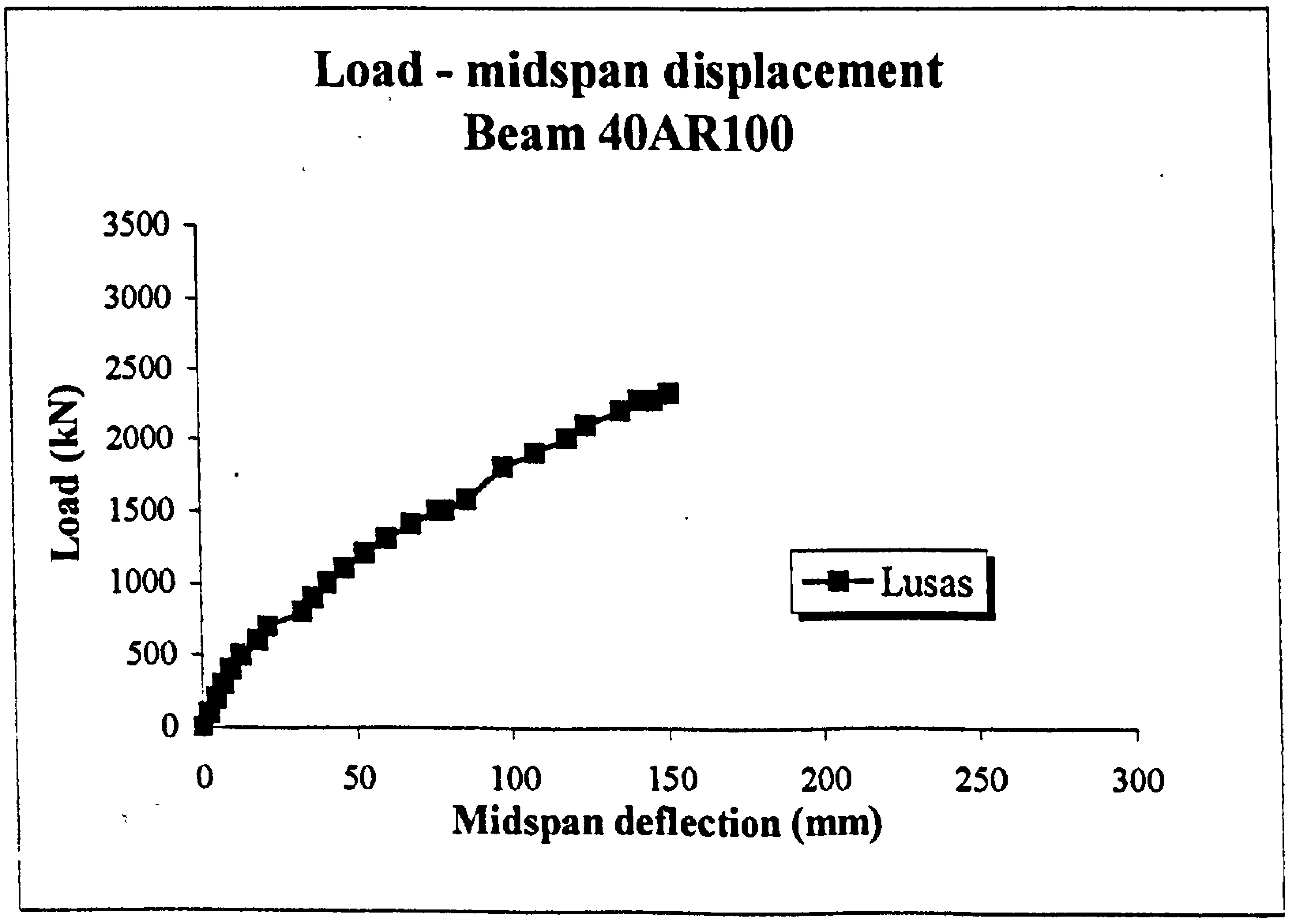


Fig. 5.23 Load-displacement curve for the beam 40AR100.

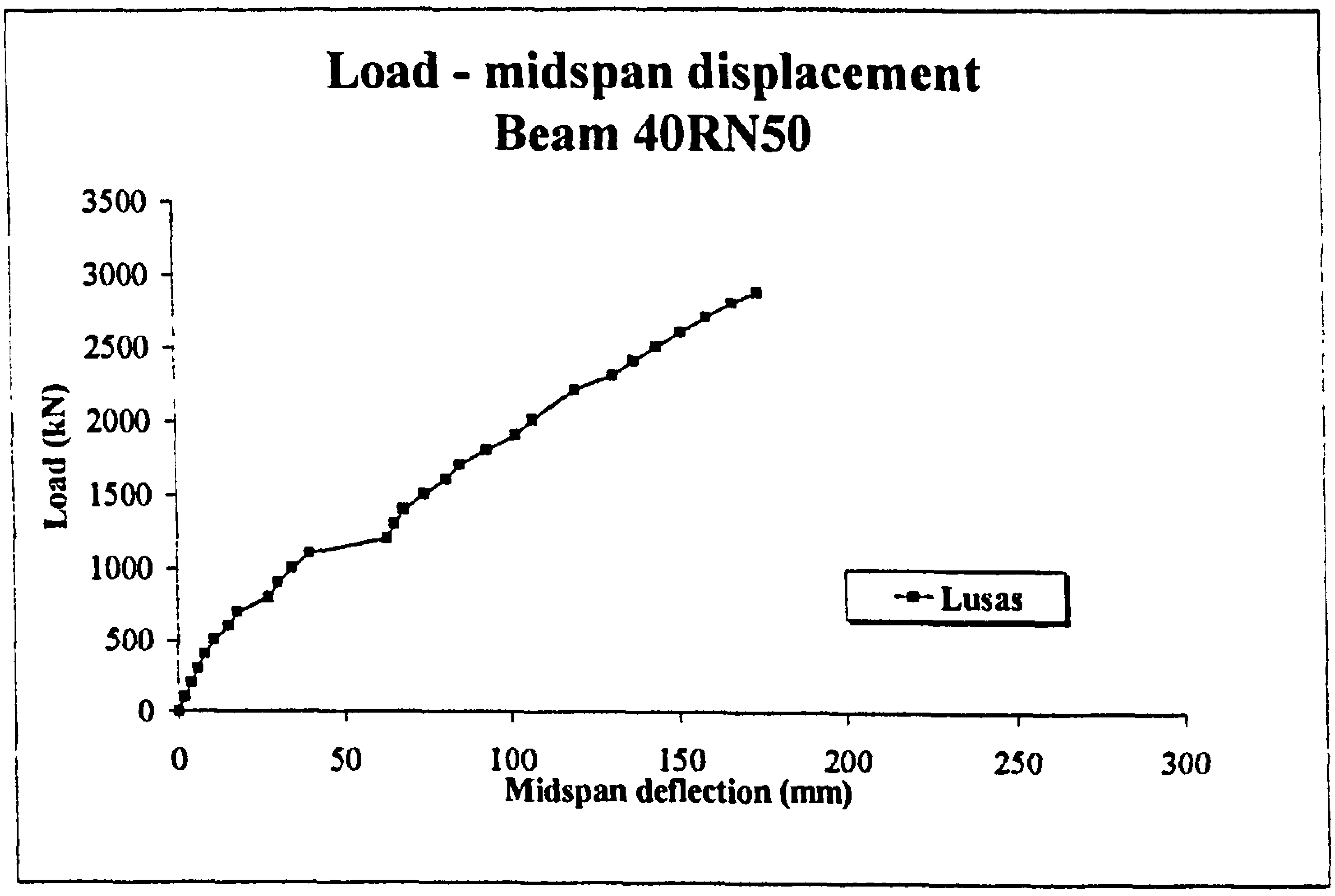


Fig. 5.24 Load-displacement curve for the beam 40RN50.

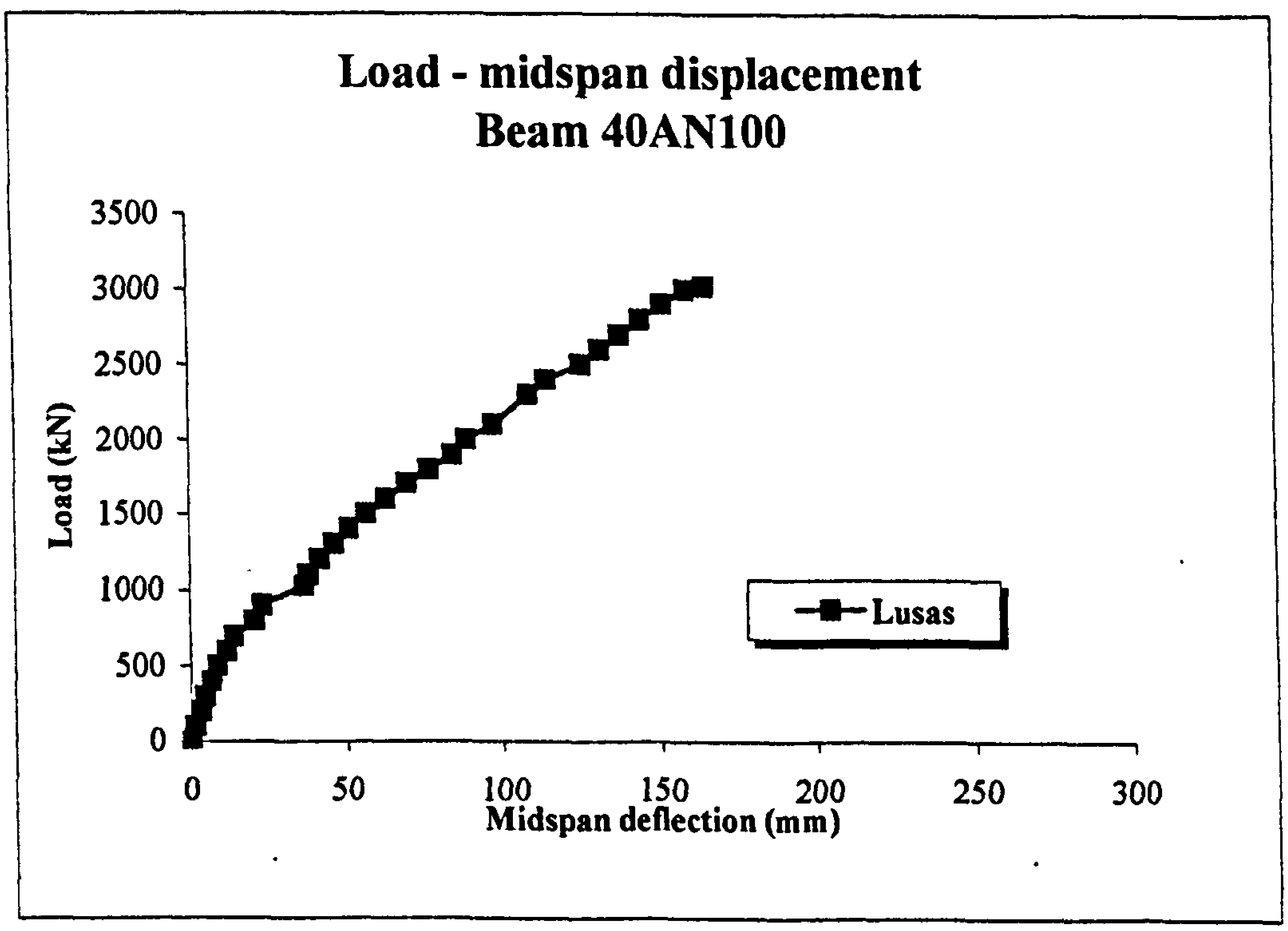


Fig. 5.25 Load-displacement curve for the beam 40AN100.

5.4 Comparison of results

This experimentation has demonstrated that is possible to produce full scale pre-stressed structural element made with Recycled Aggregate Concrete.

5.4.1 **Comparison between experimental and theoretical results**

In this section a comparison between the experimental data recorded during the beam test and the theoretical results obtained from a non-linear analysis using LUSAS finite element program is reported. This comparison has been made in terms of load – deflection curves.

From the following diagrams (Figure 5.26, 5.27, 5.28) it can be concluded that the theoretical simulation follows the real experimental behaviour but without reaching the eventual failure.

This should be ascribed to the ε_p parameter that represents the failure limit in the non-linear material modelling called *concrete* in LUSAS.

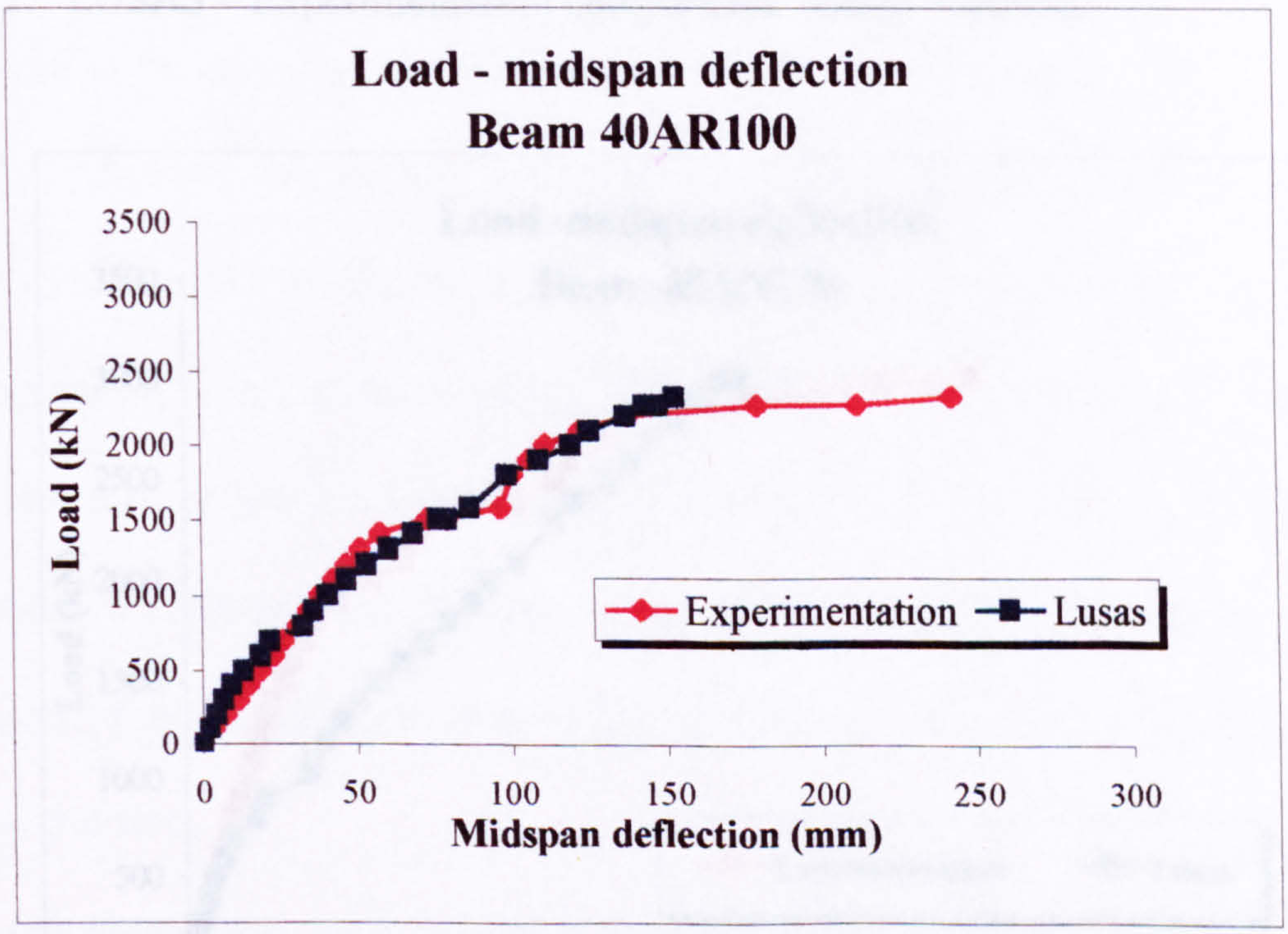


Fig. 5.26 LUSAS – Experimentation comparison. Beam 40AR100.

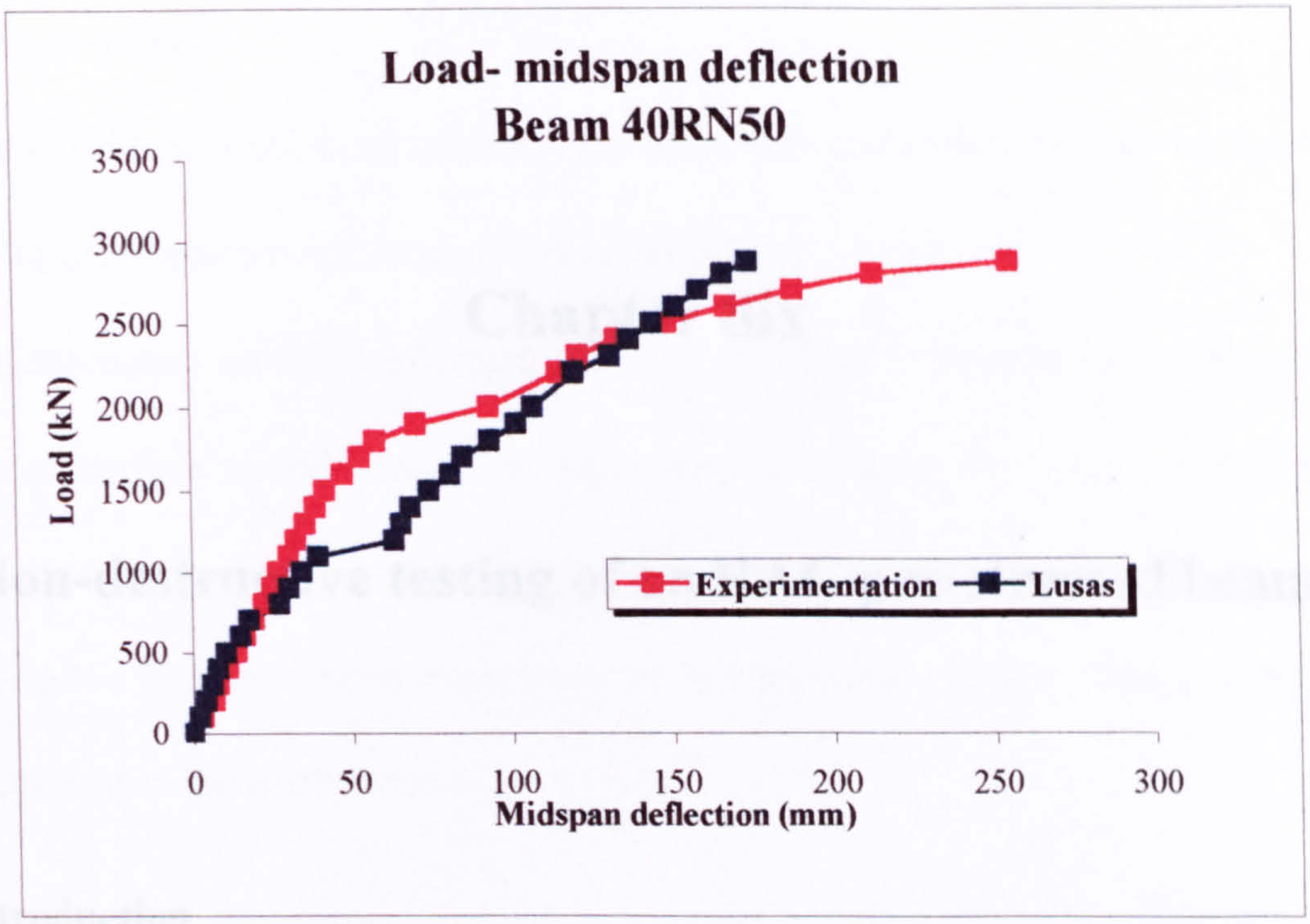


Fig. 5.27 LUSAS – Experimentation comparison. Beam 40RN50.

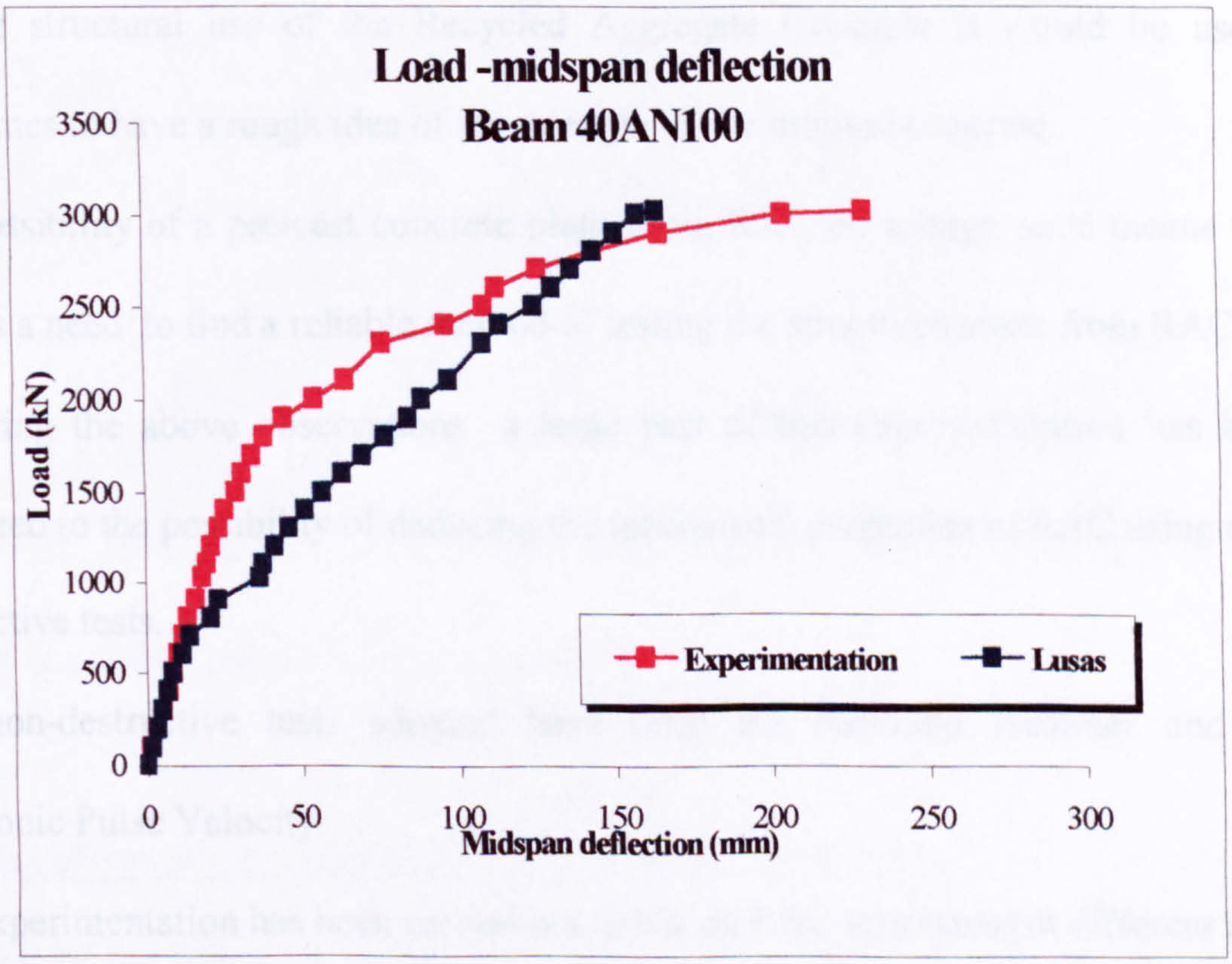


Fig. 5.28 LUSAS – Experimentation comparison. Beam 40AN100.

Chapter Six

Non-destructive testing of an RAC pre-stressed beam

6.1 Introduction

The compressive strength of the original concrete in the demolished structure processed in the recycling site plant is almost in all cases unknown.

For the structural use of the Recycled Aggregate Concrete it would be useful sometimes to have a rough idea of the strength of the original concrete.

The possibility of a pre-cast concrete plant using RAC on a large scale means that there is a need to find a reliable method of testing the structures made from RAC.

Following the above observations a large part of this experimentation has been dedicated to the possibility of deducing the mechanical properties of RAC using non-destructive tests.

The non-destructive tests adopted have been the Rebound Hammer and the Ultrasonic Pulse Velocity.

The experimentation has been carried out firstly on RAC specimens at different ages. Ultrasonic pulse velocity has been detected by placing the two transducers on opposite faces of cubes (direct transmission) and by placing glycerine paste between the transducers and the specimen's faces to attain a better transmission of the ultrasonic impulse.

The transit time measurement recorded (in µsec) has been taken as the average value of a series of measurements carried out on each cube.

The measurements have been carried out using the direct transmission technique and indirect or surface transmission has been used as comparison. The test equipment performance has been checked before each test on an aluminium reference bar.

More detailed tests have been carried out on full scale pre-stressed beams made from RAC reported in the next section.

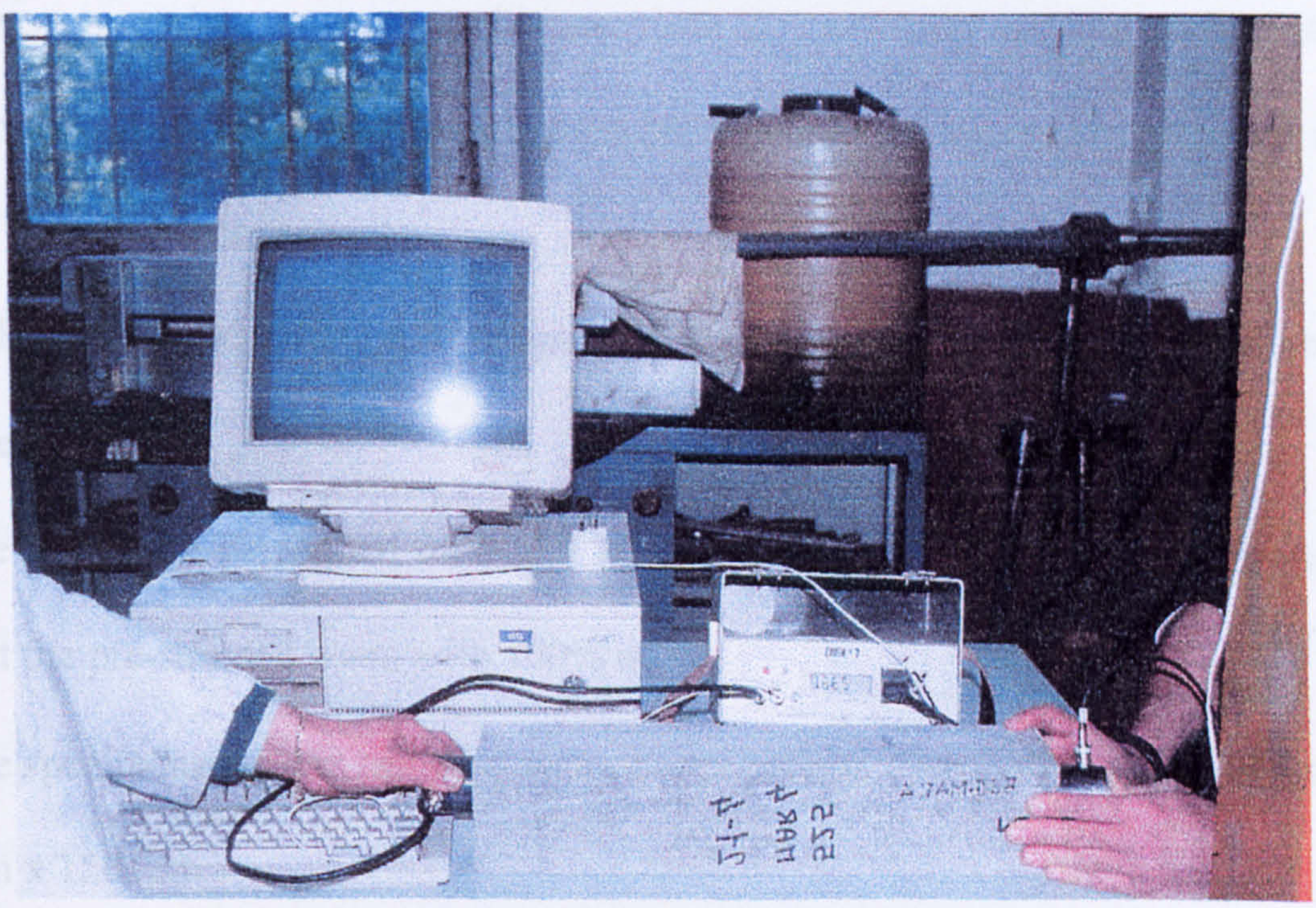


Fig. 6.1 Ultrasonic pulse velocity test equipment.

6.2 **Pulse ultrasonic velocity as function of the imposed stresses**

This part of the investigation has been carried out to check the effect of imposed axial stress on cubes on the ultrasonic pulse velocity.

In fact the experimentation normally carried out in laboratories on unloaded cubes does not reproduce the conditions of concrete in a real structure which will be under stress.

It seems therefore important to check if this stress could influence the ultrasonic pulse velocity in order to be able to interpret results from in-situ tests.

The induced stress has been applied with a uni-axial compressive test machine.

The measurement has been taken using Pundit equipment with the transducers placed orthogonally to the applied load direction.

It was appropriate to carry out this test since the next stage was the evaluation of the compressive strength on a pre-stressed beam where, of course, the imposed stresses is a basic condition.

In Figure 6.2 the variation of the pulse ultrasonic velocity versus the stress condition is reported.

The tests were performed on a 525 MAR 2 specimen that was the batch chosen to cast the pre-stressed beam with 100% of RA.

The specimens were in this case water cured and the dimensions were 150 mm x 150 mm x 150 mm.

Tests carried out on specimens from different mix-designs, different cube dimensions and at different ages have given almost the same curve as shown in Figure 6.2.

Those diagrams are reported in Appendix A.

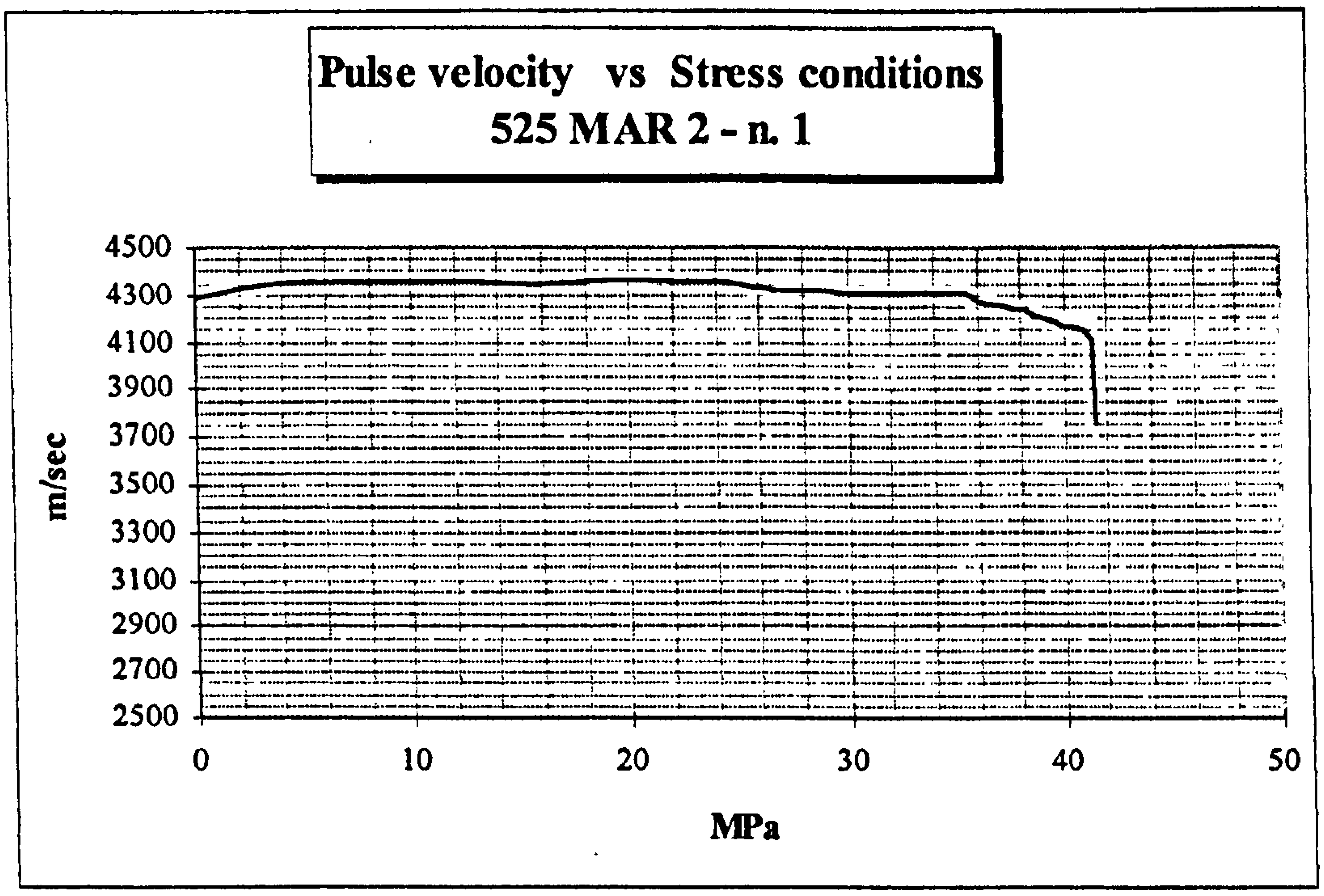


Fig. 6.2 Tendency of the pulse ultrasonic velocity under imposed stress conditions.

From the above diagram it can be noticed that for the first part (almost 10% of the failure load) there is a little increase of the ultrasonic transit velocity into cubes.

This is probably due to the fact that in the unloaded cube interstices arising from poor vibration could be present or it could due to the micro cracks coming from the shrinkage during the curing period. These micro cracks are closed during the initial phase of the load application and this phenomenon helps the passage of the ultrasonic wave.

For the next 75% of the failure load, the pulse velocity remains almost constant. After this value the micro cracks, formed during the initial load application become macro-cracks.

This means that there is more air between the cracks and since air acoustic impedence is lower than concrete the pulse velocity decrease.

6.3 Effect of reinforcing bars on ultrasonic velocity in RC elements

During the experimentation, particular attention has been paid to the ultrasonic pulse transmission in reinforced concrete.

The effect of the presence of reinforcement in the concrete element could influence markedly the evaluation of the compressive strength of concrete with the indirect method and this has been checked.

In fact if the ultrasonic pulse is close to a reinforcement bar it will be detected faster by the receiving transducer lowering the transit time and giving an evaluated compressive strength higher than the real situation.

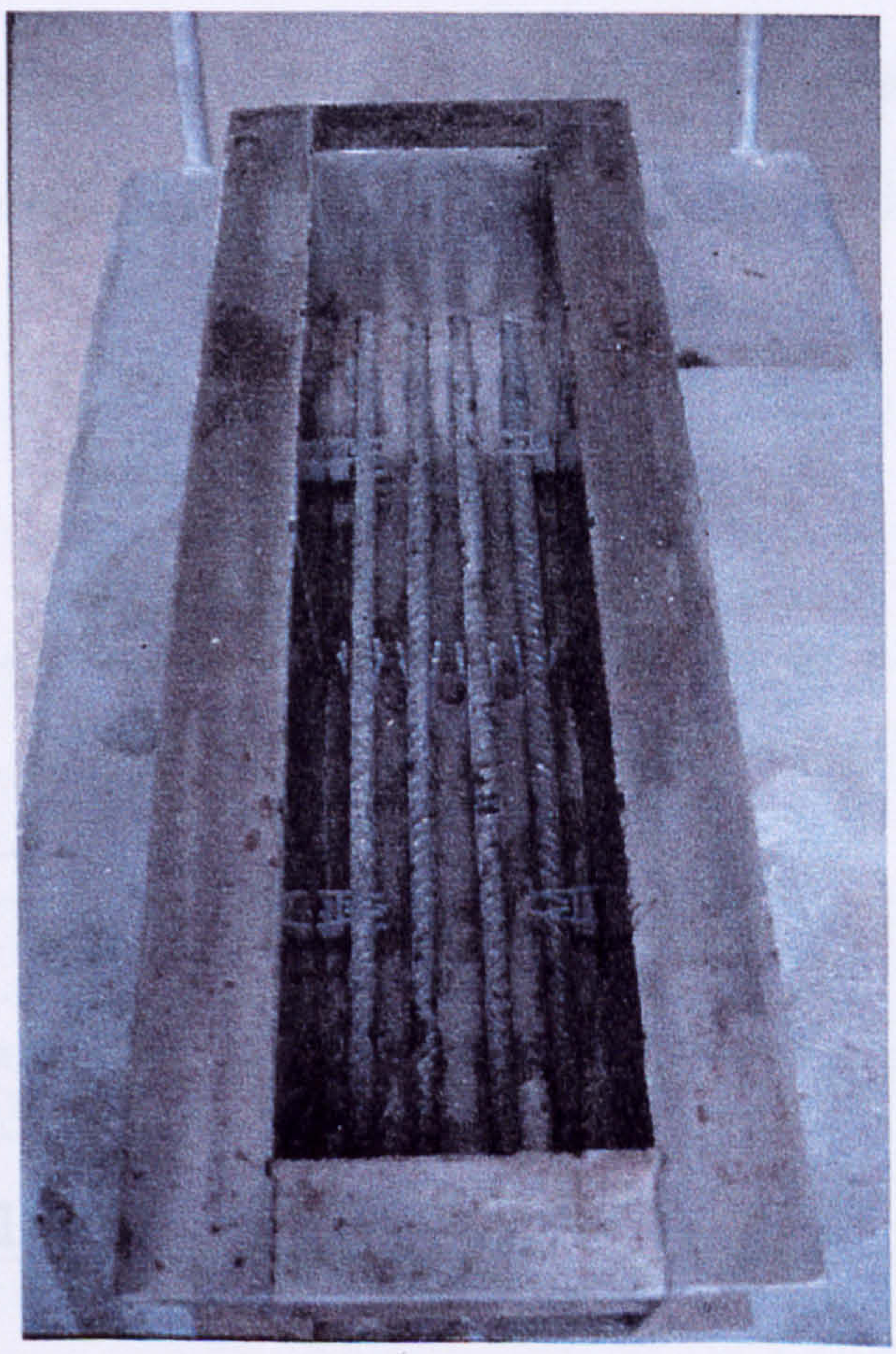


Fig. 6.3 Disposition of the reinforcement bars in the specimens.

The aim of this part of the experimentation has been to investigate the distance to the reinforcement bar that will influence the transmission time of the ultrasonic wave in the concrete.

For this reason four specimens with dimensions of 16 cm x 16 cm x 64 cm, with different mix design, have been investigated. They were reinforced with four bars of 18 mm diameter.

In Fig. 6.4 the position of the bars on the bottom of the framework are shown.

In Figure 6.5 the course of the ultrasonic pulse velocity measured at different heights of the specimen for the mix 40AR100 are reported.

The measurements have been taken according with the scheme of Figure 6.4.

All test results are reported in Table 6.1 for the mixes used to cast the three pre-stressed beams.

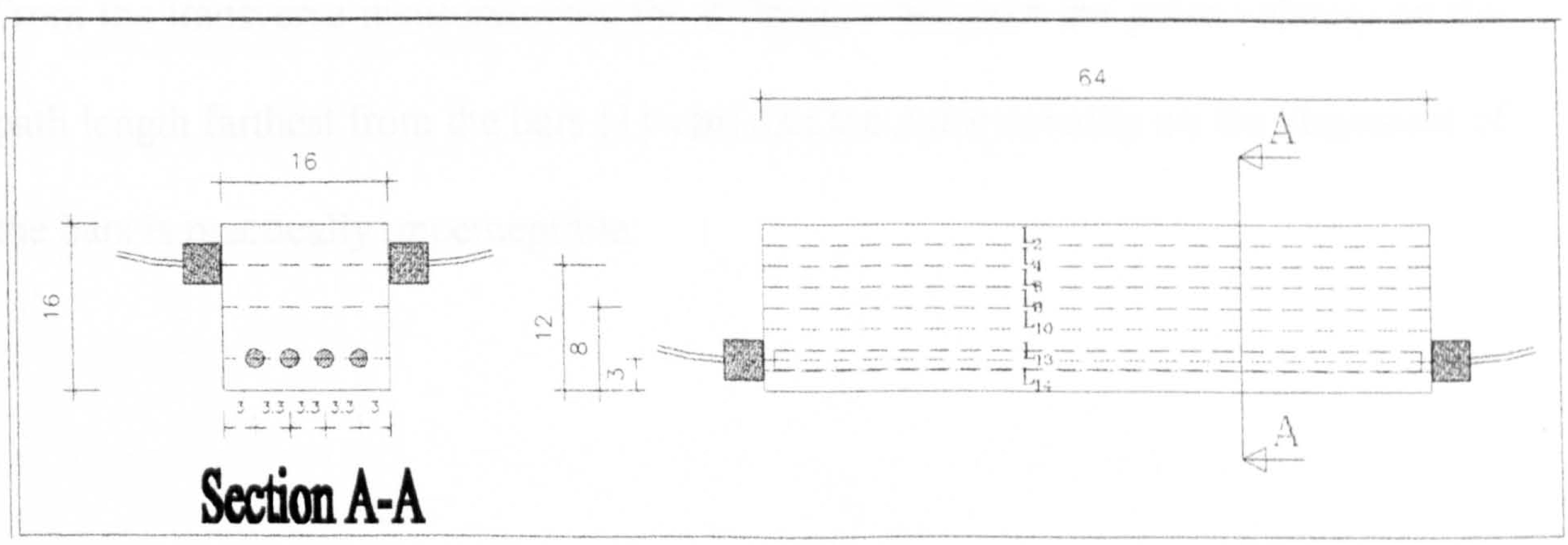


Fig. 6.4 Path length investigated on reinforced specimens (direct transmission).

		40AR100		40RN50		40AN100	
Path length	mm	Time (µsec)	Vel. (km/sec)	Time (µsec)	Vel. (km/sec)	Time (µsec)	Vel. (km/sec)
L_2	640	168	**3.810**	167	**3.832**	152	**4.211**
L_4	640	163	**3.926**	167	**3.832**	150	**4.267**
L_6	640	160	**4.000**	163	**3.926**	148	**4.324**
L_8	640	160	**4.000**	160	**4.000**	145	**4.414**
L_{10}	640	152	**4.211**	145	**4.414**	130	**4.923**
L_{13}	640	130	**4.923**	125	**5.120**	128	**5.000**
L_{14}	640	145	**4.414**	127	**5.039**	137	**4.672**

Tab. 6.1 Longitudinal pulse velocity measurements in reinforced specimens (beams).

It seems evident that nearing the position of the reinforcement bars the ultrasonic pulse velocity clearly increases.

From the longitudinal measurements carried out it appears that if the distance from the reinforcement is greater than 4 cm the presence of the bar is not relevant.

From the transverse measurements the difference between the pulse velocity on the path length farthest from the bars (11 cm) and the pulse velocity on the alignment of the bars is practically imperceptible.

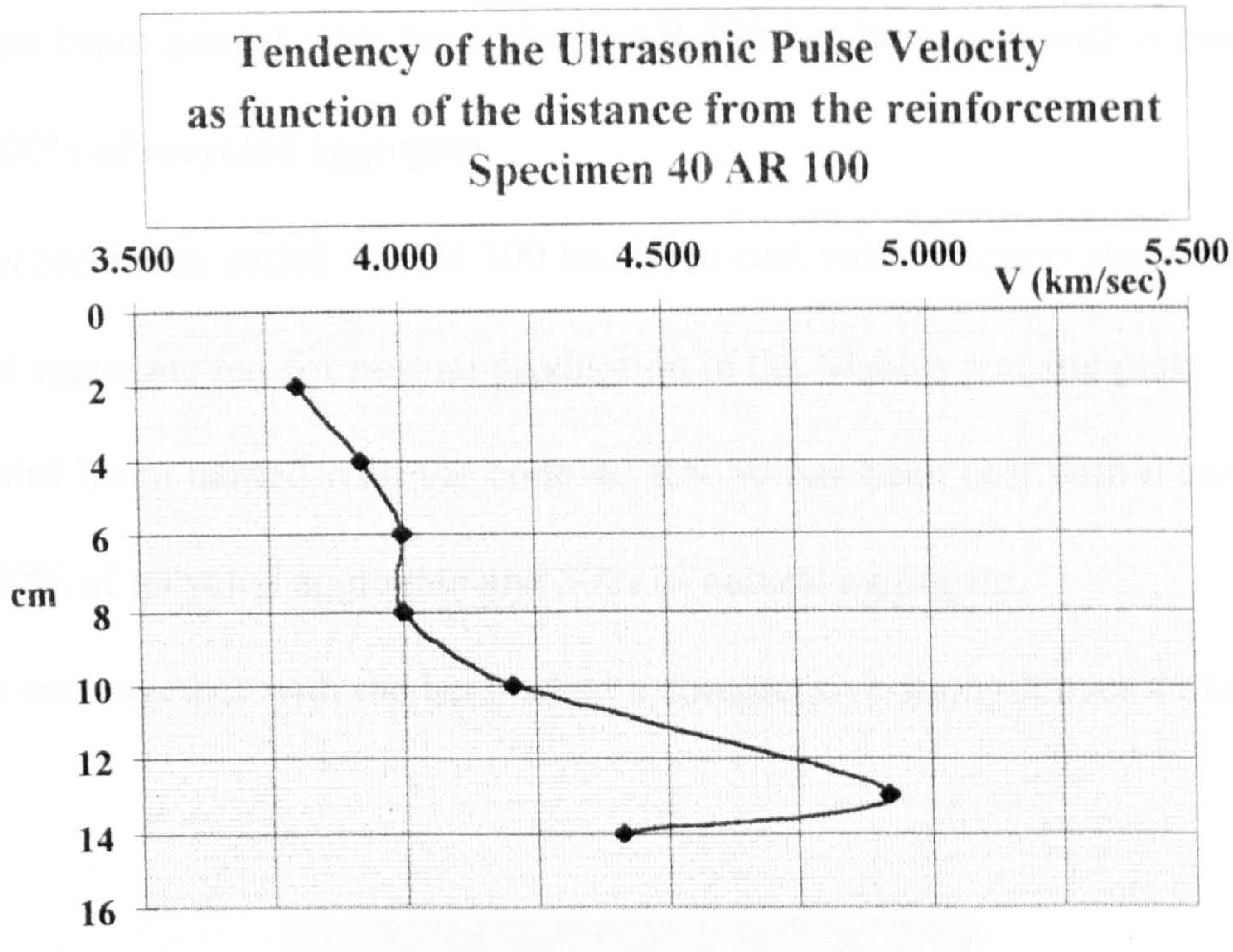


Fig.6.5 Variation of the ultrasonic velocity through the height of the specimen 40 AR 100.

6.4 Non-destructive testing on RAC prestressed beams

The experimentation reported in this paragraph has been a forerunner to the later work in detecting the crack patterns during the test to failure of the pre-stressed beams made from RAC.

An investigation using rebound hammer and ultrasonic pulse velocity tests on different zones of the beams have been performed at different ages (3, 7, 28 days).

This part of the investigation has been carried out to deduce the compressive strength of concrete using non-destructive tests and to compare the results with the values coming from compressive strength cube tests cast with the same batch of the beams.

The mix-design named 525 MAR 2 has been the one chosen to cast the pre-stressed beams (with a section of 40 cm x 70 cm and a span of 1500 cm) because it had the best mechanical performances of the previous experimentation (see Chapter Four).

The first beam named with the code 40 AR 100 has been cast with a concrete made with 100% of recycled aggregate.

The second beam coded 40 AN 100 has been cast with concrete made with 100% of natural aggregate has for normal production in the Mabo s.p.a. site plant.

The third beam named with the code 40 RN 50 has been cast with a concrete made with 50% of recycled aggregate and 50% of natural aggregate.

Cubes cast together with the beams had a compressive strength over 40 MPa.

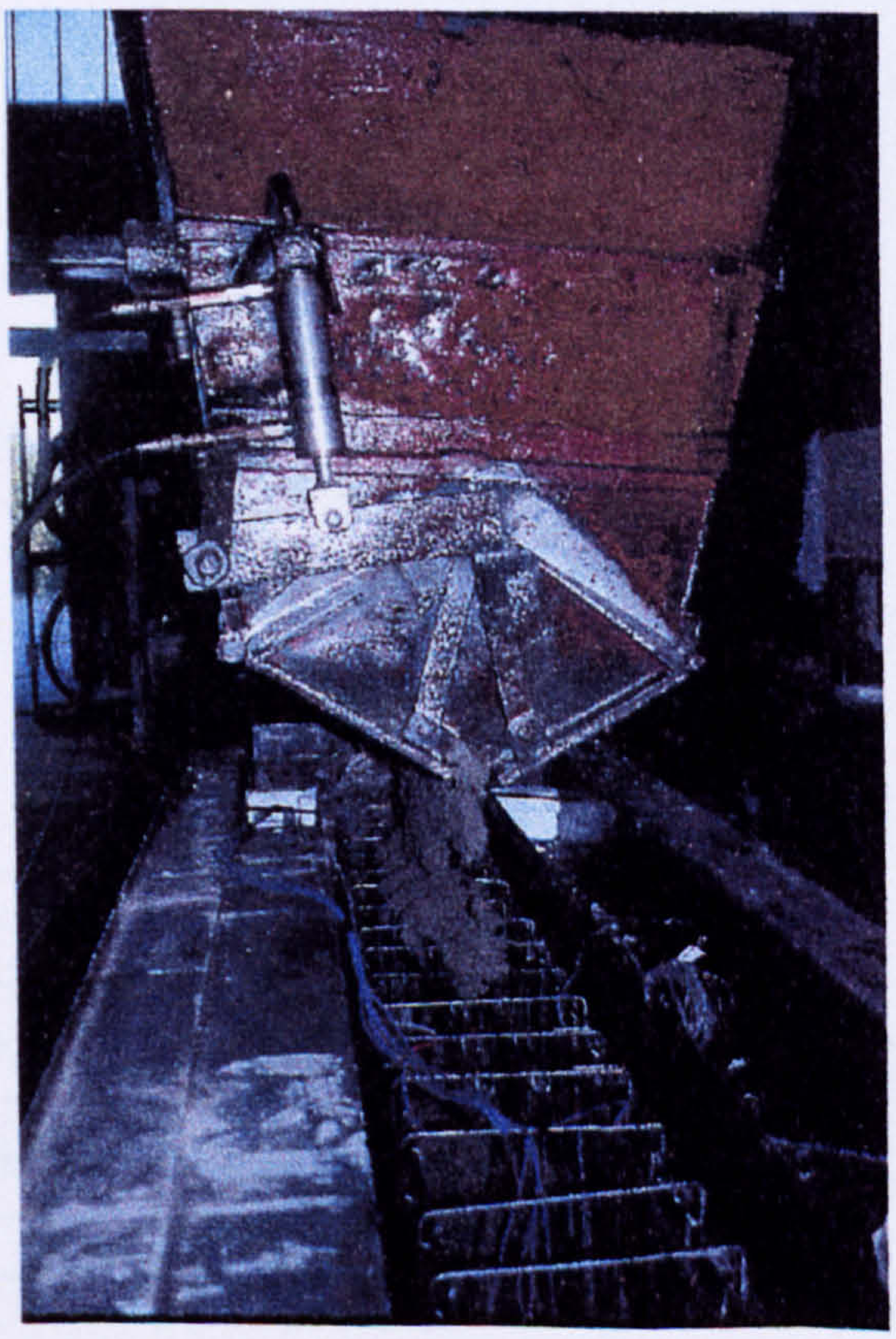


Fig. 6.6 Beam casting phases.

Compressive, tensile, flexural strength and Modulus of Elasticity tests carried out on specimens cast together with the beams are reported in Chapter Five.

In Figure 6.6 a beam casting phase is shown. The reinforcement bars and the pre-stressing tendons together with the strain-gauges are also shown.

In Fig. 6.7 the position of the points where the ultrasonic pulse velocity has been measured is reported together with a beam section that shows the path length of the ultrasonic wave inside the beams.

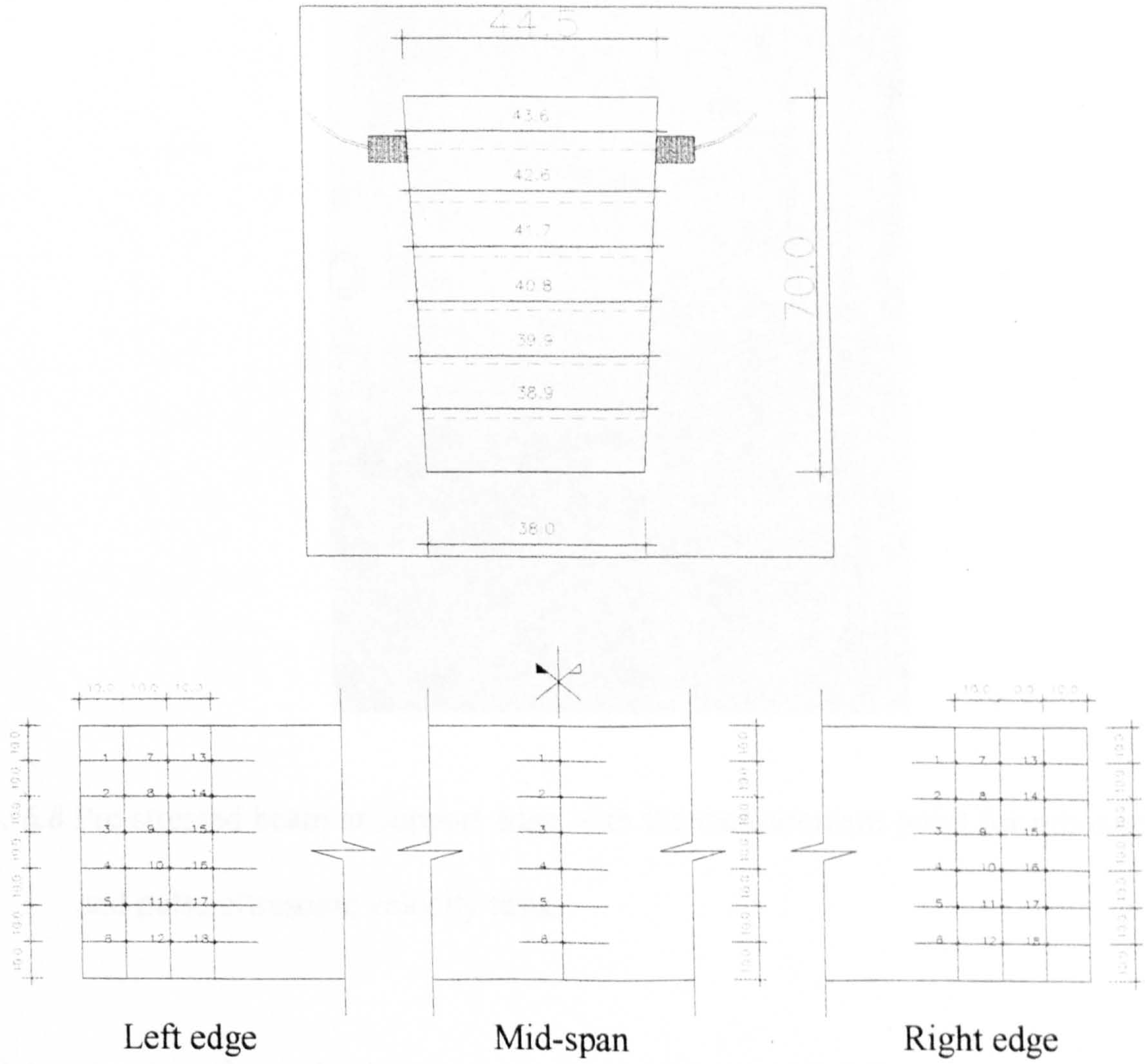


Fig. 6.7 Position for measurements for rebound hammer and pulse ultrasonic velocity tests.

During test measurements the transducers have been placed in positions such that the reinforced bars and pre-stressing tendons would not influence the path length of the ultrasonic wave.

The measurement point grid is of 100 mm x 100 mm (Figure 6.8).

The concrete surfaces on which the measurements were taken have been treated before the test to avoid any unevenness due to casting phases.

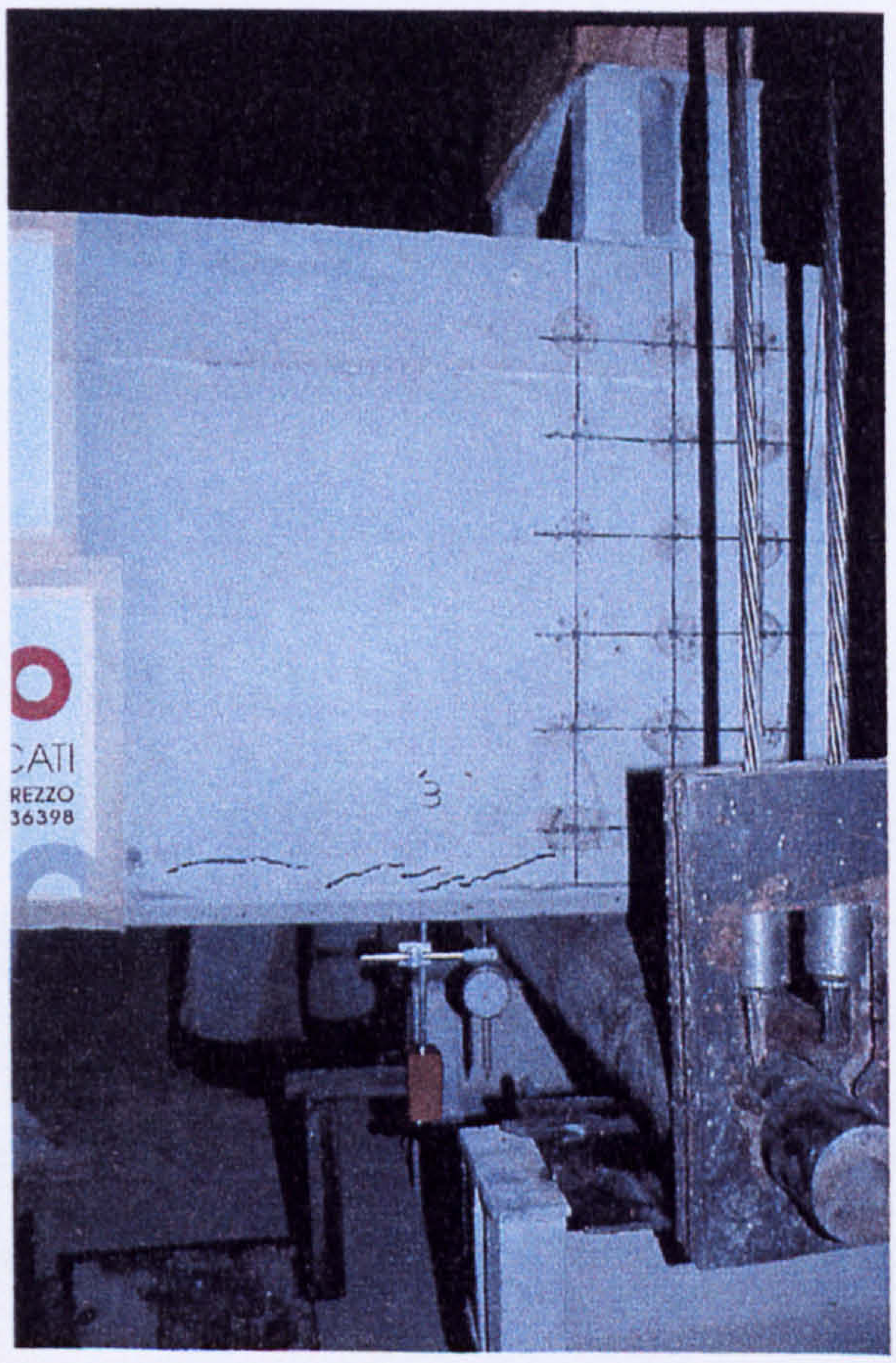


Fig. 6.8 Pre-stressed beam at support edge with the measurement point for rebound and pulse ultrasonic velocity tests.

Full test results of non-destructive tests on specimens and full scale pre-stressed beams are reported in Appendix A.

In particular compressive strength (Rc) (on 100 mm x 100 mm x 100 mm cubes), the rebound hammer number (I_R) measured on cubes and beams (on the previous shown grid), the pulse ultrasonic velocity (V) measured on cubes and beams (on the previous shown grid) are reported in Table 6.2.

The rebound hammer number (I_R) and the ultrasonic pulse velocity (V) are an average of a series of different measurements carried out on cubes and beams tested.

		40AR100			40RN50			40AN100		
Test		3 days	7 days	28 days	3 days	7 days	28 days	3 days	7 days	28 days
Rc (Mpa)	**on cubes**	33.9	36.6	44.4	34.0	42.4	45.8	43.3	47.8	59.4
I_R	**on cubes**	32.4	35.3	36.8	34.3	37.5	38.1	36.5	38.5	39.7
I_R	**on beams**	33.8	37.4	38.3	36.5	38.7	39.5	37.1	39.2	40.9
V (m/sec)	**on cubes**	3680	3730	3845	3526	3825	3889	4065	4225	4372
V (m/sec)	**on beams**	3702	3720	3751	3493	3955	4168	4108	4265	4427

Tab. 6.2 Comparison of the compressive strength R_c, the rebound number I_R, and the ultrasonic velocity V evaluated on specimens and on full scale beams.

6.5 Correlation rules between Compressive Strength (R_c), Ultrasonic Pulse Velocity (V) and Rebound Number (I_R)

Compressive strength test results coming from crushing cubes (100 mm x 100 mm x 100 mm) have been used to check the reliability of results achieved with the non-destructive methods (rebound hammer and pulse ultrasonic velocity test) on the full scale pre-stressed beams and on cubes.

The correlation rules between the Compressive Strength Rc the Rebound Number (I_R), the Pulse Ultrasonic Velocity (V) that have been used to evaluate the figures reported in Table 6.3 are given in the next page.

The Sonreb Method is a way of evaluation of the Compressive Strength of concrete using results from both the Hammer test and the Pulse velocity test. Results from this method are also reported in Table 6.3.

Test	R_C		**40AR100**			**40RN50**			**40AN100**		
			3 days	7 days	28 days	3 days	7 days	28 days	3 days	7 days	28 days
Crushing test	R_C Mpa	**on cube**	33.9	36.6	44.4	34.0	42.4	45.8	43.3	47.8	59.4
Hammer test	R_{C1} Mpa	**on cube**	29.8	37.6	42.4	34.7	44.8	46.9	41.4	48.5	53.4
	R_{C2} Mpa	**on cube**	26.9	32.6	35.9	33.6	41.2	42.7	42.2	47.7	51.1
	R_{C1} Mpa	**on beam**	33.3	44.3	47.9	41.4	49.2	52.4	43.4	51.4	58.6
	R_{C2} Mpa	**on beam**	29.6	37.1	39.3	38.7	44.2	46.3	36.5	41.4	45.5
Pulse Velocity test	R_{C3} Mpa	**on cube**	28.7	31.8	40.3	25.3	42.6	47.6	37.6	49.5	58.5
	R_{C3} Mpa	**on beam**	31.2	30.1	33.2	23.9	53.4	77.4	40.1	50.18	63.3
Sonreb method	R_{C4} Mpa	**on cube**	29.3	34.6	41.4	23.4	33.0	35.4	37.7	46.8	53.4
	R_{C4} Mpa	**on beam**	32.3	36.7	39.9	24.5	38.0	45.7	39.8	48.5	58.2

Tab. 6.3 Comparison of the compressive strength R_c evaluated with destructive and non-destructive tests on cube and full scale beams.

Correlation rules that connect the Compressive Strength R_C, ultrasonic pulse velocity V and the Rebound Hammer Number I_R are exponential and in particular:

- correlation rule R_c-I_R

$$R_{C1} = k \cdot e^{c \cdot I_R} \qquad [\,6.1\,]$$

where R_C is expressed in MPa, and I_R is the average rebound number.

- correlation rule R_c-V

$$R_{C3} = a \cdot e^{b \cdot V} \qquad [\,6.2\,]$$

where R_C is expressed in MPa, and the ultrasonic pulse velocity V is expressed in km/sec.

The rules [6.1] and [6.2]are then combined into the Sonreb method with the use of others coefficients.

$$R_{C4} = A \cdot e^{(B \cdot V + C \cdot I_R)} \qquad [\,6.3\,]$$

where:

$A=(a \cdot k)^{0.5}$ $\qquad B=b/2$ $\qquad C=c/2$

Another correlation rule R_c-I_R commonly used is:

$$R_{C2} = s \cdot I_R^{\,t} \qquad [\,6.4\,]$$

In the previous correlation rules there are coefficient *a*, *b*, *c*, e *K* that are function of the batch characteristics, the curing environment, the type of cement, and the type of aggregate used.

These coefficients have been determined during the present investigation and they are reported in Table 6.4. In Table 6.5 the same coefficients coming from another experimentation [45] are reported.

Mix Code	k	c	a	b	s	t	A	B	C
40AR100	2.25	0.08	0.015	2.060	0.010	2.27	0.184	1.0300	0.04
40RN50	2.25	0.08	0.110	1.430	0.012	2.27	0.475	0.7150	0.04
40AN100	2.25	0.08	0.055	1.745	0.011	2.27	0.352	0.8873	0.04

Tab. 6.4 Coefficient values for the rules between RC, IR, and V determined during this PhD research.

Type of Aggregate	k	c	a	b	s	t	A	B	C
Natural	7.25	0.08	0.060	1.44	-	-	0.67	0.72	0.04
Recycled	7.25	0.08	0.008	2.06	-	-	0.24	1.03	0.04

Tab. 6.5 Coefficient values for the rules between RC, IR, and V determined by Ravindrajah [45].

Comparing the two tables the differences between the coefficients coming from the two different experimentation can be noticed.

These differences are attributable to the different starting test conditions (batch characteristics, curing environment, etc.) and to the different Recycled Aggregate used. In particular the percentage of natural aggregate in the batch seems to be an important factor on the variation of the values of constant **a** and **b**. The test results coming from the indirect methods (indirect transmission) are shown in histograms in Appendix A.

From the above results (Table 6.3) it can be concluded that the values of V and I_R are lower in RAC than NAC.

The use of RA in concrete seems not to affect the correlation rule [6.1] but on the other hand it does influence the correlation rule [6.2].

The Sonreb combined method gives results close to the values attained for crushing test on cubes for the mixes AR "All Recycled" and AN "All Natural".

The differences of values the compressive strength are almost 10 to 15%, better than the single method (rebound or ultrasonic).

This is due to the fact that the Sonreb combined method attenuates the intrinsic imprecision of the single methods that are due to the curing conditions, the moisture content, and to the type of aggregate used.

More disperse are the values relative to the mix 40 RN 50 with differences of 30% to 40%.

6.6 Detection of crack patterns on RAC pre-stressed beam using ultrasonic pulse velocity

The investigation described in the previous paragraph has been a preparatory study to the experimentation reported in this paragraph.

In fact during this last part of the study the ultrasonic method has been used to detect the depth of the cracks occurring during the pre-stressed beam test.

Comparing these results with the experimental results has checked the validity of this method.

The indirect transmission has been the ultrasonic method used.

The beam zone investigated has been the mid-span one.

Fig. 6.9 Reinforcement bars and the prestressing tendons inside the beam.

In Figure 6.9 the reinforcement bars and the pre-stressing tendons are shown. It can be noticed that reinforcement (bars and tendons) present in the mid part of the bottom of the beam are sufficiently far from the zone investigated with the ultrasonic method.

In this way the path length of the ultrasonic wave is not diverted by reinforcements.

Load rate for beam test was of 1000 kg (500 kg each jack). When cracks have been noticed they have been marked and coded with a capital letter (see Figure 6.10).

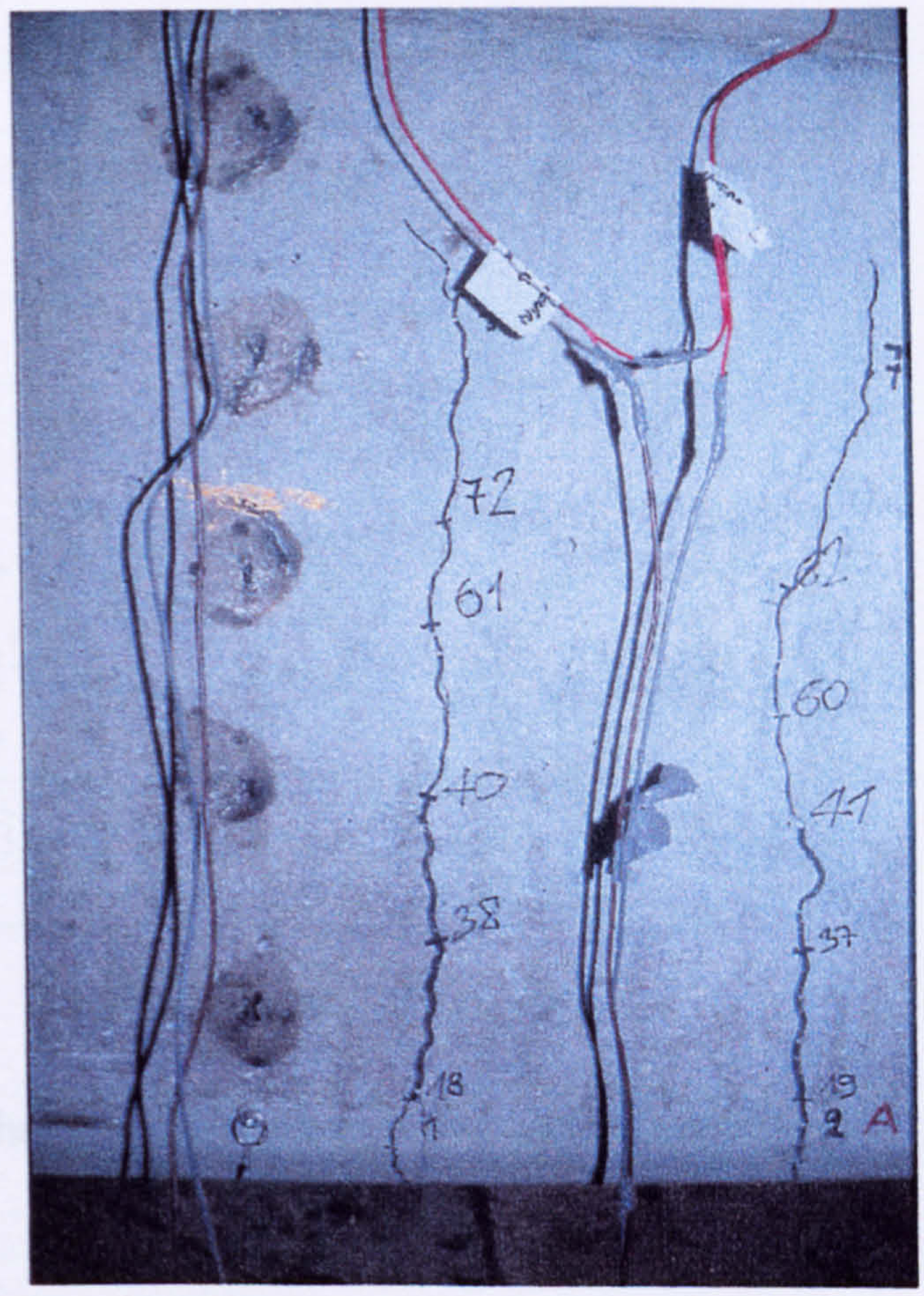


Fig. 6.10 Crack detection in the beam mid-span.

After that the ultrasonic transducers have been placed astride the crack layer with a distance of the presumed crack depth.

The distance between the two transducers was 20 cm.

To improve the ultrasonic wave transit, glycerine paste has been used.

The transducers have been placed in the mid part of the beam intrados where the ultrasonic wave are not influenced by the presence of reinforcement.

With the transducers in this position the ultrasonic wave is transmitted along a presumed path as reported in Figure 6.11.

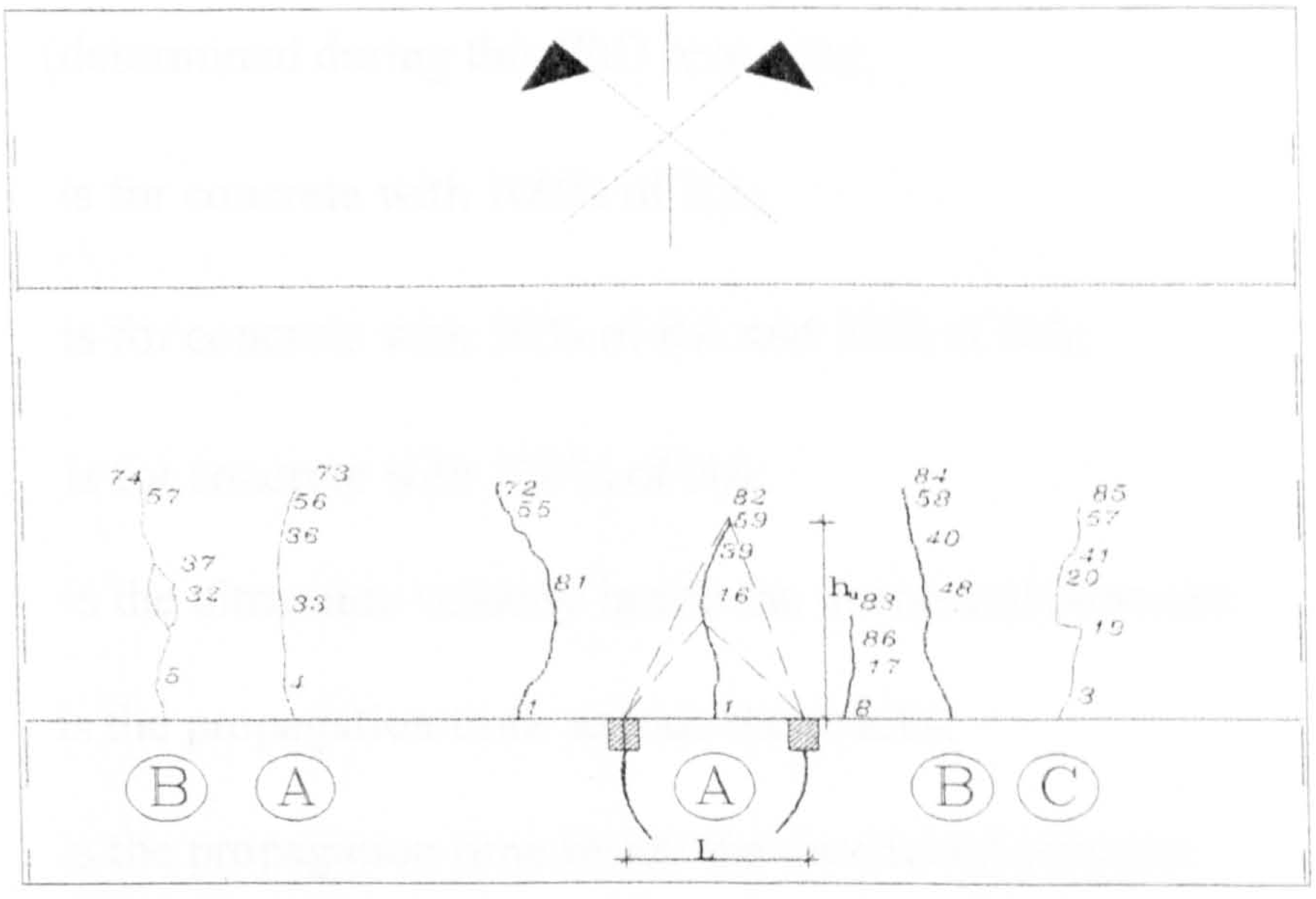


Fig. 6.11 Path of the ultrasonic wave (------) on the crack layer.

The real path of the ultrasonic wave is very difficult to define for the refraction and reflection wave phenomena due to the shape of the cracks.

Moreover it is possible that there are contact points inside the crack layer that allow the undesired energy transmission of the sonorous wave.

As can be seen in the following Tables 6.6 to 6.8 (where a comparison between the optical and ultrasonic crack measurement is reported) that this inconvenience sometimes can give an erroneous measurement using the ultrasonic method.

The crack depth has been determined with the following relation, that is a partial modification of the relation in [46]:

$$h_u = \alpha \cdot V_0 \cdot \sqrt{t_i^2 - t_0^2}$$

where:

α is a calibration coefficient for the evaluation of the cracks height (determined during this PhD research);

$\alpha = 1.3$ is for concrete with 100% of RA;

$\alpha = 1.1$ is for concrete with 50% of RA and 50% of NA;

$\alpha = 0.7$ is for concrete with 100% of NA;

V_0 is the ultrasonic velocity inside the uncracked concrete;

t_i is the propagation time astride the cracks;

t_0 is the propagation time inside the uncracked concrete.

$\varepsilon = 315$					P = 157.5 KN			
Crack	Left edge				Right edge			
	t_1 average	Optical reading	Ultrasonic reading	Δ	t_1 average	Optical reading	Ultrasonic reading	Δ
	μsec	h_O (mm)	h_V (mm)	$h_O - h_V$	μsec	h_O (mm)	H_V (mm)	$h_O - h_V$
A	91.4	310	341	-31	75.0	350	237	113
B	87.1	270	316	-46	71.8	310	214	96
C	87.8	340	319	21	86.9	340	314	26
D	86.0	320	309	11	88.9	330	326	4
E	86.6	340	312	28	92.6	340	349	-9
F	80.8	330	276	54	87.4	330	317	13
G	77.5	280	255	25	85.7	360	307	53
H	91.8	340	343	-3	89.5	260	330	-70
I	92.4	280	347	-67	88.9	320	326	-4
L	90.7	220	337	-117	86.6	250	312	-62

Table 6.6 Comparison between the height of crack measures determined with the optical method and the ultrasonic one. Beam 40AR100.

The measurements have been taken under two different load levels.

The first measurement was taken with height of cracks of almost 20 cm.

The second one (reported in Tables in the Appendix A) when the crack height was over the half height of the beam.

$\varepsilon = 360$					P = 180 KN			
Crack	Left edge				Right edge			
	T_1 average	Optical reading	Ultrasonic reading	Δ	t_1 average	Optical reading	Ultrasonic reading	Δ
	μsec	h_O (mm)	h_V (mm)	$h_O - h_V$	μsec	h_O (mm)	h_V (mm)	$h_O - h_V$
A	86.4	210	250	-40	83.2	190	235	-45
B	75.9	190	198	-8	85.7	150	247	-97
C	86.7	210	252	-42	82.8	220	233	-13
D	86.1	180	249	-69	87.9	250	258	-8
E	87.5	230	256	-26	96.4	40	297	-257
F	88.6	230	261	-31	89.3	140	264	-124
G	84.2	130	240	-110	89.9	210	267	-57
H	87.5	120	256	-136	90.6	190	270	-80
I	87.0	170	253	-83	87.8	210	257	-47
L	84.6	60	242	-182	86.1	180	249	-69

Table 6.7 Comparison between the height of crack measures determined with the optical method and the ultrasonic one. Beam 40RN50.

$\varepsilon = 370$					P = 185 KN			
Crack	Left edge				Right edge			
	T_1 Average	Optical reading	Ultrasonic reading	Δ	t_1 average	Optical reading	Ultrasonic reading	Δ
	μsec	h_O (mm)	h_V (mm)	$h_O - h_V$	μsec	h_O (mm)	h_V (mm)	$h_O - h_V$
A	113.7	200	283	-83	118.8	140	299	-159
B	81.5	250	178	72	111.7	100	277	-177
C	116.0	220	290	-70	84.8	90	190	-100
D	111.3	220	276	-56	104.8	140	255	-115
E	85.2	150	191	-41	108.6	150	267	-117
F	89.4	210	205	5	108.5	150	267	-117
G	84.6	220	189	31	110.2	190	272	-82
H	107.3	190	263	-73	81.8	90	179	-89
I	108.3	180	266	-86	81.5	40	174	-134
L	111.3	150	276	-126	67.2	40	124	-84

Table 6.8 Comparison between the height of crack measures determined with the optical method and the ultrasonic one. Beam 40AN100.

In Figures 6.12 to 6.14 the comparison between the real crack patterns and the ultrasonic detection of the crack patterns are reported for the three pre-stressed beams..

In these Figures the crack codes mentioned in the Tables 6.6 to 6.8.are adopted.

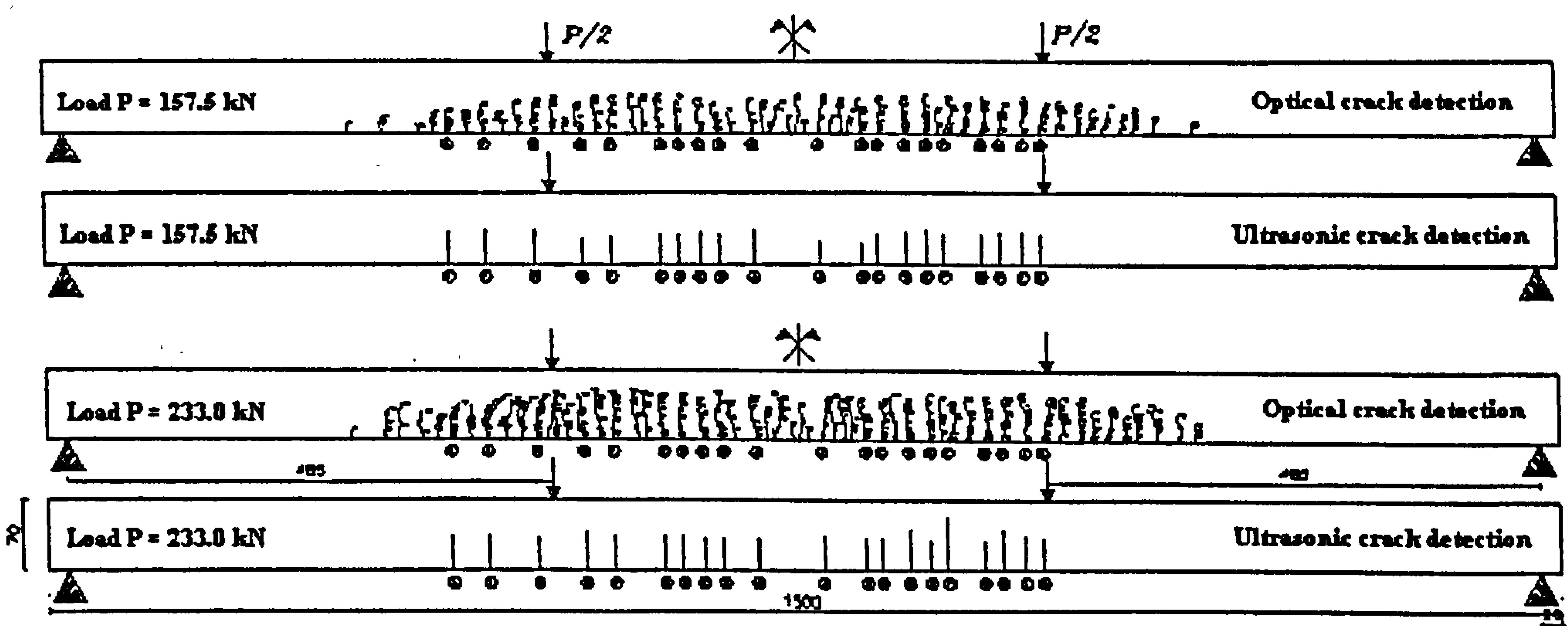


Fig. 6.12 Comparison between the real crack patterns and the crack patterns evaluated with the ultrasonic method for the beam 40AR100.

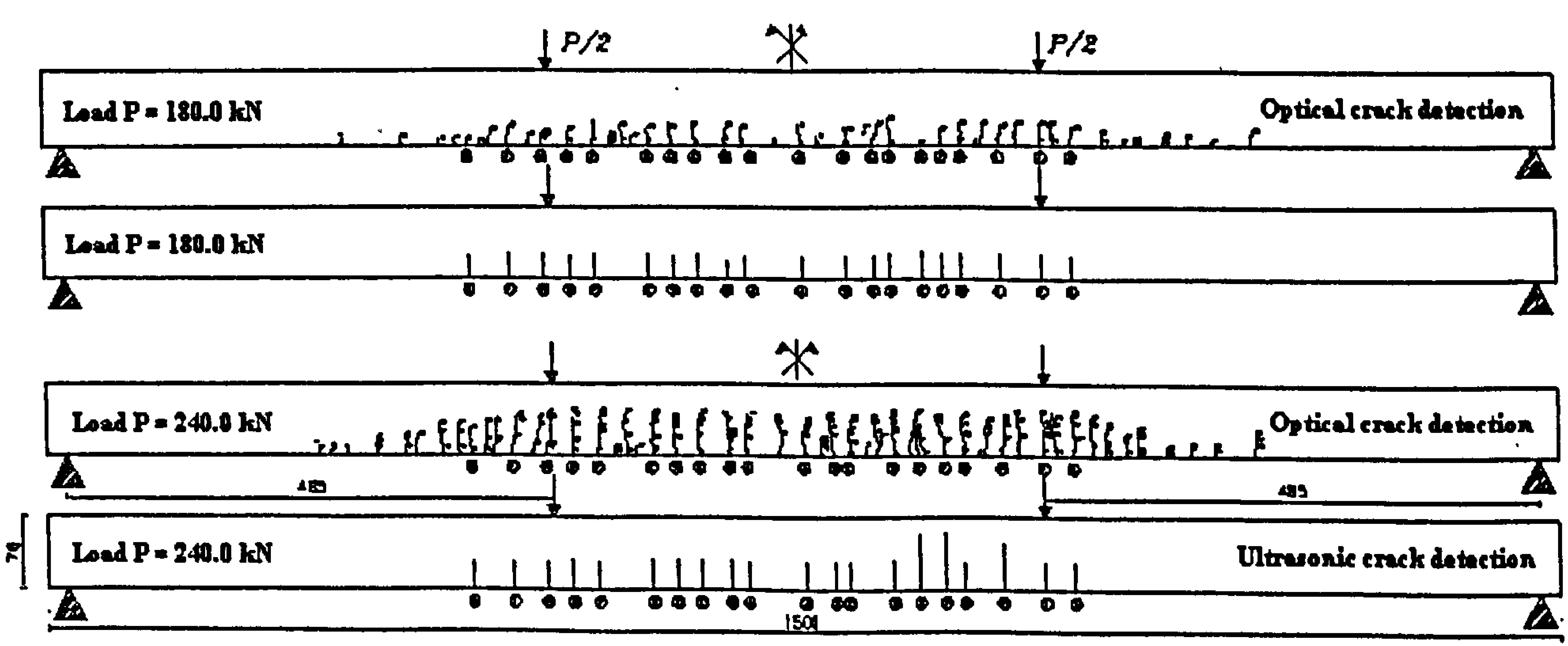


Fig. 6.13 Comparison between the real crack patterns and the crack patterns evaluated with the ultrasonic method for the beam 40RN50.

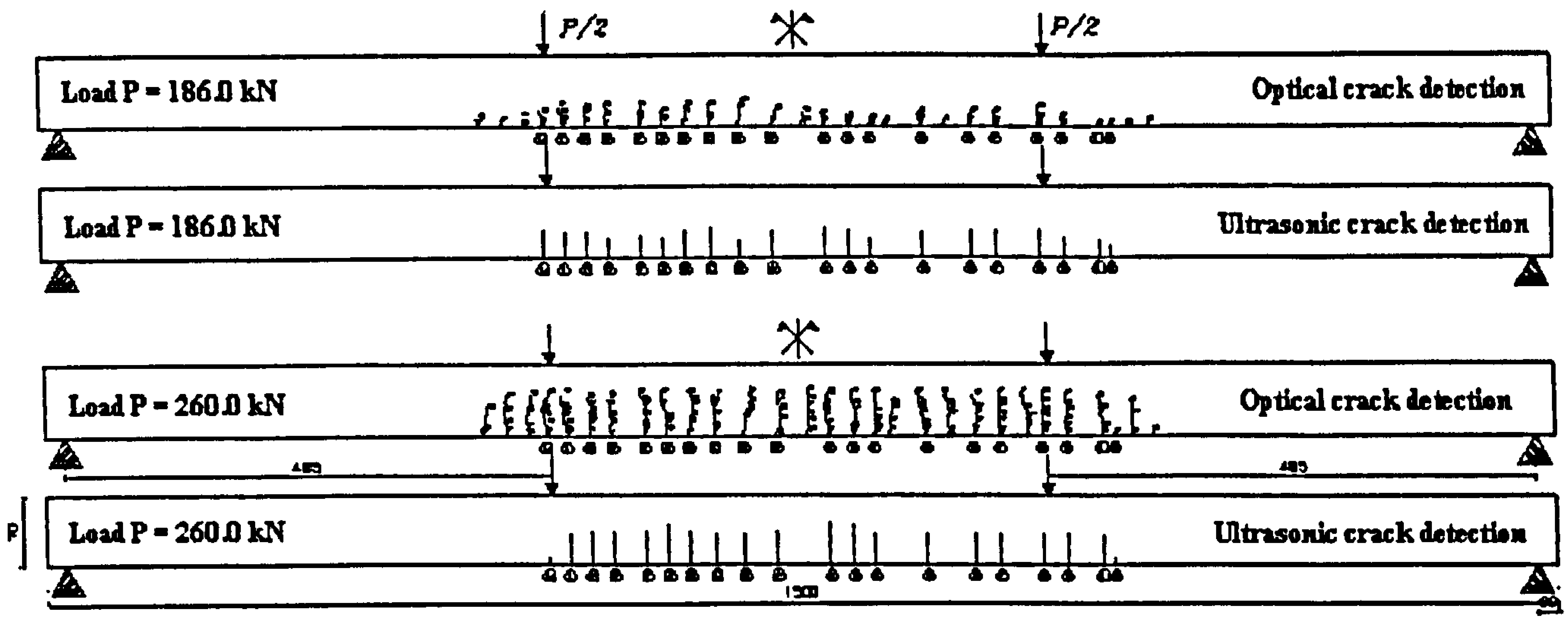


Fig. 6.14 Comparison between the real crack patterns and the crack patterns evaluated with the ultrasonic method for the beam 40AN100.

From the previous study it can be concluded that this non-destructive investigation can be usefully used to check the real condition, for safety evaluation, of the in-situ concrete structures.

In this way if a concrete structure should be demolished the strength of the original concrete could be known for a recycling use of it.

In fact at the moment, except for pre-cast plant concrete waste material, the compressive strength of the original concrete in the demolished structure processed in the recycling site plant is almost in all case unknown.

Moreover this non-destructive method could be used to monitor the service life of a prototype structure made from RAC.

Chapter Seven

Conclusions

7.1 Introduction

In this Chapter conclusions derived from the study are reported. Economic and environmental aspects using RAC are discussed. Suggestions and recommended areas of future study are given.

7.2 Conclusions

1. From the physical and chemical analysis it appears that the density of the Recycled Aggregate is lower and the water absorption is higher than the Natural Aggregate.

2. Particular attention should be paid to the determination of the water absorption value of the recycled aggregate.

   In fact this extra water can influence markedly the workability of the concrete and therefore slump design values may not be attained.

During the first moments of the mixing, a rapid absorption of water from the old cement paste attached to the Recycled Aggregate occurs and that causes a considerable loss of workability.

3. The concrete produced with Recycled Aggregate, attained mechanical performances comparable with conventional concrete and thus allowed the production of pre-stressed structural elements.

4. A prefabrication plant offers the best possibility of obtaining uniformity of casting and curing in the production of structural elements with RAC so that the results obtained from these laboratory tests can be repeated.

5. Because carbonation of the old cement paste is one of the most common events that can occur in Recycled Aggregate it is advisable to use Recycled Aggregate Concrete designed with low water/cement ratio to reduce the pore system of the concrete [43].

6. Wet curing (high humidity period) is recommended for the Recycled Aggregate Concrete so that the hydration of the cement continues and reduces the depth of carbonation.

7. The type of cement used to make the Recycled Aggregate Concrete is important (such as blended cement: silica and fly ash cement). The blended cement leads to

a lower $Ca(OH)_2$ content in the hardened cement paste and a smaller amount of CO_2 is required to remove $Ca(OH)_2$ by producing $CaCO_3$.

8. RAC with higher strength help to restrict carbonation.

9. Prefabrication can permit the use of wet cured RAC (e.g. steam curing) and give the high compressive strengths required in the production of pre-stressed elements [42].

10. To reach the mechanical strength suitable for structural use of RAC, it is necessary to use mixes with a blending of natural and recycled aggregate and/or to use high strength cement.

11. To get consistency in attaining the target strength for Recycled Aggregate Concrete it is necessary follow a mix-design procedures specially modified for RAC.

12. It has been demonstrated that using a 52.5 Portland cement it is possible to reach a designed target strength of 50 MPa with 100% of Recycled Aggregate. This is a confirmation of the possibility of RAC for structural purposes under a compressive strength point of view.

13. It has been demonstrated that it is possible to produce a RAC concrete suitable for pre-stressed elements application [41].

14. The use of RAC in pre-cast work can guarantee a better quality of the Recycled Aggregate Concrete than in-situ and it can reduce the risk of errors in mix proportioning.

15. The large scale test results show that it is possible to produce pre-stressed beams using concrete with 100% and 50% of Recycled Aggregates.

16. The beams made from RAC with 100% of RA and 50% of RA performed satisfactorily in both casting and testing phases.

17. To attain this result it is very important to have a correct mix-design procedure together with the exact control of the mix components.

18. The mechanical behaviour of pre-stressed beams made from RAC is clearly influenced by the E-value.

19. A marked recovery of deformations has been noticed in the beams cast with a concrete with 100% of RA compared with the one with 100% of NA.

20. The investigations carried out during this PhD research on mechanical properties of hardened RAC show that it is possible to conclude that RAC can be considered as a construction material.

Further study should be carried out to check its behaviour with time (creep and durability).

Nevertheless at the moment what stops a real application of this technique is a «recycling culture» that is today accepted only for the day by day waste products (glass, paper, etc.).

21. The recycling of concrete wastes inside a pre-cast plant can guarantee a better quality of Recycled Aggregate Concrete.

22. Problems concerned with the disposal of the demolition wastes can be solved inside a pre-cast plant.

23. The recycling process inside a pre-cast plant could be economically convenient if simple recycling equipment is adopted.

7.2.1 Commercial aspects of developing sources of RAC

The main advantages coming from recycling in prefabrication are a guarantee of the RAC production.

In previous works it has been shown by the author that RAC is suitable for pre-stressed elements and how the production of RAC in a pre-cast plant [40], [41] could solve most of the actual limits of this type of concrete.

These could be summarised as follows:

- known sources of Recycled Aggregate and consequently of the strength of the original concrete;

- limitation of the contaminants;
- strict respect for the mix proportions with a possible controlled use of admixtures;
- use of high strength cement (e.g. 52.5 Portland cement);
- use of low w/c ratio;
- benefits to the mechanical strength from the wet curing (steam) of RAC;

7.2.2 Environmental aspects using RAC

The recycling of concrete involves first of all environmental aspects. The benefits coming from concrete recycling are evident for the environment.

Firstly it offers a method of disposing of demolition wastes. The second one is strictly involved with the quarry and the landscape.

But above all the recycling of concrete should be economically justified if it is encouraged as a construction material.

7.2.3 Suggestions and recommended areas of future studies

It is proposed to extend the study to include investigation into:

a) the carbonation of concrete with high w/c ratio

b) how the bonding surfaces can be a catalyst for the propagation of cracks

c) RAC pre-stressed walls

d) RAC pre-stressed hollow core slab

e) the time dependant mechanical properties of RAC (creep)

f) fatigue under cyclic loads

g) resistance of chemical attack of RAC

h) interaction between the chemical substances present in the admixtures and the Recycled Aggregate that are used for in-situ casting.

i) the behaviour of Recycled Aggregate Concrete made only with concrete wastes from a pre-cast plant;

j) economic investigation on recycling plant costs.

REFERENCES

References

1. **Nixon, P.J.**
Recycled concrete as an aggregate for concrete - a review. First state-of-the-art report RILEM TC-37-DRC.
Materials and Structures (RILEM), Vol. 11, No. 65, Sept.-Oct. 1978, pp. 371-378.

2. **Hansen, T.C.**
Recycled aggregates and recycled aggregate concrete. Second state-of-the-art report developments 1945-1985 RILEM TC-37-DRC.
Materials and Structures (RILEM), Vol. 19, No. 111, May-June 1986, pp. 201-246.

3. **Hansen, T.C.**
Recycling of Demolished Concrete and Masonry
E & FN Spon / Chapman & Hall, 1992.

4. **Dhir, R.K., Henderson, N.A. and Limbachiya, M.C.**
Sustainable Construction: Use of Recycled Concrete Aggregate
Proceedings of the International Symposium organised by the Concrete Technology Unit, University of Dundee. London, 11-12 November 1998.
Thomas Telford, 1998.

5. **ASTM C-33-99ae1**
Standard Specification for Concrete Aggregates
American Society for Testing and Materials.

6. **Hansen, T.C. and Narud, H.**
Strength of recycled concrete made from crushed concrete coarse aggregate.
Concrete International - Design and Construction, 5, No.1, pp.79-83,1983.

7. **Hasaba, S., Kawamura, M., Toriik, K. *et al.***
Drying shrinkage and durability of concrete made of recycled concrete aggregates.
Traslation of the Japan Concrete Institute, 3, pp. 55-60 (Additional information obtained from background report in Japanese).

8. **B.C.S.J.**
Proposed standard for the use of recycled aggregate and recycled aggregate concrete.
Building Contractors Society of Japan. Committee on Disposal and Reuse of Construction Waste (English version published in June 1981).

9. **Kreijger, P.C.**
Hergebruik van Boun-en Sloopafval als Toeslagmateriaal in Beton.
TH-Eindhoven, Afdeling Bouwkunde, Rapport M83-1, 1983.

10. **Mukai, T., Kikuchi, M. and Ishikawa, N.**
Study on the properties of concrete containing recycled concrete aggregate.
Cement Association of Japan,1978, 32nd Review, (in English).

11. **B.C.S.J.**
Study on recycled aggregate and recycled aggregate concrete.
Building Contractors Society of Japan. Committee on Disposal and Reuse of Construction Waste. Summary in *Concrete Journal*, Japan, 16, No.7, pp.18-31 (in Japanese), 1978.

12. **Wesche, K. and Schultz, R.**
Beton aus aufbereitetem Altbeton. Technologie und Eigenschaften
Beton, 32, Nos. 2 and 3, 1982.

13. **Soshiroda, T.**
Recycled concrete.
Proceedings 9th Congress of CIB, Stockholm, 1983.

14. **Kikuki, M., Mukai, T. and Kozumi, H.**
Properties of concrete products containing recycled aggregate.
Ibid. Ref. 5, pp.595-604.

15. **Kashino, N. and Takahashi, Y.**
Experimental studies on placement of recycled aggregate concrete.
Ibid. Ref. 5, pp.557-564.

16. **Yanagi,k., Hisaka, M., Nakagama, M. and Kasai, Y.**
Effect of impurities in recycled coarse aggregate upon a few properties of the concrete produced with it.
Ibid. Ref. 5, pp.613-620.

17. **Yamato, T., Emoto, Y., Soeda, M. and Sakamoto, Y.**
Some properties of recycled aggregate concrete

Ibid. Ref. 5, pp.643-651.

18. Mukai T. and Kikuchi M.
Properties of reinforced concrete beams containing recycled aggregate.
Ibid. Ref. 5, pp.670-679.

19. de Vries, P
Concrete Re-cycled: Crushed Concrete as Aggregate.
Proceedings of the International Conference "Concrete in Service of Mankind". Dundee, Scotland. 24-28 June 1996. Vol. Pag. 121 - 130.
E & F.N. Spon, London 1996.

20. Hansen, T.C. and Boegh, E.
Elasticity and drying shrinkage of recycled aggregate concretes.
ACI Journal, pp.648-652, 1985.

21. Rasheeduzzafar and Khan, A.
Recycled concrete - a source of new aggregate.
Cement, Concrete and Aggregates (ASTM), 6, No.1, pp.17-27, 1984.

22. Ivanyi, G., Lardi, R. and Esser, A.
Recycling beton.
Forschungsbericht aus dem Fachbereich Bauwesen No. 33, Universitat-Gesamthochschule, Essen, 1985.

23. Foster, S.W.
Recycled concrete as aggregate.
Concrete International - Design and Construction (ACI), 1986, pp.34-40.

24. Moschettini, F.
Conglomerati cementizi di "calcestruzzi riciclati" con l'impiego di additivi (in italian).
Tesi di laurea in Tecnica dei lavori stradali, ferroviari ed aeroportuali. Facolta' di Ingegneria, Universita' degli Studi di Roma "La Sapienza", Italia, 1991.

25. Samarai, M.A.
The disintegration of concrete containing sulphate contaminated aggregates.
Magazine of Concrete Research, 28, No.96, pp.130-140, 1976.

26. Di Niro, G.
Analisi non lineare di una trave in conglomerato cementizio armato prodotto con aggregati riciclati (in italian).
Tesi di laurea in Strutture Prefabbricate.

Facolta' di Ingegneria, Universita' degli Studi di Roma "La Sapienza", Italia, 1995.

27. **Dolara,E., Ridgway, P., Di Niro, G.**
L'aggregato riciclato. Caratteristiche chimico-fisiche (in italian).
Quarry and Construction. Edizioni Pei. Parma. Dicembre 1995.

28. **Dolara,E., Ridgway, P., Di Niro, G.**
Proprietà e mix design del calcestruzzo fresco prodotto con aggregati riciclati (in italian).
Quarry and Construction. Edizioni Pei. Parma. Gennaio 1996.

29. **Dolara,E., Ridgway, P., Di Niro, G.**
Analisi non lineare di una trave in conglomerato cementizio armato prodotto con aggregati riciclati (in italian).
Quarry and Construction. Edizioni Pei. Parma. Marzo 1996.

30. **Dolara,E., Di Niro, G., Ridgway, P.**
Il calcestruzzo prodotto con aggregati riciclati (in italian).
Mabogazine. Mabo Prefabbricati s.r.l.. Maggio 1996.

31. **G Di Niro, E Dolara, P Ridgway.**
Recycled Aggregate Concrete (RAC). Properties of Aggregate and RC beams made from RAC.
Proceedings of the International Conference "Concrete in Service of Mankind". Dundee, Scotland. 24-28 June 1996. Pag. 141 - 149.
E & F.N. Spon, London 1996.

32. **Norma UNI 8529, parti 1-22**
Aggregati per calcestruzzo (in italian).
Ente Nazionale Italiano di Unificazione.

33. **BS 882: 1992**
Spec. for aggregates from natural sources for concrete
British Standard Istitution.

34. **BS 812: various parts**
Testing Aggregates
British Standard Istitution.

35. **C.N.R. Anno VI, n. 27.**
Istruzioni per la determinazione dell'equivalente in sabbia.
Consiglio Nazionale delle Ricerche, 30 Marzo 1972.

36. **Di Niro, G., Dolara, E., Cairns, R.**
Recycled Aggregate Concrete (RAC). A mix-design procedure.
Proceedings of the International FIP 97 Synposium "The Concrete Way to Development". Johannesburg, South Africa. 9 - 12 March 1997.
Concrete Society of Southern Africa.

37. **Di Niro, G., Dolara, E., Cairns, R., Ridgway, P.**
Recycled Aggregate Concrete (RAC). A mix-design procedure.
Proceedings of the International Conference "Our world in Concrete and Structures". August 1996.Orchard Plaza. Singapore.

38. **Di Niro, G., Dolara, E., Cairns, R.**
The use of RAC (Recycled Aggregate Concrete) for structural purposes in prefabrication.
Proceedings of the XIIIth FIP - CEB 98 Congress "Challenges for Concrete in the next millenium" Amsterdam,The Netherlands. 23 -29 May 1998.
A.A. Balkema Publishers.

39. **M Sciotti, R Cairns, G Di Niro, E Dolara.**
Bonding surface between Recycled Aggregate and new cement paste.
Proceedings of the International Symposium organised by the Concrete Technology Unit, University of Dundee and held at the Department of Trade and Industry Conference Centre, London, United Kingdom, 11-12 November 1998.
Thomas Telford Publishing.

40. **G Di Niro, R Cairns, E Dolara.**
Properties of Hardened RAC for Structural Purposes.
Proceedings of the International Symposium organised by the Concrete Technology Unit, University of Dundee and held at the Department of Trade and Industry Conference Centre, London, United Kingdom, 11-12 November 1998.
Thomas Telford Publishing.

41. **E Dolara, G Di Niro, R Cairns.**
RAC Prestressed Beams.
Proceedings of the International Symposium organised by the Concrete Technology Unit, University of Dundee and held at the Department of Trade and Industry Conference Centre, London, United Kingdom, 11-12 November 1998.
Thomas Telford Publishing.

42. **R Cairns, G Di Niro, E Dolara.**
The Use of RAC in Prefabrication.

Proceedings of the International Symposium organised by the Concrete Technology Unit, University of Dundee and held at the Department of Trade and Industry Conference Centre, London, United Kingdom, 11-12 November 1998.
Thomas Telford Publishing.

43. **Neville, A., M..**
Properties of concrete.
Fourth Edition.
Longman, 1995.

44. **Ministero dei Lavori Pubblici. D.M. 09 - 01 – 1996.**
"Norme tecniche per il calcolo, l'esecuzione ed il collaudo delle strutture in cemento armato, normale e precompresso e per le strutture metalliche."(in italian)
Edizioni DEI. Genio Civile.

45. **Ravindrarajah R.S., Tam T.C.,**
Properties of Concrete made with Crushed Concrete as Coarse Aggregate.
Magazine of Concrete Research, 37, No. 130. 1985.

46. **Bocca, P.**
I difetti delle strutture finite rilevati mediante ultrasuoni.
Industria Italiana del Cemento.

47. **Bocca P., Cianfrone F.**
Le prove non distruttive sulle costruzioni: una metodologia combinata.
Industria Italiana del Cemento. 6/1983.