THE USE OF LETHABO FIELD 2 PFA IN PAVEMENT QUALITY CONCRETE

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DECLARATION

The writer hereby declares that the work contained herein, with the exception of quoted sources, is his own and has not been submitted to any other university.

Signed by candidate

Signature Removed

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ABSTRACT

Concrete used in pavements has to be durable to withstand the load and wear imposed by vehicles moving across it and the effects of drying shrinkage and thermal changes. Failure of the pavement by either excessive cracking or degradation of the surface results in poor riding quality and low skid resistance.

The inclusion of Pulverised Fuel Ash (PFA), otherwise known as fly ash, generally improves the quality of pavement concrete and thus extends its useful life. The PFA used for the thesis was from the Lethabo power station's second electrostatic precipitator field (Lethabo Field 2 PFA). This Field 2 PFA has a very close resemblance to the expected classified commercial Lethabo PFA of the future. PFA is characterised by its fineness. The Field 2 PFA had 7.7 percent retained on the 45 micron sieve. This was considerably finer than the current commercial Matla PFA with about 12 percent retained. It was thus expected that the higher quality Lethabo Field 2 PFA could be used to produce higher quality concrete.

The other mix materials were those commonly used in the Western Cape. The aggregates used were Cape Flats Dune sand and Malmesbury shale (hornfels). The dune sand typically has very little fines content, causing severe bleeding problems in normal concrete mixes. The crushed coarse aggregate was 13 mm and flaky in shape. Ordinary Portland cement (OPC) was obtained from the De Hoek cement factory.

The investigation was carried out in two parts. First was the development of a wide range of mixes, varying 28 day design strength (10, 20, 30, 40, 50 MPa), percentage of PFA as part cement replacement (OPC only, 15% PFA, 30% PFA, 50% PFA and 70% PFA) and the coarse aggregate content to give under-, average- and over-sanded mixes. Over this wide range of mixes, the fresh properties and development of the compressive strength were observed. Secondly, properties affecting pavement quality concrete were observed on a similar range of mixes. These properties were flexural strength, surface wear resistance by wire brush, sand blasting and ball race abrasion and the drying shrinkage.

Design of the all OPC and PFA mixes was according to the D & H Ash Resources and PCI methods. For pavement quality concrete the mixes were designed as "low slump concrete". A design slump of 50 mm was thus chosen. Despite the required increase in total cements (OPC + PFA) to water ratio for increasing PFA percentage, all mixes of equivalent 28 day compressive strength had

similar total cements content. This was because the PFA gave substantial reductions in water requirements.

Using the mix design charts based on Matla PFA and Transvaal materials, the Lethabo Field 2 PFA mixes all showed substantial strength improvements. The PFA concrete compressive strength development compared to that of OPC concrete, showed a typical pozzolanic reaction. For equivalent 28 day compressive strength, the PFA mixes had lower 7 days strength and higher 90 days strength.

This long term additional strength development, typical of all the time dependant properties of PFA concretes, is of importance in the design of pavements as allowance should be made for this additional strength. Flexural strength is more critical than compressive strength in pavements, and here this additional development was even more pronounced. For example, for an OPC mix and a 30 percent PFA mix of equal 28 day flexural strength, the 30 percent PFA mix had 50 percent more flexural strength than the OPC mix at 90 days.

In terms of the material costs for each mix in the Western Cape, the reduced water demand, equivalent total cements for equivalent 28 day compressive strength and additional long term compressive and flexural strength development of the PFA mixes, make the PFA mixes very attractive. For pavement concrete the economics in terms of the flexural strength is important. At 28 days, for equal flexural strength, the OPC, 15% PFA and 30% PFA mixes are very similarly priced. At 90 days, however, the higer the percentage PFA, the cheaper the mix. The 30% PFA mixes are about 10% cheaper than the OPC mixes for any given flexural strength.

Additional costs would be incurred for the handling and storage of the PFA as an extra material. The Transvaal cost of PFA is about one third that of OPC but transport costs to the Western Cape make it almost as expensive as local OPC. Thus if the materials costing had been done for the Transvaal, the PFA mixes would have been substantially cheaper.

Considerable increases in slump were matched by increases in workability. This was shown by the lower water requirements for PFA mixes than for the OPC mixes. Thus if equivalent water contents had been used for OPC and PFA mixes, the PFA mixes would have had a far higher slump and been more workable.

Due to the slower cementing action of the PFA, the setting time of the PFA mixes was longer than the OPC mixes. The effect of this on pavement concrete would be a delay in the surface finishing and joint sawing.

However, because the setting does not occur as rapidly in the PFA mixes, the timing becomes less critical.

Excessive bleeding in pavement concrete can cause a weak upper surface of low durability. Total bleed volume was reduced in the PFA mixes compared with the OPC mixes. The duration of the bleeding was longer for the PFA mixes than for the OPC mixes. This meant that the bleed rate of the PFA mixes was substantially lower than for the OPC mixes. With the lower bleed volume, longer bleed duration and lower bleed rate, the PFA mixes should have a far more durable upper surface than the OPC mixes.

The wear durability of the PFA concrete showed the typical characteristics of a pozzolanic concrete. At 28 days the OPC concrete was the most durable, but at 90 days the PFA concrete was. The higher the PFA content, the more durable the concrete was against abrasion wear. Since the wearing of the surface is slow and progressive with no sudden failure likely, the 90 day higher resistance of PFA concrete gives a better reflection of the long term durability of the pavement surface. Thus the PFA concretes should have more durable surfaces than the OPC concretes.

The drying shrinkage of the pavement concrete is important as it can cause transverse cracking and drastically reduce both the riding quality of the surface and the life span of the pavement. The drying shrinkage of the PFA concretes and mortars were dramatically lower than for the OPC concretes and mortars. This would mean a lower stress build-up due to restrained shrinkage, with less chance of cracks developing.

In conclusion, the use of Lethabo Field 2 PFA in pavement concrete improves most properties of the concrete. Because of the slower cementing action of the Lethabo Field 2 PFA and the subsequent substantial long term strength development, most of the short term deficiencies of the PFA pavement concrete can be turned to advantages over the long term.

For the mixes developed and the tests carried out, the use of Lethabo Field 2 PFA up to about 30 percent, is of benefit to the concrete for the use in pavements.

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GLOSSARY

OPC -	Ordinary Portland Cement.
OPC concrete -	Ordinary concrete made with no PFA.
PFA -	Pulverised Fuel Ash other wise known as fly ash.
PFA concrete -	Concrete made with both OPC and PFA as cements.
Total Cements -	The sum of the OPC and PFA in the mix.
% PFA -	The percentage of the total cements that the PFA makes up. For OPC concrete the % PFA = 0.
C/W ratio -	Total cements to water ratio.
Water requirement -	Defined by Fulton as the mass of water required to produce one cubic meter of concrete of a desired consistency.
Water demand -	Defined by Fulton as that water requirement that gives a concrete made with 19 mm coarse aggregate a slump 75 mm.
Short term -	For the thesis work this meant 7 days.
Long term -	For the thesis work this meant 90 days or longer.

CHAPTER 1

INTRODUCTION

Pulverised Fuel Ash (PFA), more commonly known as fly ash, is the residue of the finely ground coal ash from the generation of power. The first ever study of coal ashes was reported in the 11th June 1914 edition of Engineering News in the USA(1) The oxide composition was observed to exhibit a striking similarity to that of natural pozzolana. The question was raised as to whether the ash, because of its wide availability and low cost, could be used in the construction of buildings and road pavement foundations.

It was not till the early 1950's that PFA was used in road pavement concrete for the first time. The pavements were generally successful, even using relatively poor quality PFA. Since then, concrete road pavements and the use of PFA in them, have become a substantial part of road systems worldwide.

Pavement quality concrete has to be durable against two sources of loading: vehicle loading and natural forces. Vehicle loading of the concrete requires a surface that is durable against wear by wheels and strong in flexure to resist the load on the slab. Natural forces expose the concrete to freezing and thawing action, thermal expansion and contraction and drying shrinkage.

The combination of excessive load actions cause concrete pavements to fail in two ways. Firstly, deterioration of the surface by scaling or wheel abrasion action gives poor riding quality and low skid resistance. Secondly, the concrete slab can crack and fail because of imposed loads being too heavy or by restrained pavement movements such as temperature change or drying shrinkage. If these build up sufficient stress the pavement can crack and break up.

The use of PFA as a partial cement replacement improves a concrete in ways, among others, that are particularly relevant to concrete pavements. The reduction in water requirement of PFA concretes compared with OPC only concretes, has a beneficial effect on the surface durability by reducing bleeding. The lower water requirement also reduces drying shrinkage. The slower pozzolanic cementing action of PFA gives lower short term strengths but higher long term strengths. For the practical use of a pavement concrete, the cost is crucial. PFA is substantially cheaper than OPC because it is a waste product of energy production. However long distance transportation PFA can raise its price. When it is substantially cheaper, it can reduce the cost of the concrete.

Although much work has been done on PFA concretes in general and relatively little in specific pavement quality concrete problems, each researcher was working with selected mix materials which gave results specific to those materials. In particular when considering PFA, the chemical and physical properties and their effect on the concrete differ widely depending on the coal source, the burning process, the method of ash collection and its grading.

In attempting to predict the characteristic effects of the expected classified Lethabo PFA, the PFA collected from the second electrostatic precipitator at Lethabo power station (called Lethabo Field 2 PFA) was used. The other mix materials were those used commonly in the Western Cape.

The objectives of this investigation were to:

(1) develop a wide range of trial mixes using the Lethabo Field 2 PFA and Western Cape materials, for different percentages of PFA, design strength and sand to stone ratio,

(2) observe and record the effects of the PFA on the fresh and compressive strength properties of each mix,

(3) observe and record the effects of the PFA on pavement quality concrete, with regard to flexural strength, wear resistance and drying shrinkage.

The scope of the work was limited by its relevance to pavement quality concrete and the time available.

The effect of freezing and thawing was not examined because substantial amounts of work have been done solving this problem by air entrainment.

The concrete mixes were developed for 0% PFA (OPC only), 15% PFA, 30% PFA, 50% PFA and 70% PFA. The range of design compressive strengths at 28 days was 10, 20, 30, 40 and 50 MPa. Three sand to stone ratios were selected, resulting in under-, average- and over-sanded mixes.

For each research aspect, the theory, analysis and discussion of results and conclusions are contained in individual chapters, the concluding chapter links the results and the consequences they hold for pavement quality concretes.

References

(1) An investigation of the pozzolanic nature of coal ash. ENGINEERING NEWS RECORD. vol 71, No. 24, 1914.

CHAPTER 2

MATERIALS

The use of PFA in concretes, brings the number of ingredients to five, the other four being the sand, stone, water and OPC. Each ingredient, apart from its concentration in the concrete mix, imparts very specific characteristics to the mix, dependent on its origin, chemical composition and physical appearance.

In considering the range and scope of the research work on pavement concrete to be carried out, the primary goal was to assess the effect of the PFA on the mix. Lethabo Field 2 PFA had been selected at the beginning of the thesis as the pivot around which all the work would revolve. This meant that varying the type of sand or stone would add little useful to the knowledge of PFA concretes, as much is already known of the effect the the different sand and stone on normal OPC concretes. Varying the type of OPC was considered as the PFA has to react chemically with the OPC and thus chemical changes in different cements may produce different results.

Also eliminated from the work was the use of additives to the mix and curing agents. Although these are both in common use in the construction industry, they would also have led to a vast increase in the work required to cover the selected fields of study.

The exponential increase in work required to cover thoroughly the already large number of variables in the mix, was also borne in mind. After evaluating the number of variables, the rate of work possible and the time allocated to the thesis, it was decided to work with one sand, one type and size of stone and one type of cement.

The selection of materials for the mix was based on what was commonly used by the construction industry in the Western Cape. These were the cheapest and most readily available. The materials finally selected were Cape Flats Dune sand as fine aggregate, Malmesbury shale (hornfels) as the coarse aggregate and De Hoek OPC as the cement. The water for local domestic purposes was used.

2.1 Fine aggregate - Cape Flats Dune sand

The Cape Flats dune sand with a relative density of 2.63, bears the very distinctive characteristics of a wind blown and graded dune sand such as the particle shape being well rounded.

The particle size distribution is not according to the ideal grading curves as described by Fulton (1). The transporting power of the wind is relatively low and results in a sand that lack both coarse and very fine constituents. The Cape Flats sand has well over 90% of the sand lying between the 1180 and 150 micron sieves. Sand particles at less than 75 micron assist in the control of bleeding in the fresh concrete but increase water requirements and possible shrinkage. The lack of these fine particles in the Cape Flats sand causes heavy bleeding. The introduction of PFA as fines to the mix improves the grading.

Fig 2.1 below gives a typical grading curve for Cape Flats dune sand with a FM equal to 1.92 and shows the characteristics as described above.

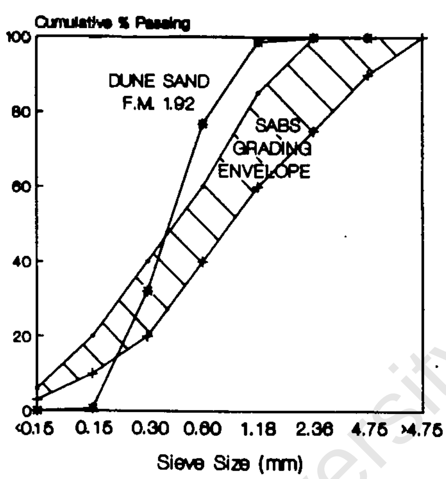

Fig 2.1 Typical grading curve of Cape Flats dune sand

During the course of the work the sand storage bins were flooded and the grading of the sand was changed slightly with the fines being washed through. The effect that this had on the grading was to reduce the Finess Modulus (FM) (FM = Sum of the cumulative percentages retained on the standard sieves sizes except the 75 micron, divided by 100). When the sand was delivered it had a FM equal to 1.92, but after the flood this dropped to 1.74, implying an increase in the fines in the mix. This resulted in the second half of the work being carried out with a water requirement of about 8 l/m3 more than the first half.

2.2 Coarse aggregate - Malmesbury Shale

The Malmesbury shale, which is a hornfels by classification, is a metamorphic transitional rock between the intrusive granite and Malmesbury group shales. This transitional nature results, when crushed, in a flaky, non "chunky" shape with quality ranging from poor to fair. The relative density was 2.68.

The size of the stone used was a nominal 13 mm stone with grading as shown below in Fig 2.2.

Sieve size mm	% Mass retained
26.5	0
19.0	0
13.2	2.2
9.5	69.2
6.7	25.9
4.75	2.1
<4.75	0.6

Fig 2.2 Stone sieve analysis.

2.3 Cement - De Hoek OPC

The selected cement for the thesis was De Hoek OPC. The cement was ordered as 100 sacks from a single production batch to ensure identical cementing properties for the whole of the thesis. The sacks were emptied into large heavy duty plastic bags inside 200 litre drums. When the bags were full they were closed and the lids placed on the drums. There was as little air as possible left in the drums when they were closed and sealed.

The chemical and physical properties are listed in Fig 2.3.

Oxide	Percent
CaO	64.0
SiO2	21.5
Al2O3	3.9
Fe2O3	3.9
MgO	1.1
Na2O	0.22
K2O	0.53
SO3	2.5
L.O.I.	2.3
SUM	99.95%

Alkali content 0.57
Specific surface area 2970 cm2/g
(modified Blaine)
Relative density 3.15
Initial set 199 min
Final set 4 h 15 min

Fig 2.3 Chemical and physical analysis of OPC.

2.4 PFA - Lethabo Field 2

2.4.1 PFA production

The use of PFA as a part cement replacement imparts to a concrete several improvements over the use of straight ordinary portland cement (OPC). These improvements are specific to each PFA and are as a consequence of its chemical, physical and morphological composition.

The chemical composition of the PFA and its physical attributes stem from its method of production. Finely ground coal is injected at high speed into a furnace at the power station. While the coal is still suspended in air, the carbonaceous content of the coal is instantly burnt off. About 20% of the remaining ash falls to the bottom of the furnace and sinters to become coarse bottom ash. The other 80%, consisting mainly of silica, alumina and iron oxide, melts and is then rapidly cooled as it is carried out of the furnace by the flue gases. The ash (now called PFA or fly ash) is extracted from these gases by either mechanical or electrostatic precipitators. Several of these precipitators are used in series to progressively remove finer ash particles from the gases before they are released into the atmosphere.

The PFA selected for use in concrete is usually of the finer portion of the material collected for reasons discussed later.

2.4.2 Physical properties of PFA

The PFA particles, as a result of their chemical composition and rapid heating and cooling have a very characteristic hollow sphere shape . This compares strikingly with the angular shape of OPC particles as shown in Fig 2.4.

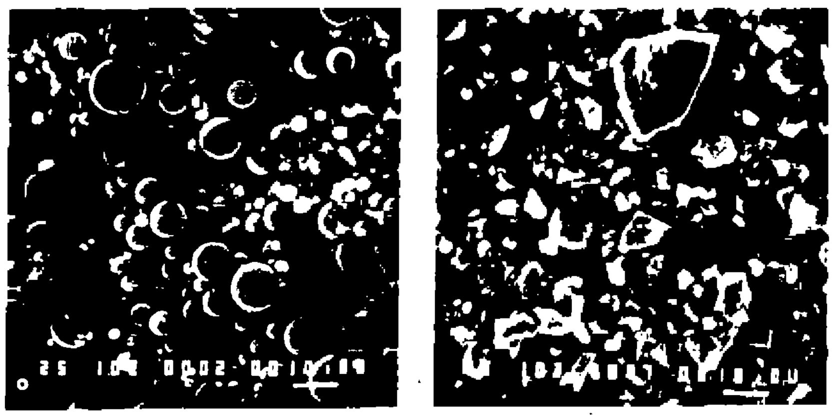

Fig 2.4(a,b) Photomicrographs of PFA and OPC.

This is due the different production methods and has a particular effect on the concrete in the fresh state. The "ball-bearing" shape of the PFA particles has a "lubricating effect" on the mix.

Another form of lubrication of the mix is due to the very fine nature of the PFA particles which can physically and stably disperse the cement flocs, thus freeing more paste to coat and lubricate the aggregates.

The lubricating of the mix makes it more workable. Thus to achieve a desired workability, the inclusion of PFA gives the mix a lower water requirement.

The size of the particles ,usually measured as the percentage retained on a 45 micron sieve, gives a measure of two properties:- firstly, the reactivity of the PFA as this is strongly dependant on how finely the PFA is divided; secondly, an idication of the specific surface area and thus the the amount of water that can be adsorbed onto the surface. For the Cape flats dune sand which lacks in fines, the high specific surface area of PFA makes up for it.

2.4.3 Chemical properties of PFA

Chemically, the PFA is a pozzolan. The ASTM C595 (5) specification gives the definition of a pozzolan as "a siliceous or siliceous-aluminous material which in itself possesses little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties."

According to the ASTM C618,[2] PFA is classified into two main types of PFA dependant on its pozzolanic properties:

(a) Low lime PFA (Class F) with CaO content at less than 10%. This possesses true pozzolanic properties, ie. it needs an activator to produce cementitious properties.

(b) High lime PFA (Class C) with CaO content greater than 10%. This possesses some cementitious properties itself in addition to the pozzolanic properties.

A true pozzolan PFA (containing less than 10% CaO) is typically made up of about 50% silica oxide, 30% aluminium oxide and 10% iron oxide with the remaining 10% made up by a range of minerals.

The full chemistry of the hydration reaction of OPC and the PFA pozzolana reactions are highly complex. In essence, the calcium hydroxide or free lime, required by the PFA is liberated in the OPC hydration reaction. Thus the PFA uses water and a byproduct of the OPC hydration to hydrate.

Since the PFA requires the free lime from the OPC hydration, it is dependent on the reaction of the OPC hydration. The pozzolanic hydration is a delayed reaction often resulting in low early (7 days) strength compared to an OPC mix for both mixes having the same 28 day strength. The pozzolanic reaction then allows the PFA concrete to gain additional strength over the equivalent OPC mix after 28 days, giving higher final strengths.

The chemical reactivity of the PFA is essentially a function of the amount of glassy phase material present. This is controlled by the nature of the source coal and the operating temperature of the furnace. Another good measure of the reactivity is the fineness of the PFA.

2.4.4 PFA standards

In South Africa there is no specification for PFA but the suppliers of the PFA ensure that it complies with ASTM C618 (2) with respect to the composition and fineness. The Matla PFA has an Agrement Certificate (3) and the classifier plant has been awarded a SABS 0157 quality assurance certificate.

As a summary of the specifications for PFA from elsewhere in the world the table below gives the details for Australia, United Kingdom and U.S.A. (Class F).

	Australia	U.K.	U.S.A. Class F
Max Loss on ignition %	8.0	7.0	12.0
SO_3 max %	2.5	2.5	5.0
MgO max %	-	4.0	5.0
Available Alkali as Na_2O max %	-	-	1.5
$SiO_2+Al_2O_3+Fe_2O_3$ min%	-	-	70
Retained on 45 micron sieve max %	50	12.5	34

Fig 2.5 PFA specifications from Australia, U.K. and U.S.A.

The significance of each of the specifications is as follows (4):

(a) Loss on ignition is a measure of the unburnt carbon left in the PFA and affects the colour. High percentages can increase water demand due to the porosity of the carbon.

(b) SO_3 and MgO contents are set at a maximum when considering concrete durability, as high SO_3 and MgO contents can lead to volume instability and loss of durability.

(c) Alkali content, taken as Na_2O and K_2O may react with some aggregates (Malmesbury Shale is one of them) producing an expansive gel. The reaction is known as the alkali silica reaction and results in the deterioration of the concrete.

(d) The sum of the major oxides (SiO_2 + Al_2O_3 + Fe_2O_3) is supposed to ensure that sufficient amounts of the reactive glassy constituents are present in the PFA. However, there is certain doubt, based on the relatively complex chemistry, that this is a valid criterion for PFA selection for concrete use.

(e) Fineness, as measured by the percentage retained on the 45 micron sieve, broadly controls the water requirement and pozzolanic activity of the PFA. This measure is considered to be the most important factor when determining the quality of the PFA.

2.4.5 Lethabo Field 2 PFA

The PFA selected for the thesis was that from Lethabo power station collected from the second electrostatic precipitator field, thus called "Lethabo Field 2" PFA. This PFA was selected as it was the PFA closest resembling the future classified and commercially available Lethabo PFA. The classifier plant to be installed at the Lethabo power station will only come on stream at the end of 1989 or in 1990. Thus work on the characteristics of the future classified PFA could only be done on the nearest semblance from a single electostatic precipitator.

In assessing the potential of the classified Lethabo PFA from the Field 2 PFA used, an examination of the relavant chemical properties and physical characteristics and their effect on a mix was required. A comparison with the current standards applied to PFAs elsewhere was also needed.

The chemical and physical analysis data for the Lethabo Field 2 PFA (used during the thesis) and commercially available classified Matla PFA are shown in Fig 2.6.

	Lethabo Field 2 %	Matla classified %
SiO_2	50.93	47.2
Al_2O_3	35.97	26.3
Fe_2O_3	3.91	4.9
TiO_3	1.80	1.5
SO_3		0.6
MnO	0.05	
MgO	1.36	3.1
CaO	4.66	12.8
Na_2O	0.32	0.6
K_2O	0.53	
P_2O_5	0.66	
Loss on ignition	0.40	1.2
Fineness - % retained on 45 micron	7.7	12.5
Relative density	2.35	

Fig 2.6 Chemical and physical analysis for Lethabo Field 2 PFA and Matla classified PFA.

Comparison of Lethabo Field two PFA with the standard specifications laid down for PFA show that it is classified as a true pozzolana with less than 10% CaO (Class F). Compared to the specification as set out in Fig 2.5, both are well within the specified limits.

Based primarily on the percentage retained on the 45 micron sieve, the Lethabo Field 2 PFA can be considered to be of a far higher quality that the commercially available Matla classified PFA. This means that the Lethabo Field 2 PFA is expected to have a higher reactivity and better cementing action than the Matla PFA.

References

(1) FULTON'S CONCRETE TECHNOLOGY. 6th rev.ed. Midrand, Porland Cement Institute, 1986.

(2) ASTM C618 - 80 Specifications for fly ash and raw or calcined natural pozzalan for the use as a mineral admixture in Portland cement concrete, American Society for Testing Materials, Philadelphia.

(3) AGREMENT BOARD OF SOUTH AFRICA. Matla PFA. Fly ash for the use in concrete. Pretoria, the Board, 1987. Certificate No. 87/160.

(4) Dhir, D.K. Pulverised Fuel Ash. CEMENT REPLACEMENT MATERIALS. ed Swamy, R.N. Concrete technology and design Vol 3. Surrey University Press 1986.

(5) ASTM C595 - 80. Standard specifications for blended cements. AMERICAN SOCIETY FOR TESTING AND MATERIALS. Philadelphia, 1980.

Chapter 3

CONCRETE MIX DESIGN, MIXES AND THEIR PROPERTIES

Mix design methods for OPC concrete vary quite widely in themselves, quite apart from the several alternative methods of incorporating PFA into the mix. For the purpose of the thesis a method was sought that was used in practice and also allowed for the use of PFA, providing easy comparison between the OPC mixes and PFA mixes.

3.1 Design of OPC concrete

The most well used method of mix design for OPC concretes in South Africa, as detailed by Fulton (1) and SABS 0100, is based on ACI Standard 211.1-81(6) It is commonly refered to as the PCI method. The mix proportions are expressed in terms of quantities of materials per unit volume of concrete rather than mere proportions by mass.

3.2 Methods of design of PFA concrete

Gopalan and Haque (2) set out the three basic methods of incorporating PFA into the mix design of OPC concrete and their conclusions drawn from each method. They thus give PFA mixes that are modifications of the OPC control mixes. These methods and conclusions are :

(a) Partial replacement of OPC. This is done on an equal volume or equal weight basis and generally leads to a lower strength.

(b) Partial replacement of both OPC and fine aggregate. This method is used to design PFA concrete of the same strength as that of OPC concrete and therefore has wider application.

(c) Addition of PFA as a fine aggregate. This method results in a small increase in compressive strength of concretes at early ages and a greater increase in strength at later ages. However, it is not usually economical to use PFA as a replacement of fine aggregate.

In selecting an appropriate method for the work to be carried out, the method (b) was employed as fully described in the D&H Ash Resourses PFA Technical Information Bulletin No.5 1985.(3) This method of design incorporated the PCI method for OPC concretes with the following modifications and fundamental differences applying to the PFA mixes:

(a) the reduction in water demand (see 2.4).

(b) the lower cementing efficiency of the PFA, which is partially offset by the lower water demand.

(c) the use of a series of curves for the replacement levels of PFA to determine the required ratios of cementitious material (total cements) to water to achieve the target strength.

(d) the increase in the stone content that is possible because of the improved workability.

Allowing for Western Cape materials, this method was followed and is explained hereafter.

3.3 D&H Ash Resourses's design of PFA concrete mixes

The D&H design method described below, is an abridged version of their PFA Technical Information Bulletin (3) and gives the steps required for the design of an OPC mix or a PFA mix.

Of particular importance is that the given design curves and water reduction values are based on Transvaal materials (OPC, sand and stone) and Matla classified PFA. Differences were thus expected between predicted values and actual values obtained for Western Cape materials and Lethabo Field 2 PFA.

The steps required are:

(a) Selection of a percentage of PFA to include in the mix based on the effect that the percentage will have on the mix. For OPC control mixes the replacement level is of course 0%.

(b) choosing a design slump of concrete, ie. consistency,

(c) selection from a table of the mass of stone required for 1m3 concrete making allowances for Fineness Modulus of sand, nominal size of stone, method of compaction and slump. Additional stone must be added to a PFA mix over and above that required for an OPC mix according to a linear relationship as shown in Fig 3.1. The stone content of the equivalent OPC mix is called the "OPC stone content".

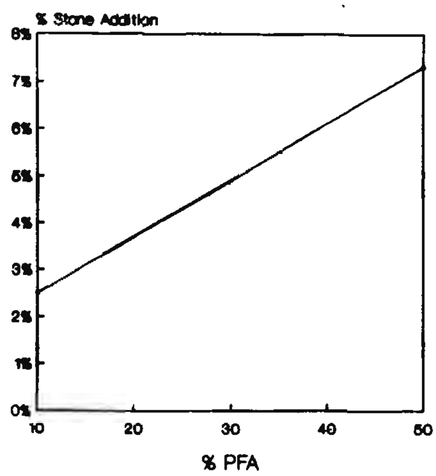

Fig 3.1 Additional stone for PFA mixes.

(d) deciding on a 28 day target strength,

(e) determining the total cement to water ratio from the design curves shown in Fig 4.1, depending on the selected PFA replacement level,

(f) Estimating the water requirement for 1m3 of concrete. This can be done either by taking the reduction of water demand compared with the OPC equivalent mix or by using data from previous mixes. Correction to this water demand may have to be made for a change of slump,

(g) calculating the amount of total cements from the total cement to water ratio and the water requirement. The amounts of OPC and PFA are then determined,

(h) calculating the volume of sand from the total volume of the stone, OPC, PFA and water, subtracted from 1m3,

(i) scaling the mix masses to yield a mix of a given volume.

3.4 Determining water requirement for mix design

Methods of determining water requirement for a specific consistency can be long and involved as detailed in Fulton (1) and Popovics Vol 1 (4), requiring basic information to establish the various coefficients for the many equations.

A consistent variable to measure water requirement against was the sand content of the mix. Fig 3.2 shows the plot used very successfully to empirically design mixes to the correct water requirement.

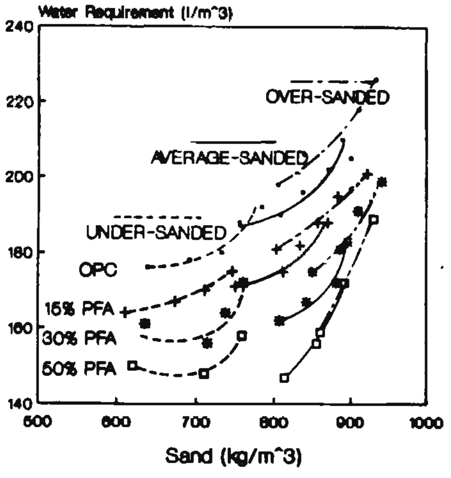

Fig 3.2 Water requirement per m3 related to sand content of mix.

The technique was make an esstimate of the water requirement. The mix was then designed and sand predicted by the plot was compared to the amount of sand calculated. If the calculated sand did not match the predicted value, the water requirement was appropriately modified and the mix design recalculated. This method quickly arrived at the correct water requirement.

3.5 Effect of additional stone for PFA mixes

It must be noted that the addition of extra stone to the PFA mixes over that for the comparable OPC control mixes, as mentioned in 3.2(d) and 3.3(c), caused a problem in comparisons of the PFA to OPC mixes where certain properties such as mortar excess and modulus of elasticity are closely dependent on the stone content in a mix.

3.6 Method of mix development

The selection of the range of mixes was a very early decision in the thesis period. At first, four potential variables were identified as options that could widely affect the properties of a mix, both as fresh concrete and over a longer period as hardened concrete. These variables were strength (total cement to water ratio), percentage PFA, sand to stone ratio and slump. The time required to obtain a reasonable amount of information on all four variables was well beyond the scope of the thesis as with each variable the number of mixes required, increased exponentially.

It was decided to limit the variables to strength, percentage PFA replacement and sand to stone ratio keeping to one design slump.

3.7 Range of mixes developed

The design slump was set at 50 mm for "low slump concrete" typical of road pavement concrete based on information on American pavement design (5).

The three chosen mix variables allowed 11 "series" of mixes to be developed. Each series had variable strength and PFA replacements and a specific OPC stone content to characterize it.

Each mix series has one of three distinct OPC stone contents (950, 1050 or 1220 kg/m3). This results in each mix series being classified as over-, average- or under-sanded respectively. Within a mix series of one OPC stone content, a narrow range of sand to stone ratios exist. Thus, an average can be calculated to represent the series.

A list of all 11 series and the mixes that were mixed, their strengths and PFA percentages, OPC stone content and classification and the tests carried out on them follows: (Full mix proportions detailed in appendix 2)

Series 1,2,3,4	Trial mixes across all three sand stone ratios for the purpose of establishing correct water demands (for 50 mm slump) across the ranges described in series 5, 6, 7 below. Trials were also carried out to set up and establish methods and limits for fresh concrete tests (vebe, mortar excess, bleeding, early set)

Series 5 OPC stone content 1050 kg/m3 = AVERAGE-SANDED

Strength (MPa)		10	20	30	40	50
	OPC	X	X	X	X	X
	15%		X	X	X	X
%PFA	30%	X	X	X	X	
	50%	X	X	X		
	70%		X	X		

Tests: Slump, Vebe, mortar excess, bleeding early set, 7 Day Comp., 28 Day Comp., 90 Day Comp., 28 Day Flex., 28 Day Wire.

Series 6 OPC stone content 1220 kg/m3 = UNDER-SANDED

Strength (MPa)		10	20	30	40	50
	OPC	X	X	X	X	X
	15%		X	X	X	X
%PFA	30%	X	X	X	X	
	50%	X	X	X		
	70%		X	X		

Tests: Slump, Vebe, mortar excess, bleeding early set, 7 Day Comp., 28 Day Comp., 90 Day Comp., 28 Day Flex., 28 Day Wire, 90 Day Wire.

Series 7 OPC stone content 950 kg/m3 = OVER-SANDED

Strength (MPa)		10	20	30	40	50
	OPC	X	X	X	X	X
	15%		X	X	X	X
%PFA	30%	X	X	X	X	
	50%	X	X	X		
	70%		X	X		

Tests: Slump, Vebe, mortar excess, bleeding early set, 7 Day Comp., 28 Day Comp., 90 Day Comp., 28 Day Flex., 28 Day Wire, 90 Day wire.

Series 8 OPC stone content 1050 kg/m3 = AVERAGE-SANDED

Strength (MPa)		10	20	30	40	50
	OPC		X		X	
	15%		X		X	
%PFA	30%		X		X	
	50%		X	X		

Tests: Slump, 7 Day Comp., 28 Day Comp., 90 Day Comp., 90 Day Flex., 28 Day E, 90 Day E, Creep.

Series 9 OPC stone content 1220 kg/m3 = UNDER-SANDED

Strength (MPa)		10	20	30	40	50
	OPC		X		X	
	15%		X		X	
%PFA	30%		X		X	
	50%		X	X		

Tests: Slump, 7 Day Comp., 28 Day Comp., 90 Day Comp., 90 Day Flex., 28 Day E, 90 Day E, Creep. 28 Day Sand, 28 Day Ball.

Series 10 OPC stone content 950 kg/m3 = OVER-SANDED

Strength (MPa)		10	20	30	40	50
	OPC		X		X	
	15%		X		X	
%PFA	30%		X		X	
	50%		X	X		

Tests: Slump, 7 Day Comp., 28 Day Comp., 90 Day Comp., 90 Day Flex., 28 Day E, 90 Day E, Creep. 28 Day Sand, 28 Day Ball.

Series 11 OPC stone content 1050 kg/m3 = AVERAGE-SANDED

Strength (MPa)		20	35	50
	OPC	X	X	X
%PFA	30%	X	X	X
	50%	X	X	X

Tests: Slump, 28 Day Comp., 90 Day Comp., Shrinkage, 28 Day Mortar Sand, 28 Day Mortar Ball, 28 Day Mortar Wire.

Abbreviations : Key to Tests

X - marks mixes made in each series.

Slump - Slump test. Design slump = 50 mm. (SABS 862)

Vebe - Vebe workability test (BS1881 Part 2).

Bleeding - See Chapter 7. (ASTM C232)

Early Set - See Chapter 7. (ASTM C403)

7, 28, 90 Day Comp. - Compression tests at 7 or 28 or 90 Days after mixing. (SABS 863) See Chapter 4.

Flex. - Flexural strength at 28 or 90 days. (SABS 864) See Chapter 5.

Wire - Wire brush abrasion. PCI method. See Chapter 8.

Sand - Sand blast abrasion resistance. (ASTM C418-81) See Chapter 8.

Ball - Ball bearing race wear resistance. (MA20 - 1986) See Chapter 8.

E - Modulus of elasticity at 28 or 90 days. See Chapter 10.

Shrinkage - Drying shrinkage. (SABS 1085) See Chapter 9.

3.8 Individual mix proportioning

The quantities required for each mix were calculated using a spread sheet programme. The required input for the calculations were:

(a) Percentage PFA in mix

(b) Total cements to water ratio

(c) Water requirement (l/m3)

(d) Stone requirement for OPC equuvalent mix (OPC stone content) (kg/m3)

(e) Actual volume of mix in litres ("pan" on spread sheet).

A typical mix example is show in Fig 3.2. The mix is a 15% PFA concrete of design target strength of 20 MPa. The total cements to water ratio (C/W) = 1.23 with a water requirement of 193 l/m3, OPC stone of 1050 kg/m3 (average-sanded) and pan volume of 20 l.

28 Day Target Strength=	20	MPa
%PFA =	15	
C/W =	1.23	
W/C =	0.81	
WaterDem=	193	l/m3
Total Cements=	237	kg/m3
FM Sand =	2.0	
Type of Sand =	Dune	
Stone Size =	13.0	mm
Stone Type =	Malmb.Shale	
OPCstone=	1050	kg/m3
Pan =	20	l

MIX NAME : DATE:

	R.D.	MASS kg	VOL abs ltr	PAN MIX kg
Water	1.00	193	193	3.86
OPC	3.15	202	64	4.04
PFA	2.35	36	15	0.71
STONE	2.68	1083	404	21.67
SAND	2.63	851	324	17.02
TOTALS		2365	1000	47.29

Fig 3.3 Typical mix design spread sheet calculations.

3.9 Mixing procedure

All materials were weighed according to the design. The cements (PFA and OPC) and water were weighed using a digital scale to 5 g accuracy (which at worst was an acuracy of 0.3% on any mix). The sand and stone were weighed on a dial scale that could easily be read to the nearest 100 g. This gave an accuracy of measurement to about 0.7%. This meant the the quantities of materials were very well controlled.

The mixing was done in a 55 l capacity pan mixer. The mixing took at least 2 min as specified by SABS 863 with the water being added slowly from the begining of the mixing till the mix was considered by looking to be at a 50 mm slump. This could either have required the holding back or addition of a small known volume of water from or to the mix. A slump test according to SABS 862 was then carried out. The margin for acceptance of the mix was a slump within the range 35 - 65 mm. If the mix required more water, ie. it had a low slump, and there was still water retained from the mix, this was added and a second slump test was carried out.

The slump being accepted, the mix was then transferred to the moulds required for the appropriate tests to be carried out.

3.10 Acceptance / rejection of mix - Quality control

The quality control system for the mixes was based on the change in volume of water required to bring the mix to exactly 50 mm slump. From the mixing procedure described in 3.9, there were two inputs that made up this volume of water.

The first input was because of the acceptance of slump of the mix if it lay within the range of 35 to 65 mm. The PCI method of concrete design gives a chart to determine the volume of water required to change a concrete of one slump to another slump. Numerically it gave two litres of water added per cubic meter of concrete for each 10 mm increase in the slump. The proportions were the same for reducing the slump. From this a theoretical change of water requirement to give exactly 50 mm slump was calculated.

The second input was the known amount of water held back from, or put in additionally to the mix to achieve an acceptable slump. This was scaled for an equivalent 1m3 of concrete.

The theoretical volume to change the slump, combined with the known amount of water withheld or added, gave a total water "correction volume". This correction volume could have been positive or negative. If the absolute correction volume was greater than 10 l/m3, the mix was rejected entirely. The mix was redesigned according to a modified water requirement and remixed with fresh materials.

If the mix was redesigned using the full correction volume to modify the water requirement, the new mix was found to be over corrected.

The explanation for this over-correction was the effect that the redesigned mix proportions, particularly sand, had on the water requirement of the new mix. For example, if a higher water requirement was needed for correction, the new mix would have more total cements and thus less sand. Since the quantity of sand determines the water requirement, by reducing the amount of sand, a smaller modification to the water requirement was actually needed.

It was found by trial and error that a modification of the water requirement by 60% of the correction volume gave very close to 50 mm slump with no water added or held back.

3.11 Actual total cements and water requirement

The design of each mix was based on the design 28 day target compressive strength. The strength is a direct consequence of the total cements to water ratio. This meant an interdependence of total cements and water requirement. They are thus discussed in conjunction.

The total cements for the range of mixes are shown in Fig 3.4 and the water requirements for the same range are shown in Fig 3.5.

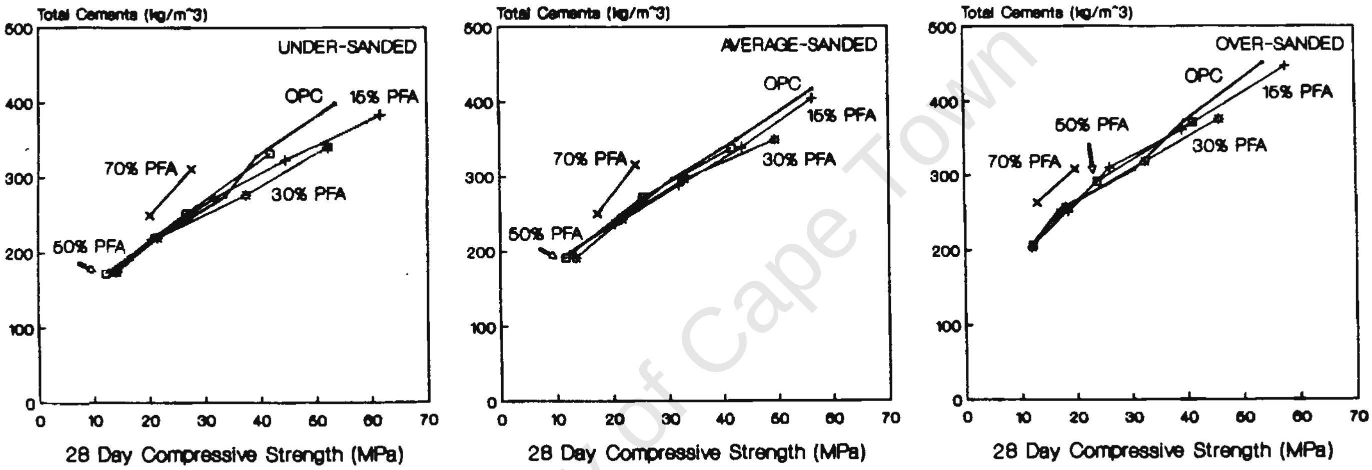

Fig 3.4(a,b,c) Total cements vs 28 day compressive strengths. Under-, Average- and Over-sanded.

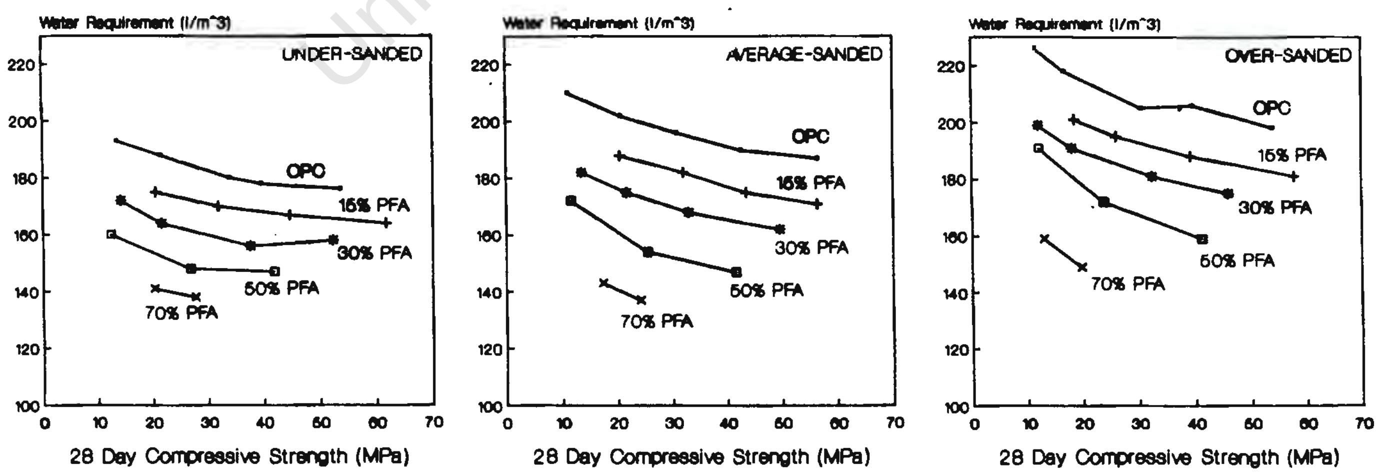

Fig 3.5(a,b,c) Water requirement related to 28 day compressive strength. Under-, average- and over-sanded.

In discussing and explaining the trends shown in Figs 3.4 and 3.5, three comparisons can be drawn. These are (a) the general trends exibited in moving from the under-sanded mixes to the over-sanded mixes; (b) the effect on total cements and water requirement that changing strength has at any particular percentage PFA level; and (c) the effect the percentage of PFA has on each sand to stone ratio.

3.11.1 Comparison between three sand to stone ratios

An increase in both the total cements and water requirement is shown when moving from the under-sanded mixes, through the average-sanded mixes to the over-sanded mixes.

This is explained by the corresponding reduction in stone content of the mixes and increasing amount of sand required to take up the volume of the reducing stone. Since the water requirement is controlled by the sand content, an increase in sand implies an increase in water requirement. To keep the total cements to water ratio constant, an increase in the total cements content parallels the water requirement trend.

3.11.2 Comparison across strength for any PFA level

With an increase in strength at any particular percentage PFA there is an increase in total cements and a decrease in the water requirement.

The explanation for this change is: with an increase in strength there has to be an increase in total cements to water ratio (C/W); for an increase in the C/W ratio there must be an increase in the total cements (water remains nearly constant as it is controlled by the sand content); but with an increase in the total cements content there must be a reduction in sand content to keep the mix volume a cubic metre; with a reduction of sand the water requirement decreases accordingly; with a reduction in water requirement the increase of total cements is limited not to affect the desired C/W ratio.

As an example of the logic, typical magnitudes of change along a single replacement level are given in Fig 3.6 for the change between 20 and 40 MPa OPC in the average-sanded series.

	20MPa	40MPa
Stone (kg/m3)	1050	1050
C/W Ratio	1.25	1.95 (+56%)
Total cements (kg/m3)	239	348 (+45%)
Sand (kg/m3)	864	812 (-6%)
Water requirement (l/m3)	202 0%	190 -6%

Fig 3.6 Table of material changes across two strength values

The total cement to water increases with strength; the cements increase, but not to the same proportions as the C/W due to the reduced sand and water contents.

3.11.3 Comparison at different PFA levels

As described in section 3.2 , the out-dated method of including PFA in the mix was a direct mass for mass replacement of OPC with PFA. Because of the weaker cementing ability of PFA, an alternative approach, where more PFA was substituted than OPC removed, is used in the D&H Ash Resources design method. This factor is incorporated in the strength design chart (Fig 4.1).

However, due to the water requirement reduction that the Lethabo Field 2 PFA gives (Fig 3.5) and the consequential reduction in total cements for a constant total cements to water ratio, <u>the actual total cements required for a given 28 day strength were equal or less than the OPC control across all PFA replacement levels, except at 70%</u> (Fig 3.4).

In examining only one of the sand to stone ratios (ie. one graph) from Fig 3.4, say the average-sanded series, it can be seen that only the 70% replacement requires approximately 15% more total cements to reach a desired 28 day compressive strength. For the other replacement levels (15%, 30% and 50%) at strengths up to 30MPa total cements are equal to those of the OPC control mix and above 30MPa up to 15% less total cements are required. The replacement level of 30% PFA gives the lowest total cement requirement for all 28 day strengths.

This result of having the equivalent (or lower) total cements in the PFA mixes than in the control OPC mixes (where originally more total cements were expected) is brought about by the significant reduction of water requirement that the Lethabo Field 2 PFA allows. This is due to the high quality and comparative fineness of the Lethabo Field 2 PFA as described in 2.4.

As a summary of the effect of PFA on the water requirement of a mix, Fig 3.7 shows the average reduction related to PFA replacement level across all the strengths on the average-, under- and over-sanded mixes.

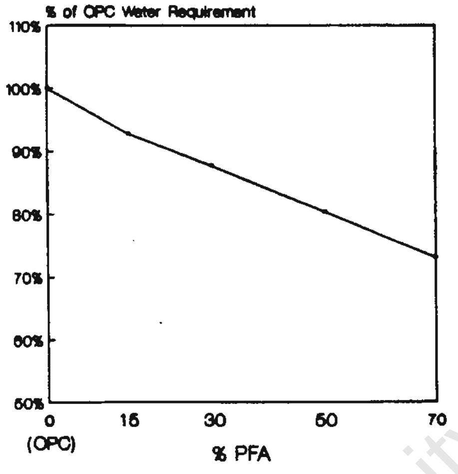

Fig 3.7 Average water requirement vs % PFA.

3.12 Conclusions

The following conclusions relating to water requirement, effect of the quantity of sand and total cements can be drawn:

(1) Water requirement for the mixes is dependent on the sand content of the mix (Fig 3.3), the % PFA and the strength of the mix.

(2) Increasing PFA replacement levels decreases the water requirement of the mix. (Fig 3.5, Fig 3.6)

(3) Due to the lower water requirement of the PFA mixes the total cements (OPC plus PFA) required for a given 28 day strength is equal to or less than the OPC control mixes at all replacement levels. (Fig 3.4)

CHAPTER 4

CONCRETE COMPRESSIVE STRENGTH DEVELOPMENT

In specifying the design strength for concrete it is common practice that the 28 day compressive strength for the concrete is given. It is this criterion and often very little else which is used to describe a concrete. Little consideration is given of either the strength prior to, or after the 28 days or the other physical properties that develop with time.

PFA concrete, due to the pozzolanic reaction with the OPC, gains subtantial compressive strength after 28 days at a greater rate, and for a longer period than OPC concrete. Making allowances for the purpose for which the concrete is to be used and the time till the concrete will be required to take its full design load, specification of an appropriate time related strength should lead to a more efficient use of the concrete's properties and make for a more economical construction.

4.1 Method of testing compressive strength

The determination of compressive strength for the thesis was done according to SABS 863.(3) The cubes that were crushed, were 100 mm which had been vibrated on a Vebe table, stored in a humid environment of at least 90% relative humidity for 24 hours then stripped and stored under water at between 22 and 25 degrees celsius till crushed.

The crushing was done in an Amsler hydraulic press at a load application of 15 MPa per minute till failure.

For each determination of compressive strength, three cubes from the same batch were crushed and the average of the three strengths was taken as the representative compressive strength.

There were three ages at which compressive strength was determined. These were 7 days, 28 days and 90 days after mixing. The crushing at 7 days was referred to as short term compressive strength, while that at 90 days was referred to as long term compressive strength.

4.2 Accuracy of cube strength testing

During the thesis two large batches of cubes were made for the exclusive purpose of obtaining statistical data on the accuracy of the crushing. They were typical mixes from the middle of the mix range. The mixes and statistical results were for:

(a) a 35 MPa design strength OPC mix with no PFA; The mix classified as being average-sanded.
Number of samples = 20
Average strength = 34.7 MPa
Standard Deviation = 1.1 MPa
Coefficient of variation = 3.28%

(b) a 35 MPa design strength mix with 30% Lethabo Field 2 PFA replacement, also an average-sanded mix.
Number of samples = 20
Average strength = 34.1 MPa
Standard Deviation = 1.3 MPa
Coefficient of variation = 3.81%

Determination of the level of control exercised is by the standard deviation of the results. For laboratory testing the top level of control is 'excellent' and limited by a maximum standard deviation of 1.4.(1) Both mixes are thus classified as having excellent control.

4.3 Comparison of design strength and actual strength

As discussed in 3.3, the method of design was as set out by D & H Ash Resources. As the basis for their design, they developed design curves relating the total cements to water ratio (C/W) to the desired 28 day compressive strength. These curves were developed for Transvaal aggregates and cements with classified Matla PFA. Their design chart is duplicated in Fig 4.1.

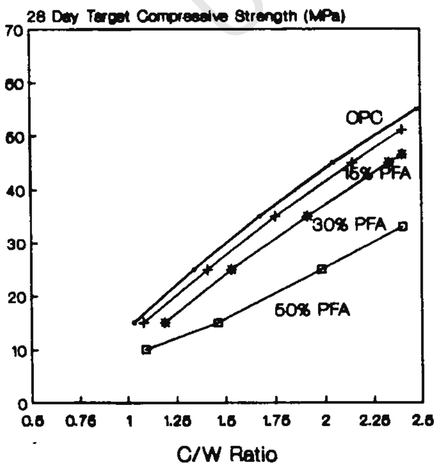

Fig 4.1 Design curves for Matla PFA and Transvaal materials (taken from PFA Technical Information Bulletin No.5 Nov 1985 - D&H Ash Resources)

The reason for the increased total cement to water ratio required for the PFA mixes is due to the lower cementing efficiency of the PFA compared with the OPC as discussed in 3.2.

Due to the work of the thesis being carried out on Western Cape aggregates and cements using Lethabo Field 2 PFA, differences between the design and the actual 28 day compressive strengths obtained were to be expected. In order to develop design curves specific to the thesis materials, design curves for each of the three sand to stone ratios were drawn up initially. These were then averaged as they were very similar, to give the modified design curves specific for the thesis work as shown in Fig 4.2.

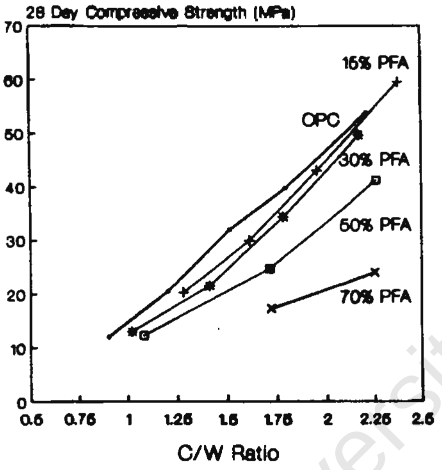

Fig 4.2 Average actual strength curves for Lethabo Field 2 PFA and Western cape materials.

In comparing Figs 4.1 and 4.2 the initial comparisons must be of the OPC control mixes. At strengths up to 35 MPa the results were identical. At higher strengths the local Western Cape materials showed marginal gains over those predicted for Transvaal materials. In short, the results were very similar for the controls and thus formed a good basis for comparison of the PFA mixes.

The comparison between the two sets of PFA mixes show marked differences, with the Lethabo Field 2 PFA performing better than the Matla PFA at any given total cements to water ratio. The Lethabo 15% PFA level has trends similar to the OPC control. At strengths lower than 35 MPa the performance is the same but at higher strengths the Lethabo Field 2 PFA develops a higher strength for a given total cements to water ratio. The 30% Lethabo Field 2 PFA level has almost equivalent strengths developed to the 15% Matla PFA level. The 50% Lethabo Field 2 PFA has strength characteristics similar to those for the 30% Matla PFA and likwise, the 70% Lethabo PFA coincides with the 50% Matla PFA level.

Another way of looking at the improvements that the Lethabo PFA gives, is to examine the two PFA's strength performances at specific total cement to water ratios as set out in Fig 4.3 below.

28 DAY STRENGTHS (MPa)

	C/W = 1.5		C/W = 2.0		
	Lethabo	Matla	Lethabo	Matla	
OPC	31.7 (100%)	31.1 (100%)	46.8 (100%)	44.6 (100%)	MPa
15% PFA	26.9 (84%)	29.0 (93%)	45.2 (97%)	41.9 (94%)	MPa
30% PFA	24.7 (78%)	25.3 (81%)	43.0 (92%)	37.6 (84%)	MPa
50% PFA	21.0 (66%)	16.1 (52%)	33.9 (72%)	25.8 (58%)	MPa
70% PFA			21.0 (45%)	MPa	

Fig 4.3 Table of relative 28 day strength performances for percentages PFA at specific total cements to water ratios.

The trends are as discussed above. The OPC control on both sets of mixes have similar strengths. The Lethabo PFA concrete, for a given percentage PFA, has a far higher strength than the Matla PFA concrete for higher C/W ratios

4.4 Strength comparison for the three sand to stone ratios

The graph shown as Fig 4.2, is the average of the three individual actual strength curves for each sand to stone ratio. These individual actual strength curves are shown in Fig 4.4.

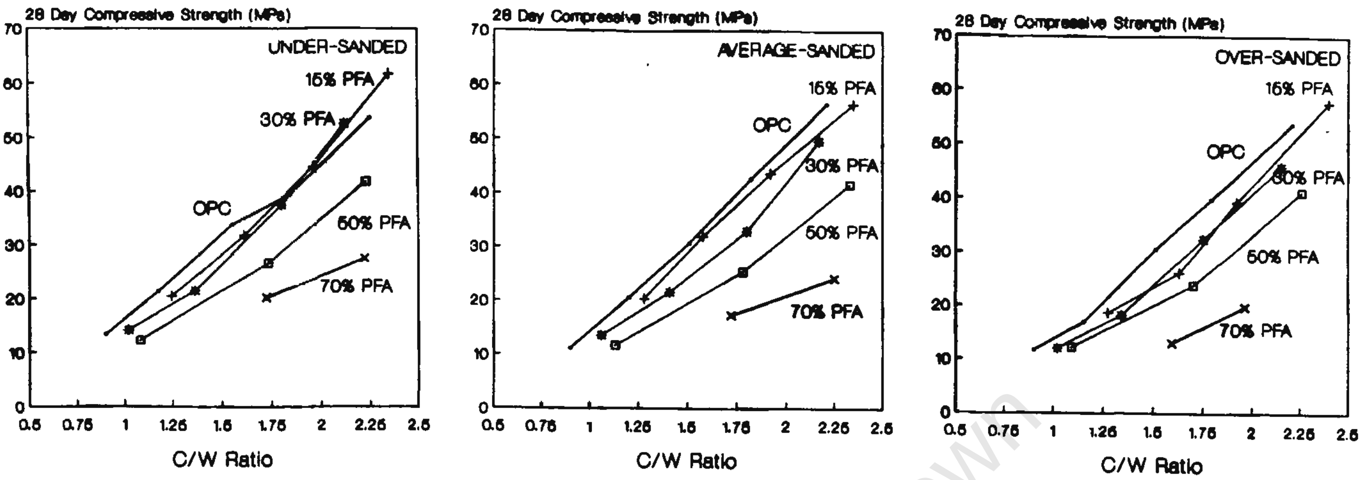

Fig 4.4(a,b,c) Actual strength curves for individual sand to stone ratios using Lethabo Field 2 PFA.

The variation between these graphs is very small giving a maximum strength variation for a given total cements to water ratio, of 0 to 3 MPa.

The under-sanded mixes give throughout the greatest strength for any given total cements to water ratio. At all PFA levels except 50%, the over-sanded mixes give the weakest strength for any given total cements to water ratio.

This trend of the under-sanded mixes being strongest, the average-sanded in the middle and the over-sanded mixes being weakest, is normal despite the fact that the under-sanded mixes contain the least total cements and the over-sanded mixes the most (See Fig 3.5 (a,b,c) and discussion at 3.10).

An explanation for this trend can be found in work quoted in Fulton (1) carried out by Erntroy and Shacklock (2). They found that for a given cement to water ratio, the compressive strength is influenced by the aggregate to cement ratio. For a given cement to water ratio, the compressive strength increased with increasing aggregate to cement ratio.

For the under-sanded mix the cement content was the lowest of the three sand to stone ratios thus the aggregate to cement ratio was at its highest. This would then give a corresponding increase in the compressive strength. Likewise for the over-sanded mix where total cements were at their highest and thus the aggregate to cement ratio is at its lowest, a decrease in the compressive strength is expected.

When the actual strength curves are drawn for the 7 day and 90 day compressive strengths, as for the 28 day curves, the under-sanded mixes are the strongest and the over-sanded mixes are the weakest. The range of variation across the three sand to stone ratios is 0 to 2 MPa at 7 days and 0 to 4 MPa at 90 days. Figs 4.5 and 4.6 give the respective actual strength curves for the average-sanded mixes for 7 and 90 days.

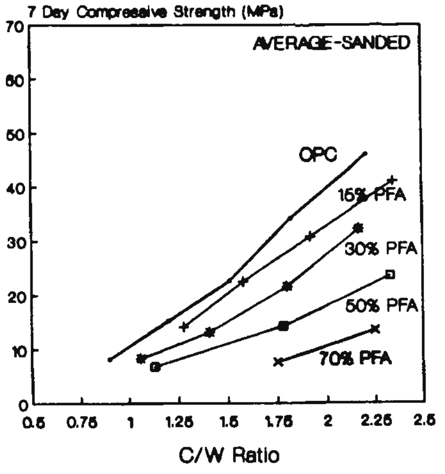

Fig 4.5 Actual strength curves for average-sanded mixes at 7 days.

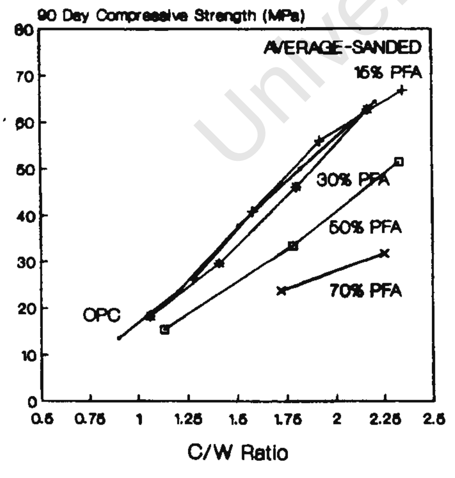

Fig 4.6 Actual strength curves for average-sanded mixes at 90 days.

4.5 Short and long term strength development

For conventional strength vs time graphs the comparative ratios of 7 to 28 day strengths and 90 to 28 day strengths must be evaluated and plotted.

Due to the relatively linear and proportional trends that the design curves exhibit in Figs 4.4, 4.5 and 4.6, across a given PFA level there is only a small fluctuation in the value of the comparative ratios for 7 to 28 day strengths. The same is true for 90 to 28 day comparative ratios. Only the average-sanded mixes are shown in Figs 4.7 and 4.8.

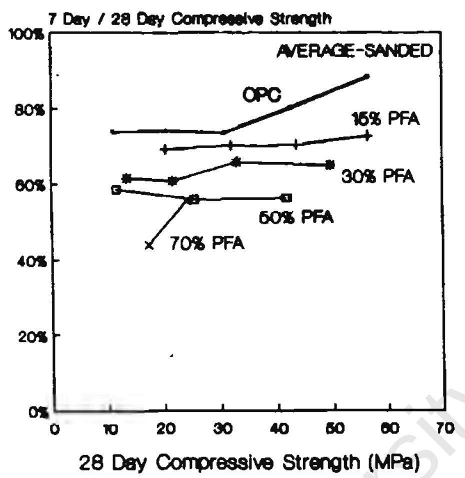

Fig 4.7 Short term (7 days) strength compared with 28 day strength for average-sanded mixes.

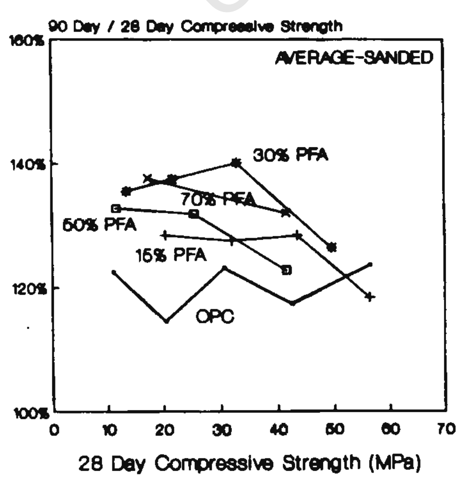

Fig 4.8 Long term (90 days) strength compared with 28 day strength for average-sanded mixes.

From these figures the very distinctive slow early strength and the increased long term strength development characteristics of the PFA mixes compared with the OPC controls can be seen.

For each PFA level, the average of short term comparative ratios across the range of strengths, represents a single value to describe the short term behaviour for that PFA level. The same can be done for the long term comparative ratios. These values are representative due to the consistency of the comparative ratio of each PFA level across the range of strengths. The clear differences between each level are shown in Figs 4.7 and 4.8.

From these values a single conventional strength/time plot can be drawn to represent the full range of strengths for each sand to stone ratio. These are given in Fig 4.9 with the comparative ratio data given for each plot.

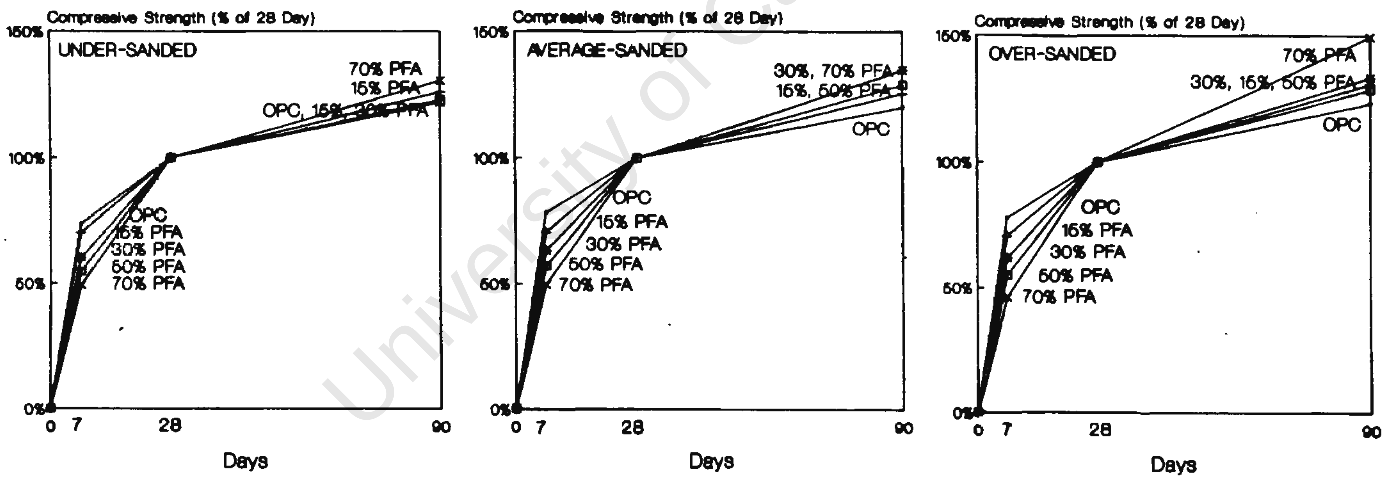

Fig 4.9(a,b,c) Conventional strength vs time plots for all strengths.

4.6 Conclusions from compressive strength development

The typical PFA characteristic of slow short term strength development and extra long term strength gain in comparison with the OPC controls, can be summarised in the following two graphs of the ratios for short and long term strength vs the percentage PFA. From these plots the effect of increasing the percentage PFA is to reduce the early strength development and to increase the long term strength development.

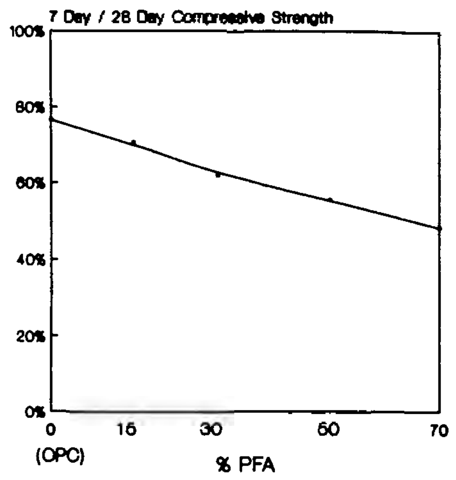

Fig 4.10 Effect of percentage PFA on short term strength development.

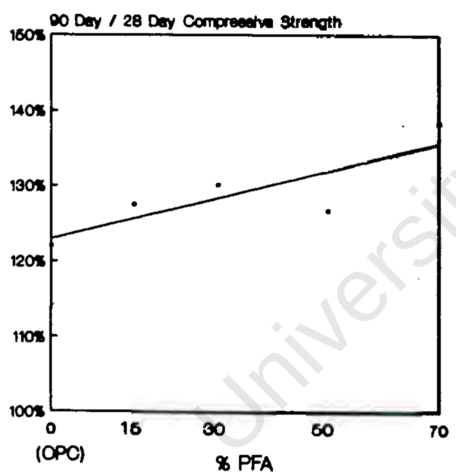

Fig 4.11 Effect of percentage PFA on long term strength development.

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CHAPTER 5

FLEXURAL STRENGTH OF PAVEMENT CONCRETE

Bending of a concrete pavement under vertical loads produces both vertical compressive and flexural stresses. However, the ratios of vertical compressive stress to compressive strength are too small to influence the design of the slab. Ratios of flexural stress to flexural strength are quite high, often exceeding 0.5. As a result the flexural stresses imposed and flexural strength of the concrete are critical in the thickness design of the concrete slab.

Flexural strength of the concrete pavement slab is also required for the distribution of concentrated loads over a wide area and the resisting of cracking due to changes in moisture or temperature (1). The Road Research Laboratory (2), in determining a measure of performance of concrete road slabs on two insitu road sites, found cracking to be the determining factor. Thus if increased tensile flexural strength can be achieved, a decrease in cracking and a lengthening of service life will enhance the performance of the concrete pavement.

Another important relevance of the flexural strength is the relationship with the elastic modulus and hence the determination of deflections in beams and slabs.

5.1 Design methods for pavements slabs

At present there are several ways of designing a concrete pavement slab. Marais (3) divides the design methods into two categories, firstly unreinforced and jointed pavements and secondly continuously reinforced. In all these methods (with possible exception of the present PCA method) a weakness is shown in the naive attitude to the potentially wide variation in properties that concrete can exhibit. Comment on these design methods is set out in Figs 5.1 and 5.2

Design Method	Concrete design criteria
American Assoc. of State Highway Officials (AASHO)	28 day flexural strength
Ministry of Transport (UK)	min 28 day crushing strength = 28 MPa

US Corps of Engineers	28 day flexural strength
Old PCA	28 day flexural strength
Present PCA	100% of 90 day flexural strength or 125% of 28 day strength

Fig 5.1 Table of concrete design criteria : Unreinforced and jointed pavements.

Design method	Concrete design criteria
Wire Reinforcement Institute	4.1 to 4.8 MPa flexural strength at 28 days
ACI	28 day flexural strength and modulus of elasticity
AASHO	28 day flexural strength

Fig 5.2 Table of concrete design criteria : Continuously reinforced pavements.

In these design procedures much attention is paid to the under-slab soil conditions and predicted traffic utilization of the pavement and only a simplistic view of the concrete is taken. The long term strength development, which can be substantial, is hardly considered.

5.2 Factors affecting flexural strength

The same factors that affect the compressive strength of concrete affect the flexural strength. However, the degree to which these factors individually affect the flexural strength is not necessarily the same as that for compressive strength.

The factors, discussed below, all affect the flexural strength, but some are not of importance for the work done on the Lethabo PFA due to mix variables selected (see 3.7). These factors are quoted from Fulton (4) and a paper by Chaston (5).

The quantity of cement, quantity of water and thus the cement to water ratio affect the flexural strength. In general an increase in the compressive strength is accompanied by an increase in the flexural strength but the rate of increase in flexural strength is usually less than that for compression.

The age at testing and the curing conditions are of particular relevance when discussing the effect of PFA on the flexural strength.

However, of particular importance is the effect of the coarse aggregates. The surface texture of different coarse aggregates of similar size were found by Kaplan (6) (according to Fulton) to change the flexural strength by up to 40%. The failure mechanism, like that for compression, is at the coarse aggregate/mortar interface. It is the aggregate properties of shape, surface texture and porosity that define this interface and cause the variations. Following this reasoning, Fulton quotes Kaplan (7) in discussing the effect of coarse aggregate proportion in the mix. An increase of coarse aggregate proportion decreases flexural strength. This is due to the introduction of more potential failure surfaces that are closer together than a similar less stoney mix.

The coarse aggregate thus plays an important role in determining the flexural strength of the concrete. For a specific aggregate, good compressive strength does not necessarily imply good flexural strength.

5.3 Flexural strength test method

The flexural strength test is one of the methods of determining the tensile strength a concrete. Other tests include direct tensile measurement and the tensile splitting tests.

The flexural strength of a concrete can be tested in three standard ways. These are the cantilever method, mid point beam loading and third point beam loading. The first two: cantilever and mid point loading, are relatively inappropriate as they test the flexural strength at a single point in the member. This is at the edge of the support for the cantilever and the midspan of the beam for the mid point loading. The results from testing a single point gives a wide scatter as the test gives the strength at a single specific point and not the weakest point across a wider part of the beam. Over a length the weakest point will always be automatically found. Thus a large number of tests is required to establish a representative mean.

The third point loading method thus provides a more accurate test of true flexural strength. The test, as detailed in SABS 864.(6) provides for the central part, equal in length to the depth of the beam, to be loaded with a uniform bending moment till failure. The loading configuration of SABS 864 is shown in Fig 5.3.

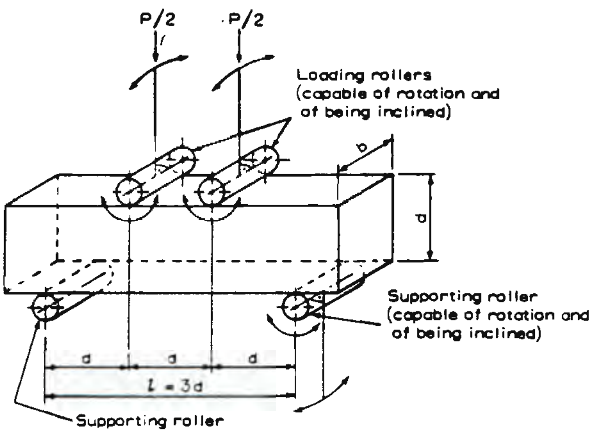

Fig 5.3 SABS 864 loading configuration.

As 13 mm stone was used, the beam mould dimensions were 100 x 100 x 500 mm as specified by the code. The loading rate was as specified in the code at 1.5 MPa per minute.

A deviation from SABS 864 was that the central part of equal moment was increased from a length equal to the depth of the beam (100 mm) to 250 mm. This was caused by difficulties in getting the existing top loading rollers close enough together. The modified loading configuration is shown in Fig 5.4.

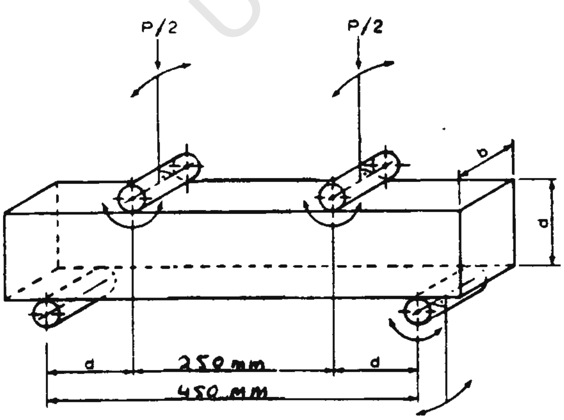

Fig 5.4 Modified loading configuration.

Two considerations allowed the acceptance of this modified loading configuration.

Firstly, the critical factors of the loading configuration, as specified by SABS 864, and illustrated in Fig 5.3, was that the "shear span" be equal to the depth of the beam. This meant that for any length of the central part the induced moment would be the same for a given load.

Secondly, with the SABS 864 configuration on the 500 mm long beam there was a unutilized overhang at each end of 100 mm. In increasing the central span to 250 mm at the expence of the overhangs, there would be a lower scatter in the results. This is for the same reason that the third point loading test was better than the mid point test. This is born out by a theoretical plot of span length against possible flexural strengths in Fig 5.5. The longer the span the less the scatter in results, but the average strength will also be lower.

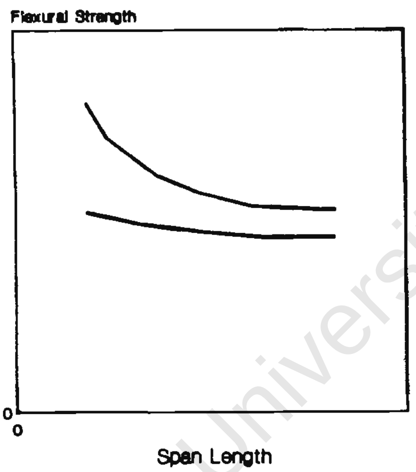

Fig 5.5 Effect of span length on scatter and average strength. (conceptual)

Thus by reducing the potential scatter of results and keeping the effective loading configuration the same as that for the code, a more accurate set of results was expected.

Fig 5.6 Photograph of a beam being loaded in the modified test rig.

5.4 Extent of flexural testing

The initial testing for flexural strength according to the modified SABS 864 was done on beams of 28 days age. Three beams were cast from each mix from the average-, under- and over-sanded mix series (series 5, 6 and 7, see 3.7). Along with the cubes for compression testing, the beams were vibrated on a Vebe vibrating table and stored in a +90% relative humidity environment for 24 hours before being stripped and cured in water for 28 days till being tested.

Inadvertently the 15% PFA beams designed for a 28 day compression strength of 30, 40 and 50 MPa of the under- and average-sanded mixes were interchanged. This meant that they were tested at the wrong ages and thus no data at 28 days is available for them.

A second set of beams were made for testing at 90 days age. These too were from average-, under- and over-sanded mix series (series 8, 9 and 10, see 3.?). They were prepared in the same way, but were cured in water for 90 days. The range of mixes was substantialy less than for those tested at 28 days with only two design compressive strengths taken for each PFA replacement level for each sand to stone ratio.

5.5 Analysis method

A comparison was made of the flexural strength with total cements to water ratio (C/W). This was done in a similar way to the analysis of the compressive strength and compressive strength developmentas in chapter 4. Actual strength curves were developed and the long term strength development assessed.

Although there is no absolute link between general compressive strength and flexural strength in general, a comparison was made. There was no change in materials for the two types of strength tests and thus no external influences induced by different aggregates was present. Of particular interest was the comparative rates of strength gain between the compressive strength and the flexural strength.

5.6 Comparison of flexural strength with C/W ratio

As for compressive strength, flexural strength vs C/W ratio curves were plotted for the three sand to stone ratios at 28 and 90 days. These are given in Figs 5.7 and 5.8 respectively.

Three comparisons can be drawn. They are (a) comparison of general trends across the three sand to stone ratios; (b) comparison at a particular percentage PFA of the effect of changing C/W ratio; and (c) comparison across the different percentages of PFA. The comparisons (b) and (c) show the same trends displayed for flexural strength as for compressive strength. Explanation for these were covered in 4.4.

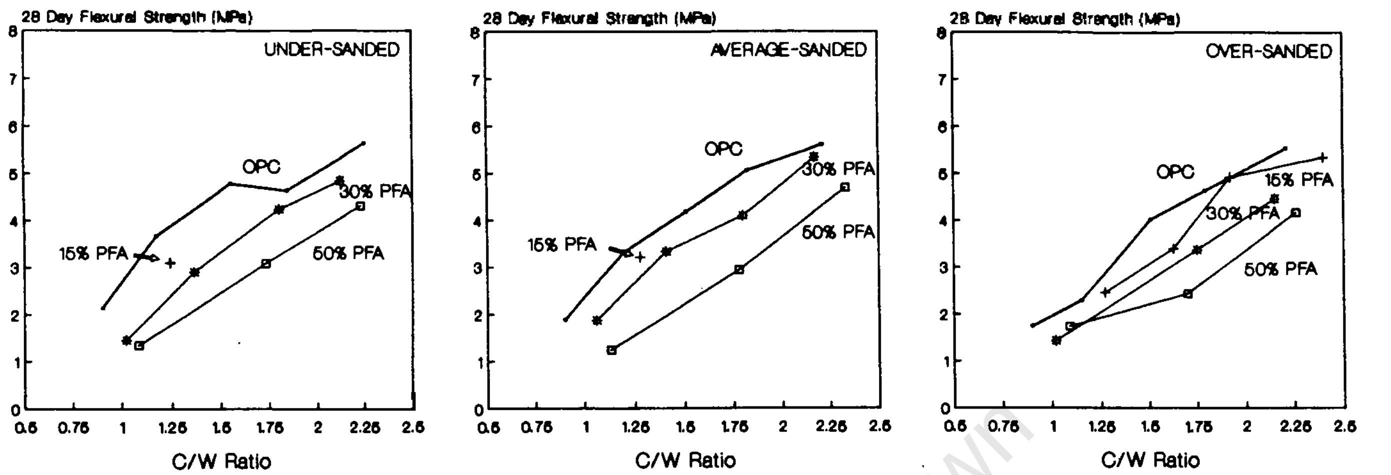

Fig 5.7(a,b,c) Actual flexural strength curves at 28 days. Under-, average- and over-sanded mixes.

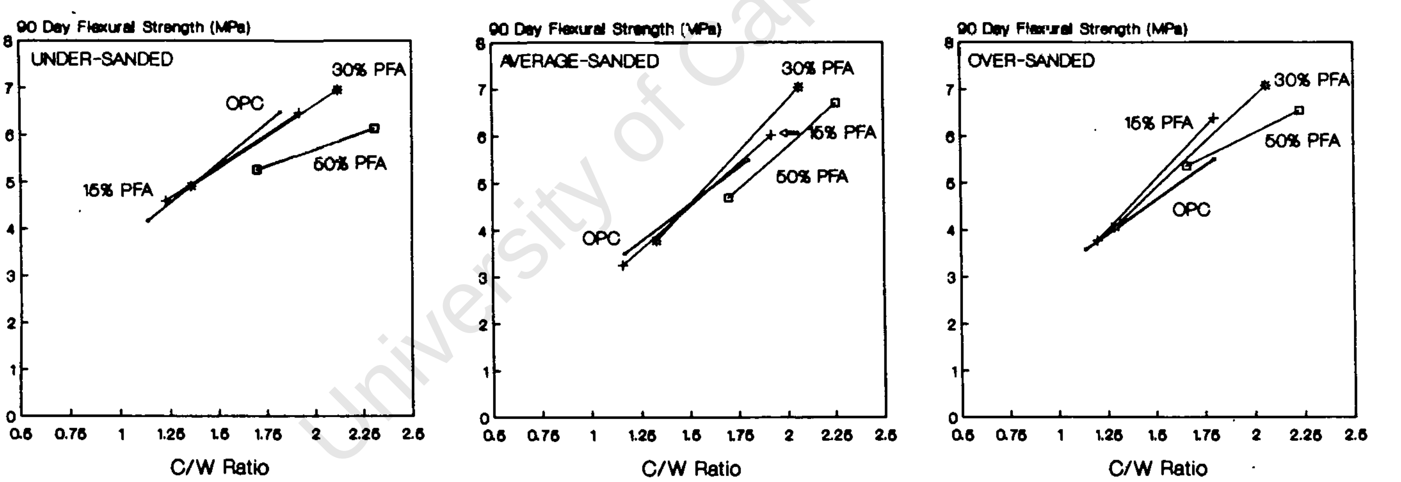

Fig 5.8 Actual flexural strength curves at 90 days. Under-, average- and over-sanded mixes.

There are slight differences across the three sand to stone ratios that need highlighting. At 28 and 90 days there is an small decrease in flexural strength at all C/W ratios when moving from the under-sanded mixes to the over-sanded mixes. This means a decrease in flexural strength with a decrease in coarse aggregate proportion of the mix. This is contrary to the work by Kaplan (7) who predicted an opposite trend.

This trend is the same for compressive strength across the same sand to stone ratios. The explanation for these trends was discussed in 4.4 with reference to Fulton and work by Shacklock.

The maximum fluctuation in the flexural strength at any C/W ratio over the three sand to stone ratios is 1 MPa at both 28 and 90 days. This fluctuation amounts to about 25% of the actual flexural strength and thus it would not be realistic to develop a single average set of curves to cover all three sand to stone ratios.

The compressive strength fluctuations were at maximum 9% (see 4.4). This means that the flexural strength is far more sensitive to coarse aggregate proportion than compressive strength.

5.7 Accuracy of mean values

At each test session for flexural strength of a mix, three beams were tested. The maximum allowable scatter allowed by SABS 864 of the three values, was for the difference between the highest and lowest values not to exceed 15 % of the mean.

Fig 5.9 shows the scatter of values for the average-sanded 28 day flexural results, the lines of Fig 5.9 being the same as Fig 5.7(b).

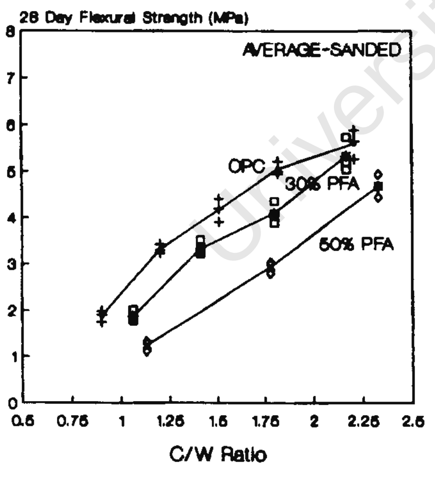

Fig 5.9 Typical scatter of flexural strength results.

At worst the maximum scatter is 0.5 MPa for a 4.2 MPa mean flexural strength, which is 12%. This is within the 15% limits specified by SABS 864.

5.8 Long term flexural strength development

As with compressive strength, the rate of strength development is of importance when comparing the effects of PFA on the mixes. As mentioned in 5.6 the fluctuations in strength across the three sand to stone ratios was relatively large. This meant that curves for long term strength development (up to 90 days) had to be considered separately for each sand to stone ratio. Due to inconsistent trends for each sand to stone ratio across the range of C/W ratios, the strength development was evaluated at C/W ratios equal to 1.5 and 2.0 only. These strength developments are shown in Fig 5.10.

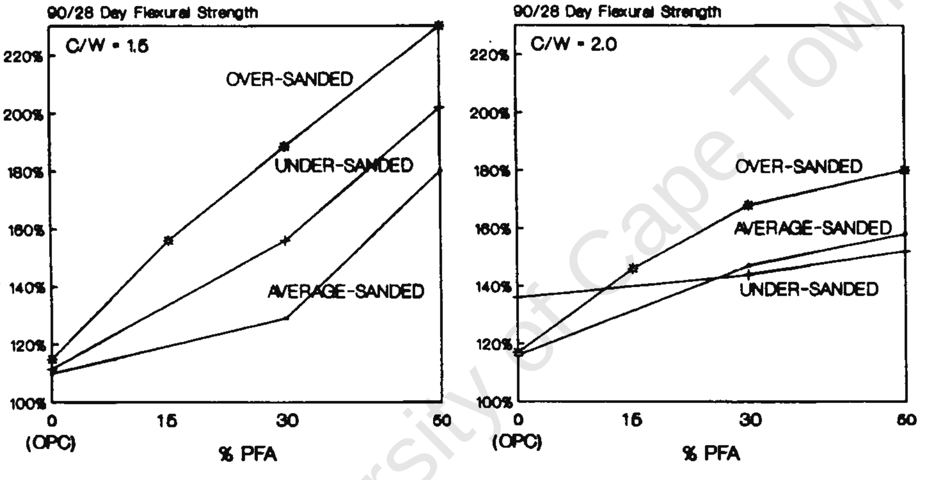

Fig 5.10(a,b) Flexural strength development at C/W = 1.5 and 2.0.

At both these C/W ratios the over-sanded mixes gained the most strength and the average- and under-sanded mixes gained similar amounts.

Of particular note is the high level of strength gain that the PFA mixes achieved compared to the OPC mixes. At both C/W ratios the OPC mixes at 90 days only managed to gain an additional 20% over their 28 day flexural strengths. The 50% PFA mixes gained between 50 and 130% over their 28 day strengths.

Thus PFA concretes gain substantial additional long term flexural strength compared to OPC concrete. This results in equal 90 day flexural strengths for equal C/W ratio.

5.9 Comparison of flexural and compressive strengths

This exceptional increase in flexural strength must be compared to the compressive strength development (see 4.5 and Fig 4.8).

The 90 day compressive strength development for OPC mixes was of the same magnitude as flexural strength (20% increase over 28 day strength). Most PFA mixes only gained a maximum additional 30% long term compressive strength compared to the maximum gain of 130% for flexural strength.

According to Chaston (5) from work on OPC concrete, early flexural strength is supposed to develop more rapidly than compressive strength and vice versa after 90 days. The slower action is attributed to the slow pozzalanic cementing action of the Lethabo Field 2 PFA which has more pronounced effects in flexural strength than compressive strength.

5.10 Conclusions

(1) Present design methods for concrete roads take very little cognisance of the potential characteristics that concrete can display. (see 5.1)

(2) Flexural strength is more sensitive to coarse aggregate content than compressive strength.(see 5.8)

(3) PFA concretes gain more additional relative long term strength than OPC concretes. (see 5.8)

(4) Flexural strength development occurs later than compressive strength development in PFA concretes.

Conclusion (3) indicates large potential advantages in in the use of Lethabo Field 2 PFA in concretes for pavements.

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CHAPTER 6

MIX ECONOMICS

In practice, the decision to use PFA concretes instead of OPC concretes is primarily based on the economics. The economic evaluation can be done easily on the basis of compressive and flexural strength. Properties such as creep and shrinkage can not easily be evaluated on a purely economic basis and thus need alternative evaluation techniques. Compressive and flexural strength can be directly related to the cost of the mix, as is done in this chapter.

The economic evaluation can be based purely on the materials costs of the mix or based on the total cost of materials, mixing, placing and curing of the concrete. Although the second and more realistic method is beyond the scope of this thesis, certain points should be made about it.

6.1 Construction economics

The economics of pavement construction is specific to application, the design and the materials used.

An estimate of the costs of the concrete work is important when considering the use of PFA concrete. Of particular importance is the proportion of the concrete costs that is made up by mix materials. The proportions, listed below in Fig 6.1, are typical of concrete road construction. They are taken from a road that used only light mesh reinforcement and relate only to the work involving the concrete. (1)

Cost item	% of cost	
Equipment	8.6%	
Mixing, transportation and placing	24.6%	
Fabric mesh reinforcement	8.4%	
Other reinforcement	4.3%	
Joints	5.6%	Concrete
Cement	22.0%	} Materials
Aggregates	24.1%	} = 46.1 %
Curing	2.4%	

Fig 6.1 Breakdown of construction costs for concrete road pavements.

From this table it can be seen that the concrete materials cost just less than half the total. Thus any saving in material costs has a dramatic effect on the total construction costs.

Over and above these costs, the cost of using PFA in the mix must make allowances for other hidden factors. These are mainly due to the learning and training, storage, use and control of an additional ingredient in the mix. There is potential risk in the use of the PFA unless it is used in a well controlled and supervised environment.

6.2 Mix cost determination

Any economic evaluation of a mix is highly dependant on the specific materials used and location of the supplies and the users. This is of particular significance when comparing costs of OPC concretes with PFA concretes.

Concrete usually uses local aggregates, cement (OPC) and water for the mix. In the Transvaal, PFA is a "local" material costing around one third the Transvaal OPC. However, in using the PFA further away from its source, the cost of transportation has to be included. This transportation cost of PFA from the Transvaal to the Western Cape raises the price of PFA to a level equivalent to the price of local Western Cape OPC.

The use of specific types of aggregate affect the economics of the mix as production and delivery costs of aggregates vary widely. The use of the different aggregates also affects the measurable properties of the mix. Economic selection between mixes using different aggregates can only be done knowing results for the different mix configurations. Thus the economic analysis for the work done with Western Cape materials and Lethabo Field 2 PFA is highly specific but gives trends within the range of mixes used.

The prices for the individual ingredients of the mixes were taken at their June 1988 values as delivered to UCT. The Leth abo Field 2 PFA was priced the same as Matla PFA as no cost estimates were available at the time. The prices are as in Fig 6.2.

Material	June 1988 Price
Cape Flats dune sand	R 13.76 /m3 (R 0.0086 /kg)
13 mm Malmesbury shale	R 36.03 /m3 (R 0.024 /kg)
De Hoek OPC	R 7.85 /50kg sack (R 0.157 /kg)
Lethabo Field 2 PFA	R 6.46 /50kg (R 0.129 /kg)
Water (Domestic supply)	R 0.50 /m3 (R 0.0005 /kg)

Fig 6.2   Table of material prices in June 1988.

From the above information the OPC and PFA are the most expensive materials per unit mass. However, the total cost should be related to the individual quantities of the materials. Taking an average-sanded 30 MPa concrete mix as an example, the mass and cost proportions of the mix are shown in Fig 6.3.

Material	% by Mass	% by Cost
Cape Flats dune sand	35%	9.6%
13 mm Malmesbury shale	45%	34.6%
Total cements (OPC+PFA)	12%	55.7%
Water (Domestic supply)	8%	0.1%
	100%	100%

Fig 6.3   Table of materials; % by mass; % by cost.

The information in Fig 6.3 shows that over half the materials cost is cements with the stone being the next most expensive item.

An important point to remember is that in the Transvaal the PFA would be one third the OPC price. The effect of this on the materials cost in the mix could be considerable for a high percentage PFA.

6.3 Compressive strength vs material costs

Figs 6.4 and 6.5 give the materials costs of the 54 mixes that were prepared, plotted against their 28 and 90 day compressive strengths respectively.

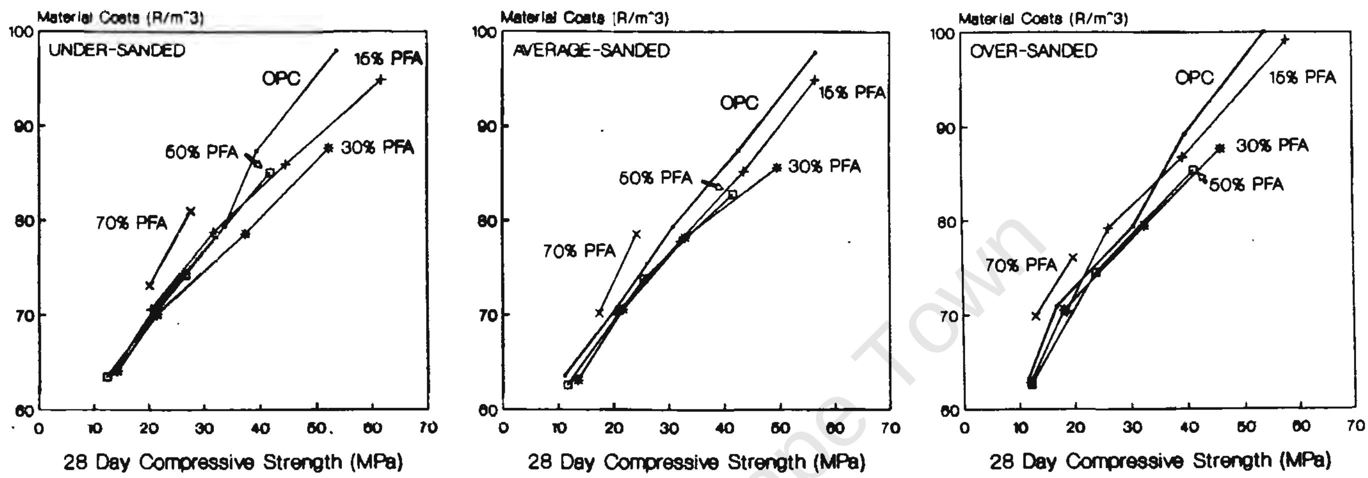

Fig 6.4(a,b,c) Materials cost vs 28 day compressive strengths.

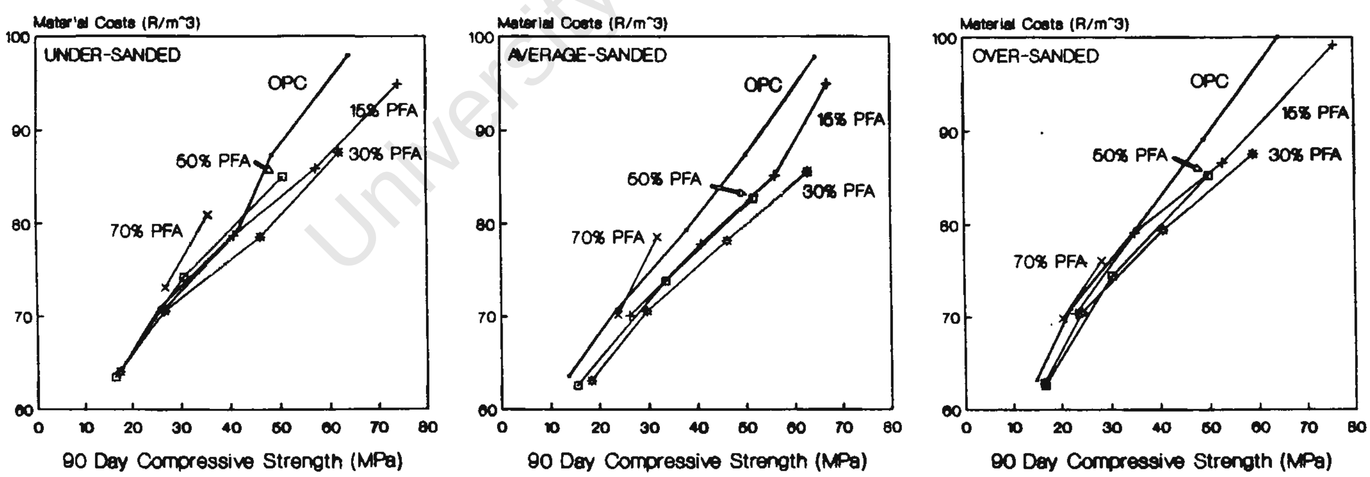

Fig 6.5(a,b,c) Materials cost vs 90 day compressive strengths.

As expected, all six of the graphs in Figs 6.4 and 6.5 show an increase in cost with increasing strength. This is due due the increased total cements content with increasing strength. (see 3.11 and Fig 3.4)

6.3.1 Comparison across three sand to stone ratios.

At both 28 days and 90 days there is little change in the costs. The under-sanded mixes are slightly cheaper than the over-sanded mixes by less than 5%.

This situation is in conflict with the theoretically expected increase in cost with increasing stone as expensive stone replaces cheap sand (see 3.11.1). The reason that this expected increase does not occur, is that the total cements decrease with increasing stone content (see 3.11 and Fig 3.4). These two factors balance each other to give little materials cost change across the three sand to stone ratios.

6.3.2 Comparison across % PFA levels

At 28 days (Fig 6.4), all the concretes are of somewhat similar cost for any given compressive strength. Two graphs show that the 30% PFA concrete is the cheapest concrete for any given compressive strength. The OPC concrete, apart from the 70% PFA concrete, is the most expensive for any given compressive strength. The cost savings for the 15%, 30% and 50% PFA concretes are in the range of 2 - 4% on the OPC concretes.

This trend is very similar to the trends displayed by the total cements vs compressive strength (see 3.11 and Fig 3.4). This similarity is explained by considering that the OPC is marginally more expensive than PFA and that the total cements content makes up half the material costs (Fig 6.3).

At 90 days (Fig 6.5), the compressive strength grows proportionally more in the PFA concretes than the OPC concretes relative to their 28 day strengths (see 4.5). This additional gain in strength magnifies the trends shown by the PFA concretes at 28 days. The 30% PFA concretes are clearly the cheapest with a saving of around 9% on the OPC concretes. The 15% and 50% PFA concretes have a saving of the order of 6% and the 70% PFA is only 1 - 2% more expensive than the OPC concrete.

6.4 Flexural strength vs materials costs

Figs 6.6 and 6.7 give the materials costs of the mixes plotted against their 28 and 90 day flexural strengths respectively.

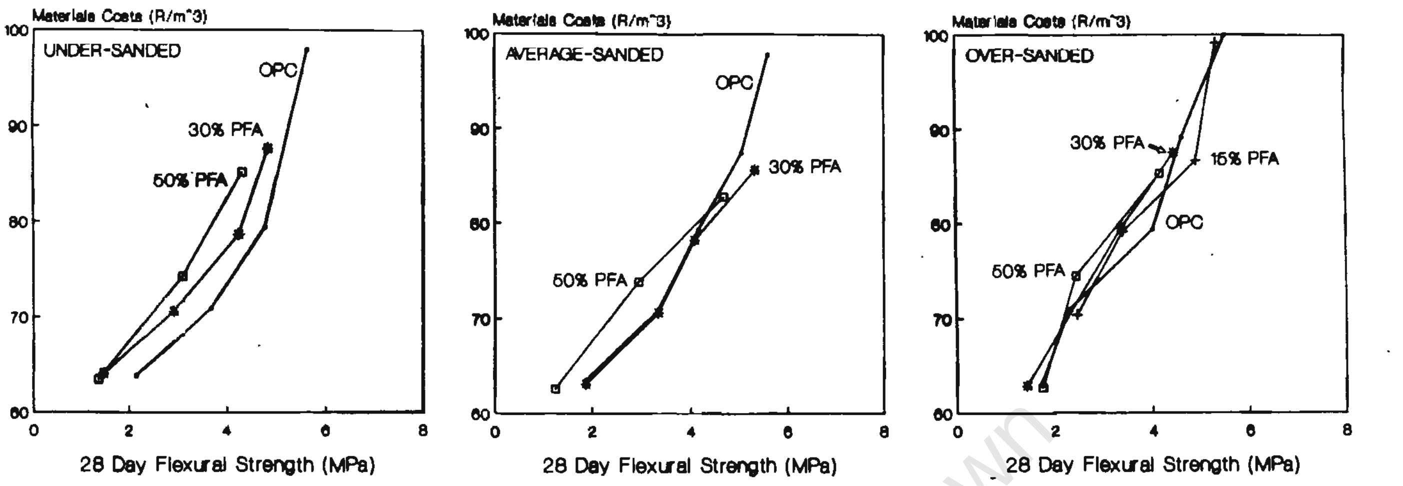

Fig 6.6(a,b,c) Materials cost vs 28 day flexural strengths.

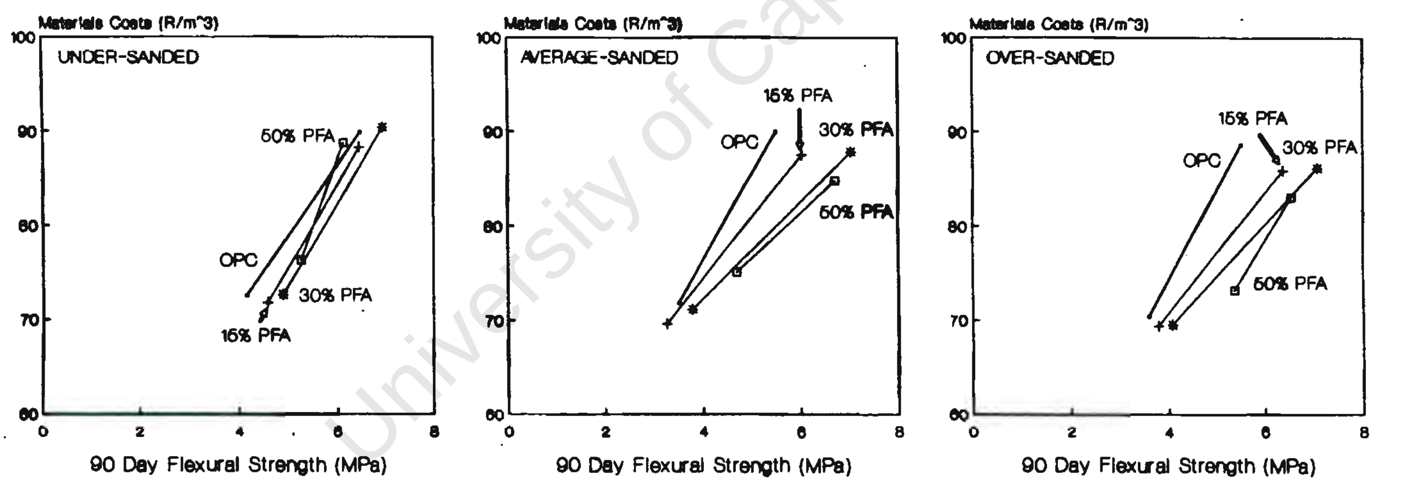

Fig 6.7(a,b,c) Materials cost vs 90 day flexural strengths.

Like the compressive strength, increased flexural strength costs more. This similar trend was expected and discussed in 5.6.

6.4.1 Comparison across three sand to stone ratios

At both 28 and 90 days there is a marginal increase in the cost of the concretes with decreasing stone content. This increase in cost is about 3% in moving from under- to over-sanded mixes.

The explanation for this small increase in costs is similar to that for compressive strength as discussed in 6.3.1.

The balance is tipped slightly so that the increase in total cements with decreasing stone content has a greater effect than the replacement of expensive stone with cheap sand and thus the cost is increased.

6.4.2 Comparison across % PFA levels.

At 28 days (Fig 6.6), the OPC mixes give about the cheapest concrete for any given flexural strength. An increase in the percentage PFA increases the cost of the concrete.

At 90 days (Fig 6.7), the trend is reversed. The higher the percentage of PFA in the concrete the cheaper the concrete for any given flexural strength.

A comparison of cost as a percentage of the OPC concrete is shown in Fig 6.8.

% PFA	Equal 28 day flexural strength	Equal 90 day flexural strength
OPC	100%	100%
15%	103%	97%
30%	104%	90%
50%	108%	88%

Fig 6.8 Table of costs as % of OPC cost.

The reason that the PFA concretes relative to OPC concretes, cost more at 28 days and less at 90 days, is due to the comparatively late development of flexural strength in PFA concretes. (see 5.9).

The consequence of this late, but substantial development of flexural strength may force the designer of the concrete pavement to select a mix costing more to give early flexural strength where early strength is indicated.

6.5 Conclusions

In comparing the OPC and PFA concrete material costs, the following can be concluded:

(1) Materials costs make up about 46% of the total construction costs (Fig 6.1).

(2) Total cements make up about 55% of the total materials costs of the mix (Fig 6.3).

(3) Changing the stone content of the mix does not changes the cost of a concrete of a given strength (6.3.1)

(4) Concrete costs based on compressive strength are proportional to the total cements content of the mix (6.3.2).

(5) The cheapest concrete for a given compressive strength is one with about 30% PFA (Figs 6.4 and 6.5).

(6) For any given flexural strength at 28 days OPC concrete is the cheapest and at 90 days the 50% PFA concrete is the cheapest.

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CHAPTER 7

FRESH CONCRETE PROPERTIES: SLUMP, VEBE WORKABILITY, MORTAR EXCESS, EARLY SET, BLEEDING

To the contractor involved in the mixing, placing and curing of concrete, the fresh properties are of vital importance. The proportions of the ingredients of the mix have already been determinedfor a specific strength, so what concerns the contractor is how the concrete will behave while he is working with it.

The fresh properties of importance are: the consistency of the mix as measured by the slump; the ease with which the concrete can be placed as measured by the vebe workability; the ease with which the surface finish can be achieved as measured by the mortar excess; the time after which forms can be stripped as measured by early set;and the bleed characteristics of the concrete that can affect the concrete quality.

If certain fresh properties of the concrete are unfavorable, the mix may be able to be modified without affecting the design strength.

The fresh properties were taken as the resulting characteristics displayed by each mix. No attempt was made (other than to set the design slump to 50 mm) to control the mix design by any other of the fresh properties.

7.1 Slump

The slump of a concrete is a simple test to establish its consistency. It is not a true measure of workability. Orchard (1) states that the improvement in workability of a mix by, for example, the addition of a pozzolana, may not be reflected in an increase in the slump and in fact water may have to be added to obtain the original slump.

Powers (2), in discussing the mechanism of the slump, invalidates the theory of no link between slump and workability. Thus uncertainty exists of the relationship between slump and workability.

Slump is the most common means by which the consistency of a concrete is measured. Experience with time, working with a concrete made with the same materials, allows a relationship to be developed between a given slump and a given workability.

The selection of a 50 mm slump (see 3.7) was to give a common consistency to the mixes across the full range of strength, percentage PFA and sand to stone ratio.

The control and acceptability of the slump of a mix was discussed in 3.9 and 3.10.

7.2 Vebe workability

The workability of a concrete mix is a very complex concept. Popovics (3) explains workability by linking it to a number of fundamental properties such as internal friction, cohesion, viscosity, yield stress, thixotropy, dilatancy, capacity for plastic strain and the tendency for segregation and bleeding. The combination of these properties control the workability. The ACI gives the definition of workability as "a property of the fresh concrete that determines the ease with which it can be mixed, transported, placed, compacted and finished".

From this definition, workability can be separated into two concepts: firstly, the ease with which a desired finish can be achieved and secondly, the amount of internal work needed for the placing and compacting.

The ability to achieve a desired finish is related to the mortar excess as a measure of "fat" in the mix, coupled with the "wetness" of the mix. The amount of internal work required for the placing and compacting can be directly related to the amount of energy needed to "move" the concrete in remoulding, as measured by the vebe workability test, for example.

The vebe test as described in BS 1881 Part 2, requires a normal slump test to be carried out inside a standard container which is firmly attached to a standard vibrating table. With the slump cone removed, a weighted perspex disk is placed on top of the slumped concrete. The vibrating table is then switched on. It vibrates at a standard frequency and amplitude, thus providing a specified energy production rate. The disk remoulds the concrete till the whole of the underside of the disk comes into contact with the remoulded concrete. The duration for this complete remoulding is measured. The time relates to the energy required for remoulding and thus relates to the workability.

As discussed earlier, there is disagreement on the relationship between slump and vebe workability. Fig 7.1 shows plots of the slump against the vebe time for the mixes from the three sand to stone ratios.

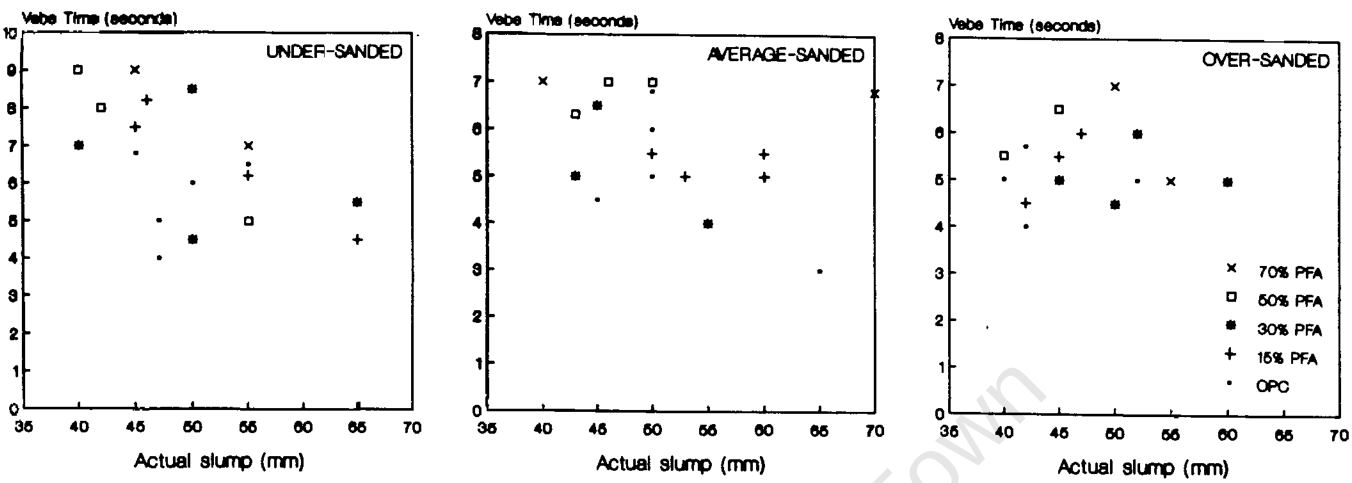

Fig 7.1(a,b,c) Plot of slump vs vebe time for under-, average- and over-sanded mixes.

From the data in Fig 7.1, a very tenuous trend of increased workability (shorter vebe time) relates to an increase in slump. It is however, not possible from the limited data to establish the effect of the different percentages PFA on the workability with respect to slump. This lack of distinction is to a certain extent, due to the additional stone placed in the PFA mixes. This stone is added to allow for the improved workability that the PFA gives (see 3.3 and 3.5).

However, because the vebe workability tended to be related to slump, it can be proven that the workability of the mixes was improved by PFA. It is shown by the fact that the 50 mm design slump was achieved with lower water requirements for the PFA mixes (see 3.11 and Fig 3.5). Thus if the PFA mixes had had the same water requirement as the OPC mixes, they would have had a far higher slump and thus far higher workabilty.

7.3 Mortar excess

Mortar excess is a measure of the "fat" in the mix. The "real" mortar excess is the depth (as a percentage of total depth) that the mortar occupies above the top of the uppermost coarse aggregate particles after full compaction. The "theoretical" mortar excess is a measure of how much excess mortar there is in the mix if the coarse aggregate were fully compacted on its own and the mortar was then allowed to fill the remaining voids.

From this it can be seen that the real mortar excess is related to how well the course aggregate packs and how much of the mix it makes up. Since only one type of stone was used the controlling factor was the amount of stone in the mix. The mortar excess was measured by poking a ruler into the top surface of the fresh concrete after compaction till it came into contact with the course aggregate - a "dipstick" method.

Since only one type and size of stone was used, the controlling factor was the amount of stone in the mix.

The results for real mortar excess for the three sand to stone ratios show direct correlation with the stone content in the mix. The additional stone and the resulting lower mortar excess for increasing percentages of PFA is clearly shown in Fig 7.2.

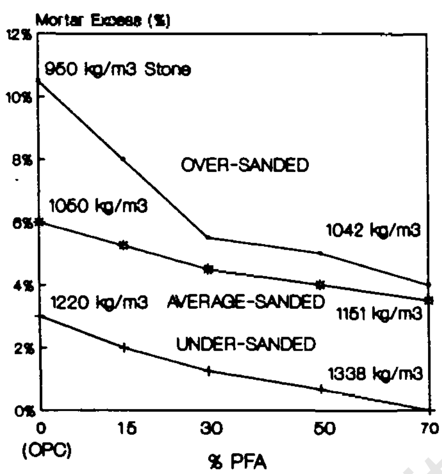

Fig 7.2 Mortar excess for three sand to stone ratios.

7.4 Early set

Early and final sets of concrete are defined in the ASTM C403(7) in terms of a pressure measure of resistance to penetration of a standard set of needles. Early set is reached when the resistance is 3.5 MPa and final set at 18 MPa

The test for final set was not done due to complications with limits of measurement. It was not necessary to know the final sets as setting trends could be determined from the early set times.

The time taken to reach early set is an indication of the rate at which the mix stiffens and gains early strength. This rate of very early strength gain is of importance as it determines the duration of bleeding (see 7.5) and how soon the mix has enough strength to allow forms to be stripped.

According to ASTM C403, the method for determining the early set of a concrete is that after mixing, the concrete is sieved to remove all aggregate larger than 4.75 mm. The remaining mortar is then placed in a mould and kept at constant humidity and temperature for the duration of the test. At intervals the maximum resisting force to the penetration of a range of standard needles is measured. Each force is converted to a stress and a strength vs time plot is developed. When the penetration resistance reaches 3.5 MPa, the concrete is considered to have reached its early set. Fig 7.3 shows the test.

Fig 7.3 Early set test.

As expected with mixes containing Lethabo Field 2 PFA, the early strength gain was slower than for OPC mixes. This was due to the slower cementing action of the pozzolana. It is shown in Fig 7.4.

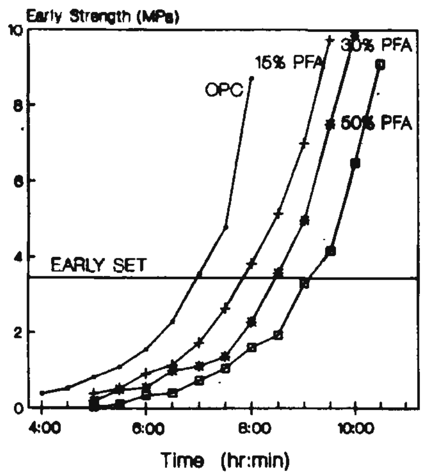

Fig 7.4 Typical resistance strength development for early set.

Fig 7.5 shows the time to reach early set for all the mixes across the three sand to stone ratios.

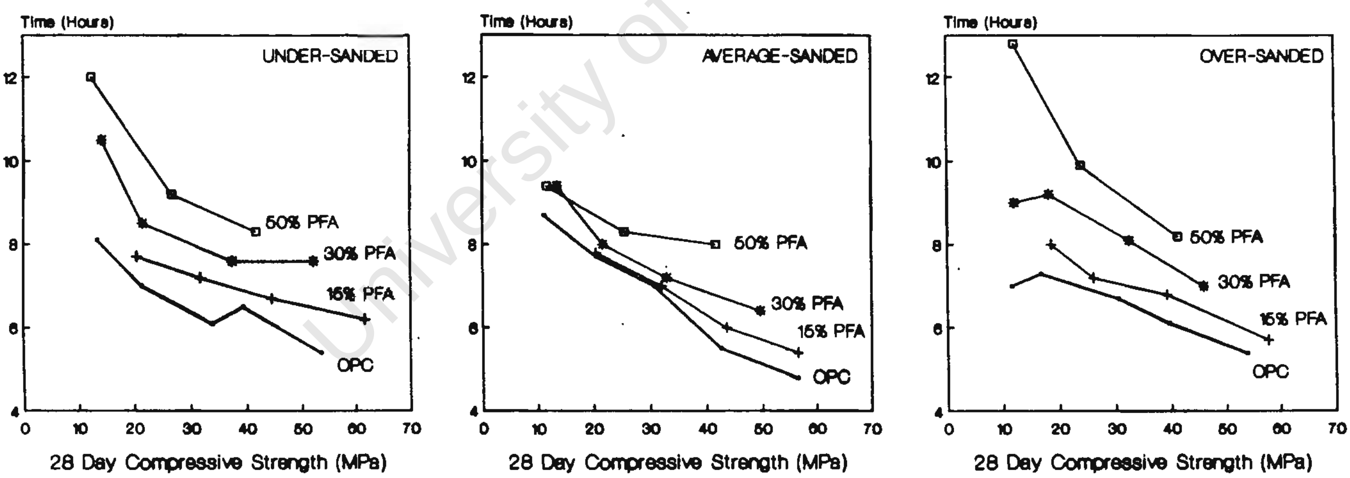

Fig 7.5(a,b,c) Time to early set (3.5 MPa).

The distinctive delay in time to reach early set for any given 28 day compressive strength for the all the PFA mixes can be seen. The higher the percentage PFA, the longer the time taken to reach early set. The time taken for the stronger mixes to reach 3.5 MPa is shorter due to their lower total cement to water ratio and greater quantity of total cements. Comparing the time to early set across the three sand to stone ratios, there is little change. This is as expected as any influence of the stone has been removed.

Summarizing the three graphs, Fig 7.6 shows the mean delays relative to OPC concrete.

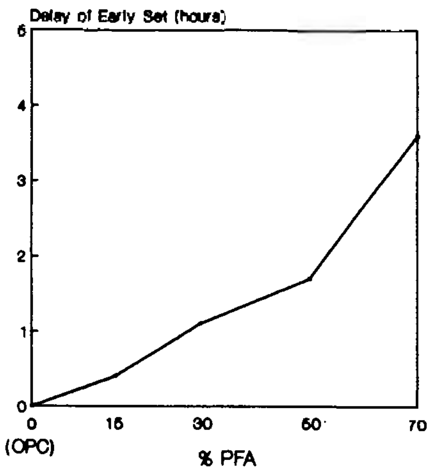

Fig 7.6 Delay of early set.

Since early set is defined as a specific strength (3.5 MPa), the time taken to reach this strength also depends on the actual 28 day strength of the mix. Fig 7.6 requires the following times of early set for the OPC concretes to set a datum for different design strengths.

28 Day Strength (MPa)	10	20	30	40	50
OPC Early Set (h)	8.5	7.4	6.7	6.1	5.2

7.5 Bleeding

Bleeding in fresh concrete is the settlement in the mix of the denser particles (cements and aggregate) and the corresponding rise of the lighter particles (water). This process of settlement continues until the mix has gained enough rigidity to set.

The visible result of bleeding is the formation of a layer of clear water on the surface of the concrete. The consequence of the bleeding of water to the surface has both beneficial and detrimental effects on the concrete and its performance.

The beneficial side to bleeding is the effective increasing of the total cements to water ratio of the mix. The increase of the total cements to water ratio in the mix raises the compressive and flexural strength of the mix.

The detrimental effects of bleeding are more numerous and of greater importance than those that are beneficial.

The water rising to the surface causes a weaker surface layer in the concrete due the local decrease in the total cements to water ratio. This weaker surface layer will result in a joint weakness if a subsequent lift is cast on top of the surface. Care must be taken to clean properly and prepare the surface before subsequent casting.

Inside the concrete, bleed water accumulates on the underside of aggregate and reinforcing. This produces similar layers of weakness and permeability.

According to Orchard (1), the anisotropy caused by the weaker layers can result in the compressive strength of up to 8% lower and tensile strength up to 15% lower across the layers.

The surface finish of the concrete can also be detrimentally affected. Over-working of the surface before bleeding has stopped may remix the bleed water into a thin layer of surface concrete which will have the tendency to flake off eventually from the main body.

These detremental effects of bleeding, producing a weaker surface to the concrete affects the wear durability of the concrete. For concretes exposed to surface wear, bleeding reduces the surface durability.

Bleeding can take place in two ways. Firstly, normal bleeding occurs where water seeps upwards uniformly over the entire surface, and secondly, channeled bleeding where the water comes to the surface through a few fast flowing channels. The channel bleeding is usually associated with leaner concretes and tends to carry the fine particles of cement to the sufave where they are deposited as laitance. It also results in higher permeability.

Three terms are used to describe the bleed characteristics of a concrete. They are:

> **Bleed capacity or bleed volume** - the total volume of water released from a unit volume of concrete by bleeding from start to finish,
>
> **Bleed rate** - the rate at which the water is released by the bleeding,
>
> **Bleed duration** - the total time elapsed from the filling of the container till when the bleeing becomes negligible.

According to Fulton (4), two factors that affect the bleeding characteristics are water content and the specific surface area of the mix. The water content of the mix obviously affects the bleeding in that the greater the water content the greater the potential for bleeding.

The specific surface area of the mix controls its water retention ability. The surface area is predominantly established by the proportion of the mix of size less than 150 micron. Water is adsorbed onto the surface of the ingredients and thus the larger the area, the more water can be adsorbed leaving less for bleeding.

This is of particular relevance to the Cape Flats dune sand. This sand lacks fines of less than 150 micron (see 2.1) and is well known for its bleeding characteristics.

The test for bleeding was the pipette method as described in ASTM C232(8) The bleed water was drawn off from the sample every 15 minutes till the bleeding became negligible. The volume drawn off was totalled to give the bleed volume. This method is shown in Fig 7.7.

Fig 7.7 Removing bleed water by pipette.

7.5.1 Analysis of bleed results

The bleed volume of the mixes can be assessed in two ways. Firstly, the bleed volume can be taken as a percentage of the original water requirement of the mix or secondly, the actual bleed volume can be considered on its own. Both these approaches give similar trends for each sand to stone ratio. However, comparing across the three sand to stone ratios, changes due to the different water requirements are apparent. The more meaningful of these two methods takes the bleed volume as the percentage of the original water requirement as shown in Fig 7.8.

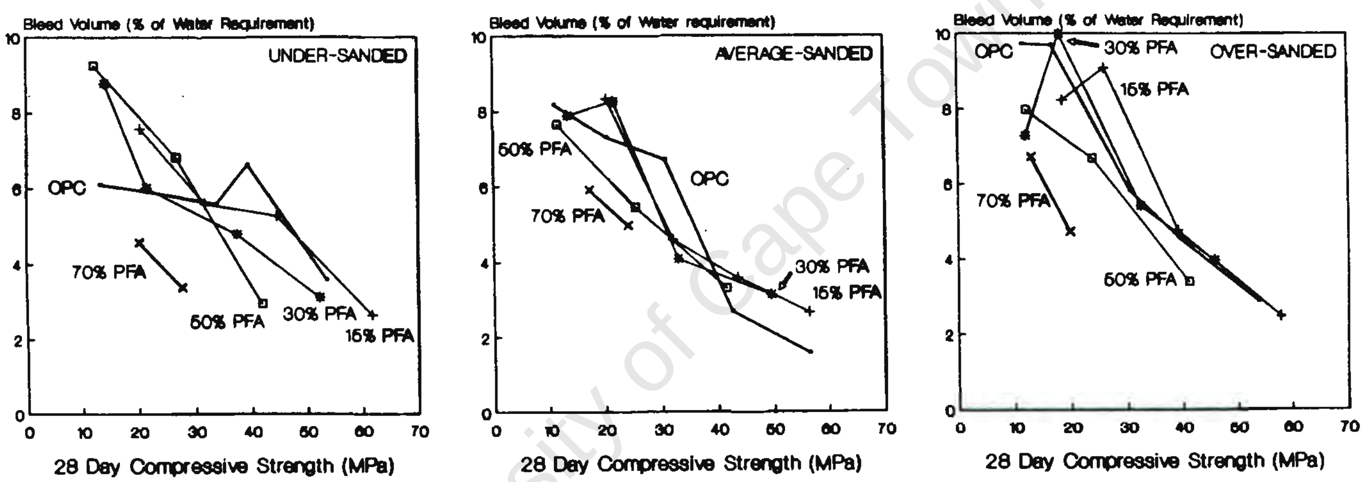

Fig 7.8(a,b,c) Bleed volume as % of water requirement.

The OPC, 15% PFA and 30% PFA mixes bleed similar proportions of their water requirements. The 50% PFA mixes tend to bleed a lower proportion of their water requirements. As however, the PFA mixes had lower water requirements than the OPC mixes (see 3.11 and Fig 3.5), it means that the PFA mixes bleed less absolute volume than the OPC mixes.

Across the three sand to stone ratios the mixes tend to bleed similar proportions of their water requirements. The over-sanded mixes had higher water requirements than the under-sanded mixes. This meant that the over-sanded mixes bled more actual volume than the under-sanded mixes. The actual bleed volumes are shown in Fig 7.9.

The reduction in actual bleed volume with increasing percentage PFA can be ascribed to the increased fines in the PFA mixes which retain the free water and so reduce bleed volume.

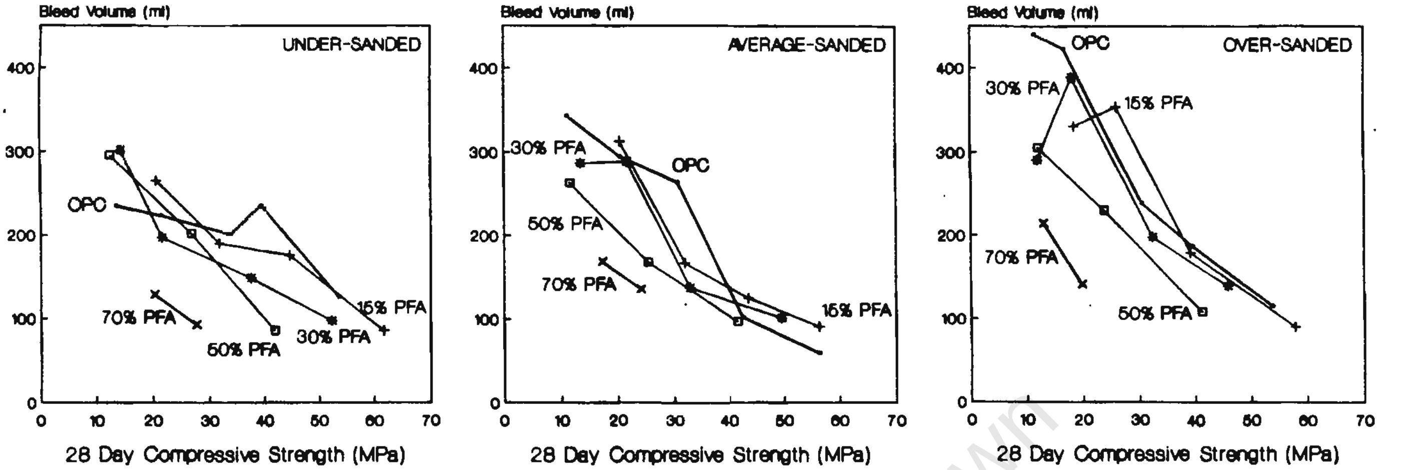

Fig 7.9(a,b,c)   Actual bleed volumes.

The rigidity or early strength gain of the mix is closely related to the duration of settlement and bleeding. The PFA mixes bleed for a longer period than the OPC mixes due to the slow pozzolanic action of the PFA. The mixes that took longer to set, have longer bleed durations. The durations for bleeding are shown in Fig 7.10.

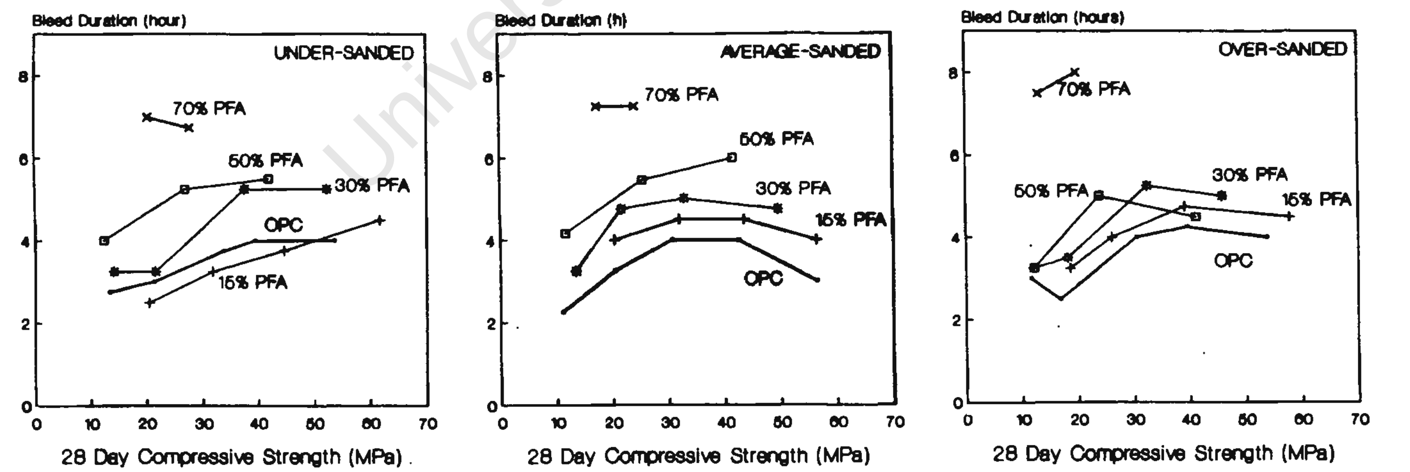

Fig 7.10(a,b,c) Duration of bleeding.

In combining the bleed volume and bleed duration, the average bleed rates for the different mixes can be calculated. These bleed rates are shown in Fig 7.11.

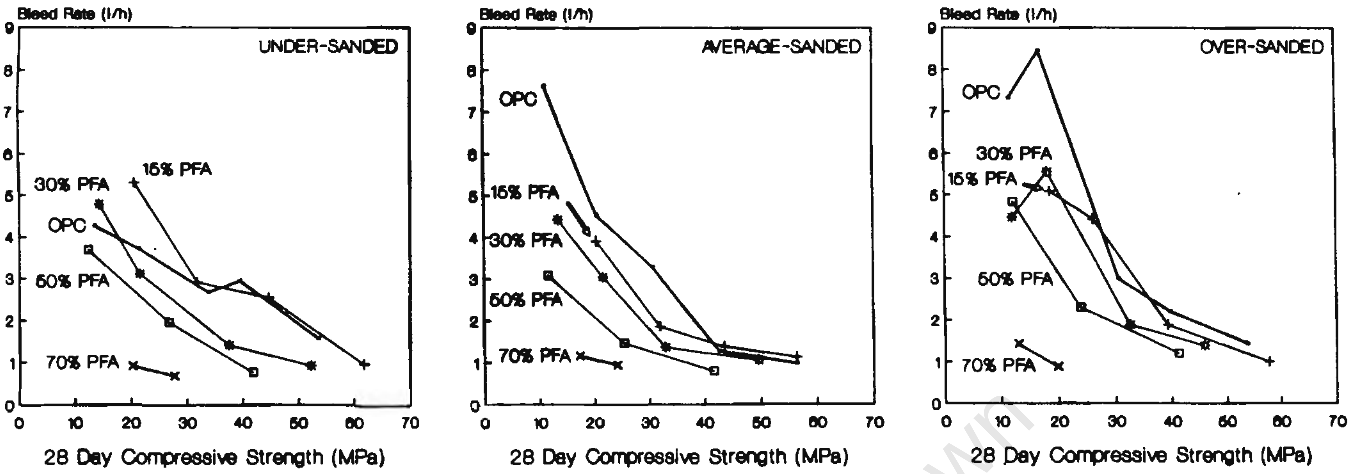

Fig 7.11(a,b,c) Bleed rates.

The lower bleed volumes and longer bleed durations of the PFA mixes result in the lower bleed rates compared to the OPC mixes.

7.6 Conclusions

The following conclusions relating to the concrete fresh properties can be drawn:

(1) Increased workability is associated with increased slump. (Fig 7.1)

(2) Mortar excess correlates inversely with stone content of the mix. (Fig 7.2)

(3) For a given 28 day compressive strength, PFA mixes reach early set later than their OPC equivalents. (Fig 7.6)

(4) PFA mixes bleed less than their OPC equivalents. (Fig 7.9)

(5) PFA mixes bleed for longer than OPC mixes. (Fig 7.10)

(6) PFA mixes bleed at a slower rate than OPC mixes. (Fig 7.11)

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(7) ASTM C403 - 68. Standard method of test for time of setting of concrete mixtures by penetration resistance. AMERICAN SOCIETY FOR TESTING AND MATERIALS. Philadelphia 1968.

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CHAPTER 8

WEAR DURABILITY OF PAVEMENT CONCRETE

For most applications of concrete, the durability is of vital importance. For pavement concrete the durability of the surface concrete is crucial.

The surface durability characteristics can be divided into two categories. These are the ability to resist (a) scaling, spalling and ravelling and (b) wearing action. The scaling, spalling and ravelling are caused by a combination of deicing agents and a freezing and thawing action as well as temperature fluctuations. The deterioration of the concrete by these influences can almost entirely be prevented by the entrainment of air in the concrete. (1, 2)

The wear resistance characteristics of a concrete are dependant to a large extent on the type of wearing action it is exposed to. Examples of different concrete applications and the wearing action that affects them are: factory floors where iron tyred vehicles are used and a grinding and rubbing action is at work; heavily trafficked road pavements wearing from high pneumatic tyre loadings with a rubbing and plucking action; hydraulic channels where water carries fine sand and stone which act as scouring agents or in spillways where cavitation erodes the surface by the implosion of vapour bubbles; marine work where attrition by waves and shingle is likely to occur.

A wide array of accelerated wear tests, as described by Alexander (3), are available for the evaluation of the wear resistance of the concrete. Each test, however, only recreates one of the wearing actions. Thus for a particular application of the concrete, an appropriate single test(or set of tests) can be applied to predict the long term wear characteristics of the concrete.

For pavement quality concrete, three tests were selected to recreate actual wearing actions. They were:

(1) Wire brush abrasion test - developed by the PCI, which gave a wear pattern "very similar to that produced under road traffic" (4). The wearing action is a picking and brushing one. (wet test)

(2) Sand blast test - ASTM C418(8) that abrades the concrete by sand blasting to imitate impinging and impacting forces. (wet or dry test)

(3) Ball race wear - an Australian test (MA 20)(9) imitating the wear imposed by steel wheel action, developed for concrete paving blocks. (wet test)

The three tests were carried out on the concretes of all three sand to stone ratios with differant percentages PFA and at different strengths. Due to problems associated with the sand blast and ball race tests (discussed later), the three tests were then also carried out on mortar cubes made from the average-sanded mixes with the aggregate larger than 4.75 mm sieved out.

8.1 Wear measurement and comparison

Wear is usually measured by an abrasion index. It is measured from volume removed over a test area. The index is a useful measure of comparison of durability for different concretes for one specific test. It is however, not comparable between different tests as the way that the each test is configured and the wearing load applied, differs significantly.

For the wire brush and sand blast tests the abrasion index required the measurement of volume change. The volume could either be determined by a mass change, converted to a volume or by a direct volume measurement. As the density of the cubes fluctuate (see Fig 8.1), it was decided to opt for a direct volume measurement. This had been done previously by others by the forcing of plasticine into the abraded volume and then weighing the plasticine. Because of the potential of getting air trapped under and inside the plasticine it was decided to use water instead and the Archimedes principle to measure volume.

This was well suited to the wire brush test as it was a test done under wet conditions. The sand blast test was also carried out wet as cubes were tested straight from the curing tanks and not allowed to dry out. This meant that the cubes were saturated. The only problem was to get the cubes at the same "surface wetness" for each volume measurement. The wire brush cubes were already wet but had loose abraded material still on them. The sand blast cubes had been slightly surfaced dried by the air jet. The cubes were thus immersed after abrasion and swirled around in clean water to remove loose material and return the surface to saturation. They were then left to "drip dry" under constant temperature and humidity (no wind) for one minute before they were immersed for volume measurement.

The accuracy of the "drip dry" method to obtain constant "surface wetness" was evaluated by first a drying period of 15 seconds and check the repeatability of volume measurement. It was then also done for a 60 second drying period. Over ten measurements with 15 seconds drying, the variation in mass was 0.2 g and at 60 seconds over ten tests was 0.1 g. Given that the smallest change in mass was 1.5 g it meant accuracies of at worst 13% and 7% respectively. The measurement of volume for all the abrasion tests thus allowed for a 60 second "drip dry" period.

For comparison against the abrasion index, the density or compressive strength at the time of testing can be used. An assessment of the possibility of using density as a comparison was done. It was found, as shown in Fig 8.1, that the concrete varied less in density than in compressive strength.

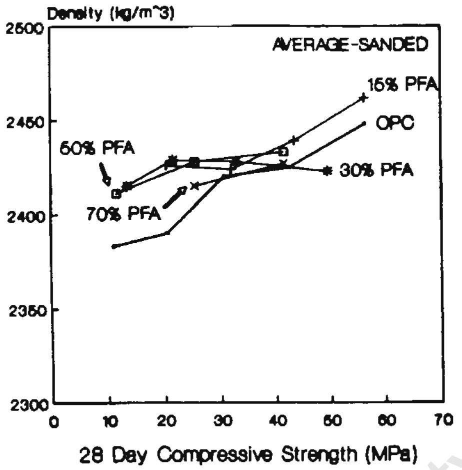


Fig 8.1 Density of concrete vs 28 day compressive strength.

From other investigators (5, 6), experience with abrasion work, a relatively close correlation was found between compressive strength and wear resistance. Because of this and the problems associated with density comparison, and the fact that the concretes were designed for strength, it was decided to compare wear against the compressive strength at the time of testing.

8.2 Factors affecting wear resistance.

From work by Spellman and Ames (6) and Chaplin (7), factors apart from strength of concrete that can affect the wear durability are: air entrainment, water content, surface finishing (timing and method), and curing.

In their work, water content of the mix (as reflected by slump), affected the wear resistance. Increasing the slump from 70 to 150 mm decreased the wear resistance by 15%. The reason given for this decrease was the higher permeability and thus weaker microstructure of the concrete.

The finish applied to the surface of their specimens obviously had an effect on the wear characteristics. Of importance was the timing of the surface finishing. The effect of the bleeding as decribed in 7.5, is of significance. The surface that received a delayed surface finishing ie. after or towards the end of bleed duration, showed more resistance to wear than those finished early. The reason for this improvement was the removal of the potentially weaker surface layer created during bleeding by the excess water. The result was a more dense and harder surface layer.

The curing method used for their concrete had a large influence on the wear resistance. Air curing leads to the premature surface drying which results in a virtual termination of hydration. This arrests any further gain in strength of the surface mortar, so reducing its abrasion resistance. Curing under a layer of polythene or under a variety of surface sealing compounds improved the wear resistance relative to air curing.

Of particular note was the importance of proper curing of their concretes made with blended cements. Poor curing gave more pronounced deterioration in these concretes than on OPC concretes.

Dhir (5), points out that there is however, disagreement between researchers on the overall effects that blended cements and PFA in particular, have on the wear durability of concrete.

8.3 Wire brush abrasion

8.3.1 Wire brush test method

The wire brush test was developed from an NBRI abrasion test (which itself was based on the similar ASTM C779 Procedure A) that used silicon carbide grit as an abrasive medium between a flat revolving disk and the concrete surface. Due to cost and complications an alternative was sought.

Using the same basic equipment, the rig was modified to meet the follow requirements: a drill press able to rotate both clockwise and anticlockwise at 400 rpm; a standard steel wire cup brush of 60 mm diameter; a water supply to control the heat generated by friction; the vertical load of 12.3 kg force (120 N); a method of determining the penetration of the wire brush.

The load of 12.3 kg force was applied by weighting the drill press lever arm. The load was monitored during the test by a electronic scale under the water catch pan. If it moved away from the 12.3 kg mark it was corrected by touching the lever appropriately. The load did not fluctuate very much and was easily held within a range of 0.05 kg force either side of 12.3 kg force. This meant that there was only a potential load fluctuation of 0.4%. the cooling water was allowed to flow out of the catch pan. This however, did not affect the scale reading as within 20 seconds of the water starting to flow at a constant rate, an equilibrium between inflow and outflow was reached. This water and the weight of the cube were tared off on the scale before the brush load was applied.

The depth of penetration was monitored by a pointer arm attached to the drill press lever. Since this was directly connected to the sliding drive shaft, its movement was directly proportional to the penetration of the brush. By taking a suitable length of pointer arm it was possible to read penetration to the nearest 1/10 mm.

The test procedure was to first find the volume of the unabraded cube as described in 8.1. Then the surface of the concrete was abraded with a vertical load of 12.3 kg force on the cup brush for 4 minutes. During this period the rotation of the brush was reversed every 30 seconds to keep the bristles from flattening too much. At each change of rotation, the depth of penetration was noted. At the end of the eight 30 second periods, the volume of the cube was remeasured to find the change that occurred during the test. The diameter of the abraded area was measured and the abrasion index was calculated as the volume removed divided by the circular area of abrasion.

Monitoring of the brush itself was important. Before using a new brush, the bristles needed to be "run in" for two reasons. Firstly the test required that the bristles be not longer than 20 mm. All the brushes had beristles of about 21.5 mm when new and therefore had to be worn down to 20 mm. Secondly, the bristles also needed evening up as their lengths differed slightly.

The limit of wear on the brush was set at 5 mm. Thus when the bristle length reached 15 mm, the brush was replaced with a new one. This limit was necessary, as with the shortening length, the abrading action of a more rigid bristle became more aggressive. Ensuring that the bristles were still long while in service, meant a well controlled rate of abrasion.

There was however, difficulty in the measurement of the diameter of the abraded area. For weak and strong concretes the cross-sections through their abraded areas are as shown in Fig 8.3.

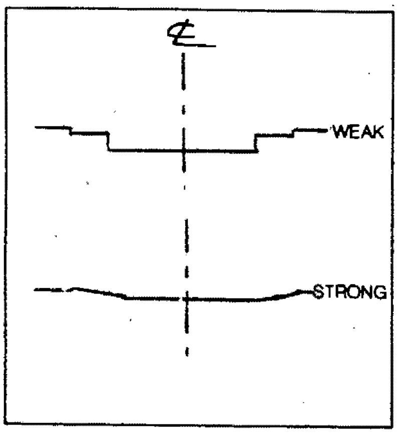


Fig 8.3 Diagrammatic cross-section through weak and strong concretes after wire abrasion.

In weaker concretes the limit of the main abrasion is well defined with only a few edge bristles causing the "outer ring" of minor abrasion. In the stronger concretes this "outer ring" was not at all easy to distinguish from the inner main area of abrasion. Thus the diameters for the stronger concretes were always measured (incorrectly) to be larger than the weaker ones. Since the abrasion index was dependant on the diameter of this area, incorrect indices would have resulted. For this reason and that the same diameter brush was used throughout, it was decided to use only the volume of material removed as a comparison and not the abrasion index.

8.3.2 Extent of wire brush testing

As set out in 3.7, the wire brush testing was carried out in three distinct phases. Firstly the testing was done at 28 days over the full range of the three sand to stone ratios (series 5, series 6, series 7). Secondly it was repeated at 90 days on the under- and over-sanded mixes. Thirdly, due to problems with the sand blast and ball race test (descibed below in 8.4 and 8.5), a series of mortar cubes were prepared and tested for wear resistance by the three abrasion tests.

At each wire brush test one cube from the mix was tested. To start with, the top cast surface and three sides were abraded. However, the testing of the top surface was eventually stopped as wide fluctuations in the results occurred. This was because before stripping the cubes, the tops were often trimmed leaving a surface that was not the true cast surface.

By testing the sides of the cubes, it was guaranteed that each surface was uniform and thus appropriate for testing. Three sides were tested and their wear characteristics averaged to represent the mix.

The monitoring of the depth of penetration of the brush turned out to be relitively pointless. This was due to the bristles of the brush and the stone in the mix.

Firstly, the bristles of the cup brush were manufactured slightly angled for drag in a clockwise rotation. This meant that if the brush was rotating clockwise the bristles lay relatively flat. When rotating anticlockwise, the bristles had a slight lifting action as their initial angle pushed the cup away from the surface. This discrepancy in sequential penetration readings resulted in step-like increases of abrasion.

Secondly, due to the picking and brushing action of the bristles on the mortar around the stone, the bristles were not necessarily able to lie on the mortar surface but were perched on top of the projecting stones.

Thirdly, there was the difference in stone content with different percentages of PFA (see 3.5 and Fig 3.1). This meant that the amount of mortar exposed to abrasion and the amount of stone hindering the bristles from reaching the mortar surface, varied.

These problems are reflected in Fig 8.4. For any given volume removed, the brush has to penetrate deeper between the more densly packed stones of the 50% PFA mixes than for the OPC mixes.

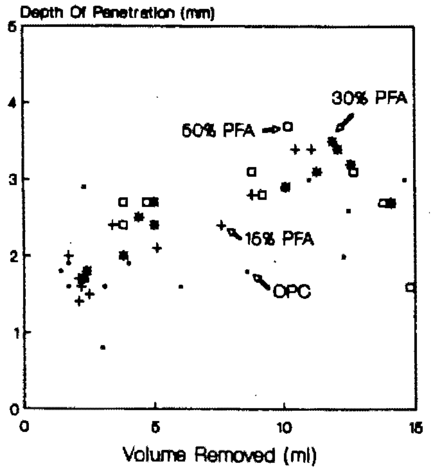


Fig 8.4 Depth of penetration vs volume removed.

8.3.3 Analysis of wire brush results on concrete

Comparison of the volume removed at 28 days by wire abrasion with the 28 day compressive strength is shown for the three sand to stone ratios in Fig 8.5. Photographs of the range of cubes are shown in Fig 8.6. A close up of four cubes taken from across the strength range are shown in Fig 8.7.

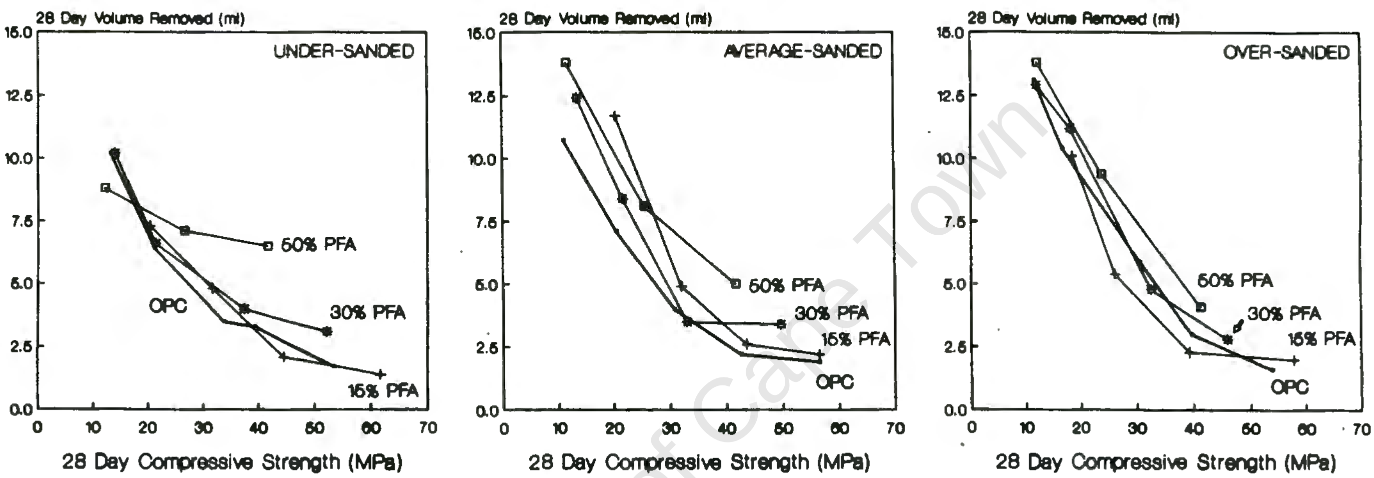


Fig 8.5(a,b,c) 28 day wire abrasion volumes vs 28 day compressive strength. Under-, average- and over-sanded.

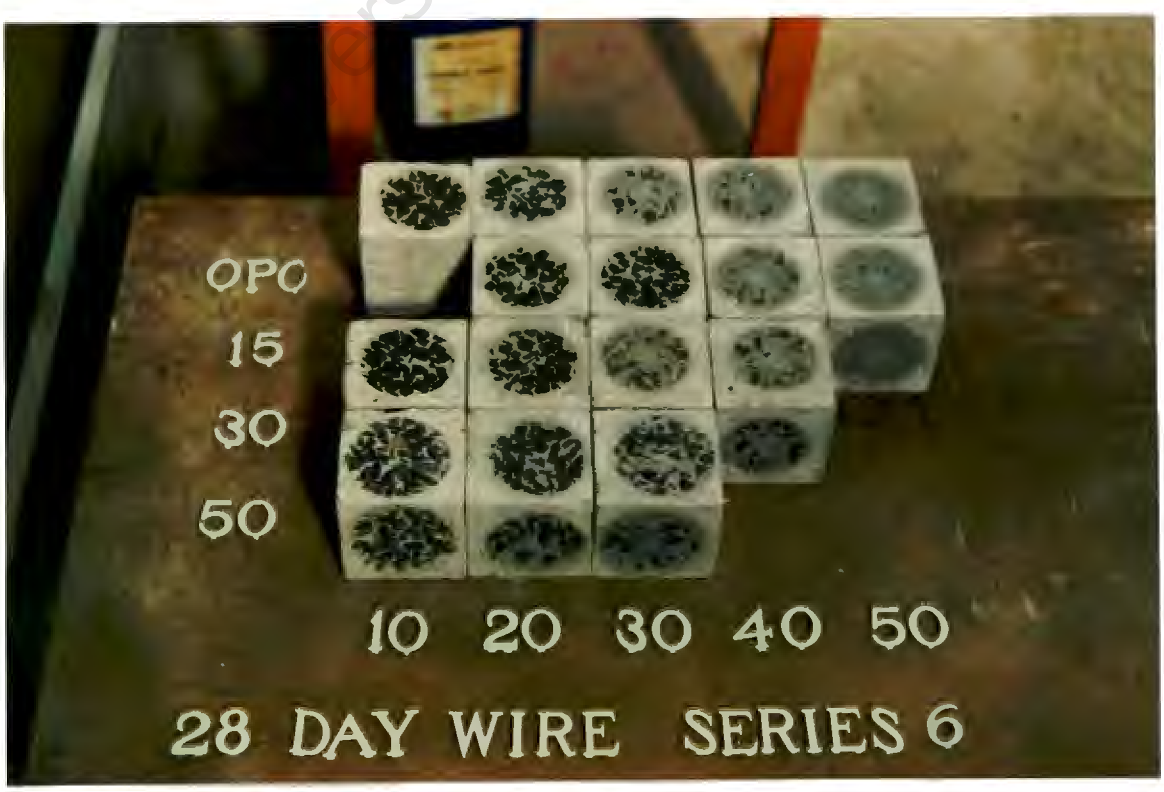


Fig 8.6(a,b,c) 28 day wire abraded cubes. Under-, average- and over-sanded.

Fig 8.7 Close up of wire abraded cubes.

At 28 days the abrasion resistance of all the concretes is very closely related to compressive strength. The stronger the concrete, the less the volume removed by abrasion and thus the more resistant it is. This is as expected.

There is no definite indication of which percentage PFA gives the most resistant concrete. For the under- and average-sanded mixes, the OPC concrete tends to be the most resistant. For the over-sanded mixes the OPC concrete is the second most resistant and the 15% PFA concrete is the strongest.

On the other hand, the 50% PFA mixes tend to be the least resistant with the largest volumes abraded.

The overall trend at 28 days is that the the lower the percentage of PFA in the mix, the more resistant the concrete is.

In an attempt to quantify this trend, consider a given abrasive resistance level and compare the corresponding compressive strengths of the OPC and 50% PFA concretes. It is found that for the equivalent abrasion resistance, the 50% PFA concrete has to have a compressive strength around 10 MPa more than the OPC concrete.

Comparing across the three sand to stone ratios, the trend is that for weaker concretes (less than 35 MPa) the more sand in the mix, the lower the abrasion resistance of the mix. Above 35 MPa the sand to stone ratio has no clear effect.

This is expected due to the effect of the stone resisting the abrading action. In Fig 8.7 the nature of the abrading action can be seen. Where the mortar is weak enough to allow exposure of the stone, the bristles make little impression on the stone but remove the mortar from between the stone. The line of distinction when the stone becomes exposed, can be seen from the photographs in Fig 8.6 to be between the 30 and 40 MPa design strengths.

Although it is not visible in the photograph, the same happened on a micro scale with the cement and very fine sand being brushed away from the larger sand particles leaving them exposed like the stone.

Since the stone particles interfere with the brush action, the further the bristles reach down, the more difficult it is to abrade the mortar. The bristles are continually being deflected and forced upwards when hitting the surrounding stone. In the under-sanded mix, the stone is more closely packed than the stone in the over-sanded mix. Where the additional stone is exposed it is clearly more difficult for the bristles to abrade the mortar. Therefore for weaker concretes of under-sanded mixes, the resistance to abrasion is greater than for over-sanded mixes of the same strengths.

When the cubes for the under- and over-sanded mixes were tested at 90 days some interesting new trends appeared. Fig 8.8 shows the volumes of material abraded at 90 days vs their 90 day compressive strength. Fig 8.9 shows the photographs of the under- and over-sanded mixes after 90 day testing.

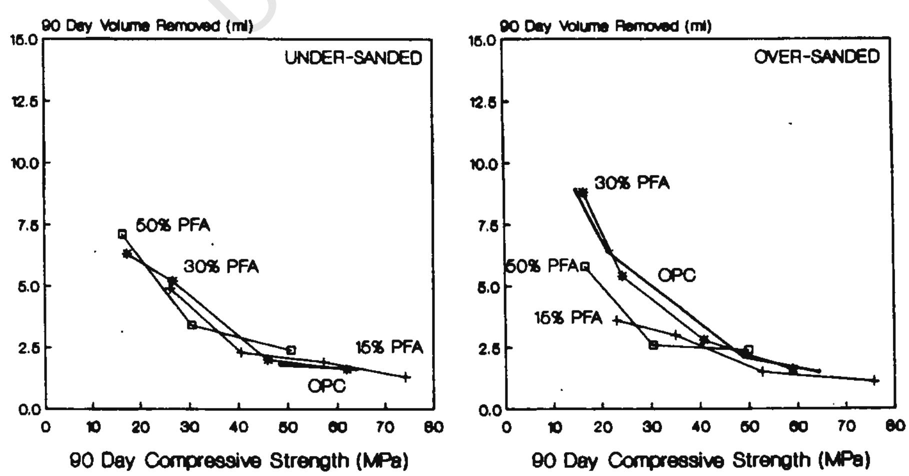
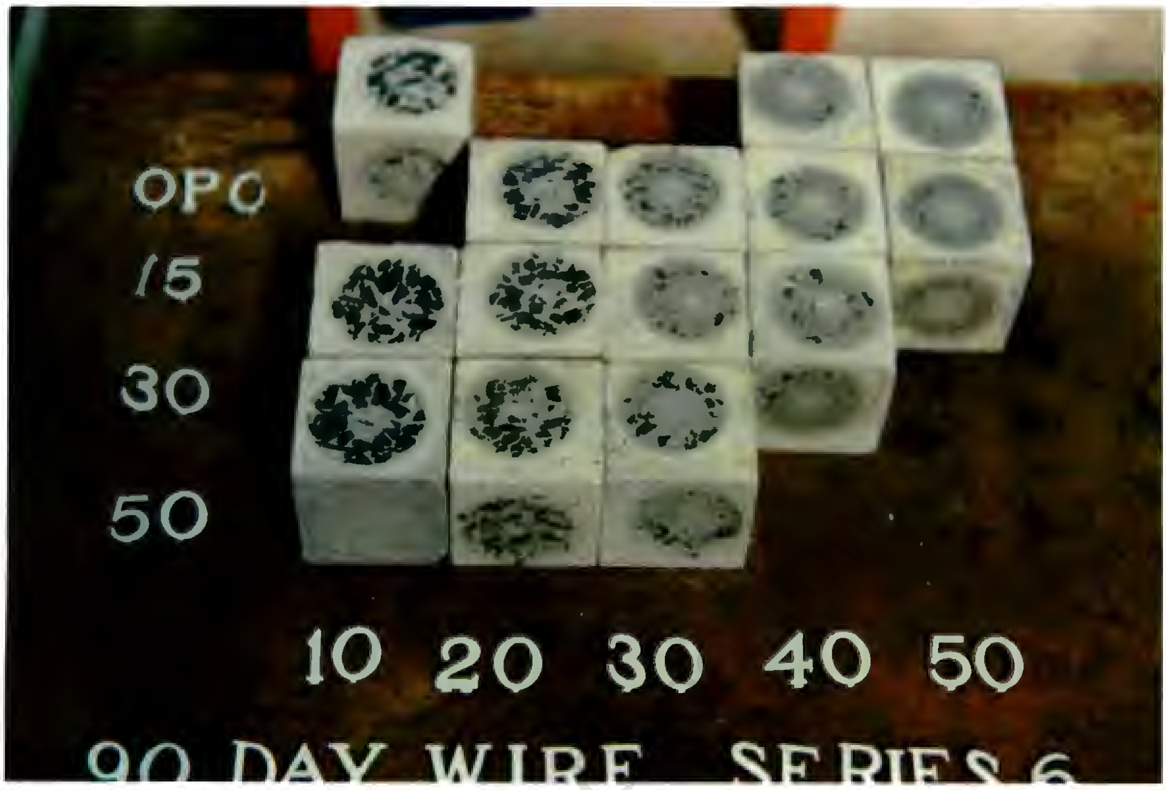


Fig 8.8(a,b) 90 day wire abrasion volumes vs 90 day compressive strength. Under- and over-sanded.



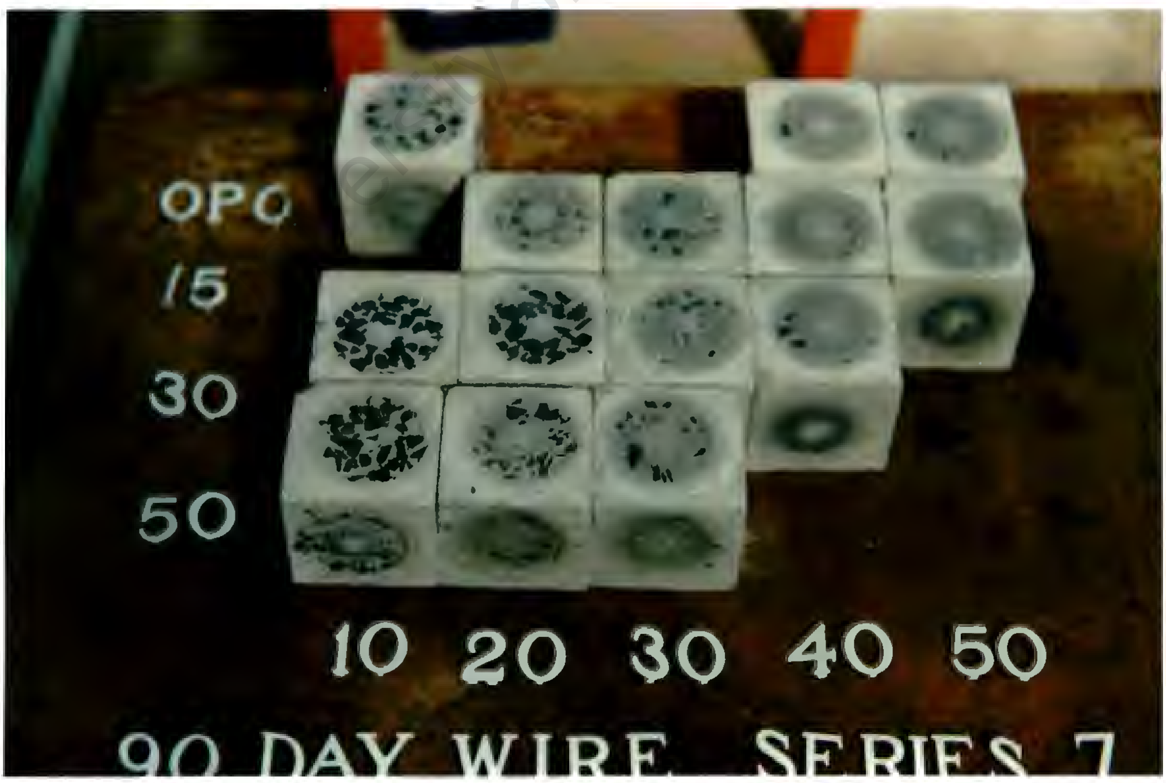


Fig 8.9(a,b) 90 day wire abraded cubes. Under- and over-sanded.

Without examining the graphs in Fig 8.8 too closely the most striking feature is the much narrower range of volumes removed for any 90 day compressive strength. The 50% PFA mix is now the far more resistant to abrasion compared to the other mixes. This is a change of the 28 day result.

This dramatic narrowing of the gap or even reversing of results requires closer comparison of 28 and 90 day trends for each PFA level. These are shown for the over-sanded mixes in Fig 8.10.

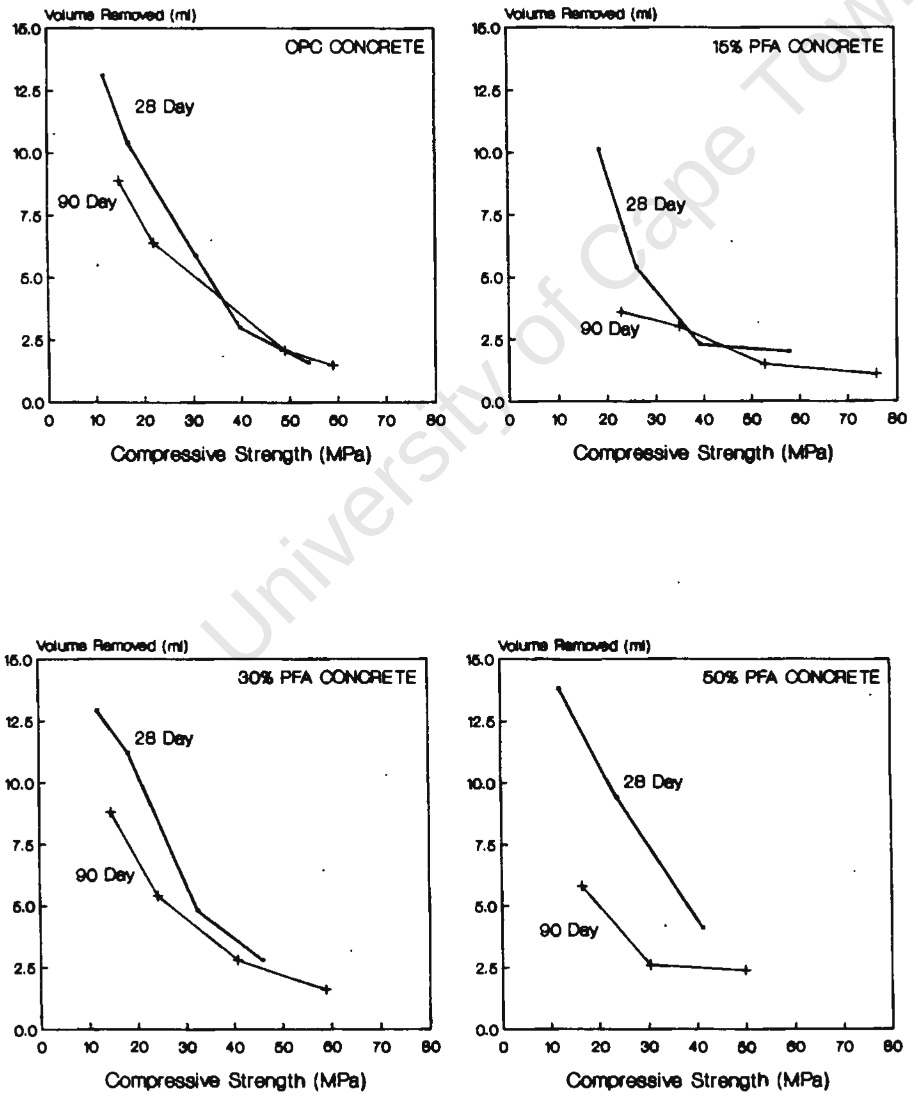


Fig 8.10(a,b,c,d) Comparison of 28 and 90 day wear characteristics.

Starting with the OPC graph (Fig 8.10(a)), the 28 and 90 day trends are very similar. The 90 day results show a slight improvement of abrasion resistance against compressive strength relative to that for 28 days. For the increasing percentages of PFA (Fig 8.10(b,c,d)), there is an increasing improvement in abrasion resistance for the 90 day results over the 28 day results.

If the abrasion resistance was only related to compressive strength, the 28 and 90 day lines for each PFA percentage should have been superimposed. The improvements shown at 90 days indicate a development of abrasion resistance that occurs at a different rate to compressive strength. The abrasion resistance increase is greater than compressive strength increase after 28 days.

This resistance increase trend relative to compressive strength is very similar to the increase of flexural strength relative to compressive strength. This growth in flexural strength was described in 5.8 and 5.9. Not only is the growth similar for abrasion resistance and flexural strength, but the characteristics displayed for the different percentages of PFA are the same.

It is thus reasonable to presume that the abrasion resistance as displayed by the wire brush method can be more closely related to flexural strength than to compressive strength.

However, the only proof of this is that of logic reasoning and comparison of separate, but similar trends. No conclusive numeric or graphic proof is able to be presented because of the limited range at which the 90 day flexural strengths were tested and the low strength range at which the difference between 28 and 90 day was noticeable.

This reasoning is acceptable when the mechanism of the abrading action is closely examined. As the bristle moves across the mortar surface it does not exert a compressive force. Instead it "plucks" particles which implies a failure in tension. The flexural strength is also a measure of tensile strength and thus has somewhat similar characteristics to the wire abrasion resistance.

8.3.4 Analysis of wire brush results on mortar

As mentioned in 8.3.2, a range of average-sanded concrete mixes were made (see 3.7 - mix series 11). The fresh concrete was then sieved to remove the aggregate greater than 4.75 mm. This mortar was then cast in 100 mm cubes. By removing the stone from the mix it was hoped to highlight the extent of the effect that the PFA had on the mortar under abrasion. The results from the wire abrasion tests done at 28 days are shown in Fig 8.11. The values are plotted against the compressive strength of cubes made of the same concrete.

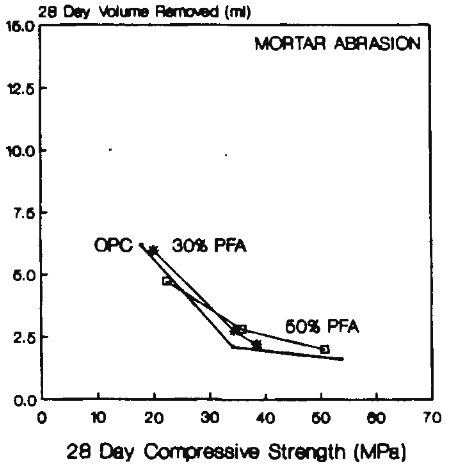


Fig 8.11 28 day mortar wire abrasion volume vs 28 day concrete compressive strength.

Comparing Fig 8.11 with Fig 8.5(b), the results were very similar at compressive strengths higher than 35 MPa. This was because the stone was not exposed in these stronger normal concretes so both sets of cubes were being tested for their mortars only.

Below 35 MPa the stone content affected the results. The variability of the results is far smaller for the mortar than for the concrete. The volumes removed were also far smaller for the mortars, ie. they appeared to be more resistant to abrasion. The reason for this is not clear. It can however, be suggested that the stone interference forces the bristles to move laterally in addition to following the circular path. This additional travel length provides more abrading action. There is no proof of this though.

At 28 days there is no real difference in the abrasion resistances of the mortars for different percentages PFA. However, at 90 days the results based trends for normal concrete (Fig 8.10), the PFA mortars are expected to be far more resistant to abrasion for any given compressive strength than their equivalent OPC mortars.

8.3.5 Conclusion on wire brush abrasion

Since failure of a pavement surface by wear is slow, progressive and not sudden, the long term beneficial effects of the increased abrasion resistance of PFA mixes will extend the life of the pavement.

8.4 Sand blast abrasion

8.4.1 Sand blast test method

The sand blast test as set out in ASTM C418(8)(appendix 1(a)), involves the abrading of the concrete surface by blasting a specified area of it with 600 g of graded sand in one minute with air at a pressure of 4.2 kPa. An abrasion coefficient, Ac, is calculated as the volume abraded divided by the area of abrasion.

The ASTM standard sets out the specifications for the air supply, delivery nozzle and shield dimensions. The sand required for the test is a natural silica sand passing the 841 micron sieve and retained on the 595 micron sieve. The local sand used was the Philippi standard sand which was graded to these specifications.

Work done at WITS University on cement paving blocks used this test, but modifications made it possible to abrade two specimens at the same time. The WITS equipment on loan to UCT is shown in Fig 8.12.

As described in 8.1 the volume of the cube was measured before and after the abrasion test by Archimedes principle. The abrasion coefficient was not used as all the tests were carried out with the size of the test area the same and since the basis of abrasion for the wire brush tests was volume, it would be more sound to keep the desciption the same.

8.4.2 Extent and results of sand blast testing

The sand blast tests were carried out on a limited range from the under- and over-sanded mixes. The extent of the range of mixes is set out in 3.7 (series 9 and series 10).

Fig 8.12 Sand blast test equipment.

At each test of a mix at 28 days, two cubes were abraded. Three sides were abraded on the one cube and two sides on the other. The effect of the sand blast can be seen in Fig 8.13 as the circular hole in the centre of the face. The outer abraded ring is the abrasion from the ball race test, as described later in 8.5.

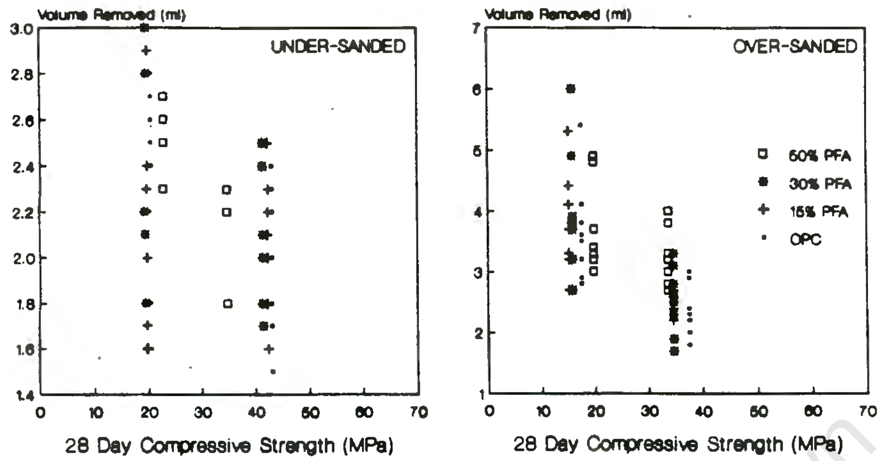


Fig 8.14 Scatter of results for sand blast abrasion of under- and over-sanded concrete mixes.

Fig 8.15 Over-sanded concrete cubes after sand blast and ball race abrasion.

In respect of the wide scatter, the test is very poorly designed. Considering that most real life concretes are made with 19 mm stone, the abrasion area is too small. To be meaningful the specimens need to abraded over a far larger area to reduce the blocking effects that the stone may have.

Because of these area and stone related problems and similar experience with the ball race test (see 8.5), average-sanded mortar cubes were made. An average-sanded fresh concrete was sieved through a 4.75 mm sieve and 100 mm cubes were cast. They were tested at 28 days for abrasion resistance. The results of this mortar sand blasting are plotted against the 28 day concrete compressive strengths in Fig 8.16.

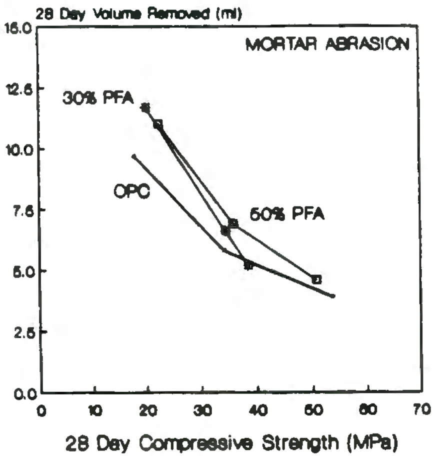


Fig 8.16 Sand blast abrasion volumes for 28 day average-sanded mortars.

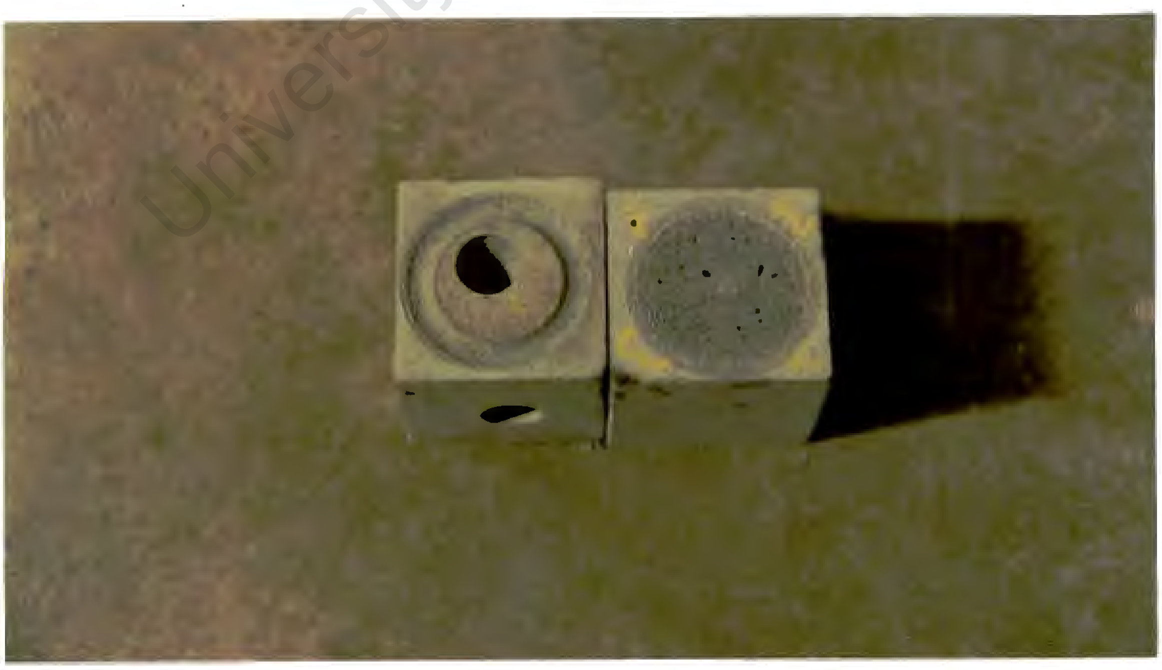


Fig 8.17 Close-up of mortar cubes after all three abrasion tests.

From Fig 8.17, it is shown that with no stone to impede the abrading action of the sand blast, the abrasion forms a deep cone. This shows that abrasion by sand blasting is not a surface test, but rather a measure of the abrasion resistance of the whole concrete body.

The volumes removed, as shown in Fig 8.16, indicate that at 28 days, the PFA concretes are less abrasion resistant. Using the same quantitative measure as previously for the wire abrasion: for any given volume removed, the 50% PFA concrete has to have a 28 day compressive strength of the order of 8 MPa more than the OPC concrete.

8.4.3 Conclusions on sand blast abrasion

The sand blast tests on concrete were inaccurate and inappropriate due to the interference of the stone with the abrading action. The results on 28 day mortars gave similar trends as those presented by the 28 day wire abrasion test.

8.5 Ball race abrasion

8.5.1 Ball race test

The ball race abrasion test was originally developed for the concrete paver industry in Australia. Under the designation of MA 20,[9] it is a test that simulates the abrasion by steel wheels. A thrust race with 12 steel balls of 15.83 mm diameter is driven into the surface under a constant "force" of 14.5 kg at between 1000 and 1050 rpm. The duration limits of the test are set at 5000 revs or 1.5 mm penetration, which ever comes first. (appendix 1(b))

The test was carried out wet with a water supply to the chuck of the machine. The penetration of the race into the concrete was measured by a dial guage fixed to the slide shaft. Timing of the test was done by monitoring the revs with an electronic counter. The full test rig can be seen in Fig 8.18.

Fig 8.18 Ball race abrasion test rig.

The test method required that the ball race be run for three seconds to "seat " itself. It was found however, that during the first few seconds the rate of penetration was the fastest. Since the sides of the cube were uniform it was decided not to seat the ball race, but rather run the test from the first revolution.

The depth of penetration of the ball race was recorded after every 1000 revs.

Since the surface area of the balls that came into contact with the concrete, increased with depth of penetration, the abrasion index calculation was the square root of the revs reached divided by the depth of penetration.

8.5.2 Analysis of ball race abrasion results

The same cubes that were used for the initial sand blast tests on the under- and over-sanded concretes were also used for the ball race tests. Fig 8.13 and Fig 8.15 show the results of the two abrasion tests on the cubes. The ball race test caused the outer abrasion ring.

In this test the stone caused a very wide scatter of results. The range of scatter was so wide that the range of results for the weaker concretes entirely overlapped the results for the stronger concretes.

When even a single ball came into contact with a hard stone, the lifting of the ball to get over the stone and released the abrading pressure off the surrounding mortar. Depending on how much stone was exposed in the narrow path of the race, the wear rate was affected.

Because of the very wide scatter, the tests on the concrete were rejected and ignored. The ball race test is not applicable for testing concrete due to the size and hardness of the stone present.

When the ball race test was applied to the mortars, it was found that the 1.5 mm penetration limit was controlling the test. Thus for these tests the number of revolutions of the race required to reach a penetration of 1.5 mm, was recorded.

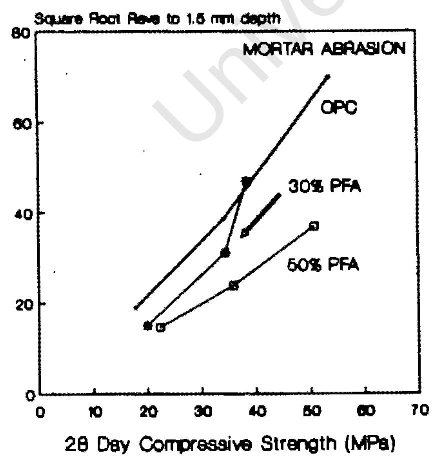


Fig 8.19 Ball race abrasion results for average-sanded mortar.

The abrasion index was calculated as the square root of the revolutions divided by the depth of penetration. Since for all the tests the standard penetration was 1.5 mm, the comparison of results was a plot (Fig 8.19) of square root of revolutions required for 1.5 mm penetration with 28 day concrete compressive strength.

From these results it is clear that the OPC mixes require more revolutions than the PFA mixes to reach a 1.5 mm penetration. This means that at 28 days, the OPC mortars are the most resistant and increasing percentages of PFA reduce the abrasion resistance.

8.5.3 Conclusion on ball race abrasion

The ball race test was not appropriate for concrete but gave satisfactory results for mortars. The trends shown by the test on 28 day mortars are similar to those of the sand blasted mortars and wire brush.

8.6 Conclusions

The following conclusions can be drawn relating to abrasion resistance according to the three test methods used:

(1) Stone can cause wide scatter in abrasion test results if tests are not carried out over a large enough area to reduce effects of their localised random distribution. (Fig 8.14)

(2) At 28 days all three tests showed that the higher the percentage of PFA in the mix, the lower the abrasion resistance of the mortar.

(3) Abrasion resistance of PFA concretes, as measured by wire brush tests, improves after 28 days proportionally more than compressive strength. (Fig 8.10)

(4) Abrasion resistance, as measured by wire brush abrasion, could be more clearly related to flexural strength than to compressive strength of the mix. (8.3.3)

(5) The sand blast test gives a measure of the abrasion resistance of not just the concrete surface but also the inside. (8.4.2, Fig 8.17)

(6) The ball race test is not an appropriate test for concrete but gave adequate results for mortars. (8.5.2)

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CHAPTER 9

DRYING SHRINKAGE

A thorough knowledge of drying shrinkage and thermal properties of a concrete are crucial for the correct positioning of joints to prevent cracking under induced stresses if the concrete is restrained. In prestressed concrete work, allowance must be made for losses in prestressing that the drying shrinkage causes.

Stresses induced by the shrinkage, can be divided into two types. Firstly, an initial shrinkage occurs while the concrete is still plastic due to the reaction of cement and water, loss of water through evaporation, loss of water through the form work and in the case of road pavements and foundations, a loss of water to the underlying gravel or soil. These "plastic shrinkages" can lead to plastic shrinkage cracks. Secondly, shrinkage occurs due to the further drying of the concrete after it has set. This shrinkage may build up stresses and can result in development of shrinkage cracks.

Control of the cracking by insertion or sawing of joints is important for the protection of reinforcement and to maintain surface continuity for good riding quality of the pavement (1).

Of the potential shrinkage that can occur from the time of casting to three years later, 20% occurs while the concrete is still plastic. The other 80% occurs after the concrete has set and is due to the drying shrinkage (2). Because of the high proportion of shrinkage due to the drying of the concrete, an evaluation of this shrinkage potential is necessary.

Although laboratory accelerated shrinkage tests can establish the ultimate possible drying shrinkage, they may not reflect the real life shrinkage, as curing conditions may vary. These conditions can range from allowing the concrete to dry out completely to keeping the concrete saturated. The tests may also not give a true indication of the shrinkage rate, because of the effect that the size of sample has on the rate.

The tests however, give a strong indication of the relative drying shrinkage potentials for different types and strengths of concretes. This is particularly useful for assessing the effect of the Lethabo Field 2 PFA on concrete.

9.1 Drying shrinkage test method

The test method selected to compare and estimate the ultimate drying shrinkage potential of the range of mixes was SABS 1085.[5]

The test required the casting of prisms of 280 x 50 x 50 mm in size, with a metal anvil at each end for the accurate measurement of length. The concrete prisms were cured in the mould for 24 hours in a +90% relative humidity environment and then demoulded. They were then cured in water at 22 degrees celsius for a further six days. They were then removed from the water and surfaced-dried before being measured with a 300 mm micrometer screw gauge. The drying out of the prisms was done in a large oven at 55 degrees celsius. A relative humidity in the range 15% to 25% was maintained in the oven for the full drying period by using silica gel crystals.

Fig 9.1 Drying shrinkage prisms in oven with silica gel, micrometer and wet and dry thermometer.

The prisms were removed from the oven three times a week and allowed to cool for two hours in the temperature room at 24 degrees Celsius. After two hours the prisms had reached room temperature and therefore were no longer changing length due to cooling. They were then measured using the same micrometer screw gauge and returned to the oven for further drying. These measurements were continued until a change of less than 2 microns in two successive measurements was observed.

The ultimate drying shrinkage was calculated as the overall change in length divided by the distance between the inner ends of the anvils.

9.2 Extent of drying shrinkage tests

The range of mixes used for the drying shrinkage was of an average sand to stone ratio. (see 3.7 - series 11) There were nine mixes made with OPC, 30% PFA and 50% PFA with three 28 day design strengths (20, 35 and 50 MPa).

Fulton (3) discusses the effect of aggregate on the shrinkage of the paste (total cements and water). There are two effects: dilution and restraint. Dilution refers to the fact that shrinkage of the concrete will decrease with increasing aggregate concentration, while restraint refers to the fact that the concrete shrinkage will reduce with increasing aggregate stiffness.

Due to the increase in stone content with increasing percentage of PFA in the mix (see 3.5 and Fig 3.1), it was decided to do drying shrinkage tests on mortar as well as on concrete.

From each mix, three mortar prisms and three concrete prisms were cast. The mortar prisms had [illegible] the aggregate larger than 4.75 mm sived out of the concrete.

9.3 Analysis of results

Murdock and Brook (2) maintain that the magnitude of the drying shrinkage is more dependant on the water content of the mix than any other factor. A comparison of the shrinkage with water requirement of the mix is appropriate when only OPC concrete is considered. Due to the large reduction in water requirement that the Lethabo Field 2 PFA gives (see Fig 3.5), this is inappropriate comparison when OPC mixes with PFA mixes. This is shown in Fig 9.2 for the concrete mixes and 9.3 for the mortar mixes.

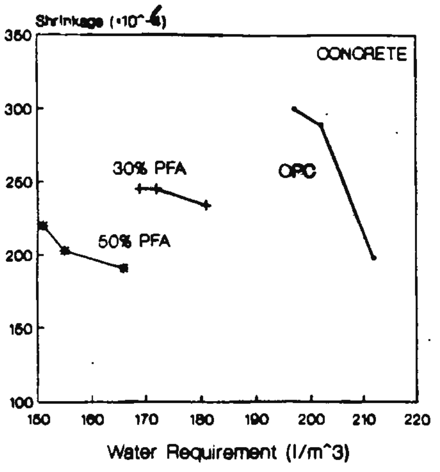

Fig 9.2 Drying shrinkage for concrete vs water requirement of mix.

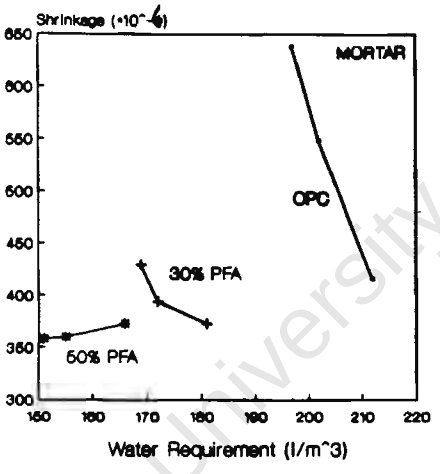

Fig 9.3 Drying shrinkage for mortar vs water requirement of mix.

An alternative comparison is of shrinkage vs the total cements in the mix. The total cements in the PFA mixes, as shown in Fig 3.4, are practically equal across all percentages of PFA for any given 28 day compressive strength. Plots of the ultimate drying shrinkages for the concrete and mortar prisms against total cement content are shown in Fig 9.4 and 9.5 respectively.

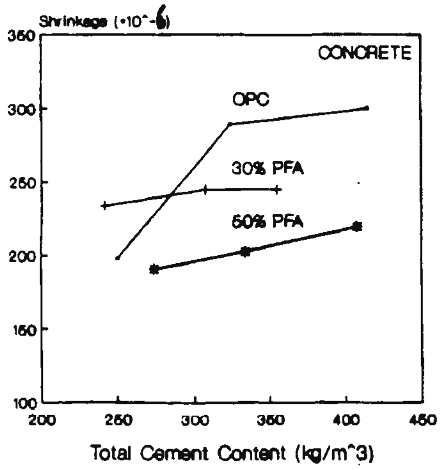

Fig 9.4 Drying shrinkage for concrete vs total cements in mix.

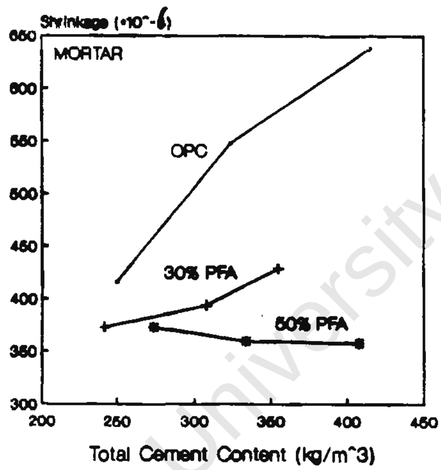

Fig 9.5 Drying shrinkage for mortar vs total cements in mix.

From Fig 9.4 and Fig 9.5 the distinct trend at each PFA level is an increase in shrinkage with increase in total cements, as expected. Referring to Fulton's dilution and restraint of the paste by aggregate, the weaker a mix is, the more diluted the paste is. The weaker mixes have less paste and more aggregate than stronger mixes as shown in Fig 3.4.

The overall effect for both concrete and mortar is that the higher the percentage of PFA in the mix the lower the shrinkage. At 350 kg total cements in the concrete, the 30% PFA mix has 16% less shrinkage and the 50% PFA mix has 30% less than the OPC mix. For the mortars, the 30% PFA mix has 27% less shrinkage and the 50% PFA mix has 35% less than the OPC mix.

In explaining this reduction of drying shrinkage with increasing percentage PFA, Dhir (4) notes that the reason for the reduction has not been fully explained by many other researchers. He suggests that with the addition of PFA and reduction of the water requirement, a finer paste structure is produced. As a result of this, the loss of pore water within the paste system is restricted and consequently the drying shrinkage is reduced.

The effect of the stone on the mix can be seen by the relative magnitudes of the shrinkage of the concrete and mortars. By removing the stone, the shrinkage increases considerably as there is less resistance.

For concrete pavements, the reduction of drying shrinkage in the PFA mixes implies a lower stress build-up. This in turn means a lower potential risk of shrinkage cracks. The lower stress allows the joints in the pavement to be spaced further apart in the PFA concrete than in the OPC concrete.

9.4 Conclusions

In comparing the OPC and PFA drying shrinkages of concrete and mortar, the following can be concluded:

(1) Although water requirement of the mix may be the main controlling factor for drying shrinkage, it is inappropriate when comparing PFA mixes with OPC mixes. (Fig 9.2 and Fig 9.3)

(2) For all PFA and OPC concretes, an increase in total cements gives a higher drying shrinkage. (Fig 9.4)

(3) Stone in the concrete reduces the potential drying shrinkage of the mortar by restraining the concrete. (Fig 9.4 vs Fig 9.5)

(4) Increasing the percentage of PFA in the mix decreases the drying shrinkage of both the concrete and the the mortar. (Fig 9.4 and Fig 9.5)

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CHAPTER 10

ELASTIC MODULUS AND CREEP

Although the elastic modulus and creep are not directly required as design considerations for pavement concrete, a limited study of the effect that the Lethabo Field 2 PFA has on these concrete properties was carried out.

From the outset, it should be noted that the results on the elastic and creep properties gave very limited amounts of information. Firstly, only a limited number of mixes were made and were spread too thin across the possible range of variables required to cover the subject.

Secondly, only three cylinders from each mix were cast. This gave one cylinder for creep loading, the second to act as a companion shrinkage cylinder and the third cylinder for the E (elastic modulus) evaluation. If more cylinders from each mix had been used for each test, it would have meant accurate mean values, rather than single sample values.

Thirdly, the creep test had to be terminated after about three months. At this stage the cylinders were still showing high creep rates. Although much is known about the creep of OPC concrete, little is known of the effect PFA on long term creep trends, thus long term trends could not be predicted.

10.1 Elastic Modulus

10.1.1 Factors affecting the elastic modulus

The elastic modulus (E) of a concrete is proportional primarily to the compressive strength. Work by Davies (as quoted by Fulton (1)) on E values of South African concretes showed a best fit curve that related E to the square root of compressive strength. There was however, a large range of values lying up to 20% on either side of this best fit line.

There are other factors which affect the E to varying degrees. These factors according to Fulton are: the coarse aggregate proportion which gives rigidity to the mix; the type of coarse aggregate; the cement water ratio; the degree of hydration; the moisture content; the temperature of the concrete and the voids in the mix.

Dhir (2) states that from several investigations on a wide range of types of PFA, that there was no significant effect on the elastic modulus of concrete.

10.1.2 Extent of elastic modulus testing

The testing for elastic modulus was done for the three sand to stone ratios. For each ratio, mixes of OPC, 15% PFA, 30% PFA, 50% PFA and of two design compressive strengths were prepared. (see 3.7 - series 8, series 9, series 10).

The tests were done at 28 days for all the mixes described above. At 90 days, only the average-sanded mixes were tested.

10.1.3 Test method for elastic modulus

The method used was close to that described in BS 1881 Part 5 1970. The concrete cylinder from each mix was 103 mm diameter and 300 mm in length. Its top surface had been capped to give give parallel ends for even compression under load. The cylinder was removed from its water curing the day before the test and allowed to surface dry. The targets for measurement were then glued to the cylindrical surface which had been brushed clean. The targets were spaced 100 mm apart in the middle third of the cylinder length. Two sets were glued diametrically opposite each other.

The loading was done with a ball and socket joint placed between the cylinder and the loading plate at the upper end. This was to accomodate any out-of-squareness and subsequent uneven strains that may be imparted to the cylinder in loading. The load range was from a lower value of 10 kN to an upper value of 60 kN. This meant a stress range of 6.00 MPa. The maximum stress was 7.2 MPa which meant that for all the the concretes, the stress was still in the linear portion of the stress strain curve.

The cylinders were "exercised" and bedded in to reduce the initial small amount of creep that may occur. This exercising involved the loading of the cylinder to 60 kN, holding it there for a minute and then releasing it to 10 kN. This cycle was repeated three or four times till the strain readings were repeatable. If the strain readings were repeatable but there was a difference in strain between the targets on opposite sides, the cylinder was unloaded. It was repositioned and re-exercised till the strain readings on opposite sides were within 10% of each other. The strain was then measured for the 6.00 MPa stress range and repeated four times. A mean of the four was taken and the elastic modulus calculatedas the stress change / strain change.

10.1.4 Analysis of elastic modulus results

Fig 10.1 shows the relationship between elastic modulus and compressive strength at 28 days. Fig 10.2 shows the relationship for the average-sanded mixes at 90 days.

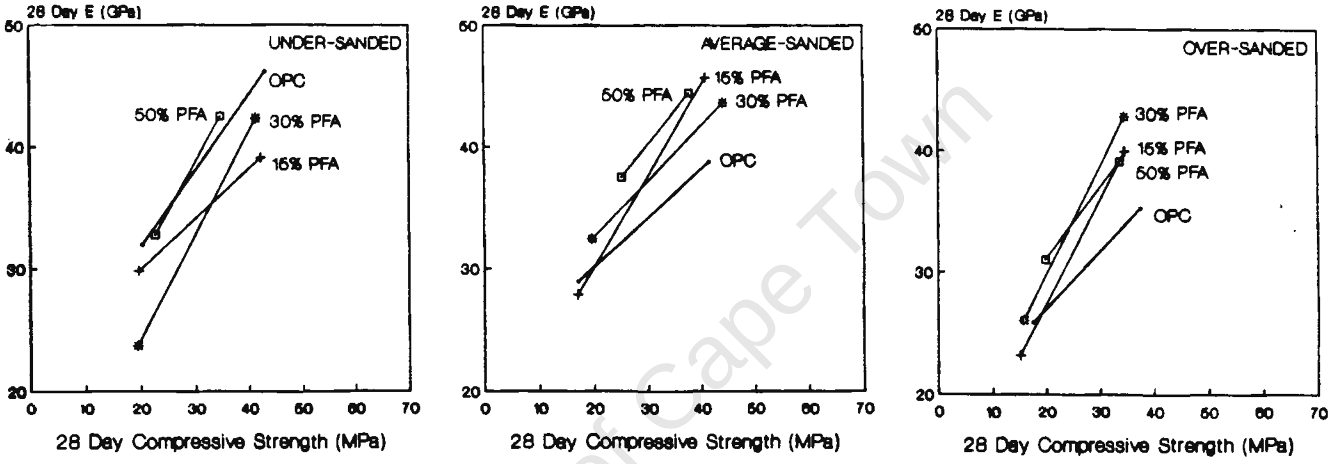

Fig 10.1(a,b,c) 28 day elastic modulus. Under-, average- and under-sanded.

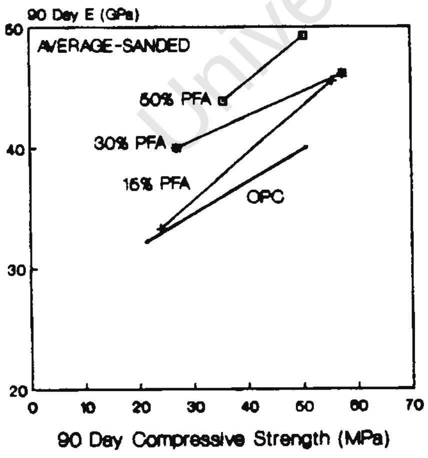

Fig 10.2 90 day elastic modulus for average-sanded mixes.

At all percentages of PFA there is the expected increase in modulus of elasticity with increasing compressive strength.

The effect that the PFA has on the elastic modulus is not clearly defined. For the average- and over-sanded mixes the trend is relatively clear with increasing PFA giving a significant increase in elastic modulus (approx. 10% increase for 30% PFA and 20% increase for 50% PFA). For the under-sanded mix the trend is not the same. The OPC and the 50% PFA mixes have similar elastic modulii and the 15% and 30% have lower elastic modulii. The general trend at 28 days is that an increase in PFA, increases the elastic modulus.

Across the three sand to stone ratios it appears that the coarse aggregate proportion has only a small effect on the elastic modulus. Thus the increase in the elastic modulus with increasing PFA might not be ascribed only to the additional stone content in the PFA mixes but also to the effect of the PFA itself.

Comparing the average-sanded elastic modulii at 28 and 90 days, the trend of increasing elastic modulus with increasing PFA content is the same. With the increase in compressive strengths with time, the elastic modulii have also increased as expected.

From these observations there are two important advantages that the PFA mixes have over the OPC mixes. In structural concrete it is usually beneficial to have a high elastic modulus as this limits deflection under load. Firstly, the PFA mixes give higher elastic modulii at any given compressive strength and secondly the PFA mixes continue to gain proportionally more strength with time than the OPC mixes (see Fig 4.9). This combination implies that in the long run, the elastic modulus for PFA mixes will be substantially higher than for an OPC concrete of the same 28 day compressive strength.

10.2 Creep

A full knowledge of the potential creep that a concrete will develop with time is of importance for calculating the long term stresses and estimating deflections and crack widths.

The resilience that creep gives to concrete is also sometimes of benefit as it can relieve stresses caused by differential structural movement and restrained shrinkage.

10.2.1 Factors affecting creep

The mechanisms that control the creep of a concrete are not fully understood. Several theoretical mechanisms have been suggested to explain the creep and creep related properties of concrete. It is beyond the scope of this work to expound on these theories, other than to outline the general idea of creep and point out the factors that affect it.

In simplistic terms, creep is related to the movement of water within the hardened cement paste of the concrete due to stress induced by some form of load. In loading the hardened cement past and relocating the water within it, new bonds are formed between cement gel particles that were previously separated by the water. This results in a plastic strain known as creep.

Because of the water movement, creep is thus also related to the shrinkage of concrete during drying out. A distinction is drawn between basic creep and drying creep. Basic creep is the creep that occurs due to an applied load with no drying taking place. Total creep is the sum of the basic creep and drying creep.

Creep in concrete occurs at all stress levels. This means that creep strain is stress and time related. Thus the primary factors that cause the creep in concrete are the stress under which it is loaded and the duration of the load.

The strength and stiffness (elastic modulus) of the concrete control the creep. Both of these factors are affected by the cement to water ratio of the mix (see Chapter 4 and 10.1 respectively). Increases in strength and stiffness, reduce the potential for creep.

A second time related factor which affects creep, is the age at loading. The creep is reduced by having a more mature and stronger concrete when loaded.

The curing conditions have a two-fold effect. Firstly, with dryer curing conditions, the shrinkage of the concrete due to moisture loss is higher than for wet curing. This increases the drying creep. Secondly, water is required for the long term hydration of the cement paste so that the concrete can carry on gaining strength. If the drying conditions are severe enough to effect the strength gain of the concrete, the obvious result is an increase in basic creep.

The aggregate properties can also effect creep. Increasing the aggregate proportion in the mix tends to reduce the creep by providing rigidity to the concrete. If however the aggregate itself has a elastic modulus below 70 GPa (1), this can increase the creep. This is because the aggregate with a low elastic modulus offers less restraint to the creep of the hardened cement paste.

Dhir (2) indicates that PFA in a concrete mix tends to reduce the creep. He suggests that the mechanism for this is the reduction in water demand that PFA allows. Since creep is linked to the relocation of water within the cement paste, the reduction of water in the paste and the higher total cements to water ratio (see chapters 3 an 4) mean that there is less potential for creep.

10.2.2 Extent of creep testing

The creep tests were done on the same set of mixes that were used for the elastic modulus tests. (see 10.1.2).

From each mix only one cylinder was tested for creep and one was used as a companion cylinder to monitor drying shrinkage.

The cylinders were only loaded at one age - 28 days.

Due to the failure of a seal on the rig containing the two 50% PFA over-sanded mix cylinders and there being no other rigs to place them in, these tests were aborted.

10.2.3 Creep test method

The cylinders for creep and shrinkage were capped to give parallel ends and had targets glued to their cylindrical surfaces. The shrinkage cylinder had two sets of targets set at 180 degrees apart. The creep cylinder had three sets at 120 degrees apart. Each set had the two targets spaced 100 mm apart in the central third of the cylinder length.

The creep rigs, as shown in Fig 10.3, contain two cylinders in tandem per rig.

A cylinder from each of two mixes was loaded into each rig. Initial unloaded measurements for each of the cylinders was taken before the rigs were pressurised to 5.55 MPa which is less than one third of the lowest cylinder strength. A second set of measurements were made on the creep cylinders immediately after loading to determine the elastic strain. An inital set of measurements was also made on the two companion shrinkage cylinders.

Fig 10.3 Creep rigs.

Three further measurements were taken on the first day of loading. After this the cylinders were measured every day for the first week, every week for the first month and then at two weekly intervals. These measurements continued till the concrete had been under constant load for three months.

At each measurement for the cylinders, the total creep strain was calculated. This was done by first taking the mean of the creep measurements from the three target sets. The mean of the two shrinkage measurements was then deducted from the mean creep measurements to give the total creep strain.

10.2.4 Analysis of creep results

Because the tests were ended while the creep rate was still considerable and not knowing the long term creep trends for PFA concretes, no long term prediction of creep could be made. To compare the mixes, the total creep strain at 100 days after loading was taken to represent each mix as shown below.

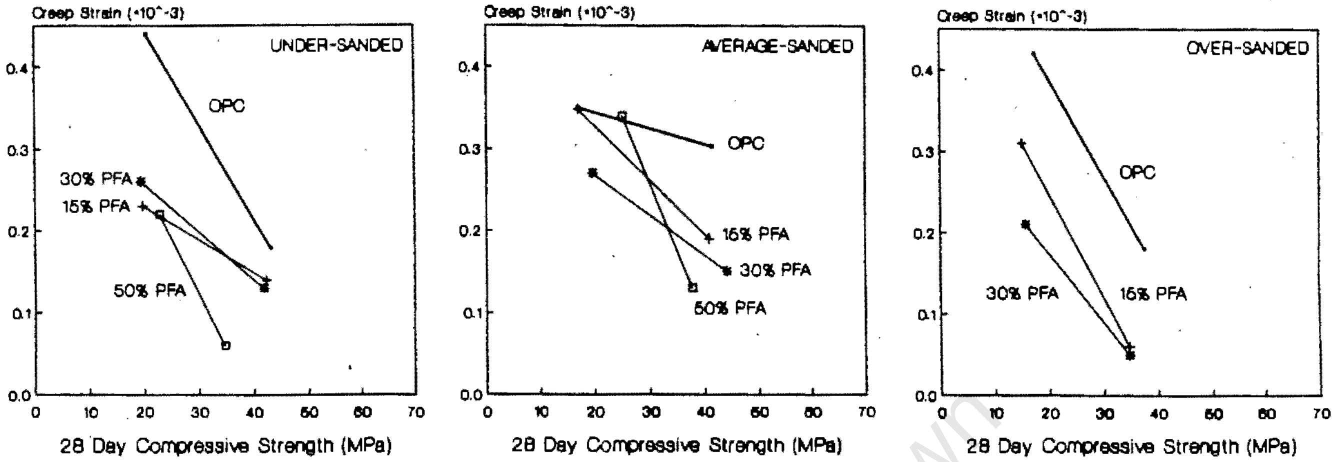

Fig 10.4(a,b,c) Total creep strain vs 28 day compressive strength. Under-, average- and over-sanded.

Fig 4.10 shows the expected trend of reduction in creep strain with increasing strength. It also shows the very significant trend of reduction in creep strain with increasing percentage PFA. A single value at 50% PFA for the average-sanded seems to be an outlier.

Explanation of the results is not possible due to the wide scatter. However, to give some idea of creep strain reduction of PFA mixes compared to OPC mixes they show a 20% reduction for 15% PFA; a 30% reduction for 30% PFA and a 50% reduction for 50% PFA.

Comparing across the three sand to stone ratios, no effect of the changing coarse aggregate proportion was detected due to the wide scatter of results.

To eliminate the various stress levels at which the concrete can be loaded, the specific creep and creep factor are used. These two values for a particular concrete, are established knowing the long term total creep. Since the tests were stopped early, the 100 days after loading values were used.

The specific creep strain is defined as

$$Cc = \varepsilon_c / \sigma$$

where σ is the stress from the load and ε_c is the total creep strain.
The creep factor is defined as

$$\phi = \varepsilon_c / \varepsilon_e$$

where ε_e is the initial elastic strain exhibited when the concrete is loaded. Using the fact that elastic strain is stress divided by elastic modulus (E), the creep factor can also be defined as

$$\phi = Cc * E$$

Thus combining the creep data from Fig 10.4 and the elastic modulus data from Fig 10.1, the creep factors can be calculated. They are shown in Fig 10.5.

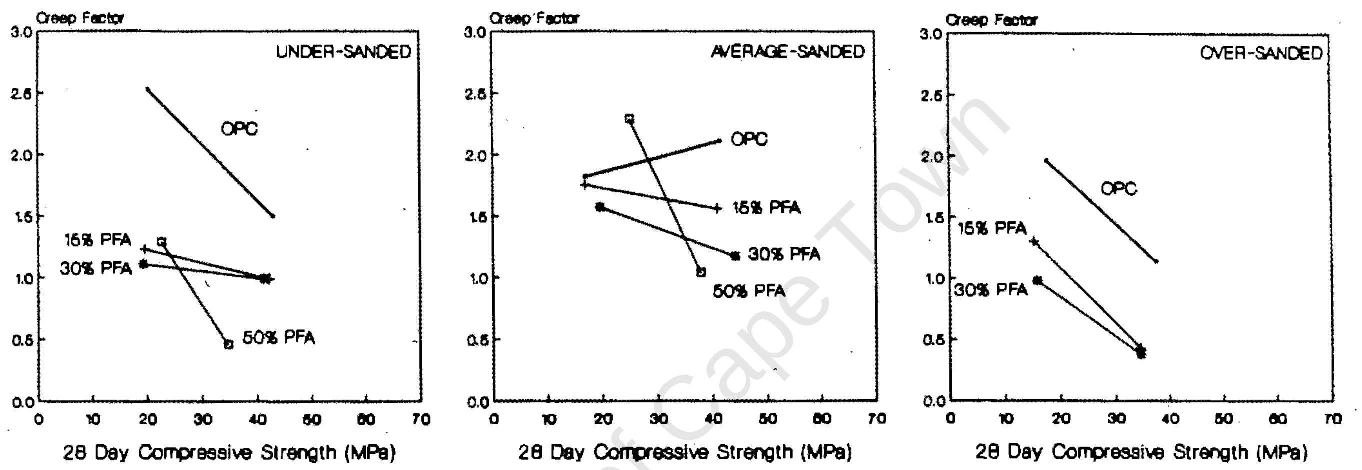

Fig 10.5(a,b,c) Creep factors vs 28 day compressive strength. Under-, average- and over-sanded.

From Fig 10.5 the changing coarse aggregate proportion does not seem to have an effect on the creep factor. However, the overall trend is a reduction in creep factor with increasing percentage of PFA.

These reductions in creep factor are very substantial. Therefore the inclusion of PFA into the mix is of benefit to structural concrete with the reduction of creep strain and creep factor.

10.3 Conclusions

In comparing the PFA and OPC concrete for elastic modulus and creep, the following can be concluded:

(1) Elastic modulus increases with increase in compressive strength. (Fig 10.1, Fig 10.2)

(2) For any given compressive strength, an increase in the percentage of PFA gives an increase in elastic modulus. (Fig 10.1, Fig 10.2)

(3) Creep strain for concrete loaded at 28 days is significantly reduced with use of PFA. (Fig 10.4)

(4) The creep factor reduces with increasing percentages of PFA. (Fig 10.5)

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CHAPTER 11

REVIEW ON PAVEMENT QUALITY CONCRETE PROPERTIES MADE WITH LETHABO FIELD 2 PFA

A review of the individual findings and conclusions is required to relate the work to pavement quality concrete and the effect that Lethabo Field 2 PFA has on it in particular.

11.1 Materials

The main constituents were representative of materials commonly used in concrete in the Western Cape (Chapter 2). The Lethabo Field 2 PFA had physical and chemical characteristics similar to those expected for the classified Lethabo PFA when available in the near future. By the measurement of percentage ash retained on the 45 micron sieve, a good indication of the activity of the PFA was obtained. This Lethabo Field 2 PFA was finer (8.1% retained) than the currently available Matla PFA (12.5% retained). This predicted a better cementing ability of the Lethabo Field 2 PFA.

11.2 Mix proportions

This good cementing ability of the Lethabo Field 2 PFA was borne out by the proportions of the materials required in each of the mixes developed (Chapter 3). Because of the substantialy lower water requirements of the PFA mixes, compared to the OPC mixes, equivalent total cements was achieved for the PFA and OPC mixes for equal 28 day strength.

Typical pavement concretes are usually under-sanded mixes to achieve skid resistance and of 30 to 35 MPa 28 day design strength.

11.3 Fresh properties

With mixes of equivalent 28 day strengths, having equivalent total cements, the fresh properties (Chapter 7) showed some of the effects of the reduced water requirement for the PFA mixes. For pavement quality concrete probably the most critical fresh property is the bleeding, because of the detrimental effect on the top surface of the concrete. With heavy bleeding the surface can be come weak and will detrimentally affect its wear characteristics.

The PFA mixes bled less volume of water and also bled for a longer period than equivalent OPC mixes. The combination of these is beneficial to a pavement concrete. The lower bleed volume means less chance of surface weakening, the longer bleed duration, in addition, means that the evaporation will be able to remove most of the bleed water.

The slower setting time of the PFA concretes in pavements may delay the timing of the surface finish application, but the timing may be less critical.

11.4 Loading characteristics of pavements

In considering the properties of pavement concrete after it has set, the loading characteristics must be allowed for.

Concrete gains strength with time and therefore pavements cannot take load before the strength has developed to a desired level. During construction a concrete should only be exposed to traffic loading after the development of the desired level of strength. Industrial pavements are usually only loaded when fully completed. There are however occasions when the concrete pavement will have to be opened to traffic soon after casting and thus early strength (7 days or less) is of importance.

This means that although the concrete design is based on 28 day strengths, short term and long term strengths should also be considered.

These considerations are of importance when examining long term progressive failures such as cracking and wear. The long term characteristics (90 days or more) give a better reflection of the true durability of the concrete.

11.5 Compressive strength

The compressive strength (Chapter 4) is a well accepted standard for comparison of concretes.

PFA concretes gain compressive strength slower than the OPC concretes before 28 days, but after 28 days the PFA concretes gain additional compressive strength compared to their OPC equivalents. By using an PFA concrete of the same 28 day compressive strength as an OPC concrete in a pavement, a loss of early strength, but a higher long term strength is achieved.

11.6 Flexural strength

The flexural strength, is the critical strength in a concrete pavement. Like the compressive strength, the development of flexural strength in the PFA concretes is slower than in OPC concretes. This highlights the potential damage that accidental early loading can cause to the pavements.

This slow start is made up for by substantial gains in long term flexural strength for the PFA concretes over those for OPC concretes. These gains are best shown in the materials cost comparison below.

11.7 Materials cost

For pavement concrete, the comparison of properties such as compressive and flexural strength are shown up best by using economic comparisons. (Chapter 6)

The costing of each mix was done for the materials only. This was because any estimate of such items as storage and handling of another material was prone to wide fluctuations. Some relevant factors are the use of ready-mixed or site-mixed concrete, the use of preblended or site blended cements.

The materials costs were for June 1988 for Western Cape materials. This locality was of particular importance when costing the PFA. In the Transvaal most PFA is about one third the cost of OPC. Due to railage costs, in the Western Cape its cost is only 20% lower than OPC. Thus any cost savings or small losses shown by PFA concretes in the Western Cape would have translated into substantial savings if the Transvaal cost of PFA was allowed for.

For the equivalent flexural strength at 28 days, the PFA concretes are more expensive than the OPC concretes. At 90 days, the trend is reversed due to the substantial additional gains in flexural strength of the PFA concretes over the OPC concretes.

Thus for pavement concretes, the inclusion of around 30% PFA gives substantial cost savings for a given 28 day design compressive strength.

11.8 Wear resistance

Due to the wearing of a concrete surface being a long term reduction in quality, the 90 day trends should be taken as controlling rather than the 28 day results.

Under the wire brush abrasion test, physical wear characteristics were similar to real life wear from pneumatic tyres. At 28 days the OPC concrete was the most resistant, with a decrease in resistance with increase in percentage PFA. Similar trends were noticed for the wearing action under steel ball bearings and by sand blasting.

At 90 days the wear trend was completely reversed. The OPC was the least resistant and by increasing the percentage of PFA (up to 50% PFA tested), the wear durability improved substantially.

Thus by including PFA into the mix of a pavement concrete the long term wear durability of the pavement is improved for equivalant 28 day compressive strength.

11.9 Drying shrinkage and creep

The drying shrinkage of a pavement concrete is important as it can cause mainly transverse cracking and thereby drastically reduce both the riding quality of the surface as well as the life span of the pavement by breaking it up.

In tests carried out on both concretes and mortars, the drying shrinkage was drastically reduced by the inclusion of PFA in the mix. This implies lower stress build-up due to restrained shrinkage and means less chance of cracks developing. Thus the quality of the concrete pavement is improved.

Although creep does not greatly affect pavement concrete, it was found that PFA reduces the creep potential and increases the elastic modulus of a concrete. Both of these imply a concrete that deforms less under load.

11.10 Further work to be done

Several areas of further work are required on concrete pavements. Wear durability could be tested by methods that simulate the steel wheel wear better than the ball race test. This wear durability should concentrate on 90 day testing of samples rather than 28 day testing.

With regard to flexural strength, a study of the strength characteristics under cylic and impact loading would possibly gave better simulation of real traffic loading.

Many of the tests should also be carried out with concrete samples exposed to dry curing. This could be of major consequence to PFA concretes which rely on continued water presence for long term hydration. In particular the surface properties such as wear durability, may be affected.

Although not relevant to Western Cape concretes, the effect of freezing and thawing should be considered in the testing of pavement concretes.

11.11 Conclusion

The use of Lethabo Field 2 PFA into pavement concrete improves most properties of the concrete. Because of slower pozzolanic cementing action of the Lethabo Field 2 PFA and the subsequent long term substantial strength development, most of the few short term deficiencies of PFA pavement concretes are turned to advantages over the long term.

From the scope of tests carried out and mixes developed, the use of Lethabo Field 2 PFA up to about 30 percent, is of benefit to the concrete for use in pavements.

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LITERATURE SURVEY AND COMMENTS

Subsequent to submission and examination both the internal and external examiners felt that the literature survey was not up to requirements and lacked current journal references.

The original literature search was done in four main areas. The first was a Dialog search done through the UCT library of large data bases in America. This turned up a substantial number of potential references on very general aspects of PFA, but on refining the search to material relevant to the thesis topic only, six references were of use. These however, were just reports on the insitu results of PFA concrete pavements.

The second aspect of the search was through material held in the UCT library. From this source Fulton's Concrete Tecknology and Popovic's Fundamentals of Portland Cement Concrete: A Quantitive Approach; Vol 1 Fresh Concrete and a number of other minor references were obtained.

The third part of the search was through papers and books that people owned privately. Material was obtained in particular from Des O'Neill, Mike de Kock and Mark Alexander. In particular the chapter by Ravindra Dhir in the book Cement Replacement Materials, the third volume in the series Concrete Technology and Design was of benefit.

The fourth part was a search through the material at the PCI libraries in Cape Town and Halfway House.

Following the examination of the thesis a number of further conference proceedings were examined for relevant information. The journals of the American Concrete Institute (ACI), the British Concrete Society (Concrete), the American Society for Testing and Materials and the Magazine of Concrete Research were searched and although a few articles and papers were found that were of some relevance to the work of the thesis, there was no new information that could be added to the thesis.

In general the author found that a large number of articles have been written on the general subject of PFA and its application. The problem with these articles was that in attempting to descibe a wide a range of PFAs and their applications, they did not cover anything in particular detail and they tended to repeat each other.

At the other end of the scale there were quite a few articles that were at the same level as the research of the author. These articles tended to report results with insubstantial explanations. These type of references helped in directing the reseach approach but did not explain problems exposed by results of the author's research. Many of the explanations were based on Fulton's and Dhir's work.

Appendix 1 (a)

Sand Blast Abrasion
ASTM C418

APPROVED AS
USA STANDARD A37.124-196
BY USA STANDARDS INSTITU
UDC 666.97.620.1

Standard Method of Test for ABRASION RESISTANCE OF CONCRETE[1]

ASTM Designation: C 418 – 68

This Standard of the American Society for Testing and Materials is issued under the fixed designation C 418; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval.

Scope

1. This method of test is intended for determining the abrasion resistance characteristics of concrete by subjecting it to the impingment of air-driven silica sand. It is intended for use as a basis for the development of informed judgment.

Apparatus

2. (*a*) *Scales.*—The scale shall have a capacity of 5000 g or more. The permissible variation at a load of 5000 g shall be ±5 g.

(*b*) *Weights.*—The permissible variations on weights used in weighing shall be as prescribed in Table I. The permissible variations on new weights shall be one half of the values given in Table I.

(*c*) *Sand Blast-Apparatus.*—The sand blast apparatus shall consist of an injector type gun. The gun shall have a high-velocity air jet fed by a suitably controlled rate of flow for the abrasive material. The nozzle shall consist of cold-rolled bar stock, 38 mm (1½ in.) long, drilled to 6.35 ± 0.02 mm (0.250 ± 0.001 in.) through the center. The walls of the nozzle shall have a 45 deg bevel on the inside at the upper end. A compressed a supply of approximately 7 kgf/cm² (psi) shall be available and equipped wi a pressure control device. Provision shal be made to collect the spent abrasi and dust. Suitable jigs and clamps shal

TABLE I.—PERMISSIBLE VARIATIO ON WEIGHTS.

Weight, g	Permissible Variations on Weights in
1000	±0.50
500	±0.35
300	±0.30
250	±0.25
200	±0.20

be provided to hold the test specimen in fixed position with relation to the dis charge end of the nozzle.

(*d*) *Shield.*—The metal shield shall be made from 1.90 to 0.91 mm (14 to gage) galvanized or similar stock least 152 mm (6 in.) square or in diam eter if circular having an opening 28.7 0.25 mm (1.13 ± 0.01 in.) in diameter the center. Suitable clamps shall provided to hold the shield fixed on th test specimen during the test.

NOTE 1.—Opening 28.7 mm. in diameter equivalent to 6.45 sq cm. or 1 sq in.

(*e*) *Abrasive.*—The abrasive shall natural silica sand from Ottawa, Ill., graded to pass a No. 20 (841-μ) sieve and retained on a No. 30 (595-μ) sieve.

[1] Under the standardization procedure of the Society, this method is under the jurisdiction of the ASTM Committee C-9 on Concrete and Concrete Aggregates. A list of members may be found in the ASTM Year Book.

Current edition accepted Sept. 13, 1968. Originally issued 1958. Replaces C 418 – 67 T.

aration of Specimens

Immerse the specimens in water for and then surface dry with a damp to obtain a saturated, surface-dry ition at the time of test.

ation of Apparatus

Adjust the air pressure to 4.2 ± kgf/cm² (60 ± 1 psi) and collect abrasive for a period of 1 min. Ad- the rate of flow of abrasive to 600 ± per min.

1: Caution.—The abrasive shall be or replaced and a new nozzle installed every 60 min operating time in order that grading and flow may be maintained.

edure

Place the weighed specimen with surface to be tested normal to the axis and at a distance of 3 ± 0.1 from the end. Clamp the specimen, shield attached, firmly in place. the surface to the blast for a of 1 min. Repeat this on at least different spots on the surface. merse the specimen in water for clean, surface dry with a damp and weigh.

ation

(a) Calculate the weight of abraded ial, as follows:

$$W = W_1 - W_2$$

:

= weight of abraded material, grams,

= weight of saturated, surface-dry specimen before test, grams, and

= weight of saturated, surface-dry specimen after test, grams.

Calculate the bulk specific gravity concrete, saturated, surface-dry basis follows:

$$D = \frac{B}{B - C}$$

:

= bulk specific gravity of concrete, saturated, surface dry,

B = weight of saturated, surface-dry specimen in air, grams, and

C = weight of saturated specimen in water, grams.

(*c*) Calculate the volume of the abraded material, V, in cubic centimeters, as follows:

$$V = \frac{W}{D}$$

where:

W = weight of abraded material, in grams, and

D = bulk specific gravity of concrete (saturated, surface-dry basis).

(*d*) Calculate the abrasion coefficient loss on a volumetric basis, expressed in cubic centimeters per square centimeter, in order to compensate for variable densities of specimens, as follows:

$$A_c = \frac{V}{A}$$

where:

A_c = abrasion coefficient, cubic centimeters per square centimeter,

V = volume of the abraded material, cubic centimeters, and

A = area of surface abraded, square centimeters.

Report

7. Report the abrasion coefficient loss to the nearest 0.01 cu cm per sq cm.

Significance

8. This method covers the laboratory evaluation of the relative resistance of concrete surfaces to abrasion. This procedure simulates the action of water-borne abrasives and abrasives under traffic on concrete surfaces. It preforms a cutting action which tends to abrade more severely the less resistant components of the concrete. Adjustments in the pressure used and the type of abrasive permit a variation in the severity of abrasion which may be used to simulate other types of wear.

Appendix 1 (b)

Ball Race Abrasion
MA 20

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Ball Race Abrasion
MA 20

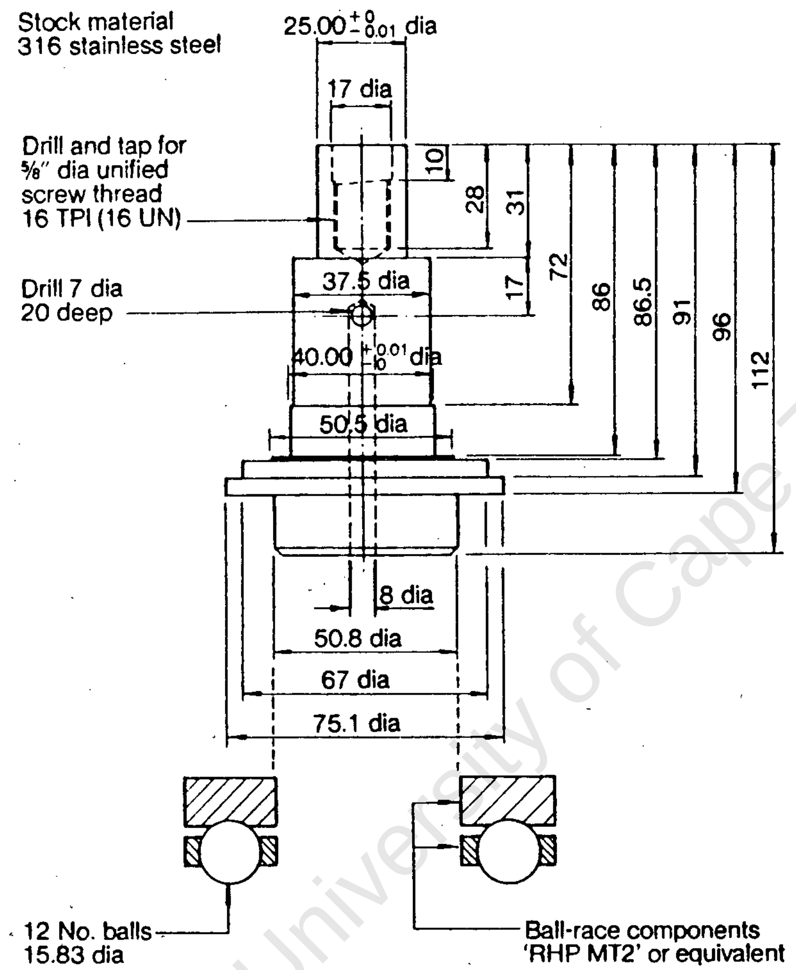

Figure D2 *Detail of chuck and ball-race*

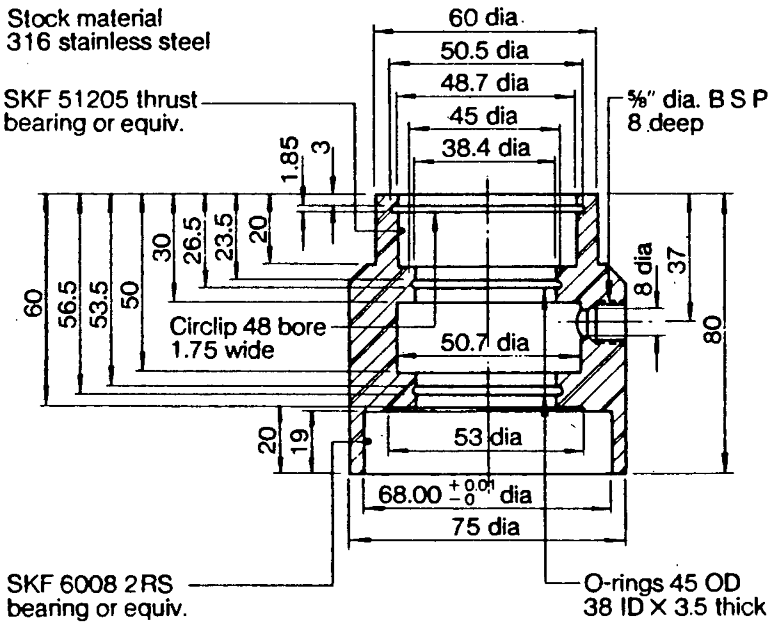

Figure D3 *Detail of chuck casing with water hose connection*

The arrangement of the apparatus is detailed in **Figure D1**.

The output shaft of the drill shall be set at a speed setting of between 1000 and 1050 revolutions per minute.

When the test drill is rigged, the total sliding mass of the drill and attachments shall be 14.5 kg with a tolerance of ± 0.25 kg. The test rig shall be maintained in this condition. Before testing, the rig shall be checked to ensure that there are no external factors affecting the sliding mass.

It is most important that the drill-stand shaft and guide bar be lubricated to ensure free sliding at all times.

D.4 Sampling of Units

Five units, representing the lot, and selected in accordance with Appendix B, shall be used as test specimens.

D.5 Procedure

Commentary D.5

If the drill and drill stand appear to wander from side to side causing the ball-race to alter its path on the paving-unit surface, then either the drill has been set at the wrong speed or the guide mechanism is worn and needs replacement.

Results from the test are valid only if a clearly defined circular impression has been made on the paving-unit surface upon completion of the test.

1 Check that the mass of drill and fittings is free to slide on the drill-stand shaft with no obstruction.

2 Place and clamp the dry specimen on the drill-stand base-plate. Place the ball-race on the specimen and lower the chuck onto the ball-race. Leave the drill unclamped on the shaft.

3 Ensure water is flowing at a sufficient rate to clear grinding debris.

4 Run the drill for approximately three seconds to seat the ball-race.

5 Lower the dial-gauge plunger onto the bearing surface of the drill bracket and rotate the chuck by hand through one revolution in each direction.

6 Set the dial-gauge zero to the mean of the needle reading.

7 Run the drill, stopping it at approximately every 1000 revolutions and measure the penetration. Continue the test until the ball-race has completed 5000 revolutions, or until the dial-gauge has indicated a penetration of greater than 1.5 mm, whichever occurs first.

8 Penetration shall be measured by rotating the chuck by hand through one revolution in each direction and noting the mean dial-gauge reading.

9 Note the number of revolutions of the ball-race.

D.6 Calculation of Results

The abrasion index of each specimen shall be calculated from the following expression:

$$I_a = \frac{\sqrt{R}}{P}$$

where:

I_a = abrasion index (calculated when the ball-race revolutions equal 5000 or the penetration equals 1.5 mm whichever occurs first)

R = ball-race revolutions, in thousands

P = penetration, in millimetres.

The abrasion index representing the sample shall be the minimum value obtained from the five specimens and shall be denoted I_{amin}.

D.7 Records

The following information shall be recorded:

- ☐ Identification mark
- ☐ Date of manufacture
- ☐ Date of test
- ☐ The sliding mass of drill and fittings in kilogrammes
- ☐ The intermediate and final numbers of ball-race revolutions for each specimen (if this figure was measured as rpm × minutes, then these figures shall be recorded)
- ☐ The intermediate and final penetrations in millimetres, for each specimen appropriate to the ball-race revolutions recorded as nominated above
- ☐ The abrasion index of each specimen
- ☐ The minimum abrasion index of the sample.

D.8 Report

If a report is prepared it shall include the following information:

- ☐ Identification of the lot, lot size and manufacturer
- ☐ Age of units, in days, at the date of test
- ☐ Date of test
- ☐ The abrasion index of each specimen of the sample
- ☐ The minimum abrasion index of the sample.

APPENDIX 2

Mix proportions for :

Average-sanded series 5 and 8

Under-sanded series 6 and 9

Over-sanded series 7 and 10

MIX PROPORTIONS (kg/m^3 concrete)

AVERAGE-SANDED - SERIES 5 AND 8

		TARGET 28 DAY STRENGTH (MPa)				
		10	20	30	40	50
OPC	Water	210	202	196	190	187
	OPC	192	239	296	348	416
	PFA	0	0	0	0	0
	Stone	1 050	1 050	1 050	1 050	1 050
	Sand	879	864	837	812	766
15% PFA	Water		188	182	175	171
	OPC		201	245	287	344
	PFA		35	43	51	61
	Stone		1 083	1 083	1 083	1 083
	Sand		855	838	808	760
30% PFA	Water	182	125	168	162	
	OPC	134	170	207	243	
	PFA	57	73	89	104	
	Stone	1 102	1 102	1 102	1 102	
	Sand	881	854	829	797	
50% PFA	Water	172	154	147		
	OPC	95	136	168		
	PFA	95	136	168		
	Stone	1 127	1 127	1 127		
	Sand	878	838	803		
70% PFA	Water		143	137		
	OPC		75	95		
	PFA		175	221		
	Stone		1 151	1 151		
	Sand		865	806		

MIX PROPORTIONS (kg/m³ concrete)

UNDER-SANDED - SERIES 6 AND 9

		TARGET 28 DAY STRENGTH (MPa)				
		10	20	30	40	50
OPC	Water	193	188	180	178	176
	OPC	173	219	275	328	398
	PFA	0	0	0	0	0
	Stone	1 220	1 220	1 220	1 220	1 220
	Sand	784	758	725	691	635
15% PFA	Water		175	170	167	164
	OPC		185	232	274	326
	PFA		33	41	48	57
	Stone		1 259	1 259	1 259	1 259
	Sand		738	702	670	630
30% PFA	Water	172	164	156	158	
	OPC	123	154	194	239	
	PFA	53	66	83	102	
	Stone	1 280	1 280	1 280	1 280	
	Sand	760	745	701	636	
50% PFA	Water	160	148	147		
	OPC	87	126	166		
	PFA	87	126	166		
	Stone	1 309	1 309	1 309		
	Sand	758	710	634		
70% PFA	Water		141	138		
	OPC		75	93		
	PFA		175	217		
	Stone		1 338	1 338		
	Sand		682	633		

MIX PROPORTIONS (kg/m^3 concrete)

OVER-SANDED - SERIES 7 AND 10

		TARGET 28 DAY STRENGTH (MPa)				
		10	20	30	40	50
OPC	Water	226	218	205	206	198
	OPC	202	252	307	372	450
	PFA	0	0	0	0	0
	Stone	950	950	950	950	950
	Sand	940	922	908	856	796
15% PFA	Water		201	195	188	181
	OPC		213	264	307	379
	PFA		38	47	54	67
	Stone		980	980	980	980
	Sand		911	883	857	798
30% PFA	Water	199	191	181	175	
	OPC	142	179	223	263	
	PFA	61	77	96	113	
	Stone	997	997	997	997	
	Sand	942	922	877	841	
50% PFA	Water	191	172	159		
	OPC	103	145	185		
	PFA	103	145	185		
	Stone	1 019	1 019	1 019		
	Sand	931	896	836		
70% PFA	Water		159	149		
	OPC		79	92		
	PFA		184	216		
	Stone		1 042	1 042		
	Sand		942	929		

APPENDIX 3

LITERATURE SURVEY AND COMMENTS

Subsequent to submission and examination both the internal and external examiners felt that the literature survey was insubstantial and lacked current journal references.

The original literature search was done in four parts. The first was a Dialog search done through the UCT library of large data bases in America. This turned up a substantial number of potential references on very general aspects of PFA, but on refining the search to material relevant to the thesis only, six references were of use. However, these were just reports on the insitu behaviour of PFA concrete pavements.

The second part of the search was through material held in the UCT library. From this source Fulton's Concrete Technology and Popovic's Fundamentals of Portland Cement Concrete: A Quantitive Approach; Vol 1 "Fresh Concrete" and a number of other minor references were obtained.

The third part of the search was through papers and books owned privately. Material was obtained in particular from Des O'Neill, Mike de Kock and Mark Alexander. In particular the chapter by Ravindra Dhir in the book "Cement Replacement Materials", the third volume in the series Concrete Technology and Design was of benefit.

The fourth part was a search through the material at the PCI libraries in Cape Town and Halfway House.

Following the examination of the thesis a number of further conference proceedings were examined for relevant information. The journals of the American Concrete Institute (ACI), the British Concrete Society (Concrete), the American Society for Testing and Materials and the Magazine of Concrete Research were searched and although a few articles and papers were found that were of some relevance to the work of the thesis, there was no new useful information that could be added to the thesis.

In general the author found that a large number of articles have been written on the general subject of PFA and its application. In attempting to describe a wide a range of PFAs and their applications, these articles did not cover anything in particular detail and they tended to repeat each other.

On the other hand ~~of the scale~~ there were quite a few articles that were at the same level and similar to the research of the author. These articles tended to report results with insubstantial explanations. These references helped in directing the research approach but did not explain problems exposed by results of the author's research. Many of the explanations were based on Fulton's and Dhir's publications.