THE DEVELOPMENT OF
HIGH FLYASH CONTENT CONCRETE
(Volume I - Script and Appendices)

Thesis submitted to the Department of Civil
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ABSTRACT.

This Thesis traces the development of high flyash content concrete. This concrete contains a high proportion of flyash in the cementitious content, usually between 50 and 80% by volume.

The concrete was initially developed as a roller-compacted hearting (or interior) concrete for dams. Concrete with a low workability, a low thermal (and non-thermal) movement, and relatively high tensile strain capacity was required. In 1976, the Author had proposed that this could be obtained by using a concrete with a high paste content and that the high paste content should be obtained by using large quantities of flyash and a low cement content. This concept was studied in depth during a CIRIA project, which is described. For roller-compacted concrete, the optimum proportion of flyash in the cementitious content was found to be between 70 and 80%. The properties of such a concrete were found to be suitable for use in dams. High flyash content concrete has now been proposed for two dams, one in America and the other in the U.K.

In the Thesis, a trial mix programme is described which extended the concept of a high flyash content in concrete into the workability range of immersion vibration. During this study, it was postulated that there is a relationship between the flyash contribution to compressive strength and the water/cementitious ratio, in the same fashion as that proposed for cement by Abrams in 1912. It is shown that the contribution of flyash to strength is more sensitive to the water content than the contribution of the cement, and that the conventional methods of mix design for flyash in concrete may not be using flyash to its best advantage.

The Thesis concludes with a description of a placement of

high flyash content concrete in a road and storage area. The sub-base of the road was placed through a paver-finisher without roller compaction and was a concrete in which flyash made up 75 to 80% of the cementitious content. The pavement-quality concrete was compacted by immersion vibration and flyash made up 60 to 65% of the cementitious content. Both concretes were extensively tested and have performed very well.

It is concluded that high flyash content concrete is a new material, and that it should have a use in many forms of Civil Engineering construction, in particular dams and roads. Usually, there are substantial economic advantages in such a use, and the long-term in-situ properties are generally better than those of similar conventional concretes.

FORMAT AND ACKNOWLEDGEMENTS

This Thesis is written in three main parts, the first deals with the development of high flyash content concrete for dam construction. It describes the work carried out in a CIRIA project for which the Author was the instigator and Project Engineer. The second part of the Thesis deals with a series of laboratory mix programmes carried out after the completion of the CIRIA project, to investigate the different contributions of flyash and Portland cement to cube compressive strength. The programme also aimed to ascertain if it was possible to extend the concept of high flyash content concrete from the low workability (suitable for roller compaction) generally used during the CIRIA project, to a higher workability suitable for use in structural members and pavement-quality concrete.

The third part, describes the construction of some access roads and a storage area at Didcot Power Station. High flyash content concrete was used in both the sub-base of the roads and the pavement-quality concrete.

The Figures and Tables are bound into a separate Volume for ease of viewing while reading the script in Volume I.

A substantial number of people were involved in all the work described above, particularly in the CIRIA project. This project cost approximately £135,000 and the majority of the work was completed in less than a year.

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NOTATION AND ABBREVIATIONS

The method of mix design developed for the roller-compacted hearting concrete for dams is based on the optimum packing of all the voids in the concrete (see Section 4). Consequently the mix design is based on the volumes (not weights) of the ingredients. The final batch weights are obtained by the multiplication of the volumes of each ingredient by their relative densities. The following factors have been used in the mix design.

a	coarse aggregate content (by absolute volume)
b	fine aggregate content (by absolute volume)
C_c	cement/cementitious* ratio - $c/(c + f)$
C_f	flyash/cementitious* ratio - $f/(c + f)$
C_w	water/cementitious* ratio - $w/(c + f)$
c	cement content (by absolute volume)
f	flyash content (by absolute volume)
m	mortar fraction (by absolute volume) (i.e. $b + c + f + w$)
P	paste/mortar ratio - (p/m)
p	paste fraction (by absolute volume) (i.e. $c + f + w$)
w	free water content (by absolute volume)

* Cementitious content is the content of cement-sized material (i.e. cement + flyash)

The water/cementitious ratio (by volume) - C_w - cannot be compared directly with the conventional water/cement ratio (by weight) - w/c - because of the different relative densities of the cement and flyash. However, in order to make a comparison easier, the equivalent water/cementitious ratio (by weight) has been plotted against the water/cementitious ratio (by volume) on Figure 1 for four different flyash/cementitious ratios - $C_f = 0, 0.4, 0.6$ and 0.8 .

In addition the following standard abbreviations have been used:

C of V	coefficient of variation
Ed	dynamic modulus
Esc	static modulus in compression
Esr	static modulus in flexure
Est	static modulus in direct tension
FST	CIRIA intermediate fullscale trial mix identifier
fc	cube compressive strength*
f'c	cylinder compressive strength*
fr	flexural strength*
fs	splitting tensile strength*
ft	direct tensile strength*
G	Brynchir quartzite gravel
H	Hingston Down granite coarse aggregate and Blackpool sand
K	Kingston granite coarse and fine aggregate
k	thermal conductivity
M	Moorcroft limestone coarse and fine aggregate
MBF	CIRIA Wimbleball fullscale trial mix identifier (Milton Brook Facing)
MBH	CIRIA Wimbleball fullscale trial mix identifier (Milton Brook Hearting)
m.s.a	maximum size of aggregate
RD	relative density
SD	standard deviation
t.a.f.	theoretical air-free density

* Measured in MPa ($= 1 \text{ MN/m}^2 = 1 \text{ N/mm}^2$)

In the mixes made during the CIRIA project, the cements are identified by letters A, B or C, and the flyashes by letters W, X, Y or Z.

PART A

DEVELOPMENT OF HIGH FLYASH CONTENT CONCRETE

FOR USE IN DAM CONSTRUCTION

1. INTRODUCTION

1.1. TYPES OF DAM

There are essentially two types of dam, the fill type (a gravity structure made of earth and rock materials) and the concrete dam. The latter can be further classified into gravity, arch, buttress or a combination of these. High flyash content concrete were initially developed for use in concrete gravity dams. It is probable that arch gravity dams, even thick arch dams, could be constructed using the methods and materials described in this Part of the Thesis.

The concrete gravity dam is constructed of solid concrete, and is designed so that its weight is sufficient to ensure stability against all external forces. The arch dam transmits the water load through arching action into the foundations and the abutments of the structure, so its weight is of less importance. The arch gravity dam combines the two types of structure. The principal external force is the pressure of the impounded water, but other forces can act on the structure (e.g. silt pressure, ice pressure and earthquake forces).

The conventional shape for a concrete gravity dam is shown on Figure 2. The slope of the downstream face varies according to the density of the concrete and the foundation conditions, but is generally between 0.60 and 0.80. The upstream face is usually vertical or at a small slope (say 0.10). The dam can either be straight or curvilinear in plan.

1.2. DEVELOPMENT OF METHOD OF CONSTRUCTION OF CONCRETE DAMS

Concrete dams similar to modern structures were being built at the turn of the century, but Hoover Dam, USA (see Figure 3), which was constructed in the early 1930s, was virtually the base from which most of present-day design and

construction techniques have evolved. The unprecedented size of the structure was such that the investigations undertaken were very elaborate.

A study was made of the effect on the constructional procedures of the heat of hydration, the strength of concrete, the fineness and composition of the cement, the cement content, the maximum size of aggregate, and the curing temperature. The results of the investigation by the US Bureau of Reclamation[1] led to the acceptance of a properly controlled monolithic method of construction which is still prevalent today. Figure 4 shows such a dam under construction.

Hoover Dam was built in a series of blocks, separate from each other and sized to minimise the adverse effects of the heating of the mass concrete, resulting from the heat of hydration of the cement, and of the subsequent cooling of the concrete. Cooling of aggregates was also introduced, as was the use of an embedded pipe cooling system[1].

The method of construction of concrete gravity dams has changed little since the design and construction of the Hoover Dam. Nevertheless, minor improvements to the mix design methods and the use of admixtures have enabled the cementitious content (total content of cement-sized materials, including pozzolans) of the heating concrete to be reduced[2]. The UK and Europe have lagged somewhat behind the USA in this reduction of cementitious content[3 to 7]. A further change has been the increasing use of artificial cooling of the concrete which has enabled the size of the monoliths to be increased[8,9].

1.3. TEMPERATURE GENERATION IN CONCRETE DAMS

One of the major problems in concrete dam construction is the heat generated by the cement hydration in large pours. The resulting temperature rise in the concrete would not create problems if the concrete were free to move, but because there is always some form of restraint, cracking can occur. Three different forms of cracking have been reported[6,10,11]:

1. early thermal cracking, caused by temperature differentials within a lift of concrete whereby the slower cooling of the centre of the pour is restrained by the outside, which has cooled more rapidly
2. cracking caused by a temperature differential between the previous lift of concrete and the new lift
3. long-term thermal cracking, caused by the difference between the maximum temperature generated in the centre of the dam and the long-term ambient temperature. This form of cracking can occur many years after the dam is completed

1.4. NEED FOR A CHANGE IN THE METHOD OF CONSTRUCTION

Although the foundation conditions and shape of valley, together with the required capability to pass floods, generally dictate the type of dam, there are sites on which more than one type of dam could be built. While the design and construction methods of concrete gravity dams have remained relatively unchanged, there has been a general improvement in the efficiency of building fill type structures, particularly the rockfill dam. This has led to a decline in the proportion of dams being built of concrete as shown on Figure 5[12].

Concrete dams have some intrinsic advantages over the fill-

type structure. In particular, they are resistant to erosion[13,14], and there have been no reports of a concrete dam being severely damaged by a movement of its foundations as a result of earthquake[15]. Maintenance is usually less than that of a fill-type structure. Concrete dams can also be overtopped both during construction and after completion without serious damage[16]. This ability has led certain bodies to apply different design criteria to the spillways of concrete and fill dams, the latter having to have a capacity of up to 30% greater than the spillways of the former[17]. A new method of construction for concrete dams thus seems to be required. Any such new method must not only be economic, but must also reduce some, if not all, of the technical problems inherent in a large concrete mass.

The initial stages of this Thesis were therefore aimed at reducing construction costs by investigating improved methods of construction and the materials required for those methods.

2. DEVELOPMENT OF THE ROLLER-COMPACTED CONCRETE DAM

2.1. MATERIALS AND METHODS OF CONSTRUCTION

In the past 20 years, horizontal placement has been introduced as an alternative to the monolith method of construction. The first dam constructed by this method was Alpe Gera Dam in Italy in the early 1960s[18,19]. Recently roller compaction has been suggested for the hearting concrete as an alternative to immersion vibration[20,21]

Essentially three different materials are at present being considered for roller compaction in dams:

1. lean concrete with a cementitious content of approximately 100 to 120 kg/m³, of which up to 30% (by weight) can be replaced by flyash
2. high flyash content concrete designed for high density (i.e. minimum air voids) and good bond between layers
3. cement-stabilised 'as dug' material, sometimes called 'rollcrete'

There is overlapping of each type of material and each is designed for a slightly different purpose and has different properties. For any material to be used in the body of a dam, not only must the properties of the concrete be understood, but there must also be a method of construction which fulfils all the requirements of the particular dam under consideration.

During the various investigations into roller compaction for dams, four different methods of construction have evolved:

1. a lean concrete dam with a separate upstream watertight wall without joints in the hearting (or interior) concrete
2. a lean concrete dam with vertical contraction joints cut through each layer soon after placement
3. a high flyash content concrete dam without joints in the hearting and with the roller-compacted concrete retained by facing elements horizontally slipformed using an offset paver
4. a dam using cement-stabilised 'as dug' material with or without an upstream watertight membrane depending upon the requirements of the dam

The development of the various methods (and materials) during the last 20 years is shown on Figure 6.

At the present time (1982) dams are known to be under construction, or in the design and tender stages, using three of the four methods (i.e. 2, 3 and 4 above).

2.2. DAM CONSTRUCTION USING LEAN CONCRETE WITHOUT JOINTS IN THE HEARTING

The first suggestion that a roller-compacted lean concrete could be used in a dam was in 1970[20]. The first full-scale trial was conducted in 1971 at Tims Ford Dam, USA, by the Tennessee Valley Authority (TVA)[22].

The next series of fullscale trials, which also included a small laboratory investigation, was undertaken at the Waterways Experiment Station at Vicksburg, Mississippi, by the US Army Engineers during 1972/3[23]. This was part of a value-engineering study for Elk Creek Dam. As a follow-up to this study, a larger series of fullscale trials was

carried out at Lost Creek Dam by the North Pacific Division of the US Army Engineers in May 1973[24]. It has been from this series of trials that most of the work on roller-compacted concrete has been generated. However none of the above investigations proposed a method of construction for dams using the material.

In 1965, a prototype dam (Manicouagan I) was constructed using horizontal placement by Hydro-Quebec in Canada[25]. The concept of this particular method of construction was that a concrete gravity dam could be considered as three separate sections: an upstream impervious barrier of rich concrete, a downstream facing of large pre-cast concrete blocks and a central core of lean concrete. In 1973 this concept was combined with the roller compaction of a lean concrete for the hearting, and a method of construction proposed[26]. Because it was considered that lean concrete was not sufficiently impermeable and that leaching could take place, the upstream watertight membrane was backed by a series of drains. In this case the upstream face consisted of a series of interlocking precast blocks (see Figure 7), with a poured bitumen watertight membrane.

u An extensive laboratory investigation of lean concrete at the University of Newcastle-upon-Tyne[27] confirmed that the material could be subject to leaching and that the bond between horizontal layers could not be guaranteed. Two different profiles for dams were suggested[28,29] (see Figures 8 and 9), both with a watertight upstream membrane and a no-fines drainage zone. The upstream concrete diaphragm in the gravity dam (Figure 8) could be vertically slipformed in a similar fashion to a method proposed in 1972[30]. The latter consists of a vertically slipformed upstream wall backed by roller-compacted lean concrete with a downstream face consisting of precast concrete blocks. In all cases, contraction joints were proposed for the upstream

barrier.

Cost comparisons were made between dams built by this method and two conventional concrete dams, one an arch gravity dam and the other a straight gravity dam with embankment wings[28]. In each case the conventional dam was cheaper, mainly because of the small size of the structures, the volume of concrete being only 17 000 m³ in each dam.

Three further placements have been made with lean concrete, all on dam sites, but none were intended as a watertight structure. In 1976, the floodway sill of Moose Creek Dam, part of the Chena River Lakes Project in Fairbanks, Alaska[31] was constructed in roller-compacted concrete. This was approximately 610 m long, 12 to 13 m wide and 1.5 m thick, and replaced what was originally to be riprap on the downstream side of a sheet-piled overflow weir. Approximately 14 500 m³ was placed in four lifts. In 1978 a similar quantity of roller-compacted concrete was placed at Bonneville Dam, Washington State, to protect a horizontal rock surface exposed by excavation for the second-stage power station[32].

In April and May 1978, approximately 12 000 m³ of lean concrete were placed and roller compacted in an access ramp at Itaipu Dam on the Brazilian/Paraguayan border[33]. A total cementitious content of 117 kg/m³ (of which 30% by volume was flyash) was used.

2.3. DAM CONSTRUCTION USING LEAN CONCRETE WITH JOINTS IN THE HEARTING

The design of this type of dam originated with the construction of Alpe Gera Dam between 1961 and 1964[19]. Instead of using the conventional monolith type of construction, the dam was built in 700-mm layers laid from one side of the valley to the other. The concrete was

transported from the batcher to the top of the dam using a funicular railway system on one side of the valley. From the hopper at the bottom of the railway, concrete was transported across the top of the dam in dump trucks. It was then spread by bulldozers and compacted using banks of immersion vibrators on the back of tractors. Joints were cut through each layer with a vibrating blade soon after the concrete had set. The upstream and downstream faces were cast against conventional shutters using richer concrete. The dam was made watertight by a complete covering of the upstream face with a steel sheet. The final structure was therefore similar to a monolith dam, as it consisted of a number of blocks, although in this case the 'blocks' were formed after the concrete had been placed rather than by shutters. It was reported in 1973[34] that the dam and its successor, the Quaira della Miniera Dam had both behaved in a totally satisfactory manner up to that date.

In 1974, the Japanese Ministry of Construction initiated a research programme aimed at reducing the costs of concrete dams. In Japan, there is a very high rainfall and significant seismic activity, and it was felt that concrete dams, although at the time more expensive than fill-type dams, had certain inherent advantages over the latter. A desk study concluded that the Alpe Gera method of construction, combined with the roller compaction suggested by the Lost Creek trials, was the most profitable line that research could follow[35].

In 1976, the upstream cofferdam of Okawa Dam was used as a fullscale trial of the method of construction proposed. The upstream, and after some unsuccessful trials with plate compactors, the downstream faces of the dam were formed against shutters using concrete compacted by conventional immersion vibrators. This followed soon after the roller compaction of the hearting. The joints in the hearting

concrete were cut using vibrating blades, and plastic sheets were inserted into the joints to stop them closing[36].

These trials were followed by a series of other fullscale trials of roller-compacted concrete[37]. At the present time (1982), the Shimajigawa dam, which is 88 m high[38], and a large base slab at Okawa Dam[17] have both been completed using this method of construction. On the former, a cableway was used to transport the concrete from the batching plant to a mobile concrete hopper on the top of the dam. From here the concrete was transported to the location at which it was to be placed. Before the concrete was placed, a layer of mortar approximately 15 mm thick was sprayed onto the surface of the previous lift. The concrete was then levelled in a 500-mm layer using a bulldozer. The concrete was finally compacted by a 7-tonne duplex vibratory roller, using two passes without vibration and nine passes with vibration. An exposed aggregate finish was produced on the concrete soon after compaction using low pressure water jets and a mobile road sweeper, or scabbled at a later age using a 'polisher'.

After the concrete had set (2 to 5 hours after placing) the joints were cut from the upstream to the downstream face with a vibrating joint-cutting machine. A separating board was then inserted into the bottom two-thirds of the joint. A complicated system of water bars was used in the joints in the facing concrete on the upstream and downstream faces of the dam, stainless steel plates having to be welded together in sections on site.

2.4. DAM CONSTRUCTION USING HIGH FLYASH CONTENT CONCRETE WITHOUT JOINTS IN THE HEARTING

The idea for this type of dam was introduced in 1974[39], combining the horizontal placement concept for the hearting with its compaction by vibratory roller. An additional

feature was the proposal to form interlocking facing elements for the dam by using an offset slipform paver running on top of the hearting (see Figure 10). At that time, it was intended that the contraction joints in the hearting would be induced with formers rather than cut after placement of the concrete. A watertight membrane was also to be placed behind the upstream facing elements.

The hearting concrete, which was to have a low Portland cement content and a high flyash content, is the material from which HFCC has been developed, and is the starting point for this Thesis.

In 1976 some roller-compacted concrete was placed in the foundation of a transformer/generator area at the Tamar Treatment Works in Cornwall[40] using various flyash contents of up to 80% of the cementitious content.

At about the same time the TVA were conducting two similar fullscale trials. The first was small, and was carried out to investigate the compaction of concrete against formwork using small rollers[41,42]. The second was a placement of 6 780 m³ in the foundations of a turbine building at the Bellefonte Nuclear Power Station in Alabama. Two different mixes were used, a normal roller-compacted concrete and a bedding mix to be spread on any cold joints[43,44]. Both these placements contained mixes with high proportions of flyash in the cementitious content, up to 80% in the case of the normal roller-compacted concrete.

Following a visit by the Author, a fullscale trial of the technique of forming a face of a dam using horizontally slipformed elements was carried out at the Waterways Experiment Station by the US Army Engineers in 1977[45,46]. In the event, the facing elements were cast by using conventional concrete compacted with immersion vibrators

between shutters, but the shape of the facing element was investigated and roller compaction was carried out against the elements at an early age (see Figure 11). The opportunity was taken to compare roller-compacted concrete designed using the lean concrete approach (see Sections 2.2 and 2.3) with a high flyash content concrete. The performance and properties of the high flyash content concrete were found to be better than those of lean concrete[46].

In 1977, using the experience from the various trials, a method of construction was proposed, using horizontally slipformed facing elements and a high flyash content hearting concrete with no construction joints[47]. The hearting was designed to have a low heat generation, and thus low thermal movement, and a relatively high tensile strain capacity at later ages. It was also designed to have minimum air voids and good bond between layers and was considered to be, in effect, the watertight element of the dam.

This method of construction was planned to be used for Milton Brook Dam, near Plymouth, for the South West Water Authority[48]. Unfortunately after tenders had been received by the Client, and after it had been proved that the roller-compacted concrete dam was cheaper than a conventional dam, the dam had to be postponed due to central Governmental financial cuts[49].

Upper Stillwater Dam in Utah, USA, is now likely to be the first dam constructed using high flyash content concrete. A successful trial was completed in the summer of 1981 (see Figure 12). Excavation for the dam started in the autumn of that year[50].

2.5. THE OPTIMUM GRAVITY DAM AND DAM CONSTRUCTION USING CEMENT-STABILISED 'AS DUG' MATERIAL (ROLLCRETE)

The concept of an optimum gravity dam was introduced in the early 1970s[51,52]. It was postulated that there is an optimum dam somewhere in the middle ground between a conventional concrete dam which has a small volume of an expensive material, and an earthfill dam which has a large volume of inexpensive material. By stabilising 'as dug' material, it is possible to steepen the slopes of a fill structure. As the cement content is increased, the cohesion of the material also increases and, with it, the slope at which the material can be placed. Consequently, it was suggested that for a particular structure, there is an optimum cement content, and this is the optimum gravity dam.

The use of cement-stabilised 'as dug' material had been suggested earlier for a soil-cement dam in the early 1960s[53]. The core of the cofferdam for Shihmen Dam, Taiwan, was constructed using 'rollcrete', a material with a maximum size of aggregate of 75 mm and a cementitious content of approximately 120 kg/m^3 , half of which (by weight) was flyash. It was laid in 300-mm layers, and it was initially proposed to roll the material with 50-tonne rubber-tyred rollers. In the event, its only compaction was performed by the running of the dump trucks and the passage of the bulldozers used for spreading.

At Tarbela Dam in Pakistan a large volume of 'rollcrete' has been placed from 1975 to the present time. The concrete is being used as rock replacement following erosion of the original material[54,55]. Initially, the 'rollcrete' was produced by passing as-dug material through a 230-mm grating on to a conveyor, adding cement from a further conveyor, and transporting the material to the top of a simple mixing tower, where water was added. The tower consisted of a series of baffles, and as the material dropped through the

baffles it was mixed. The cement content was between 110 and 140 kg/m³. During the first year (1975) the material was very variable[56] and so for the following years a drum mixer (or series of mixers) have been used[57].

In the mid 1970s, the first optimum gravity dam, Zintel Canyon Dam in Washington State, USA, was mooted[58]. This was to be a flood alleviation bank which would not generally retain water, only being filled in times of flood. The requirements for the dam material were therefore somewhat lower than those required of a material for a dam continuously subjected to a full head of water. Two different concretes were to be used: a roller-compacted concrete approximately 2.5 m wide on the outside skin, and a cement-stabilised backfill for the heart (or interior) of the dam. The former had a cement content of approximately 120 kg/m³ and the latter only 60 kg/m³. Both were to use aggregates with a maximum size of 75 mm. Three different profiles for the dam were considered initially[59], and one further possibility was considered at a later date[60]. These profiles are shown on Figure 13. In the event, difficulties with the financing of the dam precluded its construction. A further dam, also a flood alleviation bank, at Willow Creek, Oregon, USA, is now being built using this method of construction. Trials for this dam were undertaken at the laboratories of the North Pacific Division of the US Army Engineers in August 1980[61] and construction started in 1981.

A smaller flood alleviation structure was completed early in 1982 near Newton Abbot, Devon[62] by the South West Water Authority. This dam was built with a roller-compacted lean concrete core and fill shoulders (see Figure 14). The latter acted as supports during construction so that no formwork was required for the lean concrete. As the dam was to retain water only irregularly it was not considered to be

important that bond between the layers should be guaranteed. Cores taken through the roller-compacted concrete have shown that bond was not good between layers placed with an exposure time (gap between placements) of more than one day.

One further placement of a cement-stabilised 'as dug' material was conducted in May 1979. The cofferdam for Revelstoke Dam in Canada was completed with an erosion-resistant cap[63]. Two mixes were used: a bedding (or bonding) mix, and a normal mix. The former was used for placement on cold joints and spread in a 100-mm layer, to be followed by sufficient of the normal mix to give a total layer thickness (after compaction) of 300 mm. The concrete was mixed in a soil-cement plant ('Pugmill') with cement being added volumetrically direct onto the conveyor transporting the aggregate. There were considerable difficulties with the control of the material, and the properties were variable[63,64].

2.6. COMPARISON OF METHODS OF CONSTRUCTION

A comparison has been made of the four methods of construction in Table 1. The type of construction chosen for a dam will depend upon the unit cost of the materials at the site, and upon the design criteria of the dam. Generally, if the dam is to retain water and if flyash is not expensive, the high flyash content concrete dam without construction joints will be the cheapest method. However, if the dam is a flood alleviation bank and flyash is not available, the optimum gravity dam containing cement-stabilised 'as dug' material may be the most economic solution.

3. REQUIREMENTS OF CONCRETE IN DAMS

3.1. TENSILE STRAIN INDUCED BY THERMAL MOVEMENT

When concrete is placed in any large structure, the heat generated by hydration causes the concrete to expand. At early ages when the concrete is still plastic, the majority of the stress caused by this expansion is dissipated by creep. As the modulus of the concrete increases, and the ability to creep decreases, some compressive stresses are built into the concrete if its movement is restrained. These compressive stresses are advantageous because they will partially counteract the tensile stresses created during the eventual contraction of the concrete when cooling takes place to the long-term ambient temperature. In a dam, the time for the concrete to reach its maximum temperature varies, but it is generally near to its maximum at an age of 2 to 3 days. The time to cool can vary from 2 to 3 years for a small dam to many 10s of years for a larger structure. During this cooling the concrete is likely to be partially restrained and the contraction of the concrete, although partly offset by creep, will cause the concrete to be subjected to tensile strain.

A number of organisations have measured the expansion and contraction of concrete, and the strain induced by thermal movement. Given certain criteria these are all now predictable[65,66].

Artificial cooling of concrete is common on large dams[2,67] and on a large scale is economic[68]. If the concrete can be cooled to a sufficiently low temperature, and if the temperature rise is kept low, the placement temperature plus temperature rise can be made equal to the long-term temperature of the surroundings. No tensile strains would then occur in the concrete. This was achieved during the

rolled-concrete placement at Itaipu[33,69].

On smaller dams, cooling is not normally necessary nor economic. Figure 15 shows diagrammatically the temperature rise and the tensile strain induced by thermal movement in the centre of a conventional mass concrete dam when the concrete is not cooled. It follows that:

1. any reduction in temperature rise and thus thermal movement will be advantageous
2. any delay in reaching the maximum temperature will be advantageous as this will allow the modulus to increase and thus compressive stresses to develop
3. high early modulus is not necessarily a disadvantage if the temperature is still increasing
4. the maximum tensile strain capacity of the concrete is not required for some considerable time
5. any decrease in coefficient of thermal expansion will reduce the thermal movement and thus tensile strain induced in the structure

3.2. REQUIREMENTS OF ROLLER-COMPACTED CONCRETE FOR DAMS

Structural concrete mixes are usually designed to satisfy workability, compressive strength and durability requirements[70,71]. The most desirable requirements for the hearting concrete of a rolled concrete dam are:

1. high density (i.e. within the limitations of the available aggregates, the density should be as high a proportion of the t.a.f. as possible)
2. good bond between layers, leading to high tensile

strength across the joints and low permeability

3. the ability to be laid without the need for vertical contraction joints and yet to be able to tolerate both early- and long-term thermal movements without detrimental cracking
4. workability suitable for compaction by vibratory roller
5. sufficient cohesion to permit transportation without segregation

High compressive strength is not normally required. Low heat generation, high tensile strain capacity, and low permeability are more important factors. A new basis for mix design was therefore introduced to take account of these principal requirements.

4. BASIS OF DESIGN OF HIGH FLYASH CONTENT HEARTING CONCRETE FOR ROLLER COMPACTION

4.1. BACKGROUND TO MIX DESIGN

All mix design methods have limitations, since the range of materials which can be used in concrete is so wide that any relationship established by tests can only serve as a general guide. Final mix proportions must be established by trial mixes, and subsequent adjustments usually have to be made on site.

4.2. DENSITY AS A PROPORTION OF THE THEORETICAL AIR-FREE DENSITY

Over recent years the cement content used in concrete dams has been reducing steadily[2]. There are two main reasons for this: first to reduce the heat generated within the structure, and second to reduce the cost, since cement has the highest unit cost of all the materials used. This emphasis on lowering cement content is even more apparent in the various trials of roller-compacted concrete.

Density as a high proportion of the t.a.f. implies a minimum volume of air voids. It is common practice to optimise the coarse aggregate content of a mix[72], but the analysis of the filling of the voids below this level is rarely considered. If optimum packing is to be obtained, the voids in the sand (or fine aggregate) must also be filled, yet with the low volumes of cementitious content used of late this does not seem to have been occurring. In order to achieve this optimum packing, absolute volumes are used for all the materials during mix design procedures described in this Thesis. Appendix A contains details of all the fullscale trials of roller-compacted concrete reported to date. During an analysis of the results of these trials[47], it was apparent that the volume of paste used in the majority was insufficient to fill the voids in the sand.

Figure 16 shows the relationship between the density of the concretes, taken from these fullscale trials, as a proportion of the theoretical air free density, and the paste/mortar ratio (P), where this is defined as the ratio of the paste fraction of the concrete to the mortar fraction. The paste fraction is the absolute volume of the cementitious material and the free water. The mortar fraction is the absolute volume of the fine aggregate, the cementitious material and the free water. There is a significant drop in the density (as a percentage of the t.a.f.) below a P of between 0.35 and 0.40.

The porosity (i.e. the air voids in a compacted state) of a typical concreting sand is also between 0.35 and 0.40. Therefore, below a paste/mortar ratio of approximately 0.35, all the voids in the fine aggregate are not being filled and a high density cannot be achieved.

4.3. BOND BETWEEN LAYERS

An analysis was also made of the bond between the layers of concrete in the fullscale trials. This revealed the possibility of a relationship (see Figure 17) between the tensile and shear properties at the joints (as a proportion of the same properties of the parent material) and the paste/mortar ratio[47]. Data are limited, but the form of the relationship is similar to that found on Figure 16. The properties at a plastic joint (i.e. one which is almost immediately covered) are better for a given P than at a joint exposed for one day, as could be expected, and it is clear that the bond across a joint improves as the paste/mortar ratio increases.

The improvement in bond across a joint is probably attributable to the increased fluidity of the concrete with a high paste content. In addition, the high paste content may 'prime' the surface of the previous lift.

A design criterion used during this investigation was that the direct tensile strength across a joint should be similar to that of the parent material in a conventionally placed dam. A typical figure for the latter is 2 MPa at 28 days and 3 MPa in the long term. To achieve this a minimum paste/mortar ratio of between 0.40 and 0.44 is probably necessary for a roller-compacted hearting concrete where the joint is exposed for less than 1 day. If the exposure time is increased, the paste/mortar ratio must be also increased.

A single mix with a high paste/mortar ratio is considered to be preferable to the use of a 'bedding mix', because, in all the trials where bedding mixes have been used, there were problems with the control of the material and with the separation of the two types of concrete[43,64].

4.4. PROPORTIONING OF PASTE

There are contradictory requirements for the proportioning of the paste because a minimum paste content is necessary to fill the voids in the fine aggregate while as low a cement content as possible is needed to reduce the heat generation. To overcome this problem, the paste content can be increased by the addition of substantial quantities of flyash, which is in the same size range as the cement and generates less heat than cement in the short term. Flyash is also believed to be pozzolanic so that it could contribute to the long-term properties of the concrete.

In general, flyash particles are spherical (as shown on Figure 18) and are thus a better shape than cement, which tends to be made up of flaky particles (as shown on Figure 19). This means that, for a given workability, the water content can generally be reduced when flyash forms part of the cementitious content instead of cement.

Figure 20 shows the water/cementitious ratio required for a given workability when an increasing proportion of the cementitious material is flyash. With flyash making up 80% by volume of the cementitious content of a concrete (i.e. $C_F = 0.8$), the water/cementitious ratio, C_W , can be reduced by up to 0.20 when compared with a plain concrete with no flyash (i.e. $C_F = 0$).

The use of flyash as a major ingredient in the paste requires a new approach to the design of concrete. Most of the existing methods which deal with flyash in concrete, only consider it to be a partial cement replacement[75]. The Tennessee Valley Authority has used flyash in proportions of up to 60% by volume of the cementitious content in dams, but this has been for concrete compacted by immersion vibration rather than by roller compaction[76].

Most of the structural properties of a hardened concrete are improved by a reduction in water content. Therefore, in addition to the flyash, the great majority of mixes studied in the project contained a water-reducing admixture.

4.5. OPTIMUM COARSE AGGREGATE CONTENT

Coarse aggregate is one of the cheapest constituents of concrete. Therefore the aggregate fraction of a mix should be as high as possible commensurate with the properties required. A limited fullscale trial was therefore conducted to study the effect of the coarse aggregate content on segregation during transportation and spreading, and on the response of the concrete to roller-compaction. This is described in detail in Section 16.2.2 of this Thesis. A practical limit to the coarse aggregate fraction with the particular materials being used (Moorcroft crushed limestone, maximum size 40 mm) was found to be between 0.56 and 0.59 (by volume) - see Appendix B for details of the

materials. This is similar to the amount suggested elsewhere[77]. To allow for batching errors on site, a limit of 0.53 to 0.55 was subsequently used in the majority of mixes containing this aggregate. It was found that the solid fraction of aggregate occupies approximately 0.53 of the total volume when Moorcroft aggregate is tested according to the loose bulk density test[78]. Therefore it is suggested that an initial value for the coarse aggregate fraction of a mix can be arrived at by dividing the loose bulk density of the aggregate by its relative density.

4.6. MIX DESIGN METHOD

The method used to design high flyash content concrete was introduced by the Author in 1977[47]. The method is similar to that reported by ACI Committee 207 in 1980[77].

The procedure is as follows:

1. Determination of coarse aggregate content, a , and mortar content, m ,:
Divide the loose bulk density by the relative density of the coarse aggregate to obtain an initial value for a , the coarse aggregate content. It follows that the mortar fraction, m , is $(1 - a)$.
2. Determination of paste content, p :
A minimum paste/mortar ratio, P , is required for bond at the joints and to yield a high density. The volume of paste, p , is $(P \times m)$.
3. Fine aggregate content, b :
The fine aggregate b , is $(m - p)$.
4. Proportioning of paste.
The ratios of flyash and water to cementitious

content, C_f and C_w , are fixed from considerations of the tensile strain capacity (see Section 10) and heat generation (see Section 11). The cement/cementitious ratio, C_c is $(1 - C_f)$. Thus the ratio $c : f : w$ is found. As the paste fraction, p , is known and is equal to $c + f + w$, the volume of all the ingredients in the paste can be calculated.

5. Adjustments for workability.

An estimate of the workability of the mix can be obtained from data such as on Figure 20, and minor adjustments made to obtain the required workability.

6. Mix proportions by weight.

The proportions by weight can be calculated by multiplying the volumatic fraction of each material by the relative density of the material.

5. ASSESSMENT OF WORKABILITY AND DENSITY OF CONCRETE

In road base construction, where most roller-compacted concrete has been used to date, the workability of the concrete is generally assessed visually. The operator of the plant and the inspector are able to assess its suitability by observing whether the concrete sticks to the roller or corrugates, in which case it is too wet, or whether the material segregates and starts to shear, in which case it is too dry.

High flyash content concrete behaves rather differently from the lean concrete used in roads. When seeing the material for the first time, experienced operators consider the high flyash content concrete to be too wet, and feel that the roller will sink in. A further example of this difference between the two types of concrete is that high flyash content concrete can be compacted on a vibrating table for the production of test specimens, whereas dry lean concrete proportioned for roller compaction requires intensive vibration under pressure.

Ideally the method of construction, described in more detail later in this Thesis (see Section 15) requires the concrete to be placed at daily intervals. Removal of any of the concrete after it has been placed would be very expensive. Therefore, any method of control which enables the workability to be assessed and the concrete to be proved before it is placed, would make the whole method of construction more economical.

5.1. THE CANNON TEST

The TVA has developed a new test for measuring the workability of roller-compacted concrete[79]. This test is referred to as the Cannon Test after R. W. Cannon of that Authority who conceived the idea. The test was originally

conducted on a non-standard vibrating table at the TVA Singleton Laboratory, but during the CIRIA project[80,81,82] was adapted to use the Vebe standard cylinder and vibrating table[83,84]. The procedure is as follows:

1. the cylinder is loosely filled with uncompact concrete
2. the surface is struck off so that the cylinder is full to the brim
3. the cylinder is clamped to the vibrating table and the concrete is then subjected to vibration until there is paste all around the periphery of the cylinder as shown on Figure 21
4. the time (in seconds) to this point is noted and this is a measure of the workability of the concrete, and is called the Cannon Time.

There is a second part to the test which may be used to estimate the density of the concrete after compaction, and can also be used to approve or reject the material:

5. the cylinder is subjected to further vibration to bring the total time up to 120 seconds
6. the cylinder, which has a known volume (V_o) and weight (W_o), is then weighed ($W_c + W_o$). At this stage, the concrete is inspected and there should be a surfeit of paste on the surface as shown on Figure 22.
7. the cylinder is filled with water and reweighed ($W_c + W_w + W_o$), and the volume of water (V_w) found. The density (the Cannon Density) of the concrete after 120 seconds vibration is equal to $W_c / (V_o - V_w)$.

Table 2 compares the mean Cannon Density and the mean cube density measured at various ages for concretes having different flyash/cementitious ratios. Figure 23 shows the relationship between the density of the cubes and the Cannon Density for individual mixes. A number of densities are above 100% of the t.a.f. and this probably reflects the difficulties in obtaining a true measure of the S.S.D. relative density. For the range of flyash/cementitious ratios envisaged for roller compaction (i.e. $C_f = 0.75 \pm 0.05$), the difference is approximately 1.0%. Thus, if a minimum Cannon Density of 97.5% of the theoretical air-free density is specified, the minimum average cube density would probably be approximately 98.5%. The density of in-situ high flyash content concrete has been found to be approximately equal to that of cubes during various fullscale placements (see descriptions later in this Thesis) and a specified minimum Cannon Density has generally ensured that the concrete in those placements has had a density near to the t.a.f.

5.2. METHODS OF CONTROL USED ELSEWHERE

Both the US Army Engineers and the Japanese Ministry of Construction have used a loaded Vebe test for assessment of the workability of no-slump concrete[24,36]. This method of control has also been recommended by ACI Committee 207[77]. This test was originally developed in the UK for fibre-reinforced concrete which had a low workability[85]. A weight is placed on the plastic disc of the standard Vebe equipment. The test is then carried out in the same fashion as a standard test. It measures the workability of concrete in the range suitable for roller-compaction but is difficult to use because of the blanking of the plastic disc by the weight. The withdrawal of the plastic disc removes some paste, and this invalidates any measurement of the density of the concrete. However, an assessment of the in-situ

density of the concrete is desirable for proper control, and is possible with the Cannon Test before the concrete is placed.

Two other methods of measuring the density of the concrete after it had been placed were investigated during the CIRIA project: the sand replacement method, as used in earth-fill dams and in road construction, and a nuclear densiometer. The sensitivity of both tests, when compared with the densities measured on cores, was insufficient to determine the variations in density found in the field with roller-compacted high flyash content concrete.

5.3. INITIAL FULLSCALE TRIAL OF CIRIA PROJECT

During the CIRIA project, before the main series of mixes in the laboratory could start, it was necessary to define the range of workabilities which could be compacted by a vibratory roller, and also to define the method of assessing that workability. In a limited fullscale trial at the Cement and Concrete Association (C&CA), three small slabs, each 4 m X 2.1 m X 150 mm were compacted by a 7-tonne duplex roller. The roller was 1.98 m wide, which allowed very little clearance either side. The approximate mix proportions of the concrete were: coarse aggregate content, $a = 0.53$, paste/mortar ratio, $P = 0.45$, flyash/cementitious ratio, $C_f = 0.73$.

The water content was varied to obtain the range of workabilities required. The first slab was placed in the afternoon of the first day of the trial, using mixes with an approximate Cannon Time of 20 to 25 seconds. The concrete was spread by hand between road forms. It was then compacted with two passes without vibration (see Figure 24) and two further passes with vibration (see Figure 25). The concrete was considered to be too workable, because the roller, when vibrating but not moving, started to sink into

the concrete.

A further slab was placed the next morning, adjacent to the first slab to make a total length of 8 m. The workability of this concrete ranged from a Cannon Time of approximately 45 to 55 seconds. It was compacted using one pass of the roller without vibration, one with light vibration and four with heavy vibration. It appeared that the concrete was too stiff for easy compaction and it was concluded that a concrete having a Cannon Time in this range was not sufficiently workable for ideal 'rollability'. However, paste was noticed emerging below the side forms, which probably showed that the whole depth of concrete was being vibrated.

In the afternoon of the second day, a third slab was placed on top of the first slab. This concrete had a Cannon Time between 35 and 45 seconds. It was compacted with two passes without vibration, two with light vibration, a further one without vibration, then three passes with heavy vibration. The whole slab seemed to be very well compacted.

In order to get to the third slab, the roller had to run over the second slab placed about 4 hours previously. It was noticed that it still responded to vibration, and at the end of the trial seemed to have been better compacted and had shown no signs of distress.

The trial was very limited, but it showed that with the particular materials being used (Moorcroft crushed limestone aggregate, cement C and flyash Z - see Appendix B) the ideal rollability seemed to correspond to a Cannon Time within the range of 35 to 45 seconds. The workability of all the mixes in the subsequent CIRIA laboratory trials were designed to have a Cannon Time of 35 seconds, but a range of 25 to 50 seconds was acceptable. A pleasing feature of this small

trial was the ease with which the material, when of the right consistency, could be spread and rolled.

5.4. ACCURACY OF THE CANNON TEST

The results of the initial trial have since been proved by larger trials (see Section 15). It has been found possible to specify the range of acceptable workability for roller-compacted concrete using the Cannon test. For example, during the CIRIA fullscale trials, concrete with a Cannon Time of 35 to 45 seconds was acceptable without modification. Any concrete with a Cannon Time within the ranges of 25 to 35 and 45 to 60 seconds was also acceptable, but modifications were required to the mix proportions. Any concrete outside the overall range was not used in the trial structures. A similar form of specification was included in the documents for Milton Brook Dam.

In the CIRIA mid-term trial (see Section 15.1), a careful check was kept on the Cannon Time and also on the water contents of the aggregate. It was found possible to overcome the problem of varying moisture content in the fine aggregate more efficiently by direct reference to the Cannon Test, rather than by adjustments based on sand moisture measurements using the 'Speedy' test. Accuracies of 0.2% for the water content were obtained from calculations from the mix proportions and the workability as defined by the Cannon Test. The confidence acquired from this trial enabled a curve to be drawn for the final trial (see Figure 26), which was then used as the sole method of control. Similar relationships have been used for other placements. The cube compressive strengths of specimens taken from the concrete used in the final CIRIA trial over a period of 3 weeks in very bad weather conditions had a coefficient of variation in the order of 5%, which is generally accepted to be the level of control in a well organised laboratory experiment[86].

During the CIRIA final trial, which also served as a demonstration of the method of construction for the tenderers for Milton Brook Dam, an exercise was carried out to examine the repeatability of the Cannon Test. The same concrete was subjected to the Cannon Test with two different operators, both of whom had limited experience of the test. The variation between the two operators was only one second.

5.5. RELATIONSHIP BETWEEN CANNON TEST AND OTHER METHODS OF DETERMINING WORKABILITY

The workability of a concrete suitable for roller compaction cannot be measured by the normal methods such as slump or compacting factor. The slump of the concrete is nil, and it is also too cohesive to pass through the compacting factor hoppers, in the same way as some air-entrained concretes.

The workability of a range of high flyash content concretes was tested using both the Cannon Test and the slump and Vebe apparatus[84]. The relationship between the three methods of measurement for this particular type of concrete is shown on Figure 27. This illustration again emphasises the difference between conventional lean concrete, suitable for roller compaction, and high flyash content concrete. Whereas, in the USA, the former requires a loaded Vebe to obtain a reasonable measure of workability[24], the workability of the high flyash content concrete could be measured with a standard Vebe, but this test precludes the measurement of density possible with the Cannon Test.

6. SCOPE OF CIRIA PROJECT

6.1. MILTON BROOK DAM

Following the Author's suggestion in 1977[47] that a roller-compacted high flyash content concrete dam would be economic, a search was made for a suitable site for a prototype structure to be built. In the autumn of that year, it was suggested that the Milton Brook Dam could be suitable. This dam forms part of the Plymouth Interim (Tavy) Water Supply Scheme in the South West of England.

The Consulting Engineers for the dam, Babbie Shaw and Morton, considered that the rolled concrete dam method of construction had potential. However, before a decision could be taken on its technical and economic viability, they required that extensive testing be carried out to define the properties of the concrete mixes to be used, and also to prove the method of construction in fullscale trials.

Tenders for the construction of Milton Brook Dam were to be invited in September 1979 with construction to start in January 1980. Therefore the results of any research would have to be available by April 1979 if rolled concrete was to be considered in competition with the monolith type of construction already proposed for the dam. Early in 1978, the Client, the South West Water Authority, agreed to consider the use of rolled concrete for Milton Brook. CIRIA was approached to organise the programme of research and the Project started in April of that year. The laboratory work and fullscale trials were completed in about 12 months, and form a significant part of the work described in Thesis.

The Research Project was divided into two main parts:

1. a series of laboratory investigations carried out by the Cement and Concrete Association (C&CA) at Wexham Springs
2. a series of fullscale trials organised by the South West Water Authority (SWWA)

6.2. LABORATORY INVESTIGATION

This investigation comprised a main series of tests undertaken at the laboratory of the Construction Research Department of the Cement and Concrete Association (C&CA), tests on mixes containing alternative materials at the Central Electricity Generating Board laboratory at Carrington (CEGB), and tests on specimens made from concrete used during the fullscale trials organised by the South West Water Authority (SWWA). A variety of materials were used in the investigation as indicated in Table 3. Reference materials were chosen early in the Project as these were the most likely to be used in Milton Brook Dam. The aggregate (Moorcroft crushed limestone) was chosen because it had a lower coefficient of thermal expansion than the other aggregates available. The cement and flyash were chosen because they were the locally available cementitious materials. Additional tests on specimens to measure thermal properties were made by Taylor Woodrow Research and Development Limited.

The concrete properties examined included:

- compressive strength
- tensile strength
- modulus of elasticity in tension and compression
- tensile strain capacity
- coefficient of thermal expansion
- thermal conductivity

adiabatic temperature rise
drying shrinkage
freeze/thaw resistance

In addition, a study was made of the early-age behaviour of a high flyash content concrete for slipforming in the facing elements of the rolled concrete dam.

Triaxial testing of selected mixes was also carried out, but was limited to concretes with a maximum size of aggregate of 20 mm.

6.3. SIZE OF SPECIMENS

A maximum size of aggregate of 40 mm was selected for the CIRIA Project, because larger aggregate can give rise to segregation and are difficult to obtain in the UK. Generally, therefore, test specimens with a minimum dimension of 150 mm were used.

6.4. INITIAL SERIES OF MIXES

The first part of the laboratory trials was an investigation of the limits of the mix proportions of high flyash content concrete. Considerations of compactability and joint properties dictate a minimum paste content within the mortar ($P > 0.40$), transportation and segregation problems limit the maximum coarse aggregate content ($a < 0.55$), and there is a workability required for roller compaction. During this stage of the programme, only cubes were made and tested, and relationships were obtained between the workability (in terms of the Cannon Time) and the various factors used in the mix design (i.e. a, P, C_F and C_W).

6.5. STATISTICAL ANALYSIS OF RESULTS

All the tests were analysed using a statistical approach, because in any series there are probably some unrepresentative results, and such results are generally

termed 'outliers'. Initially a criterion after Chauvenet[87] was used. This postulates that it is reasonable under normal circumstances to reject an observation which differs from the calculated results by more than a specific factor times the residual standard deviation. However, it was ascertained that this criterion operates at a very low confidence limit (circa 80%). It was thus possible that relevant results may have been rejected. Eventually, results were considered to be outliers if they were outside a 95% confidence limit. After the rejection of an outlier, the remaining results were then restudied.

Where there are eight or more results defining a relationship, the method of least squares was used to derive the best solution. Both first degree and second degree (and even in some cases third degree) solutions were considered, and the one with the lowest residual standard deviation was chosen after taking into consideration the degrees of freedom. With less than eight points, a relationship was usually derived after taking into consideration the form of similar relationships.

7. CUBE COMPRESSIVE STRENGTH

During the CIRIA project, most tests in compression were carried out on 150-mm cubes in accordance with BS1881[88]. A limited number of 152-X 152-X 355-mm prisms were tested in compression for measurement of modulus of elasticity.

The principal age selected for testing in compression was 91 days, because it was thought that the significant flyash contents envisaged for the concrete would result in substantial long-term gains in compressive strength which may have become apparent by this age. Further tests at 28 days were made on all specimens in order to allow comparison with conventional concrete mixes for which this is the standard age of test. Additional compressive testing was carried out at 1, 2, 7 and 365 days for selected mixes. As the programme progressed, it became apparent that there was a considerable increase in compressive strength after 91 days and therefore specimens were made from all mixes for testing at 365 days during the latter half of the programme.

Cubes were also made from samples of the majority of the concretes used in the fullscale trials. In all, during the CIRIA project tests were undertaken on approximately 60 mixes containing the reference materials, and approximately 40 mixes with different materials. About 20 mixes were studied for various other purposes, such as in the design of the mix for the facing concrete and for comparison with typical concrete mixes used in conventional concrete dams.

7.1. EFFECT OF FLYASH CONTENT

Mixes with three different flyash/cementitious ratios were studied in detail, those with no flyash, and those in which flyash made up 60% and 80% of the cementitious content by volume (i.e. $C_f = 0, 0.6$ and 0.8). The second and third

proportions were chosen because they were expected to span the range of hearting concretes suitable for roller compaction. By modifying the coarse aggregate fraction, a , and paste/mortar ratio, P , it was possible to study values of water/cementitious ratio, C_w , from 0.8 to 2.0, and still maintain the workability at a level suitable for roller compaction.

Figure 28 shows the effect of the flyash/cementitious and water/cementitious ratios, C_f and C_w , on cube compressive strength at ages of 7, 28, 91 and 365 days.

For a C_f of 0.6, the cube compressive strength increases sharply at all ages with reducing water/cementitious ratio, and these mixes also show a greater increase in strength with time, when compared to the plain concrete mixes and those with a C_f of 0.8. However, the mixes with the higher flyash/cementitious ratio ($C_f = 0.8$) show the greatest increase in percentage terms with time.

For a given C_w , the high flyash content mixes have lower cube compressive strengths at all ages up to 365 days. The high flyash content concrete mixes have shown a further gain in compressive strength after the age of one year.

7.2. DEVELOPMENT OF COMPRESSIVE STRENGTH WITH AGE

With an increasing flyash/cementitious ratio, the water/cementitious ratio needed for a fixed Cannon Time decreases (see Figure 20). The development of strength with time, of two different groups of mixes were therefore examined. The mixes in the first group with a fixed coarse aggregate content ($a = 0.50$) and paste/mortar ratio ($P = 0.44$) have relatively low water contents (and water/cementitious ratios). The mixes in the other group have a higher coarse aggregate content ($a = 0.56$) and lower paste/mortar ratio ($P = 0.41$) and thus require higher water contents (and C_w 's)

to maintain the same workability. In both groups the plain concrete mixes are unsuited for dam construction, because of the high cement contents. A concrete, with mix proportions similar to that used in recent concrete dams in the UK[6,7,89] and made with the reference materials (Moorcroft crushed limestone, cement C and flyash Z - see Appendix B) has also been studied (see Mix 6, Table 17) so that a direct comparison can be made with the high flyash content concrete mixes.

The development of cube compressive strength with time of the various mixes making up each group, and also the conventional concrete, are shown in Figure 29. The strengths of the various mixes at various ages are shown in Table 4, and the method of calculating these results is shown in Appendix C.

The compressive strengths of the two plain concrete mixes are high due to the low water/cement ratio. The results are somewhat higher than was usually obtained a few years ago with the same water/cement ratio[70], and this may be an indication of the changing characteristics of modern cement. Over the past 30 years there has been an increase of nearly 80% in the cube compressive strength of a standard mix using ordinary Portland cement typical of the UK[90].

The development of the cube compressive strength with age of the high flyash content mixes is somewhat different from that of the plain concrete mixes. With the lower water/cementitious ratio mixes (Figure 29a), it is possible that the very long-term cube strength of the mix with $C_f = 0.6$ will at least equal, if not exceed, the strength of the plain concrete. At all ages, the cube strength of the mix with $C_f = 0.6$ exceeds that of the conventional dam concrete, and after an age of 28 days the mix with a $C_f = 0.8$ is approximately parallel to that of the plain concrete mix,

for both ranges of water/cementitious ratio. This suggests the two mixes are approaching their eventual maximum cube strength at the age of 365 days, however the mix with a C_f of 0.6 seems to be still gaining strength at a constant rate against the logarithm of age.

7.3. COMPRESSIVE TESTING OF PRISMS

A limited number of 152-X152-X355-mm prisms were tested in compression to obtain a value for the modulus of elasticity. From the failure loads on the prisms, the equivalent cube compressive strengths were derived[91] and these are compared with strengths measured on cubes taken from the same batch of concrete on Figure 30. Both cubes and prisms were tested in the same direction (i.e. perpendicular to the direction of casting), and the results fall near the line of equality, indicating that the method of derivation of equivalent cube strength is reasonably valid for the concretes tested.

7.4. EFFECT OF ALTERNATIVE MATERIALS ON COMPRESSIVE STRENGTH

The effect of different materials on the cube compressive strength was obtained from three series of tests:-

1. A series of mixes undertaken at the C&CA after the main programme of mixes had been completed (see Table 3). This consisted of comparing four mixes made with the reference materials (Moorcroft limestone, cement C and flyash Z) with four made with a different aggregate (a crushed granite from Hingston Down - see Appendix B), but with the reference cement and flyash. In a different series of mixes flyash Z was changed to flyash Y and similarly cement C was changed to cement A. Each set of results was then compared with the results of the mixes made with the reference materials. The results of this series of mixes are

tabulated in Table 5.

2. A series of mixes carried out at the Central Electricity Generating Board (CEGB) Laboratories at Carrington near Manchester (see Table 3). Three different aggregates, two different cements and two different flyashes were studied. All the mixes were of a similar workability and had similar flyash/cementitious ratios with C_f between 0.69 and 0.74. The water contents were varied to maintain the required workability, as were the aggregate fractions and paste/mortar ratios. C_w varied between 0.98 and 1.47.
3. The samples taken from the fullscale trials at Wimbleball provided an opportunity to study the effect of a different flyash used with the same aggregate and cement as was used in the main laboratory investigation. These results applied to only three different mixes: the facing concrete, the normal hearting, and a paste-rich hearting (which was used when the interval between placing consecutive layers was three days or more).

The analysis of the results was made in two stages. First, the properties of the mixes made with the different materials were compared with the estimated properties of a mix containing the reference materials with the same mix proportions, using information derived from the main series of tests. The cube compressive strengths were then classified as a percentage of the estimated strength of the reference mixes. Second, the water/cementitious ratio of the particular mix was compared with the estimated C_w of a mix containing the reference materials having the same aggregate fraction, the same paste/mortar ratio and the same workability. The latter comparison gave an estimate of the

water demand of the particular materials.

7.4.1. Effect of alternative aggregates

Four different aggregate combinations were used (see Table 3 and Appendix B).

1. Moorcroft crushed limestone coarse and fine aggregate, used in the reference mixes.
2. Hingston Down crushed granite coarse aggregate, with Blackpool sand, which emanates from the washing of the china clay spoils near St. Austell
3. a quartzite gravel and sand from Brynchir, North Wales
4. a crushed aggregate from Penmaenmawr, also in North Wales, known as Kingston granite

The compressive strength results of the Hingston Down mixes are compared with those of the reference mixes on Figure 31. The evidence is inconclusive, but it seems that, for a given value of C_w , the strengths, up to and including that at 91 days, are lower than those of the reference mixes, but that those at an age of 365 days are equal to, or rather higher, than the strengths of the reference mixes.

By comparing mixes of similar mix properties, it is possible to estimate that the water demand of the mixes containing Hingstone Down aggregate and Blackpool sand is approximately 30% higher than that of the Moorcroft mixes. This may be attributable more to the nature of the Blackpool Sand rather than to the crushed granite coarse aggregate. This increased water demand reduces the strength for mixes of equal workability such that it is likely that at all ages the Hingston Down mixes have a lower cube compressive strength than the equivalent Moorcroft mixes.

The mixes using the third and fourth combination of aggregates were typical hearting concretes with a C_f of approximately 0.70. The mixes were compared with those containing the reference materials using the information obtained from the main series of mixes (see Figure 28), and the results are presented in Table 6. Using these data, it is possible to compare typical hearting concretes made with each of the aggregates with constant mix proportions ($C_f = 0.715$ and $C_w = 1.210$ with no water-reducing admixture). The development of cube compressive strengths with age up to 365 days is plotted on Figure 32.

The results of the average mix with Moorcroft aggregate obtained from the CIRIA/CEGB programme fall near to those of the estimated mix containing those aggregates derived from the C&CA programme. The strength of the average Kingston mix is approximately 2 MPa less than the Moorcroft mixes until 91 days, but between 91 and 365 days there is an increase in the rate of development, such that the 365-day strength is in excess of that of the Moorcroft mixes. Similarly, the average quartzite mix has a cube compressive strength approximately 70 to 80% of that of the reference mix up to an age of 91 days, but at 365 days, exceed the strength of those mixes by 10 to 15%. This pattern is consistent, but no explanation is apparent.

Analysis of the water demand of the aggregates used in CEGB series is difficult because of the many variables in the programme. However, the demand of the mixes containing the gravel aggregates was estimated to be higher than that of the Moorcroft mixes by approximately 10%. To obtain the same workability with the Kingston aggregate, it was necessary to significantly increase the paste/mortar ratio of the mixes. It is estimated that the water demand of the Kingston aggregate was even greater than that of the gravel,

and was about 10 to 15% greater than that of the Moorcroft limestone.

The increased water content (and thus water/cementitious ratio) necessary with the alternative aggregates means that the compressive strength is reduced for a given aggregate and paste content. This in turn means that the early strengths are even lower than those shown on Figure 31, and the longer-term strengths will at best only equal that of the Moorcroft mixes.

7.4.2. Effect of alternative cements

The results of the mixes using cement A are compared with those containing the reference materials on Figure 33. Except for one mix at an age of 365 days, the cube compressive strengths of the mixes made with cement A are significantly above those made with cement C at all ages and with both flyash/cementitious ratios.

The water demand of the mixes containing cement A is also less by 5 to 10%, this further enhances the compressive strength of the concrete, as it will enable a reduction in the water/cementitious ratio for an otherwise similar mix.

The mixes containing cement B (used in the CEGB series) is compared with those containing cement C in Table 7. There is very little difference at any age. The water demand of both cements is also very similar.

7.4.3. Effect of alternative flyashes

Four alternative flyashes were studied during the CIRIA project as shown in Table 3.

The compressive test results of the mixes containing flyash Y are compared with those of mixes containing flyash Z on Figure 34. There is little difference at any

age. However the water demand of the mixes containing flyash Y was approximately 10% less than that of those containing flyash Z, and this will increase the compressive strength for otherwise equivalent mixes.

The mixes containing flyash W (used in the CEGB series) are compared with those containing flyash Z in Table 8. The early-age strengths of the mixes containing flyash W are rather lower than those with flyash Z, but the strengths at 365 days are similar. The water demand of the two flyashes is also similar.

The mid-term fullscale trial at Wimbleball (see Section 15.1) used flyash Y, as did the series of alternative material mixes at the C&CA. The development of cube compressive strength up to 91 days is shown on Figure 35, where it is compared with the estimated strength of a mix with exactly the same proportions containing the reference materials.

The estimated development of strength with age of a mix containing the reference materials for the same workability as the fullscale trial mixes is also shown on Figure 35. The mix used in the mid-term trial had a slightly lower strength than that of the nominally similar reference mix whereas the mix produced in the laboratory with flyash Y had very similar properties to that containing the reference materials at all ages. The differences were, however, small.

The estimated water demand of these mixes containing flyash Y was about 15% less than that of the mix which contained the reference materials with the same workability. This compares with the 10% decrease found in the laboratory mixes. This lower water demand in the field is a common phenomenon, and it is suggested that it is because of the

'freshness' of the flyash, the material in the laboratory having usually been stored for some time[92].

In the final fullscale trial at Wimbleball (see Section 15.3) flyash X was used in two different hearting concretes: a normal hearting and a paste-rich hearting which was used when the gap between the placement of two layers of concrete was three days or more. The development of cube compressive strength up to an age of 365 days for the normal hearting is shown on Figure 36 and compared with two mixes containing the reference materials, one with the same mix proportions, the other the same workability. A similar comparison is made for the paste-rich hearting on Figure 37. Both illustrations show a similar pattern, the mixes containing flyash X gaining strength in a very different fashion from those with flyash Z. As with the mid-term trial, the water demand of the mixes is less than the laboratory mixes (in this case by approximately 25%), so that the mixes used in the fullscale trial have significantly higher cube compressive strengths at all ages after 7 days when compared with mixes of equal workability using the reference materials.

7.5. TRIAXIAL TESTING OF SPECIMENS

Previous triaxial testing of concrete with a low cement content has suggested[27] that it behaves differently from conventional concrete. Its performance lies somewhere between that of soil cement and of conventional concrete[47,52]. Two hearting concrete mixes suitable for roller compaction were therefore tested under triaxial conditions to determine if this conclusion was applicable to high flyash content concrete. The method of test has been described elsewhere[93].

The mixes tested covered the range of flyash contents likely to be used in roller-compacted concrete, with C_f values of

0.6 and 0.8. The results of the tests are plotted on Figure 38. Also shown on the illustration are results of tests, using the same apparatus, on a conventional plain concrete with a water/cement ratio of 0.47 (by weight). It can be seen that the performance of the high flyash content concrete does not substantially differ from that of the conventional concrete mix tested, and does not equate to that of the low cement content mixes reported in the literature[27]. Further work may be desirable to simulate the triaxial stresses system likely to occur in a dam.

8. TENSILE STRENGTH

8.1. REVIEW OF TENSILE TESTING METHODS

Measurement of the tensile strength of a concrete, and indeed even a definition of that strength[94], has presented considerable difficulties over the years. The only true measurement of tensile strength is the direct or uniaxial tensile strength[95], as any other method of test measures the effect of other factors as well.

The three main methods that have been used to derive the tensile strength of concrete are:-

1. the indirect tensile or splitting test[96,97]
2. the flexural test
3. direct tensile test

The two former are standard British tests[88], but as yet the direct tensile test is not. There are a number of other non-standard tests for testing 'tensile' strength, (e.g. the testing of rings of concrete by hydraulic pressure[98]) but these were not considered for the CIRIA project because of the special equipment required. There are also variations on each of the three main methods, (e.g. the splitting of cubes, rather than cylinders, either parallel to the face[99] or on the diagonal[100]).

Several methods have been used to obtain the direct tensile strength. These include:

1. glueing plates to the end of specimens[101]
2. lateral grips on a prismatic specimen[102]
3. tensile loading using embedded steel bars[103]
4. clamping by means of truncated cones[104]
5. clamping by use of a dumb-bell shaped specimen[105]

6. gas pressure testing[106]

Certain criteria for the testing of roller-compacted concrete were necessary:

1. the method had to allow for the measurement of strain so that the tensile strain capacity could be determined
2. the preparation of specimens had to be quick, because a considerable number would be tested over a short period of time
3. specimens with a minimum dimension of 150 mm were necessary, because of the size of the aggregates to be used
4. measurement of the tensile strength of cores to be taken from the fullscale trials was required, including the measurement of the bond at joints in those cores

In addition, there was a very short time available for the development of any apparatus because of the time limitations of the CIRIA project, created due to time scale for the preparation of documents for Milton Brook Dam.

The various methods of direct tensile testing, together with the indirect and flexural tests, were analysed relative to the above criteria. This analysis is summarised in Table 9. It was apparent that the tensile strength of joints in the cores could only be measured using the glued plate technique or the gas pressure test. It was thought that the development of the glue required for the high tensile loads expected of the cores would take some time, and the gas pressure test was still under development at the time of

consideration. It was therefore decided to use both these methods for testing the cores in case either should prove unsatisfactory. For the main part of the testing in the laboratory it was necessary to have a similar test, the results of which could be compared with the tests on the cores. The most convenient direct tensile test method was to load a prism in tension through jaws which gripped laterally onto the ends of the specimens.

It was hoped that it might be possible to use the lateral gripping method to test the effect of joints in specimens. Half prisms were compacted vertically in the laboratory, using an Electric Kango system similar to that developed at Surrey University[107]. Unfortunately, this proved to be impossible because the high flyash content concrete behaved so differently from the lean concrete used previously. When the Kango hammer was used to compact a layer of concrete, the paste was driven up between the plate on the hammer and the side of the mould, and the loss of this paste would have affected the mix proportions of the concrete under test.

8.2. LATERAL GRIPPING TENSILE TESTING APPARATUS

The original method of testing prisms by lateral gripping using 'scissor grips'[102] has been further developed[108] by the use of longer plates. The two methods are compared on Figure 39. Under tension, lateral compressive forces are applied via the system of cross members and spindles to stiff side plates and the friction generated against the serrated surface allows a specimen to be gripped.

Each prism that was to be tested was 152 X 152 by 711 mm long and weighed nearly 45 kg. The tensile testing machine was designed to function with the prism in a horizontal mode as near to the floor as possible, so that it could be operated by one technician. A rectangular frame, approximately 1.5 X 0.8 m was constructed from hollow steel

sections, it was levelled and bolted to the floor as shown on Figure 40. The load was transmitted by 28-mm Macalloy bars from a 50-kN hydraulic jack to universal joints either end of the scissor grips. Steel rails were fitted inside the main frame to support the scissor grips and specimen on a level plane, and the prism and grips were able to slide on the rails during alignment and testing.

The hydraulic jack was controlled by a modified compression testing machine pump unit and the load was measured using a 50-kN load cell. Different rates of loading have been used in previous tests, ranging from 0.172 MPa/min[109] to 0.35 MPa/min[110] and 0.05 ± 0.01 MPa/second[111] (i.e. 3 ± 0.6 MPa/min). The gas pressure test method usually uses yet another rate of 1.0 MPa/min[112]. It was decided to use the last rate, because the method of test was being compared with the gas pressure test and also as it was within the range of rates used previously (see Section 16.5).

The strain was measured using two portal strain gauges (see Figure 41), located on each of the two opposite faces of the prism between the scissor grips. The gauges had a length of 200 mm and a range of ± 3000 microstrains.

The outputs from both portal strain gauges and from the load cell were fed into a data logging system. The results were punched on to a paper tape from which a plot of average stress against average strain was obtained for each prism. An example of such a stress/strain curve is shown on Figure 42.

8.3. ANALYSIS OF TENSILE STRENGTH DATA

Most of the specimens tested in tension were 28 days old. Further tests were carried out at 7 and 91 days and a few at 365 days. Cubes were made from all the mixes from which prisms for tensile testing were obtained. As with the

compressive testing, the flyash/cementitious ratios considered were the same as for the main series of mixes (i.e. $C_f = 0, 0.6$ and 0.8).

Three different relationships were statistically derived (see Section 6.5) for each flyash/cementitious ratio:

1. between the cube compressive strength, f_c , and water/cementitious ratio, C_w (see Figure 28)
2. between the direct tensile strength, f_t , and cube compressive strength, f_c
3. between the direct tensile strength, f_t , and water/cementitious ratio, C_w

After the initial relationships had been obtained, a further statistical analysis was carried out to make all three relationships compatible, and it is the latter relationships which are presented.

8.4. RELATIONSHIPS BETWEEN DIRECT TENSILE STRENGTH AND CUBE COMPRESSIVE STRENGTH DERIVED BY OTHER INVESTIGATORS

A number of relationships have been reported between the direct tensile strength and cube compressive strength of concrete. The many variables which influence this relationship, include the water/cement ratio, the size and type of aggregate and grading[108]. Other factors are the age at test and the size of specimen[113]. The direction of casting of a specimen also affects both the tensile strength and the cube compressive strength[114], and should be taken into account when comparing different sets of results.

In order to compare the present work with previous data, all the known directly comparable data are plotted on Figure 43. All the specimens were cast with the axis of loading

horizontal and compacted on a vibrating table. Other results on specimens compacted by different means (e.g. vibration under pressure[107] are not shown).

Although results are available at different ages[115,116] only those at 28 days are plotted. However, in one case, it has been suggested that only one relationship[117] is necessary for all ages of test. This relationship is also shown on Figure 43. There is a very close agreement between all the curves, considering the number of sources of data.

Other research[118] has suggested that a different relationship exists for each age, and that the development of cube compressive strength with age is different from that of tensile strength. For example, for one series of mixes at 7 days the tensile strength was 9 to 11% of the compressive strength, whereas at 365 days it was only 6 to 8%[116]. A further example of this effect, albeit for the indirect tensile strength, was a comparison of the rate of development of cube compressive strength, indirect tensile strength and modulus[119]. It was found that, at 3 days, the cube compressive strength was 30%, the indirect tensile strength 50%, and the dynamic modulus 70% of the 365-day values.

Only one of the investigations contained mixes including flyash[117] and a different relationship was found for the flyash content concrete when compared with the plain concrete mixes. However, work[120] with the indirect method of measurement found there was no change in the relationship for flyash concrete and plain concrete up to 28 days.

Thus, there seem to be different relationships between tensile strength and cube compressive strength, depending on many factors (including the age of test and possibly the flyash content).

8.5 RELATIONSHIP BETWEEN DIRECT TENSILE STRENGTH AND CUBE COMPRESSIVE STRENGTH AT 28 DAYS

All the tensile test results at an age of 28 days of the mixes containing the reference materials are plotted against the cube compressive results on Figure 44. A single relationship was derived between tensile strength and cube compressive strength, and this relationship is very similar to those previously found and shown on Figure 43.

It is possible that there are, however, three different relationships dependent upon the flyash/cementitious ratio. These are also plotted on Figure 44. With increasing flyash content, a higher tensile strength is found for a given cube compressive strength. This is contrary to previous findings[117], but in the latter work at 28 days the differences were found to be small.

8.6. RELATIONSHIP BETWEEN DIRECT TENSILE STRENGTH AND WATER/CEMENTITIOUS RATIO AT 28 DAYS

Three relationships were obtained for the plain concrete and high flyash content concrete mixes between the direct tensile strength and the water/cementitious ratio C_w . These are plotted on Figure 45. The plain concrete mixes have a higher direct tensile strength than the mixes containing flyash, except at a very low water/cementitious ratio, where the direct tensile strength of the mixes with $C_f = 0.6$ are approximately equal to that of the plain concrete mixes. However, the percentage difference between the tensile strength for mixes with $C_f = 0$ and 0.6 at a given C_w is smaller than the corresponding difference for the cube compressive strength at the same age (c.f. Figures 28 and 45).

8.7. DEVELOPMENT OF DIRECT TENSILE STRENGTH WITH AGE

All the results of tests at other ages were compared with the relationship derived from tests at 28 days. From these data, it is possible to estimate (see Appendix C) the development in direct tensile strength with age for the two series of mixes studied in detail (see Section 7.2). The series ($a = 0.50, P = 0.44$) with the lower water/cementitious ratio are plotted on Figure 46(a), and the series ($a = 0.56, P = 0.41$) with the higher water/cementitious ratio on Figure 46(b). At an age of 91 days, the direct tensile strength of the mixes with $C_f = 0.6$ are approximately equal to those of the mixes without flyash, and many exceed them at the lower water/cementitious ratio. This is partly a reflection of the different relationships found on Figure 44.

There are too few results at an age of 365 days to draw firm conclusions, but those that are available indicate that after 91 days there is continuing development in the tensile strength of the high flyash content concretes. By the age of 365 days, it seems likely that all the HFCC mixes with the lower water/cementitious ratio (Figure 45(a)) will exceed the tensile strength of the plain concrete mixes at the same age, but further tests are needed to confirm this.

8.8. EFFECT OF ALTERNATIVE MATERIALS ON TENSILE STRENGTH

Tensile testing of mixes containing the alternative materials was included in the CIRIA/C&CA programme (see Table 3 and Section 7.4). The reference materials were successively replaced with the alternative materials available near the Milton Brook dam site. The results of this series of mixes are presented in Table 5.

8.8.1. Effect of an alternative aggregate

Tensile tests were carried out on mixes in which Hingston Down aggregate and Blackpool sand were used, and the results

are compared with those of the reference mixes on Figure 47. The tensile strengths at a given water/cementitious ratio are lower at both 28 days and 91 days for both flyash/cementitious ratios, although the difference is less pronounced at 91 days. As the water demand of the alternative aggregate is higher than that of the reference aggregate (see Section 7.4.1), this reduction in tensile strength will be even more marked for mixes of equal workability.

8.8.2. Effect of an alternative cement

The results of the tensile testing of the mixes, in which cement A was substituted for cement C are shown on Figure 48. At the lower flyash/cementitious ratio the results are very similar to those of the reference mixes, but at the higher flyash/cementitious ratio there is a considerable improvement in tensile strength at 91 days. Because of the lower water demand of cement A (see Section 7.4.2) it is possible to reduce the water content (and thus C_w) for a given workability. This means that an increase in tensile strength is possible by changing from cement C to cement A, particularly at $C_f = 0.8$.

8.8.3. Effect of an alternative flyash

The results of the tensile testing of the mixes, in which flyash Y was substituted for flyash Z are shown on Figure 49. Apart from one mix (mix 109 - see Table 5), which gave low values throughout the tests, the results are close to those of the reference mixes. However, the reduced water demand of flyash Y (see Section 7.4.3) means that a reduced water content is possible for a given workability. Thus an improvement in the tensile strength would be forthcoming with a change from flyash Z to flyash Y, due to the reduced water/cementitious ratio possible.

8.9. DISTRIBUTION OF FAILURES IN TENSILE TESTING APPARATUS

An analysis was made of the location of the failures of the prisms when subjected to direct tensile testing in the lateral gripping apparatus. The prisms were 152 X 152 by 711 mm long with the side plates gripping the end 240 mm leaving approximately 230 mm between the plates. The two spindles which applied the pressure to each plate were approximately 50 mm from the end of the plates. This arrangement is shown on Figure 40 and a diagrammatic sketch is also shown at the bottom of Figure 50, which shows the location of failures together with the failure loads. Out of the 126 specimens, 98 (or 78%) failed in the central 290 mm (41% of the total length), i.e. between the influence of the spindles. This is comparable with previous data[107] for 100 X 100 mm specimens, where 74% of the specimens failed in the central 170 mm (33% of the length).

Table 10 shows the percentage of failures which occurred within the plates and within the central zone for each range of failure stress. It can be seen that the chances of the failure being within the plates increases substantially as the tensile strength of the prism increases.

Failures were located symmetrically about the centre of the specimen. This is different from previous results using similar prisms tested in a vertical mode[108], which had more failures in the top third of the prism than in the central and bottom third.

The average failure stress in each of the increments of length was considered and there was a definite tendency for the lower strength specimens to fail towards the centre of the specimen. This may mean that the measurement of the tensile strength tends to underestimate the higher strength specimens.

9. MODULUS OF ELASTICITY

9.1. METHOD OF MEASUREMENT

There are two fundamentally different ways of determining the modulus of elasticity of a concrete. The first method is non-destructive and involves subjecting a prism to longitudinal vibration to identify its natural frequency. From this frequency the dynamic modulus can be calculated. The British Standard method of this test[121] was used on a number of specimens just prior to testing in direct tension. The second method of measurement uses the relationship between the stress and strain measured during the testing to failure of a specimen. Stress/strain relationships can be obtained either during the direct tensile test or during testing of a prism in flexure. In either case the stress/strain curve is used to determine the secant modulus, either tensile or flexural (see Figure 51) This is known as the static modulus.

The static modulus can also be measured in compression. During this study, many more data were obtained for the modulus in tension and unfortunately there is a conflict of opinion on the relationship between the two moduli[107]. In addition, there is no internationally accepted way of determining the secant (and thus the static) modulus in tension. Some investigators calculate the value at fixed levels of stress, and some at a percentage of the ultimate strength[122]. Again, different percentages are used in compression and in tension. The form of the stress/strain curve under the different loading modes is also different, tensile loading producing a more linear relationship than compressive testing.

The method of calculation used in this Thesis for the static modulus in tension was a compromise between the various methods used elsewhere. The secant modulus at 50% and at

90% of the direct tensile strength, and also the modulus of the best-fit line through the curve are all averaged. Generally, the differences between the various evaluations of the modulus was small, but at early ages and with the mixes with a C_f of 0.8, both of which have lower strengths, the stress/strain curve became more curvilinear and the difference was more pronounced.

The compressive tests were conducted on half prisms and had dimensions of 150 X 150 X 355 mm (see Figure 52). Pairs of Demec studs 200 mm apart were stuck to the four sides of the prism. The specimens were then loaded in 50-kN increments and the movement of the studs was measured. The average of the results on all four faces was used for the plotting of the stress/strain curve. The secant modulus at 50% of the ultimate strength was defined as the static modulus, this percentage having been used by a number of organisations[123].

9.2. RELATIONSHIP BETWEEN STATIC MODULUS IN TENSION AND DIRECT TENSILE STRENGTH

All the measurements of static modulus on the mixes containing the reference materials are plotted against the direct tensile strength on Figure 53 and compared with a relationship previously found[107] for a cement-bound road material using aggregates from the Thames Valley - a flint gravel. At all but the lower strengths, the values of modulus of the latter mixes are greater than those measured in this study. The static modulus in direct tension is seldom measured, and the only other results traced also contained Thames Valley gravel[124]. These results fall somewhere between the two curves on Figure 53.

A possible explanation for these differences is the low percentage of voids in high flyash content concrete. It has been found[107] that loss in density in a lean concrete

causes a greater reduction in tensile strength than in modulus in tension. For example, a 2.5% reduction in density (due to incomplete compaction, or for whatever cause) reduces the tensile strength by some 25 to 30%, but the modulus by only 15%. Similar results have also been found during tests for modulus in compression[125]. Consequently, as the high flyash content concrete usually has a density very near to the t.a.f. and significantly higher than lean concrete, the modulus would be lower than that of a lean concrete of a given tensile strength. Statistically, it was found that there were different linear relationships for each age of testing as can be seen on Figure 54. The gradients of the lines decrease with increasing age, implying that the development of static modulus is initially faster than the development of tensile strength. This confirms previous findings[119,120].

The residual standard deviation of the curvilinear relationship for all ages, as shown on Figure 54, is very similar to that of the separate linear relationships for each age, and for simplicity the former has been used throughout this Thesis.

9.3. RELATIONSHIP BETWEEN STATIC MODULUS IN TENSION AND WATER/CEMENTITIOUS RATIO AT 28 DAYS

At an age of 28 days the static modulus of the mixes with the three flyash/cementitious ratios considered in detail, can be related to the water/cementitious ratio as shown on Figure 55. This Figure may be compared with Figure 45 where the tensile strength is plotted against C_w . It is apparent that increasing the flyash/cementitious ratio has less effect on the modulus at 28 days than it has on the direct tensile strength. The relationships on Figure 55 are compatible with those on Figure 45, given the relationship between the static modulus and tensile strength on Figure 53.

9.4. DEVELOPMENT OF STATIC MODULUS IN TENSION WITH AGE

From the static modulus values measured on specimens of various ages and the relationships derived for an age of 28 days, it is possible to estimate the development of static modulus in tension with age for the two series of mixes being studied in detail (see Appendix C). The estimated static modulus for the mixes in the series with the lower water/cementitious ratios ($a = 0.56$, $P = 0.44$) are plotted on Figure 56(a), and for the series with the higher water/cementitious ratios ($a = 0.56$, $P = 0.41$) on Figure 56(b). These figures may be directly compared with Figures 29 and 46. At all ages, but particularly so at early ages, it is apparent that the flyash/cementitious ratio has less effect on the modulus than on the tensile strength.

There are limited data at an age of 365 days but the indications are that there is little or no increase in modulus from 91 to 365 days. It therefore seems that the ultimate static modulus of a concrete suitable for roller compaction, decreases as the flyash/cementitious ratio is increased.

The development of static modulus is very rapid at early ages in a conventional plain concrete. This rapid development is slowed by the addition of flyash to the mix, and this may be a factor which could reduce early-age cracking.

9.5. EFFECT OF ALTERNATIVE MATERIALS ON STATIC MODULUS

The static modulus was measured during the series of tests on alternative materials carried out in the CIRIA/C&CA programme (see Sections 7.4 and 8.8 and Table 3). The results are presented in Table 5.

9.5.1. Effect of an alternative aggregate

The results of the measurement of static modulus on the mixes containing Hingston Down coarse aggregate and Blackpool sand are compared with those on the reference mixes on Figure 57. As with the tensile strength, (see Figure 47) the modulus is lower than that for the reference mixes at both 28 and 91 days for the lower flyash/cementitious ratio, but at 91 days with $C_f = 0.8$ the differences seem small.

9.5.2. Effect of an alternative cement

The results of the measurement of static modulus on the mixes containing cement A substituted for cement C are compared with those on the reference mixes on Figure 58. The modulus of three of the four mixes show little or no increase between 28 and 91 days, and all four mixes have moduli above those of the reference mixes. This higher modulus is comparable with the increase in both the compressive strength and tensile strength for the same change of cement.

9.5.3 Effect of an alternative flyash

The results of the measurement of static modulus on the mixes containing flyash Y substituted for flyash Z are compared with those on the reference mixes on Figure 59. At 7 days, there is little difference in static modulus between the two sets of results with both flyash/cementitious ratios but at 28 days (and particularly at 91 days) the moduli of the mixes containing flyash Y are higher than those containing the reference flyash Z, this increase being more pronounced with the higher flyash/cementitious ratio.

9.6. RELATIONSHIP BETWEEN DYNAMIC MODULUS AND STATIC MODULUS

The relationship between the static modulus (E_s) and the dynamic modulus (E_d) was derived for results of tests on

prisms both at an age of 28 days and also for results at all ages. The statistical 'best fit' for both relationships are second-degree curves. However, the linear relationships have standard deviations and coefficients of variation very similar to the second-degree curves, and it is suggested that the former are used for ease of calculation. The two 'best fit' linear equations are:

For 28-day results: $Est = 0.95Ed - 9.95$, which can be simplified to: $Est = Ed - 10$ (in GPa)

For results at all ages: $Est = 0.98Ed - 10.6$ or $Est = Ed - 10.5$ (in GPa)

The relationships are sufficiently similar for the latter to be used for all results. This relationship is plotted on Figure 60.

A number of similar relationships between dynamic and static modulus have been reported, the majority relating to the static modulus in compression. Examples of these relationships are:

1. $Ed = Est + 5$	or $Est = Ed - 5$	Ref. 107 (Tension)
2. $Ed = 7/6 Esc$	or $Esc = 0.86 Ed$	Ref. 127 (Compression)
3. $Esc = 1.25 Ed - 19$		Ref. 128 (Compression)
4. $Esc = Ed - 6.9$		Ref. 129 (Compression)

These relationships all lie close to or within the 95% confidence limits on Figure 60.

9.7. STATIC MODULUS IN COMPRESSION

Very few tests to obtain the static modulus in compression were carried out, because the major property required of concrete in dams is in tension - the tensile strain capacity. A reason for the compression testing was to find

if the modulus in tension was equal to the modulus in compression, as has been suggested by a number of researchers[107] or if there are major differences as has been suggested by others[130].

The static modulus in compression was calculated from the strain at 50% of the ultimate compressive stress. A typical stress/strain curve is shown on Figure 61 and it can be seen that it is very much more curvilinear than the stress/strain curve in tension. All the results of the compressive testing are shown in Table 11. The equivalent cube strengths were derived[91] and compared with the measured cube strengths on Figure 30, there was close correlation.

There are limited data, but the modulus in compression seems to be 70 to 80% of that in tension at 28 days and slightly greater than the modulus in tension at 91 days.

The relationship between the static modulus in compression and compressive strength of the high flyash content concrete mix has been compared with those derived by others[131 to 134]. There is close correlation.

10. TENSILE STRAIN CAPACITY

10.1. METHOD OF CALCULATION

The methods of estimating the tensile strain capacity of a concrete are even more diverse than the methods of testing concrete in tension. Three main methods of calculating the tensile strain capacity have been found:

1. The US Army Engineers[135,136] use the modulus in compression and the tensile strength derived from the flexural test.
2. In the USSR[137] the dynamic modulus is used together with the tensile strength as defined by the splitting test.
3. A method used to a limited extent in the UK[138] uses the dynamic modulus and the tensile strength from the flexural test, so that the values are obtained on the same test specimen.

None of these tests directly measure the tensile strain capacity. However, it is postulated[139] that as the modulus of rupture (the tensile strength calculated from the flexural test) is some 20 to 40% higher than the uniaxial tensile strength, and the modulus in compression is approximately the same percentage higher than the modulus in tension, the method used by the US Army Engineers indirectly reflects a reasonably accurate measure of the property. Neither of these suppositions is universally agreed, the modulus of rupture being variously estimated to be 50 to 60%[118] or even up to 100%[140] higher than the direct tensile strength. As was shown in Section 9, the static modulus in compression may increase at a different rate from that in tension, and could generally be less than the latter

up to an age of between 28 and 91 days.

The tensile strength measured using the splitting test is nearer to the direct tensile strength than the modulus of rupture, but is probably 5 to 12% higher[123]. The dynamic modulus is also higher (see Section 9.6), so that the second method may produce values nearer to the true tensile strain capacity. The third method produces higher values, because the overestimate of tensile strength indicated by the modulus of rupture is proportionally greater than the overestimate of static modulus by the dynamic test.

The rate of loading has a significant effect on the tensile strain capacity, because if the rate is slow enough creep of the concrete will take place[141]. Concrete in dams can be subjected to either relatively fast changes of strain (e.g. on the face of a dam as a result of changes in ambient temperatures throughout the day), or to changes taking place over many months or years (e.g. slow cooling of the centre of a dam) (see Section 3.1). The variation in temperature on the surface of a concrete can be significantly greater than the variation in air temperature[142,143]. Therefore, it is important to have a measure of the tensile strain capacity when the concrete is subjected to both slow loading and rapid loading. Of the three methods described above, only that used by the US Army Engineers[66,144] takes this into consideration. Both rapid loading, 0.28 MPa per min, and slow loading, 0.17 MPa per week, are used. It has been found that the slow loading test gives values which are 1.1 to 2.1 times that of the rapid loading at the same age[145].

The method of calculation used to derive the tensile strain capacity in this Thesis is to divide the ultimate direct tensile strength by the static modulus in tension. These values are generally lower for the same concrete than the tensile strain capacity calculated by the other methods, for

the reasons presented above. The tensile strain capacity under slow loading conditions was not considered.

A direct measurement of the tensile strain capacity was obtained at an age of 7 days, during one of the investigations of the thermal conductivity of the facing concrete (see Section 11.3). Values of between 100 and 110 microstrain were obtained at failure. This value is somewhat higher than would have been estimated from the main series of mixes (from a Figure similar to Figure 62), which, at 7 days, suggested a tensile strain capacity for the particular mix of between 80 and 85 microstrain.

10.2. RELATIONSHIP BETWEEN TENSILE STRAIN CAPACITY AND WATER/CEMENTITIOUS RATIO AT 28 DAYS

Using the data from Figures 45 and 55 the direct tensile strain capacity of mixes containing the reference material at an age of 28 days was calculated and plotted against the water/cementitious ratio, C_w , on Figure 62. As the water/cementitious ratio is decreased, the difference between the strain capacity of the mixes having the different flyash/cementitious ratios decrease, even at this relatively early age. With increasing flyash content, the water content of a mix can be reduced (see Figure 20) for mixes of equal workability. If this is taken into account, the difference between the strain capacities of mixes containing different contents of flyash is reduced even further.

10.3. DEVELOPMENT OF TENSILE STRAIN CAPACITY WITH AGE

There are few results from which to calculate the tensile strain capacity at ages other than 28 days but it is possible to make an estimate of the development of strain capacity with age of the two series of mixes previously studied (see Appendix C). Figure 63(a) shows the estimated development of strain capacity for the mixes with the lower water/cementitious ratios ($a = 0.50$, $P = 0.44$), and

Figure 63(b) for those with the higher water/cementitious ratios ($a = 0.56$, $P = 0.41$). With the former, the values of tensile strain capacity for all the mixes are effectively equal at 91 days, even when 80% of the cementitious material is flyash. It is difficult to predict what happens at later ages, but the few results available indicate that the strain capacity of the high flyash content concrete mixes continues to increase beyond 91 days while the tensile strain capacity of the plain concrete mixes stays sensibly constant. Other work with flyash content concrete of similar proportions to a conventional dam concrete has shown a similar growth in tensile strain capacity[66]. The latter work even indicated some decrease in the strain capacity of plain concrete mixes after 91 days, because the modulus continued to increase although the tensile strength did not.

Although the use of a high flyash content in concrete causes a decrease in the cube compressive strength, when compared with a plain concrete (see Figure 29), there is little difference in the tensile strain capacity of all the mixes in each series (see Figure 63).

The mix shown on Figure 63(a) with $C_f = 0.8$ (cement content approximately 70 kg/m^3 and flyash content approximately 200 kg/m^3) would be suitable for dam construction, but the mix without flyash is unsuitable because the cement content is too high at approximately 310 kg/m^3 . The long-term tensile strain capacities of these two mixes are similar. However the heat generated in any large structure, which defines the long-term strain induced in that structure (see Section 3), is several times higher with the plain concrete than with the mix containing a high proportion of flyash, thus increasing the risk of thermal cracking.

10.4. EFFECT OF ALTERNATIVE MATERIALS ON TENSILE STRAIN CAPACITY

There are limited data available for any positive comparison of the tensile strain capacity of concretes using the alternative materials available for the Milton Brook dam. However, a general assessment is necessary. The two sets of mixes considered previously ($a = 0.50$, $P = 0.44$ and $a = 0.56$, $P = 0.41$) were analysed for mixes with flyash/cementitious ratios of 0.6 and 0.8. These four mixes are likely to encompass the full range of hearting concretes which could be used in the Milton Brook dam.

Figure 64 illustrates the effect on strain capacity of substituting in turn:

1. Hingston Down crushed granite aggregate for Moorcroft crushed limestone
2. cement A for cement C
3. flyash Y for flyash Z

For mixes with the lower water/cementitious ratio and $C_f = 0.6$ (Figure 64(a)), there is very little difference in the tensile strain capacity of all the mixes at 91 days, although with the alternative aggregate (Hingston Down), the strain capacity at earlier ages is lower than that of the other mixes. For the other series of mixes with the lower water/cementitious ratios and with $C_f = 0.8$ (see Figure 64(b)), most mixes have a slight decrease in strain capacity compared with that of the mixes with $C_f = 0.6$, although with the mix containing cement A, this decrease is marginal. However, with the Hingston Down aggregate mixes, the decrease is significant.

For the mixes with the higher water/cementitious ratios and

$C_f = 0.6$ (Figure 64(c)), there is again little difference in the strain capacity at 91 days, and as with lower water content mixes, the alternative aggregate mix has a lower strain capacity at earlier ages.

Mixes with the higher water contents and $C_f = 0.8$ (Figure 64(d)), exhibit substantial decrease in strain capacity, again apart from the mix containing cement A. The mix with Hingston Down aggregate has a lower strain capacity than the other three mixes at all ages. One result at an age of 365 days suggests a continuing growth in strain capacity up to, and possibly beyond, the age of 365 days.

11. THERMAL PROPERTIES

11.1 BACKGROUND

The main objective of the CIRIA/C&CA laboratory study was to examine the tensile properties and, in particular, the tensile strain capacity of roller-compacted concrete. The actual applied strains due to external loading are very small in a concrete gravity dam[146], and thermal stresses are usually the limiting criteria. Therefore, an increase in the strain capacity was considered to be an increase in the capacity of the concrete to resist cracking. However, of equal importance is the need to reduce the tensile strains which arise from thermal movement. Any reduction in thermal movement also reduces the likelihood of cracking.

Investigations were undertaken to assess the heat likely to be generated in a massive structure and to measure the properties (e.g. coefficient of thermal expansion and thermal conductivity) which influence the thermal movement of concrete (see Table 12).

11.2. COEFFICIENT OF THERMAL EXPANSION

The coefficient of thermal expansion of a concrete is largely dependent upon the type of aggregate[147] and the moisture content of the concrete[148]. Age does not seem to be a major factor[148]. In the environment of a dam, the concrete is likely to be saturated, and it was therefore in this state that all the tests were carried out.

Limestone aggregates have low values of coefficient of thermal expansion and this was one of the major reasons for the choice of limestone aggregate for Milton Brook Dam, all the other aggregates in the area being igneous.

The coefficient of thermal expansion of pastes and of concretes were measured containing various proportions of

flyash. The specimens tested contained the reference materials (see Table 12). The results are plotted on Figure 65.

From the results of the tests on the pastes, an estimate was made of the coefficient of thermal expansion of a typical Moorcroft limestone concrete containing 20% paste and 80% aggregate. This is also plotted on Figure 65.

The measured coefficient of thermal expansion of concretes containing Moorcroft aggregates were obtained using the 'hot box' tests (see Section 11.4), and also by immersing prisms into a water bath, measuring movements of the concrete during both the heating and cooling cycles. Little or no difference was found in the coefficients during the heating and cooling, confirming previous work[143].

Figure 65 shows that the measured values compare very favourably with the predicted values calculated from the results of testing the pastes.

These results indicate that a high flyash content concrete ($0.6 < C_f < 0.8$) could slightly reduce the coefficient of thermal expansion when compared to a plain concrete. A similar reduction in coefficient (of approximately 7%) has been found in previous work with flyash in concrete with C_f as low as 0.35[136].

11.3. THERMAL CONDUCTIVITY

The thermal conductivity, k , is a measure of the rate at which heat transfers from one part of a material to another. In concrete, it is dependent upon the density of the concrete and on the proportions of the mix, together with the thermal conductivity of the individual constituents. It is significantly altered by the degree of saturation of the concrete, because the thermal conductivity of water is about

25 times that of air. Prediction of the thermal conductivity of concrete from the composition is very uncertain because of the many factors involved[149]. Consequently, the thermal conductivity of a typical hearting and a typical facing concrete was measured at the Transport and Road Research Laboratory (TRRL) using a method developed at the C&CA[150]. It was during these tests that a direct measurement of the 7-day tensile strain capacity was obtained of the facing concrete referred to in Section 10.1. The analysis of the results proved to be very difficult with a wide range of possible values of thermal conductivity fitting the data, a problem which has been reported previously[151]. A further series of tests was therefore conducted using the 'hot plate thermal conductivity apparatus'[152]. From these tests, the thermal conductivity of the hearting concrete was estimated to be in the range of 1.7 to 2.0 W/m.

11.4. ADIABATIC TEMPERATURE RISE

The adiabatic temperature rise of two series of mixes containing Moorcroft limestone and cement C with various proportions of flyash Z, were directly measured using the 'hot box' method[153] (see Table 12).

Three 300-mm cubes made of a typical hearting concrete ($\alpha = 0.53$, $P = 0.44$, $C_f = 0.73$ and $C_w = 1.04$) were tested during the initial series. The average result of the tests is plotted on Figure 66. The placing temperature was approximately 23C and rises of 15.7degC, 16.8degC and 18.2degC were recorded. A further series of tests was carried out on a plain concrete and a HFC concrete having a value of C_f of 0.6. These mixes were placed at a temperature of approximately 15C and the results of the temperature rises are plotted on Figure 66. The average result of the first series was corrected to the same placing temperature so that a direct comparison is possible.

The ultimate temperature rises of the 'hot box' tests are plotted against the flyash/cementitious ratio on Figure 67. The rises were also modified to be applicable to a concrete mix containing a total cementitious content of 250 kg/m^3 . In practice, a greater weight of cement is required for the same volume of cementitious material because of the higher specific gravity of cement. The shape of the curve is similar to that found previously[154].

The adiabatic temperature rise of the plain concrete mix (mix 3) was approximately 14°C per 100 kg/m^3 of cement C. This is very similar to previous results using the same cement[6] and to results found in the USA[155].

11.5. HEAT EVOLUTION OF CEMENT/FLYASH MIXTURES

The rate of heat evolution was measured by isothermal conduction calorimetry[156] for cements A and C by themselves and combined with flyashes Y and Z at three different temperatures. The flyash/cementitious ratios were 0.60 and 0.73, the latter being the value most likely to be used in the hearting concrete at Milton Brook. The water content was equivalent to that for a typical hearting concrete suitable for roller compaction. Readings of the rate of heat evolution were taken at 6-min intervals for at least 7 days. The total heat of hydration at any given time was found by finding the area under the rate of heat evolution curve.

In practice, for the various concretes being considered, the majority of the hydration reactions would take place at temperatures in the range of 20 to 50°C . The usual fixed constant temperatures for the rate of heat evolution test are 20°C and 50°C . These were therefore used, together with an additional temperature of 10°C to allow for low placing temperatures such as occurred at the Wembley final trial

(see Section 15.3).

The two cements were found to have significantly different patterns of heat evolution by themselves or when mixed with flyash. For example, the rate of heat of evolution curve of cement A with flyash Y contains only one peak (see Figure 68), while cement C with flyash Z generates two (see Figure 69). The effect of this on the total heat of hydration is relatively small as shown in Figure 70. For comparison, the curve for neat cement C is superimposed.

A summary of the results is given in Tables 13 and 14. Cement C has a total heat of hydration of about 315 kJ/kg compared with about 300 kJ/kg for cement A. This small difference applies also to the cement/flyash mixtures. The total heat of hydration of a paste containing 73% flyash in the cementitious material is approximately 50% of that of a paste containing neat cement.

Apart from the expected reduction in total heat of hydration, the most significant effect resulting from the inclusion of flyash in the cementitious content is the considerable reduction in the main peak rate of heat evolution. However, the auxiliary peaks in the cement C mixtures at 20°C are not reduced by the inclusion of flyash Z, and are increased by the addition of flyash Y.

The shape of the rate of heat of evolution is therefore different for the different cements, and the total heat of hydration is different for the different flyashes. It was concluded from the tests that there is still much to learn about the heat of hydration of flyash/cement mixtures.

12. PROPERTIES OF THE HEARTING CONCRETE INFLUENCING DURABILITY

The durability of a concrete is difficult to define in general terms, and it is best related to a particular application. Within the context of this part of the Thesis it is clear that the facing concrete must be resistant to cycling of freezing and thawing, while both the facing and hearting should have low permeability. In addition, the drying shrinkage and the thermal movement should be kept as low as possible in order to minimise the strain induced in the structure. Movements other than thermal are considered in this Section.

All adequately designed and well compacted concrete is 'watertight' in the sense that it is sufficiently impermeable to satisfy normal engineering requirements for a dam. A permeability of between 10^{-10} and 10^{-12} m/s is probably typical for a concrete used in dams[157], and at this level only a negligible amount of seepage emerges at the downstream face of a structure and this evaporates to leave the surface dry. The most frequent problems which arise to make a concrete permeable are cracking and faulty workmanship, particularly at joints. Seepage through cracks or joints in itself is not necessarily harmful, although it can be unsightly, but seepage through the concrete can cause leaching and deterioration if the properties of the hearting are not satisfactory[158,159]. The addition of flyash to concrete substantially reduces the effects of leakage[160]. In spite of the protection of the facing concrete, it is possible that under certain circumstances the hearting will be subjected to seepage, and as such must be 'durable' within itself.

12.1. PERMEABILITY

It is difficult to measure permeability on moulded specimens, so all the tests were carried out on samples of concrete taken from the fullscale trials or on the trial banks themselves (see Section 16). These tests showed that the concrete had a permeability of approximately 10^{-13} m/s, both in the parent material and at the joints.

12.2. SHRINKAGE

Dimensional changes occur in concrete for several reasons. Excluding the thermal movement (discussed in Section 11), there are three essentially different forms:

1. drying shrinkage, which is 'a time dependent volume change caused by drying'[143]
2. autogenous movement, which is 'a time dependent movement caused by chemical changes'
3. creep, which is 'a time dependent deformation due to load'[143] (see Section 3)

Drying shrinkage is dependent upon the size of the specimen being measured[161,162] and the changes found in laboratory samples are generally greater than the values which occur in a dam. This is particularly so because drying of the concrete in a dam is very slow. Because of the saturated state of a dam, there are some indications that an expansion, rather than a contraction, might take place[163]. Nevertheless, results of laboratory tests yield comparative values of shrinkage. The specimens for drying shrinkage testing were made during the intermediate fullscale trials at the C&CA (see Section 15). The approximate mix proportions are shown in Table 15.

Two specimens (each 100 X 100 X 355 mm) were cut from a

152 X 152 X 711 mm prism made from each mix. One was sealed in a butyl rubber lining, and the other was left to dry from an approximate age of 28 days in a room controlled at a constant temperature of 20C and a relative humidity of 65%.

The dimensional changes of the specimens are plotted on Figure 71 and it is clear that the shrinkage is very low. After one year of drying, the shrinkage is in the range of 130 to 190 microstrain, compared with 325 to 475 for measurements in the same laboratory on OPC concrete with a higher workability (50-mm slump)[164]. Drying shrinkage is usually considered to be directly proportioned to the water content[164,165], and the results of the tests undertaken(in this project confirm this hypothesis, since the water content of the mixes was approximately 100 kg/m^3 (or 4% of the total ingredients by weight). This is half the water content of the conventional concrete specimens tested in the same laboratory, and the drying shrinkage was also found to be half. The equivalent drying shrinkage of concrete in 11 dams in the USA have been reported to be between 270 and 1000 microstrain[2]. It therefore seems that the shrinkage of the high flyash content concrete suitable for roller compaction is less than half that expected with a conventional dam concrete.

It was hoped to measure the autogenous movement using the sealed concrete specimens, but the sealing was not entirely satisfactory and the results were unreliable. However, it has been reported[160] that reduction in Portland cement content reduces the autogenous shrinkage, irrespective of pozzolan content or storage temperature. Other previous work[166,167] also suggests that the autogenous movement of the particular concrete being studied is likely to be small.

13. FACING CONCRETE

13.1. REQUIREMENTS OF THE FACING CONCRETE

The facing concrete is to provide a durable outside skin (probably some 750 mm thick) for the dam while also protecting the hearting concrete from severe changes in temperature. The concrete must have a closed and dense finish, and also be compatible with the hearting so that the two materials can act as a monolithic structure[168].

The particular method of construction proposed must also be considered. Because the most economic cross-section of a concrete gravity dam generally has a vertical upstream face, it should be possible to slipform the concrete to such an alignment (see Figure 2). At an early age (12 to 18 hours) the concrete must also be sufficiently strong to be able to withstand the action of a heavy vibratory roller compacting the hearting concrete right alongside the slipformed facing element.

During the mid-term fullscale trial at Wimbleball (see Section 15.1) a conventional concrete could be made sufficiently workable to produce the required finish, but the concrete 'slumped' after emerging from the mould. Alternatively, the concrete could be designed to stand vertically but would then have insufficient workability to produce a dense finish. The concrete left the mould 1.5 to 2 minutes after vibration, and it was not possible to increase this time. If the mould were made longer, this would increase the volume of concrete within the mould and the slipform paver would then have been incapable of controlling the weight of concrete. At any speed slower than the optimum of 500 to 600 mm/min the paver becomes very difficult to steer.

Offset slipform pavers have been used to form many

kilometres of large median barriers for roads. However, the design of these barriers does not generally include a vertical face, a minimum slope of 10 vertical to 1 horizontal being common. A mix design method has been traced for concrete to be used in offset pavers[169,170]. For the section envisaged for the facing elements, the concrete designed by this method would require a very high cement content (probably between 400 to 500 kg/m³) which would create a significant thermal problem in the environment of a dam.

13.2. LABORATORY TRIAL MIX PROGRAMME

Relatively little is known about the properties of concrete at very early ages[171]. Therefore a programme was designed to test the ability of a concrete to stand in a vertical shortly after vibration.

The standard Vebe test[84] was chosen for the measurement of the workability of the facing concrete because it is particularly sensitive to mixes of low workability. The finish achieved in a given concrete is likely to be improved with a higher workability. Therefore any increase in workability (decrease in Vebe time) was considered likely to improve the quality of the finish.

The capability of the fresh concrete to stand immediately after being slipformed depends upon its shear strength, cohesion and stress/strain characteristics in the fresh state. In this state, a test specimen is in a similar condition to the 'undrained' state in soil testing. To simulate such a condition, cylinders wrapped in polythene were tested 30 min after the concrete had been mixed, the earliest practical interval commensurate with measuring workability and air content, and with making and demoulding of the cylinders. Two cylinders from each mix were tested and each was subjected to deformation at a constant rate of

approximately 25 mm/min (the strain being measured from the cross-head movement). For each mix tested failure stress and stiffness were obtained, where stiffness is defined as the initial slope of the stress/strain curve (see Figure 72).

13.3. EFFECT OF VARIATIONS OF MIX PROPORTIONS

The mix has to be particularly cohesive for the concrete to both stand at an early age and also to produce a good face. The high flyash content hearting concrete was a cohesive mix, although designed to have a different workability. It was therefore decided to use a similar method of design for the facing concrete. This led to a relatively high flyash content being used to overfill the voids in the fine aggregate. As with the hearting concrete, the facing concrete was defined in terms of coarse aggregate content, a , paste/mortar ratio, P , flyash/cementitious ratio, C_f , and water/cementitious ratio, C_w . Each of these was varied in turn to examine the effect of the variations on the relevant properties of the concrete. The only factor not varied was the flyash/cementitious ratio, because some concern was initially felt about the durability of high flyash content concrete and the C_f was fixed at 0.5.

An air-entraining agent was added to all the mixes in order to improve the resistance of the concrete to deterioration under the action of freezing and thawing. A water-reducing admixture was also used for the reasons given in Section 4.4.

13.3.1. Effect of variation in coarse aggregate content

Figure 73 shows the effect of variations in coarse aggregate content on workability, modulus and ultimate stress. The strength of the fresh concrete does not seem to be significantly influenced by a change in coarse aggregate content, but an increase above a content of 0.40 (75% of the

value for the loose bulk density) leads to a decrease in workability and a corresponding increase to the stiffness of the concrete.

13.3.2. Effect of variation in paste/mortar ratio

The effect of variation in paste content within the mortar is shown on Figure 74. An increase in paste content significantly reduces both the modulus and the strength of the fresh concrete, but it has less effect on the workability.

13.3.3. Effect of variation in water/cementitious ratio

The effect of variation of the mix water/cementitious ratio is shown on Figure 75. As was found during the mid-term trial (see Section 15.1) the use of water content as a means of achieving a change in the required properties is usually self defeating, because an improvement in one property (either workability or strength) is counteracted by a deterioration in the other.

13.4. GENERAL COMMENTS ON TRIAL MIX PROGRAMME

The principal finding of the limited mix programme was that alterations in the water content (as thus water/cementitious ratio) is not a satisfactory way of modifying either workability or early strength of a concrete unless there is a significant excess in one of the required properties. If the strength of the concrete is adequate but an increase in workability is required, a decrease in coarse aggregate content is the best solution. Similarly if the workability, but not the strength, is adequate, a decrease in paste/mortar ratio corrects the deficiencies. The water/cementitious ratio is best fixed to obtain the compressive strength (or other property) required in the hardened state, and only adjusted as a last resort.

13.5. LONG-TERM PROPERTIES OF THE FACING CONCRETE

Cubes and prisms were made from a typical mix during the programme. Selected properties of the hardened concrete containing the reference materials at ages up to 91 days are shown in Table 16. These compare favourably with those of a conventional facing concrete as used in the construction of a concrete dam (see Section 14.2).

14. COMPARISON OF LABORATORY TEST RESULTS OF VARIOUS CONCRETES SUITABLE FOR DAM CONSTRUCTION

Of the three categories of material being considered for rolled concrete dams, and as classified in Section 2, only the lean concrete and the high flyash content concrete are easily reproducible in the laboratory. 'Rollcrete' is difficult because of the inherent variability of 'as dug' material.

Laboratory mixes were therefore made of two lean concretes, which might be suitable for roller compaction for dam construction, with cement contents of 90 kg/m^3 and 132 kg/m^3 . These contained the reference materials used for the main laboratory investigation. The properties of these mixes may therefore be directly compared with the properties of high flyash content concrete. In addition, a typical conventional hearting and a typical facing concrete were also tested so that a comparison can be made with the immersion-vibrated concrete generally used in mass concrete gravity dams, as built at the present time, in the UK.

14.1. HEARTING CONCRETE

Selected properties of the six concrete mixes being compared are given in Table 17. These mixes include:

1. a lean concrete suitable for roller compaction in dams with a cement content of 132 kg/m^3
2. as mix 1, but with a cement content of 90 kg/m^3
3. a high flyash content hearting concrete, typical of those used throughout the CIRIA project
4. a mix similar to 3 as used during the final trial (see Section 15.3) but containing a different flyash

5. a modified high flyash content hearting concrete, which was designed following the work in the laboratory and in the field during the CIRIA project, and which should be cheaper than the mixes previously used
6. a typical hearting concrete using mix proportions similar to that used in recent mass concrete gravity dams in the UK[6,7] and designed for compaction by immersion vibration

It is probable that the lean concrete which has a cement content of only 90 kg/m^3 (mix 2) will not be sufficiently durable for use in dams, because of the probability of leaching[27,159]. It is included in the comparison because more data were obtained on this mix than on the lean concrete with a more acceptable cement content of 132 kg/m^3 (mix 1).

14.1.1. Density of concretes

The important values of density to consider are those of the cores, which are directly comparable because the same constituent materials were used for all the mixes (the two flyashes had very similar relative densities). Table 17 shows that the density of the high flyash content concrete (mix 4) is substantially higher than the other types of concrete, 2% higher than the conventional hearting (mix 6) and 5.5% higher than the lean concrete (mix 1). Consequently, the cross-section of a dam could be slightly reduced to take account of this difference when the different categories of concrete are being compared.

14.1.2. Cost of materials

The cost of flyash, as used in the calculations summarised in Table 17, is that applicable at the Milton Brook Dam site, and is an above-average cost because of the distance

from the nearest power station.

The high flyash content concrete (mix 3) used for most of the CIRIA project is approximately the same price as the conventional hearting and about 5% more expensive than the lean concrete (mix 1), but the latter difference could be offset by the difference in densities of the concrete. The modified high flyash content concrete (mix 5) is cheaper than the other concretes, and if the difference in density is also considered, it is 8% cheaper than the lean concrete and 10% cheaper than the conventional hearting concrete. In areas where the cost of flyash is nearer the national average of £8 to 10/tonne (1981 prices) the material cost saving is even more significant.

The cost comparison is based only on the material costs and does not allow for the different costs of placement. The latter are almost impossible to quantify without resorting to a particular situation, but the overall advantages and disadvantages of the various methods of construction, which can be used with the different materials are summarised in Section 2. The tenderers for Milton Brook Dam showed a preference for high flyash content concrete[49].

14.1.3. Estimated heat generation

It is estimated that the high flyash content hearting concrete (mix 3) when placed in 300-mm lifts each day, would generate approximately half the total amount of heat of that generated in a conventional hearting concrete (mix 6) placed in 1.5-m lifts every 5 days.

The lean concrete (mix 1) will also generate approximately the same amount of heat as the high flyash content hearting (mix 3). The modified high flyash content concrete (mix 5), with a reduced cement content, will generate only about 70% of this amount, and therefore roughly one third of the heat

that would be generated in a conventional hearting concrete. The thermal strain induced in the structure by the modified high flyash content concrete (mix 5) will be correspondingly reduced.

14.1.4. Compressive and tensile strengths

The compressive and tensile strengths of the unjointed parent material of all the mixes are adequate for mass concrete gravity dams, but the lean concrete (mix 2) may be inadequate for very high dams. The strengths of two of the high flyash content hearting (mixes 3 and 4) are also adequate for the majority of arch dams.

14.1.5. Tensile strain capacity

The tensile strain capacity has to be compared with the long-term tensile strain induced by the thermal movement caused by the heat of hydration of the concrete (Section 3.1).

The development of tensile strain capacity with age of the four laboratory mixes is plotted on Figure 76. The two high flyash content mixes show the range of strain capacity likely in the fullscale situation. At all ages, their tensile strain capacities are equal to (or better than) that of the conventional concrete, and always in excess of that of the lean concrete. The long-term tensile strain capacity of the latter concrete (mix 1) is estimated to be about 60 microstrain, approximately half that of the normal high flyash content hearting (mix 3). Both types of concrete will be subjected to about the same tensile strain due to thermal movement, whereas the conventional hearting concrete is probably subject to twice that strain. Clearly, the probability of cracking is much less for the two high flyash content concretes than for both the conventional concrete or the lean concretes.

The gain in tensile strain capacity of the modified high flyash content concrete (mix 4) is approximately 75% of that of the normal high flyash content hearting (mix 3). However, the strain induced in the structure will probably be approximately 70% of that induced by the heat generation of the normal high flyash content hearting concrete, so that the probability of cracking of the modified high flyash content concrete will be similar to that of the normal rollable hearting concrete.

14.1.6. Properties at joints

The tensile properties across the joints of the lean concrete are poor unless the layer on which the concrete is placed receives substantial amounts of treatment and a bedding mix is used[24,35,38]. In contrast, the tensile strength across an untreated joint exposed for one day in the high flyash content hearting approximates to that of the parent material in a conventional hearting concrete (approximately 3 to 3.5 MPa in each case). The properties of the joints of the modified high flyash content concrete would also be satisfactory.

At joints with intervals of more than one day, a modified high paste content mix is used ($P = 0.48$ (see Section 16.8)) so that the strength across an untreated joint left exposed for up to three days can also be made similar to that of the parent material in a conventional concrete dam.

14.1.7. Permeability

Even at the joints, the permeability of the high flyash content concrete (see Section 16.9) is lower than that of a conventional hearting concrete, and appreciably lower than the lean concrete. Joints in the latter are weak to the extent that permeability is not usually measurable[27].

14.2. FACING CONCRETE

The properties of the slipformed facing concretes studied in the CIRIA project both in the laboratory and in the field, and also a typical conventional facing concrete are given in Table 18. The cost, densities and properties of all three mixes are very similar, and all are adequate for use in the construction of dams.

15. CIRIA FULLSCALE TRIALS

High flyash content concrete was initially developed to fulfil the particular requirements of a new method of dam construction. During the CIRIA project, as well as an extensive laboratory programme being carried out (see Section 6.2), a series of fullscale trials were conducted to investigate the practicality of the methods being proposed. These trials were configured to the particular needs of Milton Brook dam, so that the Consultants for the dam, Babbie, Shaw and Morton could be satisfied that the material and methods were suitable for dam construction. The proof of the success of the project was that the tenderers for Milton Brook were allowed to opt to construct the dam using high flyash content concrete as an alternative to the conventional methods of construction.

15.1. MID-TERM TRIAL

The main purpose of the mid-term trial (held at the SWWA's Wimbleball Dam site, which had a mixing plant of sufficient size to simulate fullscale construction) was to investigate the method of using an offset slipform paver to form horizontal, interlocking sections of facing to create the outside skin of a dam. The vertical upstream facing element was considered to be particularly difficult to slipform.

The trial consisted of forming two parallel 20-m runs, each of two upstream facing elements, slipformed on top of each other (see Figure 77), with a roller-compacted hearting between the facing elements. The hearting was mixed, transported and spread with the equipment available on site (see Figures 78 and 79) and was compacted in 200-mm layers with a 7-tonne duplex vibratory roller. The offset slipform paver ran onto the hearting concrete soon after rolling was completed to slipform the second layer of the facing (see Figure 80).

The mix proportions of the concretes used are tabulated in Table 19. Two facing concretes were used to try to overcome the problems of a good finish and lack of slumping after leaving the mould (see Section 13.1).

The paver was guided by sensors which were attached to skids running on the previously slipformed facing element, but this method proved to be unsuitable, because any error in one lift was transmitted to the subsequent lift.

The trial showed that the method of construction was practicable and a number of conclusions were drawn:

1. The general shape of the mould was satisfactory and it was found possible to compact the hearting concrete against the facing elements at the early age required (i.e. between 12 to 18 hours after slipforming).
2. The optimum rate of slipforming was approximately 600 mm per minute, but it was essential that a continuous supply of concrete to the mould be maintained.
3. A new method of guiding the paver was required. Ideally, the system chosen should be remote from the construction.
4. The Cannon test was a satisfactory method of controlling the workability of the hearting concrete. A graph, showing the water content modifications required for each Cannon Time, was produced (see Section 5.4 and Figure 26) for use in the later trials.

5. A new approach to the mix design was required for the facing concrete, as it was found impossible to achieve simultaneously both a vertical face and a good dense finish with the conventional concrete mixes tried (see Section 13).

15.2. INTERMEDIATE TRIALS

Small fullscale trials were conducted at the C&CA in order to overcome the problems found during the mid-term trial. Also investigated were problems arising from the laboratory mix programme; these included defining the optimum coarse aggregate content of a mix.

15.2.1. Optimum coarse aggregate content

The maximum coarse aggregate content in a mix with a low workability is dictated by the segregation which can occur during transportation and spreading.

In this trial, four small slabs were compacted in two 150-mm layers with a single-drum pedestrian-controlled vibratory roller. Each slab contained a mix with a different coarse aggregate content, a , of between 0.50 and 0.59. The mix proportions of the concrete used are tabulated in Table 20. For the reference materials used, the optimum coarse aggregate content was found to be between 0.56 and 0.59.

15.2.2. Guidance of paver

A laser guidance system for the offset paver was tested. The system used two rotating laser beams (laserplanes) as datums, one fixed in the vertical plane just outside the line the paver was to follow, the other fixed horizontally above the paver. Receivers located on the front and back of the paver controlled the level of the machine and a third receiver controlled the line. The guidance system proved satisfactory in the trials.

15.2.3. Concrete feed to paver

A front-mounted conveyor was tested as a method of feeding concrete to the paver, so that the same transportation system (lorry-mounted skips, as used in the mid-term trial - see Figure 78) could be used in the final trial for the facing concrete as well as for the hearting concrete.

15.2.4. Crack control in facing elements

Cracks are liable to form in the outside skin of the facing concrete, when it is restrained, in most environments as a result of thermal strains caused by a fall in air temperature[172]. These cracks should be controlled so that they occur at fixed locations. It was decided to induce the cracks in the facing elements, and voids were formed by incorporating 100-mm diameter steel tubes vertically at 6-m centres in the facing concrete (see Figure 81) to create a reduced cross-section. The front of the paver mould was modified so that it was possible for it to pass over the former, which was retrieved soon after slipforming. Void formers proved to be very simple to use during the trials.

A proprietary sealing material was tested to see if it would seal the void created. Pressure tests indicated that the material was adequate for sealing the void, provided that the concrete was kept clean.

15.3. FINAL TRIAL

The two main purposes for the final trial were: to test at full scale, with readily available plant and equipment, the method of construction and the material developed during the CIRIA project; and to demonstrate the method of construction to the Contractors who were expected to tender for the Milton Brook Dam. The trial was carried out at the Wimbleball Dam site and lasted 3 weeks. A number of specific features were investigated including curing of the

concrete or its omission. The mix proportions of the concretes used are tabulated in Table 21.

A small non-overflow section of a gravity dam approximately 30 m long was built (see Figures 82 and 83). The time interval planned between layers of hearting concrete was generally 1 day, but a 3-day and a 6-day gap, simulating a weekend shutdown of a site or a breakdown, were also incorporated. The trial took place during very cold weather with temperatures as low as -6°C during the day while placing concrete and -8°C overnight. During the period there was snow and some heavy rain on the last day of the trial. A slab and two starters for the facings were cast in-situ, and construction of the trial section proceeded as shown on Figure 83.

The hearting concrete was transported and spread with the same plant as was used in the mid-term trial (see Figures 78 and 79). The concrete was laid in 150-mm or 300-mm layers with a 1:20 crossfall to shed rain water. Two different mixes were used: the normal hearting for placing on horizontal joints which were 1 day or less old, and a paste-rich mix for joints with longer exposure times. There was no treatment of joints, with the exception of the surface exposed for 6 days, part of which was scabbled. Two different vibratory rollers were used (see Figures 84 and 85), both of which could compact the hearting right against the facing elements.

When the concrete was compacted by vibratory roller, a surfeit of paste could be seen on the surface of the layer (see Figure 86). By using the Cannon Time to assess the workability of the hearting, in association with a graph produced relating Cannon Time to water content (see Figure 26) it was possible to minimise the variation in the workability during the trial.

The facing elements were constructed by an offset slipform paver guided automatically by the laser system used in the intermediate trials (see Figure 87). Void formers were built vertically into the upstream side of the trial bank to act as crack inducers. The workability of the facing concrete was assessed using the Vebe Test.

The conveyor system on the front of the paver (see Figure 88) was designed to accommodate the lorry-mounted skips, and was successful. Figure 88 also shows the cutout in the front of the upstream mould accommodating the void formers. Because of the short hours of daylight, some of the slipforming was done at night (see Figure 89). Compaction of the concrete inside the mould was by means of immersion vibrators (see Figure 90). Therefore it was necessary for the facing concrete to have suitable workability. However, the concrete left the mould without a noticeable slump (see Figure 91), and the finish without any treatment was fair (see Figure 92). In order to compare the finish achieved on leaving the mould with that obtainable by floating, half the face was left untreated and half was finished with a wood float (see Figure 93). Figure 94 shows the downstream face of the trial structure. Thermocouples were placed in the hearting concrete, and the temperature rise measured was between 7 and 9degC which approximates to that estimated in the laboratory studies (see Section 11.4).

Curing was carried out on part of the hearting using polythene sheet. No curing was carried out on the facing concrete. As far as can be ascertained, there was no difference between the properties of the cured and uncured hearting concrete, and the lack of curing of the facing concrete did not seem to be detrimental.

Generally the final trial showed that the method of

constructing a dam with a roller-compacted high flyash content hearting concrete and slipformed facing elements was practicable, and, apart from a few minor design modifications, the method and material used during the trial were more than adequate for concrete gravity dam construction.

15.4. ANALYSIS OF JOINTS IN CORES

Eighty metres of cores were extracted from the final trial bank. Bad weather at the trial site did not allow the coring to start until the end of January 1979, when the concrete was approximately 2 months old, and the coring was not finished until early April.

The position of the cores is shown on Figure 95 and those on grid lines E and D were located so that the length of water path to the nearest holes was one metre. These particular holes were used for the in-situ permeability tests (see Section 16.9).

A single-barrel core cutter was used with a barrel approximately 1.15 m long (see Figure 96). The maximum length of core was 2.15 m, so theoretically it should have been possible to extract them in two lengths. However, the majority were extracted in three lengths (see Figure 97).

All the cores were logged in detail (Figure 98 is an example) and compared with the location of the joints where appropriate (for example, see Figure 99).

A joint was defined as having failed if the core broke within 50 mm of the estimated position of the joint. Thus each joint could be considered to be 100-mm thick. The average layer thickness was just over 200 mm, and so if the core breakages were entirely random there should have been approximately an equal number of failures at the joints and

in the parent material.

All the breaks in the cores were rated for bond on the following scale (see Figure 100): 1(good), 2(reasonable), 3(some), 4(little), and 5(none). If there was no break, the concrete in the core was classified as completely bonded.

Four separate analyses were made: in the facing (Series A and H - see Figure 95), between the hearting and facing (Series B and G), the 6-day joint between the hearting placed on days 9 and 15, and the remaining joints in the hearting.

15.4.1. Joints in the facing concrete

The A series of cores contained three lifts of upstream facing element, there being two joints in each core. The H series contained two lifts of upstream facing element and part of a downstream element and thus also had two joints. There was also certain amount of hearting concrete in the H series (see Figure 101). Of the 22 breaks in the cores, only six were at joints. In fact, none of the cores in the A series broke at the joints, indicating that good bond was achieved between the lifts of the facing concrete.

15.4.2. Junction between the hearting and facing concretes

The cores in this series were drilled at slightly different angles, because of the difficulty of coring at the junction between the facing and hearting. Two examples of the line are given on Figure 101. Generally, the joints where the hearting had been compacted against the facing elements were bonded, and they did not break during the extraction of the cores. This was particularly so in the B series of cores, where the hearting concrete sloped into the facing. Although bonded, some of the joints in the G series could be seen.

However, some of the joints where the facing concrete had been slipformed onto the hearting were not satisfactory. This may be because of faults in the design of the moulds. Segregation was noted in some cores at the bottom inside corner of the facing elements, where the mould came to a point (e.g. at the bottom of core G2 - see Figure 101). A modification to the mould design may overcome this problem.

Of particular interest is the bottom of core B11, which was drilled in a very similar way to B2 (see Figure 101). In this core both the facing and the hearting concretes were placed against the conventional concrete starter. At this point, it was thought unlikely that the concrete placed during the trial would bond well with the conventional concrete placed approximately one week earlier. However, visually the concrete in the cores appeared to be monolithic, the joints being visible only because of the different colours of the various concrete mixes (see Figure 102).

15.4.3. Six-day joint in the hearting

Only one of the previous fullscale rolled concrete trials had joints with as long an exposure time as six days[43,44]. This trial used similar concrete to that being placed in the final trial. In spite of using a double-barrelled coring system to reduce the load on the cores during extraction, only one of the six cores containing a 6-day joint was extracted unbroken, and this particular joint did not withstand testing.

Following the mid-term trial, it was expected that the bond at the joints between layers with a 1-day exposure time would be satisfactory, but the bond at the joints with an exposure time of six days was expected to be poor if no joint treatment was carried out.

During the final trial, two types of roller were used: single-drum and twin-drum vibratory rollers. There were two different layer thicknesses, 150 mm and 300 mm, and two different joint preparations, no treatment and a scabbled surface. Thus there were eight different combinations.

An analysis of the joints is shown in Table 22 relative to the thickness of layer and in Table 23 relative to the joint preparation. The performance of a 300-mm layer is no worse than two 150-mm layers. From Table 23, it is apparent that a scabbled surface is significantly better than an untreated one. The effect of the different types of roller is summarised in Table 24, but there seems to be little difference between the two types.

From the analyses, a single 300-mm layer compacted with a single-drum vibratory roller (as it proved more controllable) onto a scabbled surface seems to be the best of the various methods tried. The concrete represented in cores F4 and F6 was constructed in this way. One of the joints in these cores had good bond, the other reasonable bond. Core E5 was from similar construction, but this concrete had had double compaction with the single-drum roller. This joint was completely bonded.

15.4.4. Joints in the remaining hearting concrete

The performance of the joints with exposure times of 1 to 3 days should be compared with the results of previous rolled concrete trials by others.

Three large trial placements intended for dam construction have been cored: in 1973 by the US Army Engineers[24], in 1976 by the Public Works Department in Japan[36], and also in 1976 by the TVA[43,44]. Of the cores recovered, the percentages of bonded joints were approximately 40, 80 and 40 respectively. The two former were achieved only with the

use of a bedding mix.

Table 25 contains the results of the rating of the hearting concrete joints (excluding the 6-day joint between the last two layers). Of the 163 joints in the cores, 140 (86%) were recovered whole and were classified as completely bonded. A number of the failures were deliberately broken by the coring contractor in order to extract the cores. If these are not included in the analysis, only four of the joints were unbonded (i.e. 97.5% of all joints were bonded).

Of the 59 plastic joints, 56 (95%) were bonded (58 or 98% were bonded if those deliberately broken by the coring contractor are excluded), and of 92 joints with an exposure time of one day, 72 (78%) were bonded (89 or 97% excluding those broken on purpose). The four unbonded joints were studied in detail, but there did not seem to be any common factors to explain their failure.

The performance of the concrete at the joints during the final trial was thus significantly better than had been obtained up to that time elsewhere.

16. PROPERTIES OF CORES TAKEN FROM THE FULLSCALE TRIALS

16.1. TESTING OF CORES

The testing of the cores served three main purposes: first, to check the performance of the concrete at joints (see Section 15.4); second to check durability and permeability of the concrete as placed; and third to ascertain if the properties of the material (particularly in tension) were similar to those measured in the laboratory.

In addition to the analysis of the joints, five main testing regimes were carried out to measure:

1. density
2. compressive properties
3. tensile properties
4. permeability
5. freeze/thaw resistance.

Each core was cut into specimens and the method of test for the specimen defined (Figure 103 is an example from the final trial).

16.2. DENSITY OF CORES

16.2.1. Mid-term trial

Cores were taken from the trial bank at the age of approximately one month. The locations of the cores are shown on Figure 104. The hearting concrete had been compacted only with the dead weight of the roller, due to a malfunction of the vibratory roller. Therefore, it was thought that little information could be gleaned from the cores of the hearting. Several modifications to the proportions of the facing concrete had been made, and also variations in the location and number of vibrators in the mould, so it was thought that the cores of the facing concrete would also be of little value. However, all the

plastic joints in the hearting (i.e. those between layers of concrete, between which there was no appreciable time interval) were found to have bonded, as had a significant proportion of the 1-day joints (see Figure 105).

The densities of the cores were measured by weighing in air and water while in a saturated state. These are compared in Table 26 with the densities of the cubes taken from the concrete. The densities of the cores from the hearting were close to the t.a.f. and the Cannon Density averaged 98.4% of the t.a.f., both results being similar to those obtained during the initial trial. As an added check, the densities of four cores, one containing a 1-day lift joint and the other three containing plastic joints, were measured by gamma-ray densiometry in thin layers to establish the density profile. The variation in density along each core was found to be small, and it was not possible to distinguish the joints. The average density of each core is compared with the density obtained in air and water in Table 27.

16.2.2. Optimum Aggregate trial

Four 150-mm cores were taken through the whole depth of each slab of the optimum aggregate trial at an age of seven days with an aggregate content of 0.59, and the remaining slabs at an age of approximately 14 days. All the cores were recovered whole, and Table 28 gives details of the densities.

Apart from the core cut from the slab with an aggregate content of 8.50 (which had had only two passes with vibration on the top layer as the concrete had been too workable) the coefficients of variation were less than 0.1%. It was concluded that it is possible to place roller-compacted concrete containing Moorcroft crushed limestone with a coarse aggregate content of up to 0.59. In fact, the

higher aggregate fractions gave slightly higher densities, but it was thought that segregation could occur at these higher fractions during actual construction, and that 0.56 to 0.59 was the optimum range (see Section 4.5).

16.2.3. Final trial

The densities were determined of the 99 specimens cut from the cores which were to be tested. These are compared with the corresponding cube densities in Table 29. The densities of the cubes of the normal hearting are similar to those obtained during the mid-term trial, being 99.1 and 98.9% of the t.a.f., respectively.

A detailed analysis was made of the densities of the hearting concrete to ascertain whether the depth of the lift or the type of roller had any effect. The variations found during these comparisons were so small as to be statistically insignificant.

The density of the facing concrete improved dramatically from 97.2% of the t.a.f. during the mid-term trial to over 99% during the final trial. This is probably a reflection of the better positioning and increased numbers of the vibrators in the mould, the new mix design method used (see Section 13), and the minor improvements made to the mould during the intermediate trials.

As with previous trials, the standard deviation of the density of the cores is similar to that of the cubes.

Two long lengths of cores, each approximately 1.2 m long, were tested for incremental density. One was taken from the hearting (from core F10) and one from the facing (from core H8). Each core was sawn into 25-mm slices, each slice being weighed in air and then in water. A plot of the density profile was obtained, and the summated densities of

the slices was compared with the overall density of each layer. There was close correlation. The results obtained were similar to the comparison with the gamma-ray density measurement on the cores from the mid-term trial.

16.2.4. Summary of densities

One of the main reasons for the extensive testing of cores for density was to prove, or otherwise, the relationship between the paste/mortar ratio and density (as a percentage of the t.a.f.) (see Figure 16). All the densities of the cores from the various trial banks are plotted on Figure 106 and it can be seen that all the results conform to the original relationship

16.3. COMPRESSIVE TESTING OF CORES

Cores from the three major trials were tested in compression: the mid-term trial, the optimum aggregate trial and the final trial. The cores from the first two were extracted at a relatively early age and cured in water from the time of extraction until they were tested. Cores from the final trial were stored on site after extraction, during which period they were subjected to temperatures well below freezing, conditions far worse than if the concrete had remained in the trial bank.

The majority of cores to be tested in compression were 152 mm in diameter and had a length/diameter ratio of approximately 2. The age of test varied: the optimum aggregate trial cores were tested at approximately 120 days, the mid-term trial cores at 190 days, and the final trial cores at 240 days and also 360 days. The results at the two ages from the final trial bank were so close that they probably fall within the same statistical group. All the results are shown in Table 30.

The calculation of the actual equivalent cube strength was

made according to Concrete Society Technical Report 11[91]. The latter Report suggests that the calculated equivalent cube strength calculated in accordance with BS 1881[88], which differs only from the measured core strength by a factor to take account of the difference in shape between the core and the cube, is probably lower than the strength of cubes made from the same concrete in idealised laboratory conditions and cured in a water tank. A potential equivalent cube strength is thus defined, and this is included in Table 30.

Although the normal hearting concrete mixes used in the two larger trials had similar mix proportions (see Tables 19 and 21), different flyashes were used (type Y in the mid-term trial and type X during the final trial).

The modulus was measured during the testing in compression of four of the cores from the final trial at the age of 240 days. The results are tabulated in Table 31. The actual equivalent cube compressive strengths are typical of the concrete used in this trial.

16.4. DEVELOPMENT OF DIRECT TENSILE TEST FOR CORES

A development programme had to be conducted to find a suitable adhesive and method for sticking plates to the cut faces of cores to suit the method of testing, because the expected tensile strength of the cores was higher than had previously been tested (see Section 8.1). The specimens, which were 200 mm long, were prepared by using an epoxy glue to attach 25-mm thick end plates. The optimum proportions of the glue were found to be 1:1:8 (resin:hardener:filler). Comparison was made between sticking the plates while the cores were wet or dry, testing a specimen while saturated or dry and the length of time between sticking the plates to the specimens and testing.

The maximum tensile stresses which could be applied were:

(a) bonded wet:

1. tested dry - approximately 1.6 MPa
2. tested saturated - approximately 1.8 MPa

(b) bonded dry:

1. tested dry - greater than 3.2 MPa
2. tested saturated - approximately 2.5 MPa

Although there was no clear pattern, it seemed that there was little difference between the loads taken by specimens of the same concrete, whether saturated or dry. This differs from the experience of a previous researcher[173] who found that the tensile strength of saturated specimens was greater than that of dry specimens.

Figure 107 shows a set of cores ready for testing. Tape was used at the ends of the core to allow the epoxy glue to build up a fillet around the core. The bonding side of each plate was given a rough textured finish to improve the adhesion, and a nut was welded to the centre of the outside, into which a universal joint was screwed. The cores were then tested in a Denison tension testing machine (see Figure 108).

16.5 FLUID PRESSURE TEST

At the time of the tension testing of the cores from the mid-term and the optimum aggregate trial structures, the direct tension test was insufficiently developed, and the majority of specimens failed at the interface between the core and the plate. The main object of testing the cores was to compare the value of the tensile strength across the joints with that of the parent material, so that the relationships postulated on Figure 17 could be confirmed or another set of relationships developed.

A test which produces a failure similar to a direct tension test was under development at the time of the CIRIA project[174]. This test had been introduced in the USA in the 1930s[175]. In order to obtain comparative values of the tensile strength across the joints and of the parent material, it was decided to use this test in addition to the direct tension test in case development of the latter could not be completed in time (see Section 8.1).

The test consists of pressurising a jacket around a cylinder (or core) of concrete with water or nitrogen (see Figure 109[174]). The specimen fails with a break perpendicular to the axis of the specimen in a very similar way to the direct tension test. Different rates of pressure application have been found to give different fracture pressures (see Figure 110[174]).

The rate chosen for the nitrogen gas test on the cores was 1 MPa/min (see Section 8.2), the highest speed possible for full percolation of gas into the specimen (see Figure 110).

An advantage of this test is that long lengths of core can be tested in the gas-pressure jacket (see Figure 111) and the core fractures at the weakest section, whether this is at a joint or not. The core can then be further tested by moving the jacket on to the remainder of the core. It has been postulated that the fracture pressure is equivalent to the direct tensile strength[174]. Typical cores before and after testing using this method are shown on Figure 112.

16.6 TENSILE TESTING OF CORES

Cores from three trials were tested in tension: the mid-term trial and the optimum aggregate trial cores in direct tension only, and the final trial cores in direct tension and also using the gas pressure test.

The specimens from the optimum aggregate trial structure were tested at an age of approximately 120 days (see Table 32). Apart from one containing a honeycombed area from the second layer of the slab with $a = 0.50$ (see Section 15.2.1), all failed at the concrete/plate interface. All the specimens contained a plastic joint and were tested saturated, having had the plates stuck to the concrete when dry.

The cores from the mid-term trial bank were tested at ages of approximately 190 and 360 days. There was some increase in load taken by the cores between the two ages of tests. The results of these tests are tabulated in Table 33. All the specimens were bonded to the plates when dry and tested saturated. As with those from the optimum aggregate trial, practically all the specimens failed at the concrete/plate interface, so that the test results are somewhat below the actual tensile strength of the concrete. At the time of the 360-day tests, the development of the direct tension test had progressed, and this may have contributed to the higher stresses achieved. It is also possible that the concrete gained strength between 180 and 360 days.

Table 34 contains all the results of direct tension and gas pressure tests on the cores taken from the final trial. Statistically, there was little difference between the results of the tests on the cores of the parent material and on those containing plastic joints. Consequently, the two are combined in the Table. As with the compressive testing, there was little statistical difference between the 240- and 360-day results.

The tensile stresses taken by the cores of the parent hearting concrete from the mid-term trial are rather higher than those taken by cores from the final trial, despite the

fact that all the former had failed at the concrete/plate interface. The difference is more noticeable for the tests on cores containing a 1-day joint. Similarly, the direct tension testing of the cores of the facing concrete indicated stresses in excess of 3.5 MPa for the specimens from the mid-term trial, which is in excess of the 3.0 MPa average stress on the cores from the final trial. One reason for this could have been the low air temperatures (including freezing) in which the cores were stored following extraction from the final trial bank (see Section 16.3).

16.7 RELATIONSHIP BETWEEN THE GAS PRESSURE TEST AND THE DIRECT TENSILE TEST

The standard deviation and coefficient of variation of the results of the direct tensile testing were satisfactory for the cores from the final trial, and it is these results which were used for the analysis of the properties of the joints.

All the results of the gas pressure testing are plotted on Figure 113 and compared with the direct tension test results. It is possible that a relationship exists between the two sets of results. The best fit lines are:-

1. for specimens without joints -
gas fracture pressure = $1.1 \times$ the direct tensile stress
2. for specimens with joints -
gas fracture pressure = $0.74 \times$ the direct tensile strength

This seems to imply that the joints are more permeable to gas than the parent material, and that the loading on the joints is thus increased. During the in-situ permeability

tests on the final trial bank (see Section 16.9) it was shown that there was little difference in the permeability of the joints to water compared to that of the parent material (this was also found in the laboratory). However, it is probable that gas would permeate the joints more quickly, as it does with conventional concrete (see Figure 110). Nevertheless, considerably more testing with the gas pressure test is necessary before any conclusions can be drawn on the possible relationship with the direct tension test.

16.8 RELATIONSHIP BETWEEN PASTE/MORTAR RATIO AND BOND BETWEEN LAYERS

On Figure 17, a relationship was postulated between the paste/mortar ratio and the bond between layers. Using the same format, the four sets of direct tension results (from Table 34) obtained from tests on the cores from the final trial are plotted on Figure 114 (the results of the other two trials were inadmissible because of the failure of the specimens at the concrete/plate interface).

The four sets of results are:

1,2. plastic joints:

the tensile strength taken at the plastic joint is 100% of that of the parent material at $P = 0.43$ (normal hearting concrete) - 2.13 MPa in each case, and $P = 0.48$ (paste-rich hearting) - 2.33 MPa in each case

3. 1-day joint:

the tensile strength at the 1-day joint (1.62 MPa) is 76% of that of the parent material (2.13 MPa) with $P = 0.43$ (normal hearting concrete)

4. 3-day joint:

the tensile strength at the 3-day joint (0.94 MPa) is 40% of that of the parent material (2.33 MPa) with $P = 0.48$ (paste-rich hearting)

Figure 114 shows four relationships: one for the plastic joint which is very similar to that derived from the results plotted on Figure 17, the second for the 1-day joint, which is now steeper than the previous relationship and approximately parallel to the plastic-joint curve. The other two relationships for 2- and 3-day joints are only suggestions which have yet to be proved. If, after more results become available, these relationships are shown to be correct, it would be possible to design a separate mix for each exposure time to give the properties required.

16.9 TESTS FOR PERMEABILITY

16.9.1. In-situ tests on the final trial bank

The locations of the cores in the hearting were arranged so that the core holes in which the in-situ tests were to be carried out were one metre away from each other (see Figure 95). The holes in the facing concrete were 100 to 150 mm away from the outside face.

Packers were designed to fit into the holes left by the coring. The packer comprised a perforated tube fixed between two discs 150 mm apart. Each disc consisted of a pair of metal plates either side of a rubber seal, the plates being slightly smaller in diameter than the core holes and capable of being pressed together to expand the rubber to form a seal against the periphery of the hole (see Figure 115). Water was pumped into the space between the discs and pressurised to 6 bar (approximately twice the expected pressure at the bottom of Milton Brook Dam). The fall in pressure was noted over a period of 5 minutes, and sufficient water was then pumped into the hole to return the

pressure up to 6 bar. The drop in pressure over the next 5 minutes was also noted, and again the pressure brought up to 6 bar. The quantity of water pumped into the packer during the test was measured.

These permeability tests were carried out when the trial bank was 5 months old. In each of the chosen holes, a series of tests was carried out at each joint position and at one place in the parent material so that the relative permeabilities could be assessed. Minor leaks from the pipes leading to the packer could not be completely sealed, and probably account for some loss in pressure. It was difficult to precisely measure the water loss in the majority of tests because the drop in level in the 312-mm dia. supply tank was so small (generally less than 1 mm). However, the drop in pressure was measurable, and this value was used in the analysis of the results. It was found that a 1-mm drop in level was approximately equal to a drop in pressure of 1 bar during each 5-min test. A 0.5-mm drop was calculated to be equivalent to a permeability of approximately 7.5×10^{-10} m/s.

Of the 51 tests on the hearting concrete, 42 were at joints. The results are summarised in Table 35. In all the tests, it was possible to maintain the pressure at 6 bar. The tests on the 6-day joints were particularly successful.

A total of 15 tests was also carried out on the facing concrete, of which 13 were at joints. Two joints leaked: one of these was the joint below the concrete placed at the beginning of day 8, during which the concrete had been poor. Water appeared on the face 6.7 m away from the packer. The other failure was at the joint below the concrete placed at the end of day 6, when the concrete was too dry. The leak in this case was localised.

16.9.2. Laboratory tests on cores

Only cores from the final trial were tested for permeability, all the cores being 300 mm long with a joint approximately 100 mm from one end. Again, the purpose of the test was to obtain a relative measurement of the difference in permeability of the various types of joint and the parent material, particularly of the hearting concrete.

The method of test was similar to that used to obtain the permeability of cores from a number of mass concrete dams in the USA[176]. A 13-mm diameter hole was drilled approximately 100 mm deep along the axis of the core, from the end remote from the joint. The ends of the core were then sealed and the whole specimen placed in a triaxial test chamber. Both the test chamber and the connection to the hole in the centre of the specimen were pressurised with de-aerated water after the hole had been evacuated with a vacuum pump. A differential head was then applied across the specimen and a marker dye introduced into the centre of the core. Some specimens had very small leaks, which were made visible by the dye and were tested separately. The permeability of the remaining specimens was calculated according to Darcy's Law, assuming radial flow only. A range of permeabilities was found for each specimen, and maximum and minimum values were obtained. After a successful test, the central hole was then drilled to intercept the joint and the test repeated. The tests were carried out on the cores at an age of between 6 and 8 months.

Recent investigations into the permeability of concrete suggest that tests similar to those used on the cores are not representative of the actual flow of water through concrete[177], but they are able to give comparative rates. The results of the tests by the US Army Engineers[176] indicate a range of permeabilities from 4×10^{-11} m/s to

$8 \times 10^{-12} \text{ m/s.}$

A total of 35 tests were carried out on the cores from the hearting concrete. The cores contained all the daily placements, and the joints in the cores ranged from plastic joints to the 6-day joint. The results are tabulated in Table 36, together with the results of the tests on the facing concrete. The results fall into two separate categories: the cores which had a low material permeability, and those cores which had a continuous void of some form from the centre of the core to the outside. In itself, the latter does not mean that the concrete in-situ would be permeable, because the length of water path was only 70 mm in the core, and may not have been much longer in the structure. The tests showed that the permeability of the hearting concrete does not drop significantly at the joints. This agrees with the results of the in-situ permeability tests. Although the readings were taken soon after the start of the test (and thus when the results indicate maximum permeability), the overall average permeabilities are as good as results from previous work on conventional dam concrete[176].

Assuming that the hearting concrete is monolithic, all the permeability results can be considered in broad ranges as in Table 37, from which it can be seen that the median permeability for the hearting concrete is about 10^{-13} m/s.

The results of the tests on the facing concrete are less satisfactory, as they were during the in-situ measurement. The cores tested were from the upstream facing elements, which had a horizontal top surface. It was found that the washing off of paste on these surfaces to create an exposed aggregate finish was not as well done as with the downstream facing elements. Therefore, the bond at the joint between the upstream facing elements was likely to be less

satisfactory. However, with modifications to the shape of the mould, the joint treatment, the bond and thus the impermeability at the joint is expected to improve.

16.10 FREEZE/THAW TESTING OF FACING CONCRETE SPECIMENS FROM THE FINAL TRIAL BANK

Two sets of specimens were tested for resistance to freeze/thaw cycling. Each set consisted of six specimens of facing concrete from the final trial bank. The first set consisted of cores taken from the centre of the facing elements, two of the specimens being kept as controls. Of the remaining four, two contained joints, one was of the parent material, and the fourth was taken from some conventionally-placed concrete which did not contain air entrainment. All the remaining specimens contained entrained air.

The second set of specimens consisted of prisms cut from the face of the trial bank. These were tested to see if the resistance to freeze/thaw cycling at the face was equal to that of the cores from the centre of the facing elements. One side of each of these specimens contained an outside face. Three of the specimens were from the inclined downstream face, and three from the vertical face. Again, one specimen from each group of three was kept as a control.

The specimens were subjected to cycling of freezing and thawing in the following way:

1. cores:

the two control cores were stored in water for the duration of the test. The remaining cores were stored in water for one week prior to the test, then wrapped in thin polythene ('Clingfilm'). These were then subjected to daily freeze/thaw cycling (16 hours in the freezer and 8 hours in the ambient conditions of

the laboratory). At weekends, the specimens were kept in the freezer from 17.00hrs on Friday until 09.00hrs on Monday.

2. prisms:

the two control specimens were again stored in water for the duration of the test. The remaining prisms were totally immersed in water in rectangular metal trays, with a surrounding water thickness of approximately 10 mm. The containers and prisms were then subjected to freeze/thaw cycling in the same way as the cores.

The cores were subjected to 50 freeze/thaw cycles and the prisms to 32 cycles. Two main tests were used to measure the deterioration of the specimens, the measurement of density in air and in water, and the measurement of ultrasonic pulse velocity (UPV). The UPV readings of the cores were directly obtained using the end of the specimens, but those of the prisms were indirectly obtained along the face of the specimens, because the ends were too rough to produce a reliable reading.

No signs of deterioration appeared in any of the specimens, except for the one not containing any entrained air. The variation in density of the specimens subjected to freeze/thaw cycling was equal to that of the control specimens, and even the specimen not containing entrained air showed no sign of loss of weight. However, the latter specimen did show a change in UPV, the reading dropping from 4.90 to 4.56 km/s during the 50 cycles of test. The variation of all the other specimens was again equal to that of the control specimens. The overall standard deviation of all the UPV readings on all the test specimens was 0.057 km/s which compares with the standard deviation of the

test itself on single specimens, estimated to be 0.04 km/s.

17. RELATIONSHIP BETWEEN RESULTS OBTAINED IN THE LABORATORY AND FROM CORES FROM FULLSCALE TRIALS

17.1 BACKGROUND

During the laboratory investigation of the CIRIA project, relationships were derived between the two main variables of the mixes, C_f and C_w , and compressive strength (see Figure 28), tensile strength (see Figure 45) and tensile modulus (see Figure 55).

Also a relationship was derived between compressive strength and tensile strength for concrete containing the reference materials (see Figure 44). These relationships provide a basis for comparing the results obtained in the laboratory with those obtained during testing of the cores from the fullscale trials.

During this comparison the direction of testing relative to the direction of casting of the specimens must be considered (see Figure 116) because this can have a major effect on the measured properties of a given sample of concrete. For example, research[178,179] has shown that cubes tested with the axis of casting vertical are stronger in compression by about 13% to 15% than those tested with the axis of casting horizontal. Similarly, a value of 12% has been reported for cores[180], together with 8%[114] and 18%[179]. When converting core strengths to equivalent cube strengths the Concrete Society Technical Report No 11[91] allows for the orientation of the core, by stating that 'vertical' cores are 9% stronger in compression than 'horizontal' cores.

Concrete thus behaves anisotropically, but the effect of the direction of testing in relation to the direction of casting on the behaviour of concrete in tension is less well documented. One report has suggested[114] that an opposite effect occurs, i.e. that the direct tensile strength of

specimens with the axis of casting vertical is 8% weaker than those of samples tested with the axis of casting horizontal.

The tendency for water to concentrate beneath aggregate particles when concrete is vibrated has been suggested as the reason for the anisotropic effect[181], but this has been seen to happen even with dry and stiff mixes[182].

Little data are available on the effect of the direction of testing on roller-compacted concrete. However, during tension splitting tests on cores from the original Vicksburg trials[23], four 'horizontal' cores had an indirect tensile strength of 1.74 Mpa, while the value for four 'vertical' cores was 2.72 Mpa. Thus a difference was found whereby the tensile strength differed by approximately $\pm 20\%$ about a mean value, depending upon the direction of testing. A similar difference was found at the Tims Ford placement[22], where direct tension testing of cores was undertaken.

17.2. DIRECTION OF CASTING AND TESTING OF SPECIMENS

A close inspection of the cores taken from the roller-compacted hearting concrete from the final trial, showed that the flaky limestone aggregate particles tended to orientate horizontally (see Figure 117). This means that any tensile testing parallel to the axis of casting, (as in the direct tensile test of vertical cores) would test more of the aggregate/paste interface than if a horizontal core was tested in the same way. Since tensile failures usually occur at the aggregate/paste interface this could account for the differences in strengths found at Vicksburg and Tims Ford. The cores from the facing concrete, which were conventionally consolidated with the immersion vibrators inside the paver mould, showed less orientation, implying that the effect is particularly severe under roller compaction.

The inspection of the cores did not reveal a tendency for water to gather under the aggregate particles, the few voids that occurred seemed to be randomly distributed. Also inspection of the cores after failure under direct tension did not show any appreciable difference between the surface above or below a failure junction (see Figure 118), even at joints (see Figure 119).

The mode of failure of cores, with the aggregate particles orientated parallel to the failure plane, is different from that of prisms cast on their sides with the axis of test perpendicular to the axis of casting (see Figure 116). The aggregate is somewhat less orientated, and Figure 120(a) shows a close-up of a prism failure with a relatively low failure stress (1.5 MPa). The prism has failed at the aggregate/paste interface, but in some cases the particles have had to be 'pulled' from sockets. It is apparent that this will lead to a higher 'apparent' tensile strength than if all the aggregate particles are orientated parallel to the failure plane. Figure 120(b) is also a close-up of the failure surface of a prism which failed at a high tensile stress (4.3 MPa). The majority of the failures in this case are failures of the aggregate particles.

17.3. RELATIONSHIP BETWEEN RESULTS OF TESTS ON CORES AND RESULTS OBTAINED ON MOULDED SPECIMENS

The values of tensile strength of the cores were lower than those obtained on moulded specimens. However, as can be seen from Figure 116, the cores were tested in the vertical ('weak') direction and a higher strength would be expected if they had been tested in the horizontal ('strong') direction. It is in the latter direction that the tensile strain capacity is mainly required for resistance to thermal cracking.

If the assumption is made that roller-compacted concrete has an anisotropic behaviour of $\pm 20\%$ about a mean, and conventionally immersion-vibrated concrete $\pm 5\%$, the relationship between the tensile and compressive results from tests on cores from the fullscale trials is found to be similar to the relationship for the moulded specimens.

The extent of the data available from the CIRIA project are limited, but the influence of the direction of testing may be an explanation for the differences found between the strength of the cores from the trials, and the cubes and prisms from the laboratory study.

18. CONCLUSIONS DRAWN FROM CIRIA PROJECT

High flyash content concrete, a concrete with flyash making up 60% to 80% of the volume of the cementitious material, can be proportioned so that it is a cohesive and workable material which can be well compacted in layers by means of a vibratory roller. Placing this material as the hearting between horizontally slipformed facing elements is a method of constructing a concrete gravity dam which can offer several technical and economic advantages over other methods of construction. The laboratory and field trials of the CIRIA project led to the following conclusions:

1. The mix design method developed is relatively straightforward. The main principles are that the mortar should at least fill the voids in the coarse aggregate, and that the paste should at least fill the voids in the fine aggregate.
2. Because of the low workability required for roller compaction and the cohesive nature of the high flyash content concrete, standard tests for workability are not suitable. A recently introduced test, the Cannon Test, was tested and found to be satisfactory.
3. For concrete suitable for roller compaction, the density of specimens, taken from the fullscale trials, increased as a percentage of their theoretical air-free density with increasing paste/mortar ratio up to an optimum value at which the density is a maximum.
4. No segregation was found during transportation and spreading because of the high cohesion of high flyash content concrete.

5. Compaction of cubes and prisms, using a vibrating table, produced densities close to the theoretical air-free density. Compaction in the field using a vibratory roller produced similar values of density.
6. Flyashes covering a range of fineness can be successfully used in rolled concrete, and the requirements of flyash for use in high flyash content concrete may be less onerous than those in the present draft British Standard for flyash as cement replacement in structural concrete[183]
7. For the same constituent aggregates, cement and flyash, comparison of the high flyash concrete with immersion-vibrated concrete as used at present for the hearting in concrete gravity dams, and with lean concrete being considered for roller compaction in dams, showed that:
 - (a) The density achieved in the field was equal to, or higher than, that of the immersion-vibrated concrete and greater than that of the lean concrete.
 - (b) Total heat of hydration and rate of heat evolution are similar to those for the lean concrete, but are less than those for the immersion-vibrated concrete. The induced thermal strains will compare in the same manner, because the concretes have similar coefficients of thermal expansion and thermal conductivity.
 - (c) The drying shrinkage is appreciably lower than the other concretes.

- (d) The tensile strain capacity of moulded specimens is similar to that of the immersion-vibrated concrete but greater than that of the lean concrete.
 - (e) Taking (b), (c) and (d) together, the likelihood of cracking is less than for the other concretes.
 - (f) The compressive strength and tensile strength of moulded specimens are greater than those for the lean concrete and equal to, or higher than, those for the immersion-vibrated concrete. The strengths are more than adequate for hearting concrete, and continue to increase with time in tests carried out at ages up to 365 days. This was also true for cores taken from the field trials.
 - (g) The total material cost in the UK is no greater than the alternative concretes and is likely to be less for sites close to a source of flyash.
8. In addition to low permeability in the parent material of high flyash content concrete, good bond between successive layers of hearting is necessary to achieve near impermeability. Where the period between compacting successive layers does not exceed one day, the paste/mortar ratio required for high density is sufficient for satisfactory bonding without treatment of the surface of the layer. If the period since compacting the previous layer exceeds one day, satisfactory bond at a joint may be achieved by increasing the paste/mortar ratio of the concrete in the subsequent layer. If the period between layers exceeds three days, treatment of the surface of the

lower layer may be desirable.

9. Permeability tests on the hearting concrete in the trial banks indicated low permeability of the parent material and did not detect any weakness at the joints, thus showing that the hearting was effectively impermeable to water.
10. A part of the investigation was specifically directed towards the construction of a gravity dam at Milton Brook in Devon, for which a range of materials was considered. Various combinations of these materials yielded high flyash content concrete mixes which were suitable for the purpose.
11. The mix design method outlined in Conclusion 1 was used to design a flyash concrete (50% of the cementitious content being flyash) for use as a facing concrete with high resistance to deformation while fresh, and with appreciable early strength. The facing concrete could be slipformed horizontally using an offset paver to produce the required cross-section without subsequent deformation, even on a vertical face, and to obtain a dense finish on the exposed faces.
12. The facing concrete had sufficient strength at less than one day to resist the action of a heavy vibratory roller compacting the hearting concrete against the facing element.
13. The results of the permeability measurements and freeze/thaw tests on the facing concrete indicated adequate durability. The concrete is suitable for use in dams with air entrainment as an additional safeguard.

14. The properties of the high flyash content concrete developed for the hearting and the lower flyash content concrete developed for the facing are such that they may be regarded as new materials which have applications in addition to dam construction, in particular for road construction.

PART B

ANALYSIS OF CONTRIBUTION OF FLYASH TO PROPERTIES OF
HIGH FLYASH CONTENT CONCRETE

19. ADDITIONAL TRIAL MIX PROGRAMME

The hardened properties of the high flyash content concrete described in Part A of this Thesis are better than would have been expected using conventional mix design criteria. An example of the differences between a conventional lean concrete suitable for roller compaction and a high flyash content concrete is shown in Table 38. In this Table a comparison has been made between the results of compressive testing of a typical 'dry lean concrete' as used in road base construction in the UK[184] with that of a mix with approximately the same water/cement ratio (by weight) but with flyash added. Both mixes contained the reference materials, as used in the CIRIA project (see Section 6.2).

The cube compressive strength of the high flyash content concrete at 7 days is approximately equal to that of the dry lean concrete at 365 days. It is apparent that the flyash in the mix is having a substantial effect on the compressive strength at all ages from 7 days.

19.1. LOPWELL TRIAL MIXES

So that a proper analysis could be made of the contribution of flyash to the properties of roller-compacted concrete, a further series of mixes was undertaken after the completion of the CIRIA project. The original laboratory programme had been carried out at the C&CA and the additional mixes were undertaken at the Lopwell laboratory of the South West Water Authority's Plymouth Interim Water Supply Scheme. Materials from the same sources as the CIRIA reference materials were used, and flyash/cementitious ratios of 0.2, 0.4 and 0.9 were investigated. Data were therefore available for the full range of cement and flyash contents.

19.2. EFFECT OF FLYASH/CEMENTITIOUS RATIO ON CUBE COMPRESSIVE STRENGTH

The relationships between cube compressive strength and flyash/cementitious ratio, are shown on Figure 121, for four different water/cementitious ratios, $C_w = 1.0, 1.25, 1.50$ and 1.75 . The points at $C_f = 0, 0.6$ and 0.8 were obtained from the CIRIA/C&CA laboratory programme (as shown on Figure 28), and the points at $C_f = 0.2, 0.4$ and 0.9 are the results of single mixes from the additional mix programme. All the mixes had a workability suitable for roller compaction. The coarse aggregate content, a , varied from 0.50 to 0.59 , and the paste/mortar ratio, P , from 0.35 to 0.44 .

On Figure 121 it has been assumed that when $C_f = 1.0$ (i.e. all the cementitious material is flyash) the concrete will have no strength at any age. The relationships for the 7-day compressive strength are a similar shape on all the Figures, but at 91 days, and more particularly at 365 days, the shapes of the curves are different. For example with $C_w = 1.75$ and $C_f = 0.6$, the cube compressive strength at 365 days is just over 50% of the strength of the plain concrete mixes with the same water/cementitious ratio. However, with $C_w = 1.00$, the corresponding cube strength is 80% of that of the plain concrete.

20. SEPARATION OF THE CONTRIBUTIONS OF PORTLAND CEMENT AND FLYASH TO CUBE COMPRESSIVE STRENGTH

20.1. RELATIONSHIP BETWEEN WATER/CEMENTITIOUS RATIO AND CUBE COMPRESSIVE STRENGTH

In 1892, a relationship was proposed by Feret[185] between the compressive strength of a mortar (or concrete) and the volumes of cement, water and air, as follows:-

$$S = K \left[\frac{c}{c+w+v} \right]^2 \quad (20.1)$$

where S = compressive strength

K = Feret's co-efficient

c = volume of cement

w = volume of water

v = volume of air

There was an allowance for the volume of air because at that time concretes were not always fully compacted. The average density of cores taken from the full-scale trials conducted during the CIRIA project was approximately 99% of the theoretical air-free density (see Tables 26, 28 and 29), and so full compaction can be assumed for this type of concrete. Thus the formula can be modified to include the factors used in the design of roller-compacted high flyash content concrete mixes as follows:-

$$\text{Therefore } f_c = K \left[\frac{C_c}{C_c + C_w} \right]^2 \quad (20.3)$$

$$\text{or } f_c = K \left[\frac{1 - C_f}{1 - C_f + C_w} \right]^2 \quad (20.4)$$

where f_c = cube compressive strength

For a plain concrete mix, where $C_f = 0$:-

$$f_c = K \left[\frac{1}{1+C_w} \right]^2$$

$$\text{Therefore } K = f_c(1 + C_w)^2 \quad (20.5)$$

By using the data from the plain concrete mixes, Feret's co-efficient can be calculated for each age of test using equation (20.5). An estimate of the contribution of the cement to the cube compressive strength can then be calculated using either equations (20.3) or (20.4).

On Figure 122 the calculated contribution of the Portland cement, using Feret's relationship, has been compared with the 7-day results for the four water/cementitious ratios shown on Figure 121. With $C_w = 1.50$ and 1.75 , there is very close correlation, but with $C_w = 1.25$ and, even more so, with $C_w = 1.00$, the results of the compressive tests on the cubes are higher than the calculated cement contribution. The cube compressive test results of a series of mixes containing Thames Valley flint gravel[186], have been analysed by the Author using Feret's co-efficient to calculate the cement contribution. At 7 days, all the measured cube compressive strengths are very near to the calculated cement contribution, even at $C_w = 1.0$.

A further analysis using the same method has been made elsewhere by Venaut[187] on mixes containing flyash at a constant water/cementitious ratio (0.5 by weight). Two different cements and three different flyashes were tested, and at 7 days, there was fairly close correlation between the measured compressive results and the calculated cement contribution.

Feret's relationship thus gives a very good estimate of the

7-day cube compressive strength of mixes containing flyash. It is also unlikely that flyash will make a significant contribution to cube compressive strength at 7 days[188] when the specimens are stored at 20C[189]. Consequently, it has been assumed that Feret's relationship can be used to calculate the contribution of the Portland cement to the compressive strength of high flyash content concrete at all ages. Any strength over and above that calculated in this fashion will be deemed to be the contribution of the flyash, reflecting either its physical presence, or a chemical reaction between the flyash and the cement paste.

Figures 123, 124 and 125 show the separate contributions of the cement and the flyash to the cube compressive strength at 28, 91 and 365 days respectively. At all ages, as the water/cementitious ratio decreases the flyash contribution increases substantially.

20.2. COMPARISON OF CONTRIBUTION OF FLYASH AND OF CEMENT TO CUBE COMPRESSIVE STRENGTH

Using the data on Figures 122 to 125, the contribution to the cube compressive strength of the two cementitious materials, Portland cement and flyash, have been separated. In order to compare the performance of the two materials throughout the range of water/cementitious ratios being considered for roller-compacted concrete, these contributions have been plotted against cube compressive strength in the fashion of a water/cement ratio relationship. These are shown on Figure 126, for the four ages of tests. It can be seen that above $C_f = 0.6$ (or below $C_c = 0.4$) there is a considerable decrease in the performance of both the cement and flyash. Nevertheless, for a hearting concrete mix in a dam, with $C_f = 0.75$, this is not critical, because the properties have been shown to be more than satisfactory for the purpose for which the concrete is designed.

Figures 127 and 128 show examples of the use of the curves. On Figure 127 the cube compressive strength is estimated for a mix with $C_f = 0.2$ and $C_w = 1.8$. (The latter is equivalent to a 'cement replacement' by flyash of 14% by weight in a mix with a w/c ratio of 0.61 (by weight)). The calculation is shown in Table 39.

On Figure 128 a mix with $C_f = 0.6$ and $C_w = 1.0$ is analysed. This is similar to one of the mixes used during the optimum aggregate trial (see Table 20). The calculation is shown in Table 40.

The proportions of the cementitious material in the mix detailed on Figure 127 are typical of that of a conventional concrete as used in gravity dams at the present time (155 kg/m^3 cement and 30 kg/m^3 flyash), but the water content is lower and the workability would be more suitable for roller compaction, rather than immersion vibration. The compressive strength is therefore higher than a conventional dam concrete. The paste proportions of the mix analysed on Figure 128 are rather different, the cement content being 135 kg/m^3 and the flyash content 140 kg/m^3 . The water contents in both mixes are similar (circa 110 kg/m^3) as is the workability. The more efficient use of the cementitious materials in the latter mix is clearly apparent.

20.3. ABRAMS' WATER/CEMENT RATIO

On Figure 126 it can be seen that the gradient of the relationship showing the flyash contribution is very much steeper than that of the Portland cement contribution. Abrams[190] original equation for the water/cement curve was:-

$$S = \frac{A}{B^x} \quad (20.6)$$

where S = compressive strength

x = the ratio of the volume of water to the volume of cement, i.e. C_w when $C_f = 0$

A and B are constants depending upon the quality of the cement used, the age of the concrete and the curing conditions, etc.

The factor B in equation (20.6) defines the gradient of the line and factor A is the level of the compressive strength (A/B is equal to the cube compressive strength when $C_w = 0$). All the curves on Figure 126 conform very closely to equation (20.6). For cement, the factor B ranges between 1.5 and 2.5, and for flyash, between 2.5 and 3.5.

The high cube compressive strengths found during testing of low-workability high flyash content concrete are the result of the steepness of the relationship showing the flyash contribution on Figure 126, the majority of the mixes having a water/cementitious ratio in the order of 1.0. At this level, both the cement and flyash are very efficient and at the age of 365 days, the flyash generates as much cube compressive strength as the Portland cement on an equal volume basis. As flyash has a specific gravity which is approximately two-thirds of that of cement, it will generate 50% more cube compressive strength on an equal weight basis.

To date, flyash has been considered as a 'replacement' for cement in the majority of mixes which have been tested in roller-compacted concrete trials. In these mixes, C_w has been in the range of 1.9[23,24] to 2.5[33,35]. It can be seen from Figure 126 that at this level the flyash contribution to the cube compressive strength will be very small at ages up to 28 days, and not substantial even at later ages. This confirms the importance of regarding high flyash content concrete as a new material and not as an extension of the existing mix design concepts.

21. DEVELOPMENT OF HIGH FLYASH CONTENTS IN STRUCTURAL CONCRETE

21.1 FACING CONCRETE IN CIRIA FINAL TRIAL

The compressive testing of the cubes of the facing concrete from the final fullscale trial of the CIRIA project (see Table 8) showed that the concrete had the strength normally associated with a structural concrete (7-day strength: 29 MPa, 28-day strength: 50 MPa and 91-day strength: 64 MPa). The concrete had a workability suitable for immersion vibration and a cement content of only 205 kg/m³ and flyash content of 145 kg/m³. The testing of the cores indicated even higher strengths, at the age of 6 months the actual equivalent cube strength was 78 MPa (see Table 30). It is unusual for the results of the testing of cores to indicate higher strengths than those of cubes, which are stored in idealised BS1881[189] conditions of 100% relative humidity and a constant temperature of 20C. However this improved performance of a flyash concrete in the field has been found previously[191].

21.2 CONVENTIONAL METHODS FOR DESIGN OF CONCRETE MIXES CONTAINING FLYASH

At present, there are several approaches to the design of concrete mixes containing flyash. These can be classified under three general headings:-

- (a) Partial replacement of cement. This is usually on an equal volume, or equal weight, basis and generally leads to lower strengths (both compressive and flexural) up to 3 months and greater strength after 6 months[192].

- (b) Addition of flyash as fine aggregate. This has been used to increase the sulphate resistance of concrete and can lead to a small increase in the compressive strength at 7 days and greater increases at ages of 3 months and 1 year[193].
- (c) Partial replacement of both cement and fine aggregate. This method is that in general use at the present time and concrete can be designed to have equal strength to that of a plain concrete even at early ages[76,194,195].

The one common factor with all the above approaches is that a plain concrete mix, the control mix, is the starting point. The flyash concretes are not designed as such, but are modifications of that control mix.

21.3 STRUCTURAL CONCRETE MIX PROGRAMME

In order to investigate the possible use of a high flyash content in structural concrete, a further series of mix programmes was undertaken by three different laboratories. Three different aggregates were used; the same crushed limestone as had been used in the CIRIA project and the additional mix programme, a further crushed limestone and a Thames Valley Gravel. Three different cements were used but the flyash from the same source as had been used in the CIRIA project was used in all the programmes.

The workabilities of the mixes were all suitable for immersion vibration and a maximum size of aggregate of 20 mm was used. As the high flyash content concretes are usually cohesive, the normal methods of measuring workability may not be satisfactory. In the same fashion as some air-entrained concretes, the concrete will not come out of the Compacting Factor hoppers[84]. The collapse of the cone of a high flyash content concrete in the slump test is the same

as that of a plain concrete which has apparently a significantly lower workability. Consequently the Vebe test was chosen and a workability equal to a Vebe time of 4 to 5 seconds was used for the mixes. This is equivalent to approximately a 50-mm slump in a plain concrete.

In order to obtain the full benefit of flyash in concrete it is apparent from Figure 126 that the water/cementitious ratio must be kept as low as possible commensurate with the other requirements of the concrete. A low water content is usually beneficial to most properties of the concrete, both in the fresh state and when hardened. Therefore, the high flyash content concretes tested in this series of mixes all contained a water-reducing admixture in order to reduce the water content.

It has been found (see Part A of this Thesis) that the optimum water/cementitious ratio for a roller-compacted high flyash content concrete is in the order of 1.0 to 1.2. With some flyashes, even lower water/cementitious ratios have been found to be possible, for example, in the final fullscale trial (see Table 21) a water/cementitious ratio as low as 0.75 was obtained. For the structural concrete mixes, the aim was for the water/cementitious ratio to be as near 1.0 as possible.

The three different laboratory programmes evolved as follows:

- (a) Lopwell Site Laboratory. A range of mixes with flyash/cementitious ratios of 0, 0.2, 0.4, 0.5, 0.6, 0.7 and 0.8 was investigated. Work was concentrated on mixes with a flyash/cementitious ratio of 0.6, because from preliminary work this seemed to be the optimum for immersion-vibrated concrete.

- (b) Fosroc Construction Chemicals Ltd. A range of mixes was studied with flyash/cementitious ratios of 0, 0.4, 0.6 and 0.7 at water/cementitious ratios of 0.9 and 1.1. Again work was concentrated on determining a relationship between cube compressive strength and water/cementitious ratio at a flyash/cementitious ratio of 0.6. The effect of the addition of various admixtures on one particular mix with a C_f of 0.6 and a C_w of 1.1 was studied in detail.
- (c) Stanton and Staveley. Mixes were studied, which had a water/cementitious ratio of 1.1, and flyash/cementitious ratios of 0, 0.4, 0.5, 0.6, 0.7 and 0.8.

The cube compressive strengths of the three series of mixes are plotted against the flyash/cementitious ratio for the two water/cementitious ratios of 0.9 and 1.1 on Figures 129 and 130 respectively. The correlation between the results from the different series of mixes is good considering the different aggregates being used and the differing cements.

The close similarity between two of the sets of mixes is even more noticeable when the cube compressive strengths are plotted against the water/cementitious ratio for a C_f of 0.6. This is shown on Figure 131. One of the sets of mixes contained a Thames Valley Gravel and the other a crushed limestone.

21.4. RELATIONSHIP BETWEEN ROLLER-COMPACTED AND IMMERSION-VIBRATED CONCRETES

Figure 132 shows two relationships between cube compressive strength and water/cementitious ratio for a C_f of 0.6; for roller-compacted concrete from Figure 28(b) and for immersion-vibrated concrete from Figure 131. For comparison purposes the scale for the water/cementitious ratio is offset by 0.15. There is close similarity between the two

sets of data. The compressive strengths of the immersion-vibrated concrete with a water/cementitious ratio of 1.1 (Figure 130) and the roller-compacted concrete with a water/cementitious ratio of 1.25 (Figure 121(b)) are also similar.

The offset of the scales is probably due to the difference in the size of aggregate used, 20 mm for the immersion-vibrated concrete and 40 mm for the roller-compacted concrete, and due to the difference in coarse aggregate content, the immersion-vibrated concrete having a content approximately 75% of that in the roller-compacted concrete. This difference is also applicable to conventional w/c ratio relationships[196].

It therefore seems probable that the same set of relationships shown on Figure 126 could be used for immersion-vibrated concrete with an offset water/cementitious scale.

22. DURABILITY OF HIGH FLYASH CONTENT CONCRETE

Durability is a difficult property to define without considering the requirements of a particular environment. However, the resistance of the concrete to deterioration must be dependent upon the compaction of that concrete and the porosity of the paste. One of the criteria used during the development of high flyash content concrete was to obtain a high density (i.e. that the in-situ density should be as close as possible to the theoretical air-free density). Throughout the fullscale trials the density of cores has been found to be equal to, or higher than, that of cubes and approximately 99% of the theoretical air-free density.

Recently, concern has been expressed[90] about the durability of concrete in the United Kingdom. Because of the nature of modern Portland cements, it is possible to use high w/c ratios and still obtain 28-day strengths which can conform to the majority of specifications. The stage has been reached with a conventional C30 concrete, which usually has a w/c ratio between 0.6 and 0.9 (by weight), whereby it is unlikely that all the capillary pores of the concrete will ever be filled. This can be seen in Table 41[197] which shows the time for pores to fill relative to the w/c ratio (in terms of weight) and the water/cementitious ratio (on a volumetric basis).

In a plain concrete the strength is related to the w/c ratio because both are related to porosity. A recent preliminary study[198] has shown that the relationships are still applicable with high flyash content concrete. By limiting the water/cementitious ratio of high flyash content concrete a low porosity is probable, and thereby a high strength can be achieved.

A limited number of freeze/thaw tests have been conducted on samples of high flyash content concrete[81]. It is probable that the resistance to deterioration under the action of freezing and thawing is related to the cube compressive strength at the time of test. This had previously been shown with concretes containing lower flyash contents[76].

Thus a properly designed high flyash content concrete should have a density very close to the theoretical air-free density, low porosity and a resistance to deterioration under the action of freezing and thawing probably at least equal to that of a plain concrete of the same cube compressive strength.

23. CONCLUSIONS DRAWN FROM THE ADDITIONAL MIX PROGRAMMES

The additional trial mix programme, completed after the completion of the CIRIA project, has shown that the relationships between cube compressive strength, water/cementitious ratio and flyash/cementitious, which had been postulated during that project, are applicable to high flyash content concretes suitable for compaction by immersion vibration. The following additional conclusions and reiterations of the conclusions from the CIRIA project (see Section 18) can be drawn from the results of the trial mix programme:

1. by merely using flyash as a Portland cement replacement material, the full potential and benefits of the flyash are not generally achieved, and a new approach to design is required.
2. high flyash content concrete must be considered to be a new material, and mixes cannot be designed as an extension of existing conventional mix design methods.
3. a relationship exists between the contribution of the flyash to cube compressive strength and the water/cementitious ratio, similar to the water/cement ratio introduced by Abrams in 1912. The contribution of the flyash is more sensitive to water content than the contribution of the cement, and above a certain water/cementitious ratio the contribution at early ages is nil.
4. high flyash content concrete can be designed to have high early in-situ strength, so that, with appropriate design early-age cracking should not be a significant problem.

5. significant economies could probably be made by using a high flyash content in most concretes.
6. flyashes of a different quality to that specified in the relevant British Standard had been used for the majority of trial mixes of high flyash content concrete, upto the end of the programme, with no untoward results.

PART C

EXAMPLES OF THE USE OF

HIGH FLYASH CONTENT CONCRETE

IN ROAD CONSTRUCTION

24. INITIAL TRIALS OF HIGH FLYASH CONTENT CONCRETE IN ROADS

24.1. HODDESDON PLACEMENT

As early as June 1978, the potential of high flyash content concrete for road construction was recognised. A Contractor, Fitzpatrick and Nicholls, wanted to resurface his plant yard and to construct a road to the yard. He decided to take the opportunity of using the area for a trial of high flyash content concrete. The concept was to try to modify the concrete which was being developed for roller compaction in dams so that it could be placed through a paver-finisher without roller compaction. The main advantage of this method would be the accuracy at which the concrete could be laid. Without rolling the tolerances could be reduced enabling the Contractor to cut down the thickness of wearing surface (the most expensive material) and thus reduce the overall cost of the road. There would, of course, also be the reduction in the cost of the roller (probably small). A further advantage was the strength of the concrete: it could be designed for low-early strength, which defines the optimum crack pattern, and for high long-term strength, which would improve the long-term load carrying capacity of the road.

A preliminary trial was carried out in the plant yard by the Contractor before he had access to the design method of high flyash content concrete. A mix, using 20-mm Thames Valley flint gravel, was placed through a paver-finisher (an ABG Titan 300S). A cement content of 85 kg/m^3 and flyash content of 60 kg/m^3 was tried. Two slabs of this modified lean concrete were laid, one slab was rolled and the other left unrolled. The average density obtained by the former was 95.8% of the t.a.f. and by the latter 92.2%. The 28-day cube compressive strength was 14.2 MPa. Therefore, although the roller-compacted section conformed with the DTP specification[184], the unrolled section did not.

The final placement in this trial was the road leading to the plant yard in which two 3-m wide sections were laid two days apart (meaning that the first section was trafficked after only two days). The thickness of the slabs varied between 180 and 250 mm. The concrete for this placement was designed by the Author and had a cement content of approximately 80 kg/m^3 and flyash content of 135 kg/m^3 . The concrete was placed through the same paver without rolling. Cores taken from the slabs indicated that the average density of the concrete was 96.5% of the t.a.f. Unfortunately the cube compressive strength at 28 days averaged 26.6 MPa, so that the concrete would have 'failed' to meet the DTP specification[184]. However, it was apparent that the specification could be met by reducing the cement content. The equivalent cube compressive strengths of the cores averaged 21.8 MPa, slightly less than the cubes. Additional cores (taken at an age of approximately 20 days), which were stored in air alongside the road, were tested at an age of 3 years and had an average equivalent cube compressive strength of 30.6 MPa.

The tolerances on the road were found to be $\pm 5 \text{ mm}$ (from a 3-m straight edge) and were so good that it was decided not to surface the road. It has now been trafficked for over 3 years and has shown no signs of distress. Small cracks have appeared at 4- to 6-m centres, but the edges of these cracks are not deteriorating. The only area which is not satisfactory is the longitudinal joint between the two sections of road, which had not received any treatment during the placement.

24.2. DRAX PLACEMENT

The conditioned flyash from the Drax Power Station used to be discharged from a conveyor system onto an area of compacted hardcore. During the winter, this area would

become almost impassible to traffic. In 1980, it was decided to provide a paved area of approximately 2000 m² of concrete, which was required to carry 33-tonne tipper lorries. High flyash content concrete was used as it offered the cheapest solution to the problem.

The coarse aggregate was a crushed limestone and cement from the Hope Works and flyash from Eggborough (see Appendix B for details) were used. The concrete was required to have a characteristic cube compressive strength of 25 MPa at 28 days. After initial trial mixes in a laboratory, a mix was designed with a cement content of 125 k/m³ and a flyash content of 200 kg/m³. For the placement the concrete was mixed at a ready-mix plant some 20 miles from the site and was transported in tipper lorries. The Cannon Test (see Section 5.1) was used for control of the concrete, and the mix had a mean Cannon Time of 30 seconds at the ready-mix plant.

The concrete was spread by an asphalt paver (a Blaw-Knox PK 90D) and roller compacted in two 115 mm layers by a twin-drum vibratory roller (an Ingersol-Rand DA3). The weather during the placement in November 1980 was cold and there were delays of up to four days between the placements of the layers. This resulted in little bond between the layers, but this does not seem to have affected the performance of the slab which has behaved satisfactorily since it was placed.

The mean 7-day cube compressive strength of the concrete was 26.5 MPa and the 28-day strength 40 MPa.

24.3. HEATHROW PLACEMENT

In February 1981, a section of the sub-base of a taxi-way at Heathrow was placed by Fitzpatrick and Nicholls using high flyash content concrete. The experience of the original

Hoddesdon placement was used and the Author re-designed the mix. A cement content of approximately 70 kg/m^3 was used together with a flyash content of 180 kg/m^3 , (see Appendix B for the analysis of the flyash used). As well as falling within the range of a cube compressive strength of 10 to 20 MPa at 28 days, the British Airports Authority (BAA) insisted that the concrete should also have a cube compressive strength within the limits of 7 to 14 MPa at 7 days. When compared with a conventional 'dry lean concrete', high flyash content concrete develops strength at a different rate (see Section 7.2 and Table 4). This leaves a very small 'window' in which the concrete could be designed.

Figure 133 shows the concrete after placement by the paver-finisher (an ABG Titan 300S) and the smooth surface and the good tolerances ($\pm 4 \text{ mm}$ from a 3-m straight edge) can be seen. The cube compressive strengths were satisfactory (see Figure 134) and the average density of the slabs was 97.6% which was obtained without roller compaction.

This placement confirmed that high flyash content concrete was suitable for use in sub-bases and could conform with both the BAA and DTp specifications[184].

25. ACCESS ROADS FOR DIDCOT POWER STATION

In order to improve the flexibility of the coal handling system at the Didcot Power Station, modifications were made to the coal conveyors in the summer of 1981. These modifications also required an extension to a storage area and some additional roads. The roads were to carry heavy plant (scrappers and D8's) and were to be constructed of concrete. The additional storage area was unlikely to be trafficked and required a lower specification. The coal handling area at Didcot is shown on Figure 135 and the area to be paved on Figure 136.

Due to difficulties with the access to the site - the whole area is surrounded by the rail loop which handles the coal trains - all the concrete paving had to be carried out over a period of four weekends. There was also a limit to the capital expenditure for the Civil Engineering Works. Consequently the method of placement had to be both economic and fast.

It was decided to take the opportunity to use high flyash content concrete for the placement. The material conformed with both the requirements for economy and speed.

In the UK, there are a number of different specifications for PQ concrete; of these, the BAA and DTp[184] specifications require properties at the upper and lower ends of the range of requirements. The roads at Didcot - see Figure 136 - could be split into two sections, those most heavily trafficked and those with less heavy traffic. It was therefore decided to use two grades of concrete for the placement of the 225-mm thick pavement.

The more heavily trafficked roads had a pavement designed to

conform to the BAA specification - 4 MPa minimum flexural strength at 28 days with a 1% failure rate. The less heavily trafficked areas had a pavement designed to conform to the DTp specification[184] - 1.8 MPa minimum indirect tensile strength at 28 days with a 3.5% failure rate. The latter concrete was also designed to conform to the BAA specification at 91 days. For convenience the concretes were classified as PQ(28) and PQ(91) respectively.

The roads were founded on compacted flyash as a sub-grade and 150 mm of sub-base conforming to the DTp[184] and BAA specifications - cube compressive strength at 28 days between 10 and 20 MPa and a minimum in-situ density of 95% of the theoretical density. The additional storage area was also founded on compacted flyash and consisted of 150 mm of concrete conforming with the same two specifications for a sub-base. For conveniences this concrete was classified as BM (base-material) concrete.

26. PLACEMENT AT DIDCOT

26.1. BASE MATERIAL

The base material was mixed at a ready-mix plant only two miles from the placement, was transported to the site in open tippers (covered with tarpaulins), and dumped into an ABG Titan 300S paver-finisher (see Figure 137). The paver-finisher was guided by stringlines and placed a 3-m wide strip of base material concrete which was not roller-compacted (see Figure 138).

The concrete was sprayed with a bitumen emulsion as a curing membrane and covered with polythene in order to keep the concrete free from the coal dust from the surrounding coal heaps. A close inspection of the base material during the week following the placement (before it was covered by the PQ concrete) did not reveal any visible cracking in spite of hot weather and high winds.

Certain areas of the base material could not be placed by paver as they were in restricted areas or because of interference by a sprinkler system, which was used for damping down the coal dust. These areas were placed by hand and compacted by a small vibratory roller (a Bomag 75S). Generally these areas were less satisfactory than the machine-laid concrete, as the finish was poor and surface rather irregular.

26.2. PQ CONCRETE

26.2.1. Method of placement

Although it was originally intended to mix the PQ concrete at the batch plant and transport it in tipper lorries in the same way as the base material, the Contractor opted to use ready-mix trucks as he considered that it would be easier to spread the concrete using this method.

The PQ was placed in 5- to 6-m wide strips between road forms. The transverse 'joints' were induced using a system developed by Fitzpatrick and Nicholls with bottom crack-inducers and thin formica strips pushed into the surface of the concrete to locate a straight joint at that point. No dowel bars were used as it was considered that the irregular pattern of the induced crack, would be sufficient to transfer the loads across the 'joint'. The concrete was initially compacted using immersion vibration and a vibrating beam was used (see Figure 139) to obtain the final compaction. A 'boom float' was used to obtain the initial finish and after approximately 30 minutes the specified 'brush finish' was created (see Figure 140).

To compensate for the different rate of development of strength of the two PQ concretes, the specification did not allow trafficking of the PQ(28) pavement for 7 days and the PQ(91) for 14 days. In the event, due to delays in the commissioning of the conveyor system, the roads were not heavily trafficked until some time after the specified period.

26.2.2. Workability problems

Before the main placement commenced, a small area of PQ(91) was laid in order to gauge the workability required for easy finishing. Up to that time a Vebe of 6 to 8 seconds had been considered by the Contractor to be the optimum for placement. The first batch was considered to be too workable at the plant as it had a Vebe of 2.5 seconds (slump 75 mm). When this arrived at the site it was considered to be ideal for placement. The next batch had a more satisfactory workability at the plant (Vebe 6 seconds - slump 35 mm). However, when it arrived at the site 15 minutes later, the slump had reduced to zero and the concrete was unsatisfactory. This was a feature of the concrete which continued throughout the main placement and seemed to be a

function of the flyash. When the latter was later tested in the laboratory it seemed to have a two-stage water demand. There was an initial demand for water and after a further 20 to 30 minutes a further demand.

This initial trial indicated that a higher workability was required at the plant in order to obtain a satisfactory mix at the point of placement. PQ(91) was used in the first area of road, and a further problem with workability was found. Due to the cohesive nature of the concrete and to the relatively low workability, it was found to be very difficult to discharge the concrete from the ready-mix trucks. This delayed the placement and as fairly high temperatures were prevailing at the time, compounded the problem of loss of workability. After the initial three or four batches it was decided to increase the workability of the mix so that it could be more rapidly discharged by the ready-mix trucks. In order to do this, and to maintain the properties of the concrete, the mix had to be modified. The optimum flyash/cementitious ratio, which had been found during laboratory trial mixes, was decreased from a C_f of 0.70 to 0.65, and the water/cementitious ratio, C_w , was increased from 0.86 to 0.91. The paste/mortar ratio, P , was also decreased in order to slightly reduce the cohesive nature of the concrete. After the modification, the PQ(91) concrete performed most satisfactorily, it was easy to discharge from the trucks, was easy to place and easy to finish.

A corresponding modification to the workability of the PQ(28) mix was also made, the slump of the concrete at the plant was increased from 15 $\pm$ 10mm to 50 $\pm$ 25mm. This was done by increasing the water/cementitious ratio. No other modifications were made to the mix proportions so that it was expected that the properties would be decreased slightly when compared with those obtained during the laboratory

trials. The final mix performed most satisfactorily in the fresh state. One of the factors which could have contributed to the workability problem was the shortage of flyash at the Power Station at the time of placement. All the flyash being produced was being utilised, and the material used in the PQ concrete was taken from the dust hoppers having come directly from the precipitators and was thus very hot.

26.2.3. Air entrainment problems

During the laboratory trial mixes, it was found to be difficult to achieve the specified level of air entrainment, and the same problem was also found during the main placement, the average air content being only 1.4% (range 1.0 to 2.0%). However, additional work since the placement has shown that it is possible to obtain a satisfactory level of entrained air with a different type of admixture.

27. PROPERTIES OF CONCRETE USED AT DIDCOT

27.1. COMPRESSIVE PROPERTIES

27.1.1. Base material

A number of modifications were made to the base material concrete during the placement and five different mix proportions were used. The 7-, 28- and 91-day cube compressive results of four of the five are shown in Table 42 (the fifth mix had a different flyash/cementitious ratio, 0.75 as opposed to 0.80, and the one set of cubes was poorly made).

The 7- and 28-day results are also plotted on Figures 141 and 142, which show the relationships between the water/cementitious ratio and cube strength. All the results fall quite close to the relationship derived from the results of testing of no-slump Thames Valley gravel mixes with a C_f of 0.8, which had been used at previous trials (see Section 24).

All the mixes conformed with the DTp and BAA specifications at 28 days - cube strength between 10 and 20 MPa - and all would have also conformed with the 'modified' specification at 7 days - cube strength between 5 and 12 MPa. (The DTp have accepted that high flyash content concretes develop cube compressive strength at a different rate from conventional 'dry lean concrete' and have allowed a different range for the 7-day cube compressive strength).

The development of the cube compressive strength with age up to 91 days is shown on Figure 143. The increase from 3 and 7 days to 28 days is very similar to previous data from Thames Valley gravel mixes (although the actual aggregate at Didcot was a limestone gravel). However the increase from 28 days to 91 days is less than that obtained before. For example, a cube compressive strength of the overall average

at Didcot of 14.4 MPa at 28 days, would usually lead to a 91-day strength of 22 to 23 MPa. At Didcot the overall average at 91 days was 19.7 MPa.

27.1.2. PQ concrete

After the initial few batches with the original PQ(91) mix, the PQ mixes were not substantially modified. The 7-, 28- and 91-day cube compressive results of the three different mixes are shown in Table 43.

The results are also plotted on Figures 144 (7 days), 145 (28 days) and 146 (91 days), which show the relationships between water/cementitious ratio and cube strengths derived from the results of previous testing of slumpable high flyash content concrete (structural and PQ concretes) with flyash/cementitious ratios between 0.60 and 0.70 using crushed limestone aggregate (over 40 No. mixes) (see Part B of this Thesis). Also plotted on these Figures are the results of the laboratory trial mixes, and all fall very near to the relationships.

The development of the average cube compressive strengths of the two PQ mixes is shown on Figure 147. There is a consistent increase in strength and it is probable that there will be a further substantial increase in the properties between the ages of 91 days and a year.

27.2. TENSILE PROPERTIES

Only the PQ concretes were tested in tension - the indirect tension test for compliance with the DTp specification[184], and the flexural test for compliance with the BAA specification.

27.2.1. Indirect tensile strength

The results of the indirect tensile testing of the mixes from the main placement are shown in Table 44, and the

development of the average indirect tensile strength with age for two mixes is plotted on Figure 148.

Both mixes conformed with the DTp specification at 28 days, the characteristic strengths being 2.20 and 2.16 MPa for the PQ(28) and PQ(91) mixes respectively. The difference is small because the standard deviation of the PQ(28) mix was rather high at 28 days (0.63 MPa), although not at the other ages tested (average 0.29 MPa).

The results show a continuing development of strength with age and are significantly better than those obtained during a recent series of laboratory tests at the TRRL[200] where the effect of flyash in PQ concrete was being investigated.

For example, both PQ mixes at Didcot had a 91-day indirect tensile strength of approximately 4 MPa whereas during the TRRL series of tests an average of approximately 3 MPa was obtained. This is probably a function of the different aggregates used at Didcot (crushed limestone as opposed to Thames Valley gravel) but also due to the design of the high flyash content concrete for the properties required, rather than the use of the concept of 'cement replacement'.

The relationship between the indirect tensile strength and cube compressive strength of all the PQ mixes (laboratory trials and main placement) is shown on Figure 149. There is a consistent relationship at all ages, which is similar to that found in the TRRL series of mixes[200]. However it is different from another relationship found elsewhere on plain concrete[201], which indicated higher early indirect tensile strength and lower long-term strengths for the same compressive strength (although as this relationship was derived from cylinder compressive results, the relationship between cylinder and cube compressive strength may have been a factor).

27.2.2. Flexural strength

The results of the flexural testing of the PQ concrete mixes are shown in Table 45 and the development of the strength with age is plotted on Figure 150.

A different pattern in the strength development can be seen on Figure 150, when compared with the indirect tensile and cube compressive results. This can also be seen on Figure 151, where the relationship between the cube compressive strength and flexural strength is shown. There is a consistent relationship for the 7-day and 28-day results, but at 91 days the flexural strength is higher than would have been indicated from the 7- and 28-day relationships.

These relationships have been compared with relationships derived by other investigators[200 to 203]. Where several relationships exist[203], the one applicable to crushed limestone (from Somerset) has been used. On Figure 152, these relationships are compared with those derived from the Didcot results - the TRRL results[200] generally conform to the same type of relationship.

The Didcot relationship for the 7- and 28-day results is slightly below the other relationships, but the relationship derived from the 91-day results is very similar to those derived by others. Therefore it seems that a slight increase in cube compressive strength is necessary for high flyash concrete mixes if a particular laboratory flexural strength is required at 28 days. However, if an equal flexural strength to that obtained on a plain concrete is required at 91 days the same cube compressive strength would suffice.

27.3. TESTING OF CORES

27.3.1. Compressive testing

Nine cores were taken from the main placement and tested at an age of between 28 and 33 days. The results are shown in Table 46 together with the estimated equivalent cube compressive strength according to the Concrete Society Technical Report No.11[91] and according to BS1881[88]. In all cases the core results are higher than the results measured on cubes stored under BS1881 conditions[189]. In the case of the base material considerably higher - between 30 and 55% depending upon the method of calculation and the mix. This is contrary to previous experience with cores of high flyash content base-material type concrete, which normally have potential cube strengths slightly lower than the BS1881 cubes (see Section 24).

Both PQ concretes had equivalent cube compressive strengths very similar to the cube results according to the BS1881[88] method of calculation, and higher according to the Concrete Society Report[91], this is typical of previous results of core testing (see Section 17.3).

27.3.2. Testing for density

Four cores of base material were tested at the TRRL for incremental density. It was not possible to ascertain the actual density as a percentage of the t.a.f., as the water absorbed by the cementitious reactions up to the age of 20 days is not known. However, it is possible to assess the variation in density with depth. Figure 153 shows the oven-dry density plotted against depth for the four cores tested. Core No.2 showed a high density throughout the whole depth while core No.3 showed some decrease in density with depth, these two cores were typical of previous tests on paver-laid high flyash content concrete, which have averaged a density 97% of the t.a.f. (see Section 24). However core Nos.1 and 6A showed a significant drop in density with depth, and were

unsatisfactory. It is unlikely that this was typical as previous placements have had high in-situ densities and the cores from Didcot all showed a high compressive strength.

28. ECONOMY OF HIGH FLYASH CONTENT CONCRETE FOR USE IN ROADS

28.1. BASE MATERIAL

The economy of using paver-laid high flyash content concrete in road bases is not generally the material cost saving, or the lack of rolling, which can lead to a small saving, but the accuracy at which the material can be laid leading to a reduction in the thickness of the wearing course.

The mix used at Didcot was not typical of the previous placements of high flyash content concrete base material and Table 47 shows the more typical mix from Heathrow, with typical material costs for the South East of the U.K. As can be seen there was a small material cost saving in addition to the more significant savings mentioned above.

28.2. PQ CONCRETE

The price of flyash at Didcot was untypical, so, in addition to the price at that plant, a more typical price of £10/tonne is also included in the cost comparison. In Table 48 the PQ(91) mix, which conformed with the DTP specification at 28 days, is compared with a conventional PQ concrete which was being produced at the ready-mix plant at the time of the Didcot placement using the same materials.

The very substantial cost saving of £5.75/m³, or over 20%, can be seen. Even with the modified price of flyash the cost saving is 15%. A similar exercise is shown in Table 49 where a 'modified' PQ(28) mix - in order to improve the properties to obtain a minimum flexural strength at 28 days of 4.0 MPa - is compared with a typical conventional PQ concrete conforming with the same BAA specification. The same order of cost saving is found as with the other PQ mix. Both high flyash content concrete mixes probably have better long-term properties than the plain concretes and have been

designed to have adequate early-age properties in the field, where different conditions exist compared with the laboratory. It is apparent that in addition to the material cost saving, a better product is possible.

In addition to the material cost saving, the PQ concrete had additional advantages which could lead to further savings. it was very easy to place and finish, which could lead to a reduction in construction time.

29. QUALITY OF FLYASH

The quality of the flyash (see Appendix B) used for the majority of the CIRIA project although not for the final trial (see Part A of this Thesis) and for the additional and structural mix programmes discussed in Part B, was outside the range that would be acceptable in the newly proposed British Standard for flyash in structural concrete[183]. Similarly the Didcot flyash used in the placement of PQ concrete would also not be acceptable. However all the results are typical of high flyash content concrete.

It has been shown during the various laboratory mix programmes, fullscale trials and actual placements that satisfactory concrete can be designed containing flyash which has a level of carbon content and retention on the 45 μ sieve, which was unacceptably high according to the proposed British Standard. It is the variability of 'low quality' flyash which can cause difficulties and not the level of these properties. The problems caused by this variability have been overcome in the placements described above by very tight control over the batching operations. This tight control has a defineable cost.

With a 'high quality' flyash (i.e. one with a low carbon content and low retention on the 45 μ sieve) more significant reductions in the water content of a mix are possible when compared with a lower quality flyash. With flyash mixes designed in the conventional fashion[76, 192 to 195], a contribution to strength can more easily be obtained by the use of such a high quality flyash as they enable an adequate water reduction to be made, so that the water/cementitious ratio is brought into the area at which the flyash makes a contribution to strength at 28 days (see Figure 126). With high flyash content concrete, mixes containing the majority of flyashes can be designed to have a sufficiently low

water/ cementitious ratio.

With a high quality flyash the Portland cement and flyash contents of a mix can be reduced, for equal properties, when compared with a lower quality flyash. It is the overall cost of the concrete, including the additional cost of supervision needed if the flyash is variable, that is important. In certain cases, a high quality flyash would be the most economic because of the lower cementitious content that is possible; and in other cases, the additional cost per tonne of the quality control material will be in excess of the cost of the increased supervision needed to overcome the effects of the variability of the lower quality flyash.

30. CONCLUSIONS DRAWN FROM THE DIDCOT PLACEMENT

The first major use of high flyash content structural concrete, compacted by immersion vibration, was in the pavement-quality concrete used in the access roads at the Didcot Power Station. The sub-base of these roads also contained high flyash content concrete, which was placed by a paver-finisher without roller compaction. A storage area was constructed in a similar fashion. The following conclusions can be drawn from this successful placement, some of which confirm earlier conclusions (see Parts A and B of this Thesis):-

1. The properties of both the base material and PQ concrete conformed with the DTp and BAA specification where required.
2. A very considerable material cost saving (over 20%) was achieved for the PQ concrete. There were probably other less easily definable cost savings, such as ease of use, etc.
3. Very tight control is required over the batching of the high flyash content concrete, in order to maintain a consistent material.
4. High flyash content PQ concrete can be designed to have high early strength in the field, so that, with appropriate design, early-age cracking should not be a significant problem.
5. The long-term properties of the high flyash content PQ concrete showed a significant improvement over conventional plain PQ which could lead to a longer design life for the roads in which HFCC is used.

6. The properties obtained by the high flyash content concrete are probably due to the very low water/cementitious ratios used (as low as 0.33 - by weight).
7. At the present time it is difficult to obtain a satisfactory level of air entrainment in the high flyash content PQ concrete.
8. The base material can be placed with ease and accuracy through a paver-finisher and, to date, all measurements of the in-situ density indicate that they conform with the DTp (and BAA) requirements.
9. The development of cube compressive strength with age of the base material, is different from conventional 'dry lean concrete' and a modification to the specified requirement of cube compressive strength at 7 days is required. A range of 5 to 12 MPa (compared with 7 to 14 MPa) has been accepted by the DTp and is consistent with a range of 10 to 20 MPa at 28 days.
10. Very little early-age cracking has been visually discerned in the high flyash content base material.
11. The increase in strength after 28 days of the high flyash content base material is very marked and this should lead, in the same way as the PQ concrete, to an increase in the long-term load-carrying capacity of a road.
12. There is a strong possibility that the in-situ properties of both the high flyash content concretes are better than those indicated by laboratory specimens.

13. A flyash which did not conform with the relevant British standards was used throughout all the placements with no untoward results.

The overall conclusion was that the placement at Didcot Power Station showed that high flyash content concrete should have a very significant part to play in road construction in addition to the already proven use in dam construction.

PART D

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK

31. MAIN CONCLUSIONS

High flyash content concrete was originally conceived for use as a roller-compacted hearting to fulfil the particular requirements of a new method of concrete dam construction. During the period covered by this Thesis the concrete has been further developed into the workability range of immersion vibration. It has been successfully slipformed using an offset paver in facing elements in dam construction, and has been used as both a sub-base for a road and also as pavement-quality concrete. During this development programme, a number of laboratory and field trials have been conducted and a number of actual placements carried out, these have led to the following conclusions:

1. The mix design method developed for high flyash content concrete is relatively straightforward. The main principles are that the mortar should at least fill the voids in the coarse aggregate, and that the paste should at least fill the voids in the fine aggregate.
2. Because of the low workability required for roller compaction and the cohesive nature of the high flyash content concrete, standard tests for workability are not suitable. A recently introduced test, the Cannon Test, has been used on a number of occasions and found to be satisfactory.
3. For concrete suitable for roller compaction, the density of specimens, taken from the fullscale trials and placements, increased as a percentage of their theoretical air-free density with increasing paste/mortar ratio up to an optimum value at which the density is a maximum.

4. No segregation has been found during transportation and spreading of high flyash content concrete suitable for roller compaction because of the material's high cohesion.
5. Compaction of cubes and prisms of high flyash content concrete suitable for roller compaction, using a vibrating table, has produced densities close to the theoretical air-free density. Compaction in the field using a vibratory roller have produced similar values of density.
6. For the same constituent aggregates, cement and flyash, comparison of the high flyash content concrete with immersion-vibrated concrete as used at present for the hearting in concrete gravity dams, and with lean concrete being considered for roller compaction in dams, has shown that:
 - (a) The density achieved in the field was equal to, or higher than, that of the immersion-vibrated concrete and greater than that of the lean concrete.
 - (b) Total heat of hydration and rate of heat evolution are similar to those for the lean concrete, but are less than those for the immersion-vibrated concrete. The induced thermal strains will compare in the same manner, because the concretes have similar coefficients of thermal expansion and thermal conductivity.
 - (c) The drying shrinkage is appreciably lower than the other concretes.

- (d) The tensile strain capacity of moulded specimens is similar to that of the immersion-vibrated concrete but greater than that of the lean concrete.
 - (e) Taking (b), (c) and (d) together, the likelihood of cracking in the environment of a dam is less than for the other concretes.
 - (f) The compressive strength and tensile strength of moulded specimens are greater than those for the lean concrete and equal to, or higher than, those for the immersion-vibrated concrete. The strengths are more than adequate for hearting concrete, and continue to increase with time in tests carried out at ages up to 365 days. This was also true for cores taken from the field trials.
 - (g) The total material cost in the UK is no greater than the alternative concretes and is likely to be less for sites close to a source of flyash.
7. In addition to low permeability in the parent material of high flyash content concrete, good bond between successive layers of hearting is necessary to achieve near impermeability. Where the period between compacting successive layers does not exceed one day, the paste/mortar ratio required for high density is sufficient for satisfactory bonding without treatment of the surface of the layer. If the period since compacting the previous layer exceeds one day, satisfactory bond at a joint may be achieved by increasing the paste/mortar ratio of the concrete in the subsequent layer. If the period between layers exceeds three days, treatment of the surface of the

lower layer may be desirable.

8. Permeability tests on the hearting concrete in the CIRIA trial banks indicated low permeability of the parent material and did not detect any weakness at the joints, thus showing that the hearting was effectively impermeable to water.
9. The mix design method outlined in Conclusion 1 was used to design a flyash concrete (50% of the cementitious content being flyash) for use as a facing concrete for dam construction with high resistance to deformation while fresh, and with appreciable early strength. The facing concrete could be slipformed horizontally using an offset paver to produce the required cross-section without subsequent deformation, even on a vertical face, and to obtain a dense finish on the exposed faces.
10. The facing concrete had sufficient strength at less than one day to resist the action of a heavy vibratory roller compacting the hearting concrete against the facing element.
11. The results of the permeability measurements and freeze/thaw tests on the facing concrete indicated adequate durability. The concrete is suitable for use in dams with air entrainment as an additional safeguard.

12. A relationship exists between the contribution of the flyash to cube compressive strength and the water/cementitious ratio, similar to the water/cement ratio introduced by Abrams in 1912. The contribution of the flyash is more sensitive to water content than the contribution of the cement, and above a certain water/cementitious ratio the contribution at early ages is nil.
13. High flyash content concrete can be designed to conform to all the relevant Specifications when used in both road base (and sub-base) and pavement-quality concretes.
14. A very considerable material cost saving (over 20%) can be achieved when high flyash content concrete is used as a PQ concrete. There are probably other less easily definable cost savings, such as ease of use, etc.
15. High flyash content PQ concrete can be designed to have high early strength in the field, so that, with appropriate design, early-age cracking should not be a significant problem.
16. The long-term properties of the high flyash content PQ concrete show a significant improvement over conventional plain PQ concrete which could lead to a longer design life for the roads in which HFCC is used.
17. At the present time it is difficult to obtain a satisfactory level of air entrainment in high flyash content PQ concrete.

18. High flyash content road base concretes can be placed with ease and accuracy through a paver-finisher and, to date, all measurements of the in-situ density indicate that they conform with the DTp (and BAA) requirements.
19. The development of cube compressive strength with age of high flyash content road base concrete, is different from conventional 'dry lean concrete' and a modification to the specified requirement of cube compressive strength at 7 days is required. A range of 5 to 12 MPa (compared with 7 to 14 MPa) has been accepted by the DTp and is consistent with a range of 10 to 20 MPa at 28 days.
20. Very little early-age cracking can be visually discerned in the high flyash content base concrete.
21. The increase in strength after 28 days of the high flyash content base concrete is very marked and this should lead, in the same way as the PQ concrete, to an increase in the long-term load carrying capacity of a road.
22. Very tight control is required over the batching of the high flyash content concrete, in order to maintain a consistent material.
22. There is a strong possibility that the in-situ properties of high flyash content concretes are better than those indicated by laboratory specimens.
23. The properties obtained by the high flyash content concrete are probably due to the very low water/cementitious ratios used (as low as 0.33 - by weight).

24. Flyashes which did not conform with the relevant British standards have been used in the majority of the trials and placements with no untoward results.

The overall conclusion which can be drawn from all the trials and placements of high flyash content concrete is that it is a new material and must be designed as such and not as an extension of the existing 'cement replacement' mix design methods. By merely using flyash as a Portland cement replacement, the full potential and benefits of flyash are not generally achieved. When properly designed, high flyash content concrete will make the best use of the particular properties of flyash, and significant economies could be made by using a high flyash content in most concretes.

32. RECOMMENDATIONS FOR FUTURE WORK

The work covered in this Thesis has taken high flyash content concrete from a concept to an accepted material for use in dam and road construction. However, considerable further work is required before the concrete will be generally accepted throughout the Construction Industry. The immediate future work should include:

1. the testing, both in the laboratory and the field, of high flyash content concretes containing a wide range of flyashes in combination with different cements and aggregates
2. a further investigation into the contribution of the flyash to compressive strength (and other properties) so that the relationships postulated in Part B of this Thesis can be confirmed, or otherwise.
3. the definition of the limits of the physical and chemical properties of flyash for use in high flyash content concrete. It has been shown that the proposed British Standard for flyash for use in structural concrete is not relevant for this type of concrete. The particular properties of a flyash (and cement) which result in an improved performance in high flyash content concrete need to be defined.
4. the confirmation of the relationship between the in-situ and laboratory specimens, as it is the former and not the latter which is the most important criterion. It has been shown that the in-situ properties of the majority of high flyash content concretes are rather better than indicated by laboratory specimens.

5. an investigation of new types of air entraining agent for high flyash content concrete. The present admixtures have been designed for use in plain Portland cement concrete and new materials may be needed for high flyash content concrete.

High flyash content concrete has been accepted for use in dam construction and in road bases. It has also been used in pavement-quality concrete and it is likely to be accepted for this use within the foreseeable future. Further testing is required so that high flyash content concrete can also be accepted as a structural concrete.

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APPENDICES

A: DETAILS OF FULLSCALE PLACEMENTS OF ROLLER-COMPACTED CONCRETE

The mix proportions of roller-compacted concrete placed in all the reported fullscale trials prior to the CIRIA project are shown in Table 50. The hardened properties of cores (not moulded specimens) taken from the trials are shown in Table 51.

When considering the properties at the joints, only those results in which the direction of test was the same for both the parent material and the joint, have been included in the analysis for Figures 16 and 17. For example, direct tensile testing of vertical cores is all in the same direction, but for the splitting test, the 'tensile' results of vertical cores in the parent material, cannot be compared with the splitting test results at joints in horizontal cores, because of the anisotropic effect (see Section 7. Similarly, the only comparable shear test results are transverse shear tests of vertical cores, with longitudinal shear tests of horizontal cores.

B. DETAILS OF MATERIALS USED DURING WORK FOR THESIS

B.1. Aggregates

Moorcroft crushed limestone coarse and fine aggregate was used as the reference material for the majority of the mixes during the CIRIA laboratory programme and also during all the CIRIA full-scale trials. Hingston Down crushed granite coarse aggregate and Blackpool sand were used as the alternative materials for the CIRIA/C&CA mix programme, and Kingston granite and Brynchir quartzite gravel were used in the CIRIA/CEGB programme.

Moorcroft limestone was also used during the alternative mix programme and in one of the structural mix programmes. Thames Valley gravel from the Ouse Valley was used in one of the other programmes and a crushed limestone from Nottingham in the other. Quartzite gravel from the Thames Valley was used during the Hoddesdon placement and at Heathrow and a crushed limestone from Yorkshire for the Drax placement.

At Didcot, an oolitic limestone gravel from Sutton Courtney was used for the sub-base and for the fine aggregate of the pavement-quality concrete. A crushed limestone aggregate from Chipping Sodbury was used for the coarse aggregate in the PQ concrete.

B.2 Cements

The cements used during the CIRIA project were referred to by letter; type C was the reference material, type A was used during the CIRIA/C&CA alternative material programme and type B during the CIRIA/CEGB programme. Cement from Westbury was used during the alternative material programme and one of the structural programmes. Cement from Northfleet was used for the Hoddesdon, Heathrow and Didcot placements and also during the second structural programme. Cement from the Hope Works was used during the Drax placement and cement from the Cauldon Works for the third

structural programme.

B.3 Flyashes

Details of the flyashes used during the work on the Thesis are included in Table 52. All except Flyash X did not conform to the relevant British Standard[199].

C. METHOD OF ESTIMATING PROPERTIES OF STANDARD MIXES

C.1 Estimation of water/cementitious ratio

As the flyash content increases in a concrete, the water content required for a given workability decreases because of the better shape of the flyash compared with cement. For a constant workability equivalent to a Cannon Time of 40 seconds, the water/cementitious ratio, C_w , of the six standard mixes making up the two ranges of concretes can be estimated from data on Figure 20. These estimates are shown in Table 53.

C.2 Estimation of cube compressive strength (see Figure 29)

The estimates of cube compressive strength were obtained using the flyash/cementitious and water/cementitious ratios from Table 53 and from the relationships plotted on Figure 28. The cube compressive strengths are shown in Table 4 and plotted on Figure 29. The results at 1 and 2 days are included from similar data not included in this Thesis.

C.3 Estimation of tensile strength (see Figure 46)

The estimates of direct tensile strength at 28 days were obtained using the mix proportions in Table 53 and the relationships plotted on Figure 45. These estimates, and those at 7 and 91 days which were obtained from other test data, are shown in Table 54 and plotted on Figure 46.

C.4 Estimation of static modulus in tension (see Figure 56)

The estimates of static modulus in tension at 28 days were obtained using the mix proportions in Table 53 and the relationships plotted on Figure 55. These estimates, and those at 7 and 91 days which were obtained from other test data, are shown in Table 55 and plotted on Figure 56.

C.5 Estimation of direct tensile strain capacity (see Figure 63)

The estimation of development of tensile strain capacity with age is shown on Figure 63, and strain capacity was calculated in the same way as has been used throughout this Thesis (i.e. by dividing the tensile strength by the tensile modulus). Thus the estimates in Table 56 were obtained from Tables 54 and 55.

THE DEVELOPMENT OF
HIGH FLYASH CONTENT CONCRETE
(Volume II - Figures and Tables)

Thesis submitted to the Department of Civil
Engineering of the University of Surrey in
fulfilment of the requirement for the award of the
Degree of Doctor of Philosophy.

M.R.H.DUNSTAN

September 1982.

Fig 1 Relationship between water/cementitious ratio (by volume) - C_w - and water/cementitious ratio (by weight) - $w/(c + f)$

Fig 2 Forces acting on a typical concrete gravity dam

Fig 3 Hoover Dam - a large arch gravity dam built in the 1930s

Fig 4 Typical monolith method of construction for a small concrete gravity dam

Fig 5 Total number of large dams built throughout the world by year of construction and classification

Fig 6 Development of the rolled concrete dam

Fig 7 Principle features of the dry lean concrete dam (with detail)

DETAIL OF UPSTREAM FACING

Fig 8 Profile of a 40-m high rolled dry lean concrete dam (with detail)

Fig 9 Profile of a 15 m high rolled dry lean concrete gravity/embankment section dam

Fig 10 Original proposal for placing concrete in horizontal layers using extruded curbs as in-situ shutters

Fig 11 Trial of slip-formed technique at Waterways Experiment Station (by courtesy of the US Army Engineers)

Fig 12 Trial of slip-forming technique for the rolled concrete dam at Upper Stillwater (by courtesy of the US Bureau of Reclamation)

ALTERNATIVE I

ALTERNATIVE II

ALTERNATIVE III

ALTERNATIVE IV

Fig 13 Profiles considered for the Zintel Canyon Dam

Fig 14 Holbeam flood alleviation bank under construction

Fig 15 Temperature rise and tensile strain induced in a typical dam concrete not subjected to cooling

TABLE 1: COMPARISON OF DIFFERENT METHODS OF CONSTRUCTION FOR A ROLLED CONCRETE DAM

Method of Construction	Material costs(1)	Volume of dam(1)	Impermeability of hearting	Horiz. joint preparation	Contraction in hearting joints	Contraction in facing joints	Craneage for formwork	No. of types of concrete required	Concrete separated or concurrent.(2)	Compatibility of methods of placement	Additional requirement
Lean concrete dam with joints	Low	Same	Reasonable(3)	Expensive	Expensive	Expensive	Required for formwork	3	Concurrent	Yes	Plant required for cutting joint
Lean concrete dam without joints	Low	Same	Permeable joints	None	None	Very expensive	Required for precast blocks	3	Separated	No	No fines drainage layer required behind upstream wall
High flyash content dam without joints	Similar or low(4)	Slightly smaller	Good	None(5)	None	Simple	Not required	2	Separated	Yes	Specialised pump required for facing concrete
Optimum gravity dam containing cement stabilised material	Very low	Larger	Poor(6)	None	None	None	Not required	2	Concurrent	Yes	-

Notes: (1) compared with conventional concrete as used in gravity dams

(2) timing of placing of hearting and facing

(3) bedding mix usually used

(4) depending upon cost of flyash

(5) except after gaps of more than 3 days

(6) separate expensive watertight membrane required if dam to hold water

Fig 16
Relationship between density and paste/mortar ratio

Fig 17
Relationship between bond between layers and paste/mortar ratio

Fig 18 Electron-scanning microscope photograph of typical lignite flyash

Fig 19 Electron-scanning microscope photograph of typical ordinary Portland cement

Moorcroft limestone
aggregate,
Cement C, Flyash Z.
Workability:
40 seconds Cannon Time

0.50	◆
0.53	○
0.56	△
0.59	□
0.35	◆
0.38	△
0.41	□
0.44	◆
Paste/mortar ratio (P)

Fig 20

Relationship between water/
cementitious ratio and
flyash/cementitious ratio at
a constant Cannon time for
fixed coarse aggregate
content and paste/mortar ratio

Fig 21 Vebe cylinder with paste all around the periphery
after first part of Cannon test

Fig 22 Cylinder of concrete suitable for roller compaction being weighed after subjection to a total of 120 seconds of vibration

Fig 23 Relationship between cube density and Cannon density

TABLE 2: AVERAGE DENSITY OF CUBES AT VARIOUS AGES

Cannon Density (% of the t.a.f.)		Cube densities (% of the t.a.f.)																
		7 days			28 days			91 days			365 days			Overall				
C _f	No. of specimens	Mean (%)	S.D.	No. of specimens	Mean (%)	S.D.	No. of specimens	Mean (%)	S.D.	No. of specimens	Mean (%)	S.D.	No. of specimens	Mean (%)	S.D.			
0	18	98.9	0.62	11	100.6	0.49	18	100.9	0.60	17	100.7	0.60	12	100.8	0.80	15	100.8	0.51
0.6	13	99.6	0.30	7	100.5	0.61	15	100.8	0.80	14	100.8	0.65	8	100.7	0.61	12	100.8	0.49
0.8	16	99.4	0.52	10	99.8	0.74	16	100.3	0.75	15	100.1	0.87	10	100.3	0.42	10	100.3	0.42

Fig 24 CIRIA initial full-scale trial - concrete after two passes with a 7-tonne duplex roller without vibration

Fig 25 CIRIA initial full-scale trial - concrete after two further passes of the vibratory roller with vibration

Fig 26 Control of water content used for hearting concrete at the CIRIA final trial

Fig 27 Workability of concrete as measured by Cannon test compared with other methods of measurement

TABLE 3: MATERIALS USED DURING CIRIA PROJECT

	Initial series of mixes (15 No.) at the C&CA using reference materials	Main series of laboratory mixes (45 No.) at the C&CA using reference materials	Fullscale trials at the C&CA (initial and intermediate trials)	Series of mixes (16 No.) at the C&CA using alternative materials	Series of mixes (25 No.) at the C&CA Laboratory using alternative materials	Main full trials by at Wimble
Materials						
Aggregates						
Moorcroft crushed limestone	*	*	*		*	*
Hingston Down crushed granite				*		
Bryncbir quartzite gravel					*	*
Kingston crushed granite					*	*
Cements						
Type A				*		
Type B	*	*	*		*	*
Type C					*	*
Flyashes						
Type W					*	*
Type X						*
Type Y					*	*
Type Z	*	*	*	*	*	*

Notes: (1) Final trial
(2) Mid-term trial

(a) $C_f = 0$

(b) $C_f = 0.6$

(c) $C_f = 0.8$

Fig 28
Relationship between cube compressive strength and water/cementitious ratio at ages up to 365 days for mixes with various flyash/cementitious ratios

Fig 29 Development of cube compressive strength with age of mixes suitable for roller compaction with different flyash/cementitious ratios compared with a mix suitable for a conventional concrete dam

TABLE 4: ESTIMATED DEVELOPMENT OF CUBE COMPRESSIVE STRENGTH RELATIVE TO THE 28-DAY STRENGTH FOR VARIOUS MIXES

Mix proportions	C _f	1 day		2 days		7 days		28 days		91 days		365 days	
		% 28 day		% 28 day		% 28 day		% 28 day		% 28 day		% 28 day	
		(MPa)	result	(MPa)	result	(MPa)	result	(MPa)	result	(MPa)	result	(MPa)	result
a = 0.50	0	21	30	41	59	58	84	69	100	74	107	80	116
	0.6	10	23	18	42	30	70	43	100	57	133	70	163
P = 0.44	0.8	-	-	-	-	13	59	22	100	33	150	41	186
a = 0.56	0	17	29	33	57	50	86	58	100	62	107	65	112
	0.6	3	10	9	31	19	66	29	100	39	134	49	169
P = 0.41	0.8	-	-	-	-	9	69	13	100	23	177	30	231
Conventional													
dam concrete	0.32	-	-	-	-	20	74	27	100	35	130	43	159

Fig 30 Relationship between equivalent cube compressive strength (from prism crushing) and measured/estimated compressive strengths

TABLE 5: RESULTS OF THE CIRIA/C&CA ALTERNATIVE MIX PROGRAMME

Mix No.	Mix proportions				Age at test (days)	Cube com- pressive strength f _c (MPa)	Direct tensile strength f _t (MPa)	Static modulus Est (GPa)	Tensile strain capacity X 10 ⁻⁶
	a (vol.)	P (vol.)	C _F (vol.)	C _W (vol.)					
M/C/Y/107	0.53	0.40	0.59	1.37	16	(25.5)	2.22	35.3	63
					28	31.1	2.78	35.4	79
					91	43.6	3.39*	40.5	84*
					365	50.4	-	-	-
M/C/Y/108	0.54	0.39	0.79	1.16	7	12.2	1.10	26.4	42
					27	19.6	2.03	40.4	50
					91	28.0	2.72	34.8	78
					390	32.4	3.49*	38.9	90*
M/C/Y/109	0.54	0.42	0.59	1.04	7	29.6	2.37	35.1	64
					29	39.8	3.13	42.0	75
					91	52.4	3.53*	45.2	78*
					364	62.9	3.19*	43.8	73*
M/C/Y/110	0.54	0.43	0.79	0.94	7	14.9	1.51	30.0	50
					29	25.1	2.35	34.5	68
					91	34.2	2.53*	39.3	63*
					365	40.2	-	-	-
M/A/Z/111	0.53	0.40	0.60	1.33	29	38.2	2.75	44.7+	62
					91	50.4	3.77	45.4	83
					365	59.1	-	-	-
M/A/Z/112	0.53	0.40	0.80	1.21	27	17.6	1.60	32.5	49
					91	28.0	3.15	37.0	85
					365	31.8	-	-	-
M/A/Z/113	0.53	0.44	0.60	1.13	28	41.6	3.34	42.9	78
					91	54.4	3.93	42.2	93
					365	59.3	-	-	-
M/A/Z/114	0.53	0.44	0.80	0.98	26	24.9	2.54	36.8	69
					91	36.4	3.55	36.9	96
					356	40.4	-	-	-
H/C/Z/115	0.53	0.41	0.60	1.48	29	22.9	1.85*	30.9	60*
					91	32.0	2.91	31.8	91
					365	49.1	-	-	-
H/C/Z/116	0.53	0.41	0.80	1.35	29	14.2	0.59	-	-
					91	20.2	1.68	28.3	59
					365	33.8	-	-	-
H/C/Z/117	0.53	0.45	0.60	1.17	28	34.2	2.49	28.6	87
					91	42.9	3.39*	36.1	94*
					365	57.1	-	-	-
H/C/Z/118	0.53	0.44	0.80	0.98	29	20.4	1.45	25.7	56
					91	24.4	1.87	35.2	53
					365	39.8	-	-	-

() Estimated * Failure within grips + Very scattered results

Fig 31 Relationship between cube compressive strength and water/cementitious ratio for Hingston Down aggregate/cement C/flyash Z mixes compared with mixes containing the reference materials

TABLE 6: EFFECT OF DIFFERENT AGGREGATES TESTED IN THE
CIRIA/CEGB MIX PROGRAMME

(a) Moorcroft Limestone

Mix ingredients*	Cube strengths (% of reference mix)			
	7 days	28 days	91 days	365 days
M/C/Z	125.1	119.5	116.9	100.9
M/B/Z	98.2	103.4	109.7	107.5
M/C/W	93.9	89.6	92.1	91.8
M/B/W	96.2	91.3	94.6	82.3
Average	103.4	101.0	103.3	95.6

(b) Quartzite gravel

Mix ingredients*	Cube strengths (% of reference mix)			
	7 days	28 days	91 days	365 days
G/C/Z	89.3	92.1	78.2	126.0
G/B/Z	66.2	60.7	61.9	(58.9)
G/C/W	51.8	54.2	55.1	86.2
G/B/W	85.8	85.7	88.5	134.1
Average	73.2	73.2	70.9	101.3
				113.0+

(c) Kingston Granite

Mix ingredients*	Cube strengths (% of reference mix)			
	7 days	28 days	91 days	365 days
K/C/Z	-	-	-	-
K/B/Z	85.3	96.9	95.4	-
K/C/W	87.4	82.4	85.3	110.2
K/B/W	86.7	86.4	100.8	113.6
Average	86.4	88.5	93.8	111.9

* see page 28 (Vol.I) for explanation of abbreviations

+ average neglecting outlier

() outlier

Fig 32 Development of cube compressive strength with age of typical hearting concretes made with different aggregates

Fig 33 Relationship between cube compressive strength and water / cementitious ratio for Moorcroft aggregate/cement A/flyash Z mixes compared with mixes containing the reference materials

TABLE 7: EFFECT OF DIFFERENT CEMENTS TESTED IN THE
CIRIA/CEGB MIX PROGRAMME

(a) Cement C

Mix ingredients*	Cube strength (% of reference mix)			
	7 days	28 days	91 days	365 days
M/C/Z	125.1	119.5	116.9	100.9
M/C/W	93.9	89.6	92.1	91.8
G/C/Z	89.3	92.1	78.2	126.0
G/C/W	51.8	54.2	55.1	86.2
K/C/Z	-	-	-	-
K/C/W	87.4	82.4	85.3	110.2
Average	89.5	87.6	85.5	103.0

(b) Cement B

Mix ingredients*	Cube strength (% of reference mix)			
	7 days	28 days	91 days	365 days
M/B/Z	98.2	103.4	109.7	107.5
M/B/W	96.2	91.3	94.6	82.3
G/B/Z	66.2	60.7	61.9	(58.9)
G/B/W	85.8	85.7	88.5	134.1
K/B/Z	85.3	96.9	95.4	-
K/B/W	86.7	86.4	100.8	113.6
Average	86.4	87.4	91.8	99.3
				103.0+

* see page 28 (Vol.I) for explanation of abbreviations

+ average neglecting outlier

() outlier

Fig 34 Relationship between cube compressive strength and water/cementitious ratio for Moorcroft aggregate/cement C/flyash Y mixes compared with mixes containing the reference materials

TABLE 8: EFFECT OF DIFFERENT FLYASHES TESTED IN THE
CIRIA/CEGB MIX PROGRAMME

(a) Flyash Z

Mix ingredients*	Cube strength (% of reference mix)			
	7 days	28 days	91 days	365 days
M/C/Z	125.1	119.5	116.9	100.9
M/B/Z	98.2	103.4	109.7	107.5
G/C/Z	89.3	92.1	78.2	126.0
G/B/Z	66.2	60.7	61.9	(58.9)
K/C/Z	-	-	-	-
K/B/Z	85.3	96.9	95.4	-
Average	92.8	94.5	92.4	98.3
				103.5+

(b) Flyash W

Mix ingredients*	Cube strength (% of reference mix)			
	7 days	28 days	91 days	365 days
M/C/W	93.9	89.6	92.1	91.8
M/B/W	96.2	91.3	94.6	82.3
G/C/W	51.8	54.2	55.1	86.2
G/B/W	85.8	85.7	88.5	134.1
K/C/W	87.4	82.4	85.3	110.2
K/B/W	86.7	86.4	100.8	113.6
Average	83.6	81.6	86.0	103.1

* see page 28 (Vol.I) for explanation of abbreviations

+ average neglecting outlier

() outlier

Fig 35 Development of cube compressive strength with age of the heating concrete of the CIRIA mid-term trial

Fig 36 Development of cube compressive strength with age of the normal hearting concrete of the CIRIA final trial

Fig 37 Development of cube compressive strength with age of the paste-rich hearing concrete of the CIRIA final trial

a) $C_f = 0.6, C_w = 1.5$

(b) $C_f = 0.8, C_w = 1.2$

Fig 38 Triaxial testing of typical concretes suitable for roller compaction

TABLE 9: COMPARISON OF VARIOUS METHODS OF DETERMINING THE TENSILE STRENGTH OF CONCRETE

Method of test	150-mm specimens	Special/standard moulds	Special equipment	Avail-ability of apparatus	Develop-ment time	Measure-ment of static modulus	Measure-ment of dynamic modulus	Testing of cores	Testing of joints
Splitting	Yes	Standard	No	Yes	Nil	No	Yes	Yes	No(1)
Flexural	Yes	Standard	No	Yes	Nil	No	Yes	No	No
Glued plates	Yes	Standard	Yes	Yes	Long(2)	Yes	Yes	Yes	Yes
Lateral grips	Yes	Standard	Yes	Yes	Short	Yes	Yes	No	Possible
Embedded steel bars	Yes	Special	No	Yes	Short	Yes	No	No	No
Truncated cones	No(3)	Special	No	Yes	Short	Yes	No	No	No
Dumb-bell shape	Yes	Special	Yes	No	Long	No	No	No	No
Fluid pressure	Yes	Standard	Yes	Yes	Very long(4)	No	Yes	Yes	Yes

Notes: (1) Although possible on horizontal cores, the joints in the cores are invisible and doubt must be expressed about their precise location
(2) Glue used to date unable to withstand loads envisaged
(3) All tests to date on small specimens
(4) Still under development
(5) There are few machines available for direct tensile testing
(6) Data from references 88, 91, 97 and 101 to 106

Fig 39 Comparison of two lateral gripping methods for direct tensile testing

Fig 40 Direct tensile testing apparatus showing "scissor grip" and the portal frame strain gauges

Fig 41 Detail of portal frame strain gauges used for direct tensile test

Fig 42 Example of a stress/strain curve for direct tensile testing of 152-mm prisms

Fig 43 Relationship between direct tensile strength and cube compressive strength derived by other investigators

Fig 44 Relationship between direct tensile strength and cube compressive strength at 28 days for various flyash/cementitious ratios.

Fig 45 Relationship between direct tensile strength and water/cementitious ratio at 28 days for various flyash/cementitious ratios

(a) $a = 0.50$, $P = 0.44$

(b) $a = 0.56$, $P = 0.41$

Fig 46 Estimated development of tensile strength with age of mixes with different flyash/cementitious ratios suitable for roller compaction

(a) $C_f = 0.6$

(b) $C_f = 0.8$

Fig 47 Relationship between direct tensile strength and water/cementitious ratio for Hingston Down aggregate/cement C/flyash Z mixes compared with mixes containing the reference materials

Fig 48 Relationship between direct tensile strength and water/cementitious ratio for Moorcroft aggregate/cement A/flyash Z mixes compared with mixes containing the reference materials

Fig 49 Relationship between direct tensile strength and water/cementitious ratio for Moorcroft aggregate/cement C/flyash Y mixes compared with mixes containing the reference materials

Fig 50 Analysis of position of failures of specimens in direct tensile testing apparatus

TABLE 10: ANALYSIS OF LOCATION OF FAILURE IN TENSILE
TESTING MACHINE RELATIVE TO FAILURE LOAD

Failure tensile strength (MPa)	Between spindles (211 - 500 mm)		Within grips (0 - 210 and 501 - 710 mm)		Total	
	No.	%	No.	%	No.	%
0 - 1.0	12	100	0	0	12	10
1.01 - 2.0	13	89	4	11	37	29
2.01 - 3.0	36	78	10	22	46	36
3.01 - 4.0	15	52	14	48	29	23
4.01 +	2	-	0	-	2	2
Total	98	78	28	22	126	100

Fig 51 Example of a tensile stress/strain curve showing the method of calculation of the secant modulus

Fig 52 Half prism after compressive testing

Fig 53 Relationship between static modulus in tension and direct tensile strength

Fig 54 Statistical best-fit relationship between static modulus and direct tensile strength at various stages

Fig 55 Relationship between water/cementitious ratio and static modulus in tension at 28 days for various flyash/cementitious ratios

(a) $a = 0.50$, $P = 0.44$

(b) $a = 0.56$, $P = 0.41$

Fig 56 Estimated development of static modulus in tension with age for mixes suitable for roller compaction with different flyash/cementitious ratios

Fig 57 Relationship between static modulus in tension and water/cementitious ratio for Hingston Down aggregate/cement C/flyash Z mixes compared with mixes containing the reference materials

Fig 58 Relationship between static modulus in tension and water/cementitious ratio for Moorcroft aggregate/cement A/flyash Z mixes compared with mixes containing the reference materials

Fig 59 Relationship between static modulus in tension and water/cementitious ratio for Moorcroft aggregate /cement C/flyash Y mixes compared with mixes containing the reference materials

Fig 60 Relationship between electro-dynamic modulus and static modulus in tension for specimens tested at various ages

Fig 61 Example of a compressive stress/strain curve showing the method of calculation of secant modulus

TABLE 11: RESULTS OF TESTING FOR MODULUS IN COMPRESSION ON HALF PRISMS

Mix No.	(Vol.)	(Vol.)	(MPa)	(GPa)	(GPa)	(%)	Modulus in com- pression as a % of estimated modulus in tension
	C_f	Estimated C_w	Equivalent cube strength f_c	Modulus in com- pression Esc	Estimated modulus in tension Est		
28 days							
FST(1)/7	2.75	1.20	24.9	25.4	31.0	82	
FST(1)/14	1.5	1.20	36.0	28.2	39.0	72	
FST(2)/67	2.75	1.35	15.1	19.7	27.5	72	
FST(2)/85	1.5	1.28	33.5	28.0	38.0	74	
91 days							
FST(1)/7	2.75	1.20	38.8	34.7	32.0	108

Fig 62 Relationship between tensile strain capacity and water/cementitious ratio at 28 days for various flyash/cementitious ratios

(a) $a = 0.50, P = 0.44$

(b) $a = 0.56, P = 0.41$

Fig 63 Estimated development in tensile strain capacity with age of mixes suitable for roller compaction with different flyash/cementitious ratios

(a) $a = 0.50, P = 0.44, C_f = 0.6$

(b) $a = 0.50, P = 0.44, C_f = 0.8$

Fig 64A Inferred development of tensile strain capacity with age of mixes suitable for roller compaction containing different materials

(c) $a = 0.56, P = 0.41, C_f = 0.6$

(d) $a = 0.56, P = 0.41, C_f = 0.8$

Fig 64B Inferred development of tensile strain capacity with age of mixes suitable for roller compaction containing different materials

TABLE 12: TESTS CONDUCTED TO MEASURE THERMAL PROPERTIES

Materials	Coefficient of thermal expansion of concrete		Coefficient of thermal expansion of paste		Thermal conductivity of concrete	
	'Hot Box' tests at T.W. R&D	Water bath test at C&CA	Water bath tests at T.W. R&D	Water bath test at C&CA	'Hot Wire' apparatus at TRRL	'Hot Plate' apparatus at C&CA
Aggregate						
Moorcroft crushed limestone	*	*			*	*
Cements						
Type A						*
Type C	*	*	*	*	*	*
Flyashes						
Type Y						*
Type Z	*	*	*	*	*	*

Heat of hydr tests of cem flyash mixt

Fig 65 Coefficient of thermal expansion of pastes and concretes containing the reference materials relative to flyash/cementitious ratio

Fig 66 Temperature rise under adiabatic conditions of various mixes containing the reference materials as measured in the 'hot box' apparatus

Fig 67 Temperature rise in concrete containing the reference materials with various flyash/cementitious ratios

Fig 68 Rate of heat evolution for a 40:60 (by volume) mixture of cement A and flyash Y at a constant 20C

Fig 69 Rate of heat evolution for a 40:60 (by volume) mixture of cement C and flyash Z at a constant 20C

Fig 70 Gain in heat of hydration at a constant 20C for:
cement C
cement A and flyash Y
cement C and flyash Z

TABLE 13: RESULTS OF HEAT OF HYDRATION TESTS AT CONSTANT TEMPERATURE FOR CEMENT C

C & CA experiment no.	Mix proportions			Peak rates* of heat evolution							Total heat of hydration	
	C _c	C _f	C _w	w/c (wt)	Temp. (C)	Primary			Secondary		3 days (kJ/kg)	7 days (kJ/kg)
						Rate (W/kg)	Age (h)	Rate (W/kg)	Age (h)			
Flyash nil												
308	1.0	0	1.35	0.43	20	2.6	9.0	1.6	23	260	312	312
312	1.0	0	1.35	0.43	50	19	3.5	-	-	312	316	316
Flyash Z												
275	0.40	0.60	1.18	0.44	20	1.4	12	1.4	24	152(58)+	198(65)	198(65)
306	0.40	0.60	1.18	0.44	50	11	4.6	-	-	185(59)	187(59)	187(59)
311	0.27	0.73	1.08	0.43	20	0.84	16	1.2	26	102(39)	135(43)	135(43)
310	0.27	0.73	1.08	0.43	50	8.6	5.5	-	-	135(43)	-	-
Flyash Y												
292	0.40	0.60	1.18	0.44	20	1.3	12	1.9	21	147(57)	191(61)	191(61)
307	0.40	0.60	1.18	0.44	50	5.5	4.0	12	5.9	186(60)	190(60)	190(60)
300	0.27	0.73	1.08	0.43	10	0.50	28	0.7	48	90(-)	126(-)	126(-)
291	0.27	0.73	1.08	0.43	20	0.85	14	2.4	19	100(38)	135(43)	135(43)
309	0.27	0.73	1.08	0.43	50	8.3	6.1	-	-	148(47)	157(50)	157(50)

* per kg of cementitious material
+ percentage of OPC rates in parentheses

TABLE 14: RESULTS OF HEAT OF HYDRATION TESTS AT CONSTANT TEMPERATURE FOR CEMENT A

Mix proportions				Peak rates* of heat evolution						Total heat of hydration	
C & CA experiment no.	C _c	C _f	C _w	w/c (wt)	Temp. (C)	Primary		Secondary		3 days (kJ/kg)	7 days (kJ/kg)
						Rate (W/kg)	Age (h)	Rate (W/kg)	Age (h)		
Flyash nil											
314	1.0	0	1.35	0.43	20	3.1	9.4	-	-	258	306
313	1.0	0	1.35	0.43	50	14	3.4	-	-	293	297
Flyash Z											
274	0.40	0.60	1.18	0.44	20	1.7	11	-	-	147(57)+	190(62)
305	0.40	0.60	1.18	0.44	50	6.9	5.3	-	-	196(67)	197(66)
293	0.27	0.73	1.08	0.43	20	1.1	16	-	-	112(43)	142(46)
276	0.27	0.73	1.08	0.43	50	5.7	6.1	-	-	173(59)	178(60)
Flyash Y											
295	0.40	0.60	1.18	0.44	20	1.6	13	-	-	144(56)	178(58)
302	0.40	0.60	1.18	0.44	50	6.9	5.8	-	-	191(65)	191(64)
296	0.27	0.73	1.08	0.43	10	0.67	25	-	-	97(-)	133(-)
294	0.27	0.73	1.08	0.43	20	1.25	13	-	-	117(45)	149(49)
303	0.27	0.73	1.08	0.43	50	5.0	7.2	-	-	165(56)	173(58)

* per kg of cementitious material
+ percentage of OPC rates in parentheses

TABLE 15: APPROXIMATE MIX PROPORTIONS OF SAMPLES TESTED FOR DRYING SHRINKAGE AND FREEZE/THAW CYCLING

Mix No.	Mix proportions (by vol.)				Paste proportions		
	Coarse aggregate content	Paste/mortar ratio	Flyash/cementitious ratio	Water/cementitious ratio	(kg/m ³)		
	a	P	C _f	C _w	OPC	Flyash	Water
FST 1/11	0.53	0.42 to	0.73	1.20	86	165	98
FST 1/27	0.53	0.44	0.60	1.20	123	129	102
FST 2/62	0.53	0.38	0.60	1.35	96	102	103
FST 2/92	0.53	to	0.73	1.35	67	128	101
FST 2/103	0.53	0.40	0.73	1.35	67	128	101
102	0.53	0.40	0.60	1.33	101	106	107
108*	0.54	0.39	0.79	1.16	54	151	96

* Flyash Y, all other mixes flyash Z.

Fig 71 Drying shrinkage of specimens of concrete suitable for roller compaction compared with results of conventional OPC concrete

Fig 72 Example of early-age stress/strain relationship for a slipformable concrete

Fig 73 Effect of varying the coarse aggregate content on the early-age properties of slipformable concrete

Fig 74 Effect of varying the paste/mortar ratio on the early-age properties of slipformable concrete

Fig 75 Effect of varying the water/cementitious ratio on the early-age properties of slipformable concrete

TABLE 16: PROPERTIES OF A TYPICAL SLIPFORMABLE FACING CONCRETE USING
THE REFERENCE MATERIALS

Mix proportions: $a = 0.40$, $P = 0.46$, $C_F = 0.5$, $C_W = 1.10$

Cement content = 205 kg/m^3

Flyash content = 145 kg/m^3

Cube compressive strength				Tensile properties at 28 days			
f_c				Average	Strength	Static	Strain
1 day	7 days	28 days	91 days	density		modulus	capacity
					ft	fst	
		(MPa)		(kg/m^3)	(MPa)	(GPa)	($\times 10^{-6}$)
7.3	32.4	41.8	55.6	2450	3.3	37	90

TABLE 17: COMPARISON OF MATERIAL COSTS AND PROPERTIES OF VARIOUS HEARTING CONCRETES SUITABLE FOR MASS CONCRETE GRAVITY DAMS

Parameter	Lean concrete (mix 1)	Lean concrete (mix 2)	High flyash content hearting (mix 3)	High flyash content hearting (mix 4)	High flyash content hearting (mix 5)	Type of concrete
Coarse aggregate content - $a(m^3/m^3)$	0.49	0.50	0.53	0.54	0.56	1
Paste/mortar ratio - $P(vol)$	0.35	0.35	0.44	0.43	0.41	1
Flyash/cementitious ratio - $C_f(vol)$	0	0	0.73	0.72	0.80	25
Water/cementitious ratio - $C_w(vol)$	3.18	4.75	1.00	0.89	1.20	24
Cement content (kg/m^3)	132	90	85	94	52	1
Flyash content (kg/m^3)	0	0	165	176	144	1
Cube density (kg/m^3)	2492	2521	2510	2498	2508	25
Core density (kg/m^3)	2369(2)	2356(2)	-	2497	-	24
Cost of materials (£/m ³)(6)	17.90	-	18.84	-	17.52	1
Cube compressive strength (MPa)	2 days	4	10	-	-	1
	7 days	9	20	21	8	2
	28 days	11	31	43	13	2
	91 days	12	43	56	23	2
	365 days	-	54	60	30	1
Direct tensile strength (MPa)	7 days	-	1.8	-	1.0	1
	28 days	10	2.7	-	1.6	1
	91 days	-	3.5	-	2.2	2
	365 days	1.2(1)	4.3(2)	-	2.9(2)	2
	7 days	-	60	-	40	5
Direct tensile strain capacity ($\times 10^{-6}$)	28 days	35	76	-	57	5
	91 days	-	91	-	72	5
	365 days	40(1)	115(2)	-	88(2)	8
	1-day exposure time	16-20(2)	70-80	70-80	60-70(2)	8
Direct tensile strength at joints	3-day exposure time	0	25-35(2)	25-35(2)	15-25	8
	Permeability of parent material (m/s)	10 ⁻⁵ (2)	10 ⁻⁵ (2)	10 ⁻¹³ (4)	-	8
	Permeability at joints (up to 6-day exposure time)(m/s)	not measureable(2)	-	10 ⁻¹³ (4)	-	8

Notes: (1) At 551 days. (2) Estimated from similar mixes. (3) Flyash X - All other mixes contain Flyash Z. (4) Measured on concrete at 115 days. (5) At 115 days. (6) Cement = £40/tonne, Flyash = £15/tonne, Aggregate = £6/tonne.

Fig 76 Development of tensile strain capacity with age of various hearting concretes suitable for mass concrete dams

TABLE 18: COMPARISON OF MATERIAL COSTS AND PROPERTIES OF VARIOUS FACING CONCRETES
SUITABLE FOR MASS CONCRETE GRAVITY DAMS

Parameter	Slip- formable facing (mix 7)	Slip- formable facing(2) (mix 8)	Typical facing concrete (mix 9)
Coarse aggregate content - a(m ³ /m ³)	0.40	0.37	0.49
Paste/mortar ratio - P(vol)	0.46	0.44	0.50
Water/cementitious ratio - C _w (vol)	1.10	1.19	1.42
Flyash/cementitious ratio - C _f (vol)	0.50	0.49	0.32
Cement content (kg/m ³)	206	203	225
Flyash content (kg/m ³)	145	142	75
Cube density (kg/m ³)	2448	2427	2521
Core density (kg/m ³)	-	2435	2408(1)
Cost of materials (£/m ³)(5)	21.70	21.88	21.88
	2 days	-	-
	7 days	29	32
	28 days	50	43
	91 days	64	51
	365 days	-	57
	7 days	-	-
	28 days	-	2.8
	91 days	-	-
	365 days	-	3.2
	7 days	-	-
	28 days	72	72
	91 days	-	-
	365 days	-	82(4)
	1-day exposure time	40-50(1)	-
	3-day exposure time	-	-
	Permeability of parent material (m/s)	10 ⁻¹³ (3)	10 ⁻¹¹
	Permeability at joints (up to 6-day exposure time)(m/s)	-	-

Notes: (1) Estimated from similar mixes. (2) Flyash X - All other mixes contain flyash Z.
(3) Measured on cores. (4) At 115 days. (5) Cement = £40/tonne, Flyash = £15/tonne
Aggregate = £6/tonne.

Fig 77 Cross-section of the CIRIA mid-term trial bank

Fig 78 Close-up of hearting concrete being dumped during the mid-term trial

Fig 79 Hearting concrete being spread by a front-end loader during the mid-term trial

Fig 80 First length of the second lift of the upstream facing units being slipformed during the mid-term trial

TABLE 19: MIX PROPORTIONS OF THE CONCRETES USED DURING THE CIRIA MID-TERM TRIAL

	40mm	20mm	10mm	b	c	f	w	Total
Hearting (MBH1)								
Weight (kg/m ³)	688	417	359	688	89	175	96	2512
Volume (fraction of total)	0.2538	0.1533	0.1330	0.2567	0.0283	0.0794	0.0955	1.0000
	a = 0.539 P = 0.442 C _f = 0.736 C _w = 0.89							
Facing (MBF4)								
Weight (kg/m ³)	-	737	374	906	220	91	141	2469
Volume (fraction of total)	-	0.2710	0.1384	0.3381	0.0698	0.0416	0.1411	1.0000
	a = 0.409 P = 0.428 C _f = 0.375 C _w = 1.26							
Facing (MBF5)								
Weight (kg/m ³)	-	814	410	814	218	95	133	2484
Volume (fraction of total)	-	0.2993	0.1518	0.3038	0.0691	0.0433	0.1327	1.0000
	a = 0.451 P = 0.447 C _f = 0.385 C _w = 1.18							

TABLE 20: AVERAGE MIX PROPORTIONS USED DURING THE CIRIA OPTIMUM AGGREGATE TRIAL

	a	b	c	f	w	Total
a = 0.50						
Weight (kg/m ³)	1363	760	134	137	108	2502
Volume (fraction of total)	0.5024	0.2835	0.0426	0.0639	0.1076	1.0000
	a = 0.502	P = 0.430	C _f = 0.600	C _w = 1.01		
a = 0.53						
Weight (kg/m ³)	1459	710	120	123	103	2515
Volume (fraction of total)	0.5368	0.2651	0.0381	0.0571	0.1029	1.0000
	a = 0.537	P = 0.428	C _f = 0.600	C _w = 1.08		
a = 0.56						
Weight (kg/m ³)	1556	673	108	111	97	2525
Volume (fraction of total)	0.5664	0.2510	0.0344	0.0517	0.0965	1.0000
	a = 0.566	P = 0.421	C _f = 0.600	C _w = 1.12		
a = 0.59						
Weight (kg/m ³)	1616	632	95	98	93	2534
Volume (fraction of total)	0.5953	0.2357	0.0303	0.0455	0.0932	1.0000
	a = 0.595	P = 0.418	C _f = 0.600	C _w = 1.23

Fig 81 Proposed inducement of cracks in facing elements

TABLE 21: MIX PROPORTIONS OF THE CONCRETES USED DURING THE CIRIA FINAL TRIAL

	40mm	20mm	10mm	b	c	f	w	Total
Normal hearting (MBH2)								
Weight (kg/m ³)	689	409	355	705	95	176	95	2524
Volume (fraction of total)	0.2542	0.1503	0.1313	0.2629	0.0300	0.0764	0.0949	1.0000
	a = 0.536			P = 0.434	C _f = 0.718		C _w = 0.89	
Paste-rich hearting (MBH3)								
Weight (kg/m ³)	678	414	355	654	94	225	95	2515
Volume (fraction of total)	0.2500	0.1523	0.1316	0.2440	0.0298	0.0977	0.0947	1.0000
	a = 0.534			P = 0.477	C _f = 0.766		C _w = 0.74	
Facing concrete (MFB6)								
Weight (kg/m ³)	-	661	339	954	203	141	149	2447
Volume (fraction of total)	-	0.2429	0.1257	0.3561	0.0644	0.0615	0.1494	1.0000
	a = 0.369			P = 0.436	C _f = 0.489		C _w = 1.19

Fig 83 Cross-section of the CIRIA final trial bank

Fig 84 Single-drum vibratory roller compacting hearting concrete against upstream facing unit

Fig 85 Twin drum vibratory roller compacting central strip of hearting concrete

Fig 86 Close-up of surface of hearting concrete being compacted by twin-drum vibratory roller

Fig 87 Slipform paver running automatically without operator

Fig 88 Offset paver lining up for slipforming of the first upstream facing element

Fig 89 Start of slipforming of upstream facing element on top of downstream facing element

Fig 90 Close-up of inside of downstream mould showing the location of vibrators. Note: one of the vibrators is out of sight above the mould

Fig 91 Close-up of concrete emerging from mould showing joint between upstream and downstream facing element

Fig 92 Close-up of outside face of upstream facing unit (without floating)

Fig 93 Detail of sloping downstream face after floating

Fig 94 General view of downstream face of final trial bank

Fig 95 Position of cores taken from the CIRIA final trial bank

Fig 96 Coring of final trial bank

Fig 97 Typical hearting concrete cores extracted from the final trial bank

Fig 98 Classification of hearing concrete cores taken from the CIRIA final

Fig 99 Description of hearing concrete cores taken from the CIRIA final trial bank showing the position

Core E11 broken in middle of layer H6/2, classified as good bond

Core F4 broken on 6-day joint between H9 and H15, classified as reasonable bond

Fig 100 Classification of bond in core fracture

Core E11 broken by coring contractor at 1-day joint between H7/2 and H5/1, classified as some bond

Core C1 broken at 6-day joint between H9 and H15, classified as little bond

Fig 100 Classification of bond in core fracture

Core C9 broken at 6-day joint between H9 and H15/1. Classified as no bond, mud on joint

Fig 100 Classification of bond in core fracture

Fig 101 Cross-section of the CIRIA final trial bank showing location of cores

Fig 102 Detail of bottom of core B11 showing facing and hearting concrete well bonded to the conventionally vibrated concrete starter

Fig 103 Classification of tests on hearting cores taken from the final trial bank

TABLE 22: ANALYSIS OF BOND RELATIVE TO LAYER THICKNESS

Thickness of layer	No. of joints	Number of cores with type of bond (% equal or better)				
		Completely Bonded	Good	Reasonable	Some	Little
Two 150-mm layers	12	0(0%)	5(42%)	1(50%)	0(50%)	2+1*(75%)
One 300-mm layer	12	1(8%)	4(42%)	1(50%)	1(58%)	5(100%)
						2+1*(100%)
						0

TABLE 23: ANALYSIS OF BOND OF 6-DAY JOINT RELATIVE TO JOINT PREPARATION

Treatment of joint	No. of joints	Number of cores with type of bond (% equal or better)				
		Completely Bonded	Good	Reasonable	Some	Little
Scabbled	12	1(8%)	7(67%)	1(75%)	0(75%)	2(92%)
Non-scabbled	12	0(0%)	2(17%)	1(25%)	1(33%)	5+1*(83%)
						1*(100%)
						2(100%)

*Mud on joint

TABLE 24: ANALYSIS OF BOND OF 6-DAY JOINT RELATIVE TO TYPE OF ROLLER

Type of roller	No. of joints	Number of cores with type of bond (% equal or better)				
		Completely Bonded	Good	Reasonable	Some	Little
A	8	0(0%)	3(38%)	2(63%)	1(75%)	2(100%)
A(X2)+	4	1(25%)	1(50%)	0(50%)	0(50%)	1+1*(100%)
BandA+	4	0(0%)	3(75%)	0(75%)	0(75%)	1(100%)
B	8	0(0%)	2(25%)	0(25%)	0(25%)	2+1*(100%)

* Mud on joint

+ Central strip rolled by single-drum. Edge of strip overlapped with original rolled area

A = Single-drum vibratory roller

B = Twin-drum vibratory roller

TABLE 25: ANALYSIS OF BOND BETWEEN LAYERS OF HEARTING CONCRETE AT THE FINAL TRIAL

Junction	No. of joints	Completely bonded	Cores deliberately broken by contractor					Reason--			Other fractures	
			Good	able	Some	Little	None	Good	able	Some	Little	
Half layer day 15	12	12(100%)*	-	-	-	-	-	-	-	-	-	
Day 15/9					SEE DETAILED ANALYSIS TABLES 22 to 24							
Day 9/8	24	22(92%)	-	1(96%)	-	1(100%)+	-	-	-	-	-	
Half layer day 8	12	10(84%)	1(92%)	1(100%)	-	-	-	-	-	-	-	
Day 8/7	24	10(42%)	6(67%)	4(84%)	1(88%)	3(100%)	-	-	-	-	-	
Half layer day 7	12	12(100%)	-	-	-	-	-	-	-	-	-	
Day 7/6	24	22(92%)	-	-	-	-	1(96%)	-	-	-	1(100%)+	
Half layer day 6	23	22(96%)	-	-	-	-	-	1(100%)	-	-	-	
Day 6/5	20	18(90%)	-	-	-	1(100%)	-	-	1(95%)	-	-	
Day 5/2	12	12(100%)	-	-	-	-	-	-	-	-	-	
In parent material	9	-	5(78%)	1(83%)	1(89%)	-	-	2(78%)	-	-	-	
Total	163	140(86%)	7(91%)	6(95%)	1(96%)	5(100%)	0	1(91%)	1(95%)	1(96%)	1(100%)	

* % equal to or better in parentheses

+ Some segregation

Fig 104 Location of cores in the CIRIA mid-term trial bank

Fig 105 Typical heaving concrete cores extracted from the CIRIA mid-term trial bank

TABLE 26: SUMMARY OF CORE DENSITIES FROM THE CIRIA MID-TERM TRIAL BANK COMPARED WITH THE THEORETICAL AIR-FREE DENSITY AND CUBE DENSITIES

	Average	% t.a.f.	SD	Range
Hearting (MBH1)				
Cube density (kg/m ³)	2485	98.9	24.8	2456-2545
Core density (kg/m ³)	2505	99.7	12.0	2479-2523
Facing (MBF4)				
Cube density (kg/m ³)	2426	98.3	13.9	2400-2444
Core density (kg/m ³)	2404	97.4	28.5	2341-2467
Facing (MBF5)				
Cube density (kg/m ³)	2432	98.0	14.3	2415-2456
Core density (kg/m ³)	2408	97.0	8.5	2399-2419

TABLE 27: COMPARISON BETWEEN CORE DENSITIES OBTAINED BY WEIGHING IN AIR AND WATER, AND BY USE OF GAMMA RAY SYSTEM

Density of core (kg/m ³)			
Core No.	Gamma-ray analysis	Air and water measurement	Difference
C1/3	2501	2492	+9(0.4%)
D1/3	2482	2491	-9(0.4%)
D4/2	2483	2490	-7(0.3%)
E1/3	2503	2503	0(0.0%)

TABLE 28: SUMMARY OF CORE DENSITIES FROM THE CIRIA OPTIMUM
AGGREGATE TRIAL BANK COMPARED WITH THE
THEORETICAL AIR-FREE DENSITY

	Average	% t.a.f.	SD	Range
a = 0.50				
Core density (kg/m ³)	2470	98.7	8.1	2461-2481
a = 0.53				
Core density (kg/m ³)	2489	99.0	2.3	2484-2492
a = 0.56				
Core density (kg/m ³)	2491	98.7	2.4	2489-2495
a = 0.59				
Core density (kg/m ³)	2500	98.7	1.9	2498-2503

TABLE 29: SUMMARY OF CORE DENSITIES FROM THE CIRIA FINAL
TRIAL BANK COMPARED WITH THE THEORETICAL AIR-
FREE DENSITY AND CUBE DENSITIES

	Average % t.a.f.		SD	Range
Normal hearting (MBH2)				
Cube density (kg/m ³)	2498	99.0	9.3	2459-2528
Core density (kg/m ³)	2497	98.9	13.5	2475-2559
Paste-rich hearting (MBH3)				
Cube density (kg/m ³)	2472	98.3	10.6	2450-2516
Core density (kg/m ³)	2478	98.5	7.8	2464-2495
Air-entrained Facing (MBF6)				
Cube density (kg/m ³)	2427	99.2	15.9	2374-2495
Core density (kg/m ³)	2435	99.5	20.0	2401-2506
Non-air entrained facing (MBF6)				
Cube density (kg/m ³)	2464	100.7	18.3	2436-2486
Core density (kg/m ³)	2446	100.0	16.5	2433-2470

Fig 106 Developed relationship between density and paste/mortar ratio

TABLE 30: RESULTS OF COMPRESSIVE TESTING OF CORES TAKEN FROM THE CIRIA FULLSCALE TRIAL BANKS

Type of concrete	No. of speci- mens	Actual equivalent cube strength				Potential equivalent cube strength (MPa)
		Mean (MPa)	Standard deviation (MPa)	Coef- ficient of variation (%)	Range (MPa)	
Optimum aggregate trial at 120 days						
a = 0.50	2	60.6	-	-	60.4-60.9	79.0
a = 0.53	2	52.0	-	-	51.9-52.1	67.8
a = 0.56	2	58.6	-	-	57.1-60.1	76.4
a = 0.59	2	50.4	-	-	49.7-51.2	65.7
Mid-term trial at 190 days						
Normal hearding (MBH1)	4	44.5	6.4	14	40.8-51.5	58.0
Facing (MBF4)	3	63.3	2.0	3	61.4-65.3	82.5
Final trial at 240 and 360 days						
Normal hearding (MBH2)	13	49.1	4.3	9	43.2-56.7	64.0
Paste-rich hearding (MBH3)	3	40.8	1.6	4	39.3-42.4	53.2
Facing (MBF6)	9	78.2	4.8	6	71.2-86.7	102.0

TABLE 31: RESULTS OF TESTING FOR MODULUS IN COMPRESSION OF CORES TAKEN FROM THE CIRIA FINAL TRIAL BANK

Core no.	Concrete type	Lift no. (see Figure 101)	Stress at failure (MPa)	Estimated strain at failure ($\times 10^{-6}$)	Stress at 50% failure load (MPa)	Strain at 50% failure load ($\times 10^{-6}$)	Modulus Esc (GPa)	Actual equivalent cube strength (MPa)
C10/1	Paste-rich	H15	36.9	2500	18.4	705	26.2	42.4
C3/4	Hearting	H6	45.8	2600	22.9	825	27.8	52.8
F7/3	Hearting	H8	42.4	2800	21.1	855	24.8	47.3
H2/4	Facing	F7	70.3	3300	35.2	1130	31.2	80.9

Fig 107 Cores ready for testing in direct tension

Fig 108 Core under direct tensile testing

Fig 109 Detail of jacket for fluid pressure test (by courtesy of Building Research Establishment)

Fig 110 Change in fracture pressure of the fluid pressure test with differing rates of applied pressure and different fluids

Fig 111 Core under gas pressure test

Before test, showing location estimated joint

After test, showing fractures relative to estimated location of joints

Fig 112 Core A8/5 (facing concrete) and C6/5 (hearding concrete) subject to gas pressure test

TABLE 32: RESULTS OF DIRECT TENSILE TESTING OF CORES
TAKEN FROM THE CIRIA OPTIMUM AGGREGATE TRIAL
BANK

Direct tensile stress			
Plastic joint			
Specimen no.	Approx. age	Individual	Average
	(days)	(MPa)	(MPa)
a = 0.50			
50R	120	2.15*	
50S	120	(2.99)	(2.99)
a = 0.53			
53P	120	(2.87)	
53H	120	(2.03)	(2.87)
a = 0.56			
56O	120	(2.94)	
56L	120	(2.81)	(2.87)
a = 0.59			
59C	120	(2.85)	
59D	120	(2.69)	(2.77)

() Failure at concrete/plate interface

* Failure at weak (honeycombed) plane in concrete (not a plastic joint)

TABLE 33: RESULTS OF DIRECT TENSILE TESTING OF CORES TAKEN FROM THE CIRIA MID-TERM TRIAL BANK

Direct tensile stress							
Specimen no.	Layer (see Figures 77 and 104)	Approx. age (days)	Parent material		Plastic joint		1-day joint
			Individual	Average	Individual	Average	
			(MPa)	(MPa)	(MPa)	(MPa)	
Hearting (MBH1)							
E1/1	H2.2	190	(2.82)	(2.82)			
D1/2	H2.2/H2.3	190			(2.43)		
D2/2	H2.2/H2.3	190			2.46		
E1/3	H2.2/H2.3	190			(2.42)		
F1/2	H2.2/H2.3	190			(2.05)	(2.34)	
C1/3	H2.3/H3	190					(2.22)
D3/3	H3	190	(2.32)				-
C2/4	H3	360	2.03*				
E1/5	H3	360	2.06*	(2.14)			
Facing (MBF4 and 5)							
A1/1	F1	360	(3.67)*	(3.67)			
A1/2	F2	190	(2.56)				
A2/2	F2	190	(2.61)				
G1/3	F2	190	(2.77)				
G1/4	F2	360	(3.58)*	(3.58)			

() Failure at concrete/plate interface

* Specimen retested for specimen which had failed at concrete/plate interface during previous test

TABLE 34: COMPARISON OF ALL RESULTS OF DIRECT TENSILE AND GAS PRESSURE TESTS OF CORES TAKEN FROM THE
CIRIA FINAL TRIAL BANK

Method of test	No. of samples	Mean (MPa)	Standard deviation (MPa)	Coefficient of variation (%)	Range (MPa)
Normal hearting (MBH2)					
a) Direct tensile	11	2.13	0.21	10	1.92-2.56
b) Fluid pressures	9	2.36	0.34	15	1.95-2.85
Paste-rich hearting (MBH3)					
a) Direct tensile	8	2.33	0.16	7	2.19-2.51
b) Fluid pressure	4	1.93	0.28	14	1.60-2.20
Facing (MBF6)					
a) Direct tensile	10	3.02	0.17	6	2.69-3.28
b) Fluid pressure	7	3.41	0.51	15	2.70-4.00
One-day joints (normal hearting)					
a) Direct tensile	8	1.62	0.32	20	1.21-2.19
b) Fluid pressure	6	1.23	0.31	26	0.70-1.59
Three-day joints (paste-rich)					
a) Direct tensile	3	0.94	0.41	-	0.58-1.38
b) Fluid pressure	4	0.70	0.08	11	0.65-0.80

Fig 113 Comparison between the results of the gas pressure test and direct tensile test

Fig 114 Developed relationships between bond between layers and paste/mortar ratio

Fig 115 Lower section of Packer test equipment

TABLE 35: SUMMARY OF PACKER TESTS IN CORE HOLES IN THE CIRIA FINAL TRIAL BANK

Type of joint	Average loss in pressure after two 5-min tests				6 bar not possible	Total no. of tests
	0 - 0.5 bar	0.55 - 1.0 bar	1.0 - 6.0 bar			
Parent material	7(78%)*	2(100%)	-		-	9
Plastic joint	5(50%)	3(80%)	2(100%)		-	10
1-day joint	15(58%)	5(77%)	6(100%)		-	26
6-day joint	5(83%)	1(100%)	-		-	6
Total	32(63%)	11(84%)	8(100%)		-	51

* % or equal to or better in parentheses

TABLE 36: RESULTS OF PERMEABILITY TESTING OF CORES TAKEN FROM THE CIRIA FINAL TRIAL BANK

	No. of tests	Leaks (cm ³ /min)		Permeability (m/s x 10 ⁻¹²)		
		No.	Range	Mean	Max.	Min.
Hearting						
1. Parent material	17	3	0.6 to 67	2.55	13.1	0.13
2. Plastic joints	6	0	-	2.91	12.1	0.05
3. 1-day joint	6	1	0.7	1.59	6.9	0.04
4. 3-day joint	4	0	-	14.3	55.3	0.23
5. 6-day joint	2	0	-	1.1	2.0	0.18
Facing						
1. Parent material	4	1	0.7	1.67	3.1	0.93
2. 2-day joint	1	1	268*	-	-	-
3. 4-day joint	1	1	474	-	-	-

*pour on day 8 (see Section 15.3)

TABLE 37: BROAD RANGES OF PERMEABILITY FROM THE TESTS ON CORES OF THE HEARTING CONCRETE FROM THE CIRIA FINAL TRIAL BANK

Permeability (m/s)	10-14	10-13 to 10-14	10-12 to 10-13	10-11 to 10-12	10-11
No. of readings	3	15	10	3	4

1. CUBES AND PRISMS TESTED DURING THE PROJECT

(a) Compressive (cubes)

(b) Tensile (prisms)

(c) Compressive (prisms)

2. CORES TESTED DURING THE PROJECT

(a) Compression

(b) Tensile

3. CYLINDERS TESTED ELSEWHERE

(a) Compression

(b) Tensile

~~~~~ Top surface (cast)

~~~~~ Top surface of core

Fig 116 Direction of testing relative to the axis of casting of various specimens

Fig 117 Close-up of core E5 showing orientation of the flaky aggregate particles in roller-compacted concrete

Fig 118 Fracture surfaces of core E11/3 (parent material from H6.1) after direct tensile testing

Fig 119 Fracture surfaces of core F9/3 (1 day joint between H7.2 and H8.1) after direct tensile testing

(a) With low tensile stress

(b) With high tensile stress

Fig 120 Typical failure planes of prisms subjected to direct tensile testing

TABLE 38: COMPARISON BETWEEN A TYPICAL 'DRY LEAN CONCRETE' AND A HIGH FLYASH CONTENT CONCRETE

	Mix proportions				Cube compressive strength (MPa)			
	Cement	Flyash	Water	(kg/m ³)	7 days	28 days	91 days	365 days
'Dry lean concrete'	132	0	133		13.6	16.9	17.8	19.0
High flyash content concrete	108	113	109		21.0	36.9	42.9	53.6

Fig 121 Relationship between cube compressive strength and flyash/cementitious ratio

Fig 122 Calculated cement contribution to 7-day cube compressive strength using Feret's Relationship

Fig 123 Calculated cement contribution and flyash contribution to 28-day cube compressive strength

Fig 124 Calculated cement contribution and flyash contribution to 91-day cube compressive strength

Fig 125 Calculated cement contribution and flyash contribution to 365-day cube compressive strength

Fig 126 Relationship between the flyash and cement contribution to cube compressive strength, and the water/cementitious ratio

Note : C_c = cement/cementitious ratio (by volume) $\frac{c}{c+f}$
 C_f = flyash/cementitious ratio (by volume) $\frac{f}{c+f}$

Fig 127 Estimation of cube compressive strength of a mix with $C_f = 0.2$ ($C_c = 0.8$) and $C_w = 1.8$

Note : C_c = cement/cementitious ratio (by volume) $\frac{c}{c + f}$
 C_f = flyash/cementitious ratio (by volume) $\frac{f}{c + f}$

Fig 128 Estimation of cube compressive strength of a mix with $C_f = 0.6$ ($C_c = 0.4$) and $C_w = 1.0$

Note : C_c = cement/cementitious ratio (by volume) $\frac{c}{(c + f)}$
 C_f = flyash/cementitious ratio (by volume) $\frac{f}{(c + f)}$

TABLE 39: ESTIMATION OF CUBE COMPRESSIVE STRENGTH OF A MIX
WITH $C_f = 0.2$ ($C_c = 0.8$) AND $C_w = 1.8$
(see Figure 127)

Cube compressive strength (MPa)				
Age	Cement
contribution		Flyash
contribution	Total
7 days	(0.8 x 40.4)	+	(0.2 x 0.0)	= 32.3
28 days	(0.8 x 47.6)	+	(0.2 x 8.8)	= 39.9
91 days	(0.8 x 50.2)	+	(0.2 x 28.0)	= 45.8
365 days	(0.8 x 55.4)	+	(0.2 x 38.4)	= 52.0

TABLE 40: ESTIMATION OF CUBE COMPRESSIVE STRENGTH OF A MIX
WITH $C_f = 0.6$ ($C_c = 0.4$) AND $C_w = 1.0$
(See Figure 128)

Cube compressive strength (MPa)				
Age	Cement
contribution		Flyash
contribution	Total
7 days	(0.4 x 51.6)	+	(0.6 x 16.6)	= 30.6
28 days	(0.4 x 62.6)	+	(0.6 x 34.0)	= 45.4
91 days	(0.4 x 65.2)	+	(0.6 x 55.0)	= 59.1
365 days	(0.4 x 73.2)	+	(0.6 x 69.6)	= 71.1

Fig 129 Relationship between cube compressive strength of an immersion-vibrated concrete and flyash/cementitious ratio for a water/cementitious ratio of 0.9

Fig 130 Relationship between cube compressive strength of an immersion vibrated concrete and flyash/cementitious ratio for a water/cementitious ratio of 1.1

Fig 131 Relationship between cube compressive strength of an immersion-vibrated concrete and flyash/cementitious ratio for a flyash/cementitious ratio of 0.6

Fig 132 Cube compressive strength of concrete with flyash/cement ratio of 0,6 comparing concretes suitable for roller compaction with those suitable for immersion vibration

TABLE 41: APPROXIMATE TIME REQUIRED TO PRODUCE A MATURITY
AT WHICH CAPILLARIES BECOME SEGMENTED

| Water/cement ratio
(by weight) | Time required | Water/cementitious ratio
(by volume) |
|-----------------------------------|---------------|---|
| (w/c) | | (C _w) |
| 0.40 | 3 days | 1.25 |
| 0.45 | 7 days | 1.40 |
| 0.50 | 14 days | 1.60 |
| 0.60 | 6 months | 1.90 |
| 0.70 | 2-3 years | 2.20 |
| over 0.70 | impossible | over 2.20 |

Fig 133 Heathrow placement

Fig 134 Development of cube compressive strength with age of sub-base placed at Heathrow

Fig 135 Didcot coal handling area

Fig 136 Access roads and storage area at Didcot

Fig 137 Placement of base material through paver-finisher at Didcot

Fig 138 Detail of base material concrete at rear of paver-finisher

Fig 139 Compaction of high flyash content PQ concrete at Didcot

Fig 140 Detail of 'broom finish' obtained at Didcot

TABLE 42: CUBE COMPRESSIVE RESULTS OF THE BASE MATERIAL AT DIDCOT POWER STATION

Cube Compressive Results (MPa)								
Mix
No.	Mix
proportions		7-day		28-day		91-day	
	C _F	C _W	Mean	Range	Mean	Range	Mean	Range
I								
6/6-3								
6/6-5								
7/6-3	0.80	0.98	7.0	6.0- 8.5	14.4	12.0-16.5	19.7	16.3-22.9
7/6-5								
7/6-7								
II								
7/6-9	0.81	1.01	8.0	6.0-10.0	15.5	12.0-19.5	20.2	16.5-23.9
7/6-9								
III								
7/6-12	0.81	1.02	6.0	6.0	11.7	11.5-12.0	17.7	17.7
IV								
22/6-7								
23/6-9	0.81	1.04	8.0	6.0- 9.0	15.6	14.0-17.0	20.2	17.6-22.4
25/6-1								

Fig 141 7-day cube compressive results of the base material mixes used at Didcot

water/cementitious ratio (by weight)

Fig 142 28-day cube compressive results of the base material mixes used at Didcot

Fig 143 Development of cube compressive strength with age of base material concrete at Didcot

Fig 144 7-day cube compressive results of PQ concrete used at Didcot

TABLE 43: CUBE COMPRESSIVE RESULTS OF THE PQ CONCRETE PLACED AT DIDCOT POWER STATION

Cube Compressive Results (MPa)								
Mix No.	Mix proportions		7-day		28-day		91-day	
	C _f	C _w	Mean	Range	Mean	Range	Mean	Range
Initial PQ(91)								
13/6-8	0.70	0.86	20.0	-	34.5	-	46.7	46.1-47.3
PQ(91)								
17/6-5								
18/6-4*	0.65	0.91	20.3	17.0-25.5	36.8	31.0-44.0	48.0	40.3-57.3
18/6-6								
21/6-13								
PQ(28)								
14/6-8								
20/6-2								
20/6-10	0.60	0.95	26.4	22.0-33.0	43.3	36.5-53.5	51.8	49.6-58.0
21/6-4								
21/6-8								

*These results have been neglected from the analysis of cube compressive strength as they were statistical 'outliers'

Fig 147 Development of cube compressive strength with age of PQ concrete placed at Didcot

Fig 148 Development of indirect tensile strength with age of PQ concrete placed at Didcot

TABLE 44: INDIRECT TENSILE STRENGTH OF THE PQ CONCRETE PLACED AT DIDCOT POWER STATION

| Mix
No. | Mix
proportions | | Indirect Tensile Strength (MPa) | | | | | |
|-------------------|--------------------|----------------|---------------------------------|-----------|--------|-----------|--------|-----------|
| | | | 7-day | | 28-day | | 91-day | |
| | C _f | C _w | Mean | Range | Mean | Range | Mean | Range |
| Initial
PQ(91) | | | | | | | | |
| 13/6-8 | 0.70 | 0.86 | 1.70 | - | 2.55 | - | 3.64 | 3.49-3.80 |
| PQ(91) | | | | | | | | |
| 17/6-5 | | | | | | | | |
| 18/6-4 | 0.65 | 0.91 | 1.91 | 1.42-2.55 | 3.03 | 2.55-3.68 | 3.69 | 3.20-5.00 |
| 18/6-6 | | | | | | | | |
| 21/6-13 | | | | | | | | |
| PQ(28) | | | | | | | | |
| 14/6-8 | | | | | | | | |
| 20/6-2 | | | | | | | | |
| 20/6-10 | 0.60 | 0.95 | 1.87 | 1.56-2.12 | 3.35 | 2.97-4.39 | 4.16 | 3.95-4.65 |
| 21/6-4 | | | | | | | | |
| 21/6-8 | | | | | | | | |

Fig 149 Relationship between indirect tensile strength and cube compressive strength

Fig 150 Development of flexural strength with age of PQ concrete placed at Didcot

TABLE 45: FLEXURAL STRENGTH OF THE PQ CONCRETE PLACED AT DIDCOT POWER STATION

Flexural Strength (MPa)								
Mix
No.	Mix
proportions		7-day		28-day		91-day	
	C _f	C _w	Mean	Range	Mean	Range	Mean	Range
Initial
PQ(91)								
13/6-8	0.70	0.86	2.35	-	4.05	-	5.50	5.28-5.72
PQ(91)								
17/6-5								
18/6-4	0.65	0.91	2.44	2.20-2.75	4.03	3.60-4.40	5.35	4.70-6.30
18/6-6								
21/6-13								
PQ(28)								
14/6-8								
20/6-2								
20/6-10	0.60	0.95	3.04	2.80-3.20	4.19	3.45-4.75	5.74	5.26-6.75
21/6-4								
21/6-8								

Fig 151 Relationship between flexural strength and cube compressive strength

Fig 152 Relationship between flexural strength and cube compressive strength derived by others

TABLE 46: COMPRESSIVE TESTING OF CORES TAKEN FROM THE DIDCOT PLACEMENT

Mix	Cube				Equivalent cube compressive strength from cores			
	Compressive		Strength (MPa)		Conc. Society Report*		BS 1881	
	Mean	Range	Mean	Range	%Cube	Mean	Range	%Cube
B.M.IV	16.3	15.0-17.0	25.0	23.9-25.0	154	23.7	22.5-25.0	145
B.M.V	12.7	12.5-13.0	16.9	-	133	16.5	-	130
PQ(91)	38.3	31.0-44.0	44.0	40.0-50.0	115	38.7	35.0-44.0	101
PQ(28)	45.2	40.7-50.7	54.5	52.0-59.5	121	47.2	45.0-51.5	104

* Potential cube strength (not actual)

Note: The base material core results have been separated because the properties of the slabs from which they were taken differed substantially

Fig 153 Variation with depth of density of base material cores. (By courtesy of TRRL)

TABLE 47: COST COMPARISON OF A CONVENTIONAL 'DRY LEAN CONCRETE' WITH THE HIGH FLYASH CONTENT CONCRETE USED IN THE SUB-BASE AT HEATHROW AIRPORT

	Conventional			H.F.C.C.	
	Unit	Weight	Cost	Weight	Cost
	price				
	£/tonne	kg/m ³	£/m ³	kg/m ³	£/m ³
Aggregate and sand	5.00	2260	11.30	2010	10.05
Cement	40.00	100	4.00	70	2.80
Flyash	10.00	-	-	180	1.80
SUB-TOTAL			15.30		14.65
+ Additional costs*			-		0.15
TOTAL			15.30		14.80
Saving (£/m ³)					0.50
(%)					3.3%

* The additional costs are those necessary on finer filters for the flyash silos, the additional site supervision (for the tighter control) and the higher costs associated with additional trial mixes and advice required for those mixes

TABLE 48: COST COMPARISON OF A CONVENTIONAL PQ CONCRETE CONFORMING TO THE DTp SPECIFICATION WITH THE PQ(91) MIX USED AT DIDCOT

| | Unit
price | Conventional | | H.F.C.C. | |
|-----------------------------|---------------|-------------------|------------------|-------------------|------------------|
| | | Weight+ | Cost | Weight+ | Cost |
| | £/tonne | kg/m ³ | £/m ³ | kg/m ³ | £/m ³ |
| 40 mm | 7.50 | 575 | 4.57 | 548 | 4.36 |
| 20 mm | 7.80 | 402 | 3.32 | 383 | 3.17 |
| 10 mm | 8.20 | 172 | 1.50 | 165 | 1.43 |
| Sand | 3.50 | 599 | 2.22 | 759 | 2.82 |
| Cement | 43.00 | 339 | 14.80 | 175 | 7.64 |
| Flyash | 2.00 | - | - | 225 | 0.46 |
| | (10.00) | | | | (2.25) |
| Admixture= | 0.50 | 1.40 | 0.71 | 2.65 | 1.34 |
| SUB-TOTAL | | | 27.12 | | 21.22 |
| | | | | | (23.01) |
| + Additional costs* | | | - | | 0.15 |
| TOTAL | | | 27.12 | | 21.37 |
| | | | | | (23.16) |
| Savings (£/m ³) | | | | | 5.75 |
| | | | | | (4.11) |
| (%) | | | | | 21.2% |
| | | | | | (15.2%) |

= in litres

+ Assuming 4.5% air

* see Note for Table 47

TABLE 49: COST COMPARISON OF A CONVENTIONAL PQ CONFORMING WITH THE BAA SPECIFICATION WITH THE MODIFIED PQ(28) MIX USED AT DIDCOT

| | Unit
price | Conventional | | H.F.C.C. | |
|-----------------------------|---------------|-------------------|------------------|-------------------|------------------|
| | | Weight+ | Cost | Weight+ | Cost |
| | £/tonne | kg/m ³ | £/m ³ | kg/m ³ | £/m ³ |
| 40 mm | 7.50 | 565 | 4.24 | 495 | 3.71 |
| 20 mm | 7.80 | 396 | 3.09 | 346 | 2.70 |
| 10 mm | 8.20 | 169 | 1.39 | 149 | 1.22 |
| Sand | 3.50 | 589 | 2.06 | 732 | 2.56 |
| Cement | 43.00 | 374 | 16.08 | 217 | 9.33 |
| Flyash | 2.00 | - | - | 206 | 0.41 |
| | (10.00) | | | | (2.06) |
| Admixture= | 0.50 | 1.54 | 0.77 | 2.95 | 1.48 |
| SUB-TOTAL | | | 27.63 | | 21.41 |
| | | | | | (23.06) |
| + Additional costs* | | | | | 0.15 |
| TOTAL | | | 27.63 | | 21.56 |
| | | | | | (23.21) |
| Savings (£/m ³) | | | | | 6.07 |
| | | | | | (4.42) |
| (%) | | | | | 22.0% |
| | | | | | (16.0%) |

= in litres

+ Assuming 4.5% air

* see Note for Table 47

TABLE 50: MIX PROPORTIONS OF CONCRETES USED IN REPORTED FULLSCALE TRIALS OF ROLLER -COMPACT

| Fullscale placement | | Coarse
aggregate
content

(kg/m ³) | Fine
aggregate
content

(kg/m ³) | Cement
content

(kg/m ³) | Flyash
content

(kg/m ³) | Free
water
content

(kg/m ³) | (mm) |
|--------------------------------|-------------|--|--|---|---|--|------|
| Tims Ford placement[22] | | 1649 | 605 | 56 | 77 | 77 | 75 |
| No-slump concrete[23] | Mix 1 | 1738 | 694 | 139 | 0 | 74 | 115 |
| | Mix 2 | 1775 | 617 | 139 | 0 | 80 | 115 |
| | Mix 3 | 1691 | 682 | 139 | 0 | 86 | 75 |
| Lost Creek[24] | Mix 1 | 1602 | 676 | 139 | 0 | 83 | 75 |
| | Mix 4 | 1602 | 676 | 42 | 78 | 83 | 75 |
| | Mix 5 | 1602 | 676 | 69 | 56 | 83 | 75 |
| Singleton
trials[41,42] | Mix 12 | 1602 | 759 | 65 | 23 | 71 | 75 |
| | Mix 1 | 1426 | 745 | 75 | 164 | 97 | 40 |
| | Mix 2 | 1426 | 745 | 75 | 164 | 85 | 40 |
| Okawa Cofferdam[35] | Mix P-1 | 1571 | 636 | 96 | 24 | 90 | 75 |
| | Mix P-2 | 1526 | 665 | 96 | 24 | 96 | 75 |
| | Mix P-3 | 1481 | 694 | 96 | 24 | 102 | 75 |
| | Mix P-4 | 1436 | 723 | 96 | 24 | 108 | 75 |
| | Mix P-13 | 1488 | 680 | 112 | 28 | 98 | 75 |
| Bellefonte
placement[43,44] | 201.5CFW-R | 1418 | 734 | 44 | 165 | 83 | 40 |
| | 301.5BFW-R* | 1504 | 690 | 120 | 147 | 110 | 40 |
| WES trial section[46] | | 1453 | 771 | 139 | 62 | 95 | 40 |
| Itaipu trials[33] | Mix 2 | 1596 | 813 | 62 | 27 | 83 | 75 |
| | Mix 8 | 1261 | 1089 | 88 | 25 | 100 | 40 |
| Itaipu placement[33] | | 1625 | 804 | 91 | 26 | 71 | 75 |
| Tamar TW placement[40] | | 1350 | 790 | 46 | 69 | 105 | 20 |
| Torpoint placement[73] | | 1360 | 760 | 140 | 0 | 120 | 20 |
| Typical dry lean concrete[74] | | 1262 | 841 | 117 | 0 | 133 | 20 |

+ with bedding mix, * bedding mix, = corrected to allow for oversized sand grading

TABLE 51: PROPERTIES OF HARDENED CONCRETE SPECIMENS FROM REPORTED FULLSCALE TRIALS

| | | | | | | | ft = | Tensi
direct |
|--------------------------------|-------------|-------------------------|---------|----------------|------------------------|--------------------|------|-----------------|
| | | Average core
density | | Age
at test | Compressive
testing | Parent
material | | A
join |
| Fullscale placement | | (kg/m ³) | (% taf) | (days) | (MPa) | (MPa) | | (MPa) |
| Tims Ford placement[22] | | 2494 | 97.3 | 111 to 139 | fc=24.1 | ft=1.44 | | ft=1 |
| No-slump concrete[23] | Mix 1 | 2438 | 92.9\$ | 73 | fc=21.7 | fs=2.06 | | - |
| | Mix 2 | 2568 | 98.3 | 71 | fc=25.7 | fs=3.14 | | - |
| | Mix 3 | 2559 | 98.4 | 66 | fc=27.4 | fs=2.72 | | - |
| Lost Creek[24] | Mix 1 | 2503 | 97.1 | 120 | fc=22.6 | fs=2.65 | | fs=1 |
| | | | | 360 | fc=32.9 | - | | - |
| | Mix 4 | 2489 | 97.3 | 120 | fc=15.9 | fs=1.69 | | fs=0 |
| | | | | 360 | fc=22.5 | - | | - |
| | Mix 5 | 2497 | 97.4 | 120 | fc=17.9 | fs=1.89 | | fs=0 |
| Singleton
trials[41,42] | Mix 12 | 2453 | 94.7 | 120 | fc=11.0 | - | | - |
| | Mix 1 | 2564 | 99.0 | 34 | fc=14.4 | - | | - |
| | | | | 90 | fc=27.0 | - | | - |
| | Mix 2 | 2575 | 98.7 | 34 | fc=19.8 | - | | - |
| Okawa Cofferdam[35] | | | | 90 | fc=26.7 | - | | - |
| | Mix P-1 | 2386 | 97.2 | 90 | fc=17.6 | fs=1.65 | | - |
| | Mix P-2 | 2368 | 96.9 | 90 | fc=18.5 | fs=1.84 | | - |
| | Mix P-3 | 2395 | 98.4 | 90 | fc=12.8 | fs=1.78 | | - |
| | Mix P-4 | 2384 | 98.4 | 90 | fc=19.7 | fs=2.30 | | - |
| Bellefonte
placement[43,44] | Mix P-13 | 2416 | 98.9 | 90 | fc=17.6 | fs=2.20 | | - |
| | 201.5CFW-R | 2474 | 98.1 | 90 | fc=18.0 | ft=1.54 | | ft=1 |
| | 301.5BFW-R* | 2478 | 98.9 | 90 | - | ft=2.68 | | ft=2 |
| WES trial section[46] | | - | - | - | - | ft=2.14 | | ft=1 |
| Itapu trials[33] | Mix 2 | 2625 | 96.6 | 119 | fc=13.0 | fs=1.04 | | ft=1 |
| | Mix 8 | 2550 | 95.4 | 119 | fc=14.6 | fs=1.37 | | - |
| Itaipu placement[33] | | 2515 | 91.6 | 110 | fc=13.3 | fs=1.37 | | - |
| | | | | 180 | fc=15.2 | fs=1.53 | | - |
| Tamar TW placement[40] | | 2376 | 93.5 | 90 | fc=14.0 | - | | - |
| Torpoint placement[73] | | 2354 | 95.8 | - | - | - | | - |
| Typical dry lean concrete[74] | | 2228 | 94.7 | - | - | - | | - |

\$ concrete unresponsive to vibration, some segregation, + bedding mix used, = test not in

TABLE 52: DETAILS OF FLYASHES USED DURING WORK ON THESES

| Trial/
placement | Ciria Project | | | Final
trial | Additional
mix
programme | Structural
mix
programme | Initial trials in roads | | | Disposal |
|-----------------------|----------------------|----------------|-----------|----------------|--------------------------------|--------------------------------|-------------------------|--------|----------|----------|
| | Laboratory programme | Mid-term trial | Y | | | | Hoddesdon | Drax | Heathrow | |
| Source of flyash | W | Y | Z | X | Ratcliffe | Ratcliffe | Eggborough | Didcot | Didcot | Disposal |
| Chemical Analysis (%) | | | | | | | | | | |
| Silica | 43.4 | 45.5 | 46.6-49.3 | 47.8-52.4 | | | | | 54.0 | |
| Alumina | 27.6 | 27.2 | 24.0-30.4 | 27.4-27.5 | | | | | 29.3 | |
| Iron | 9.8 | 13.5 | 6.2-9.5 | 10.0-10.8 | | | | | 6.3 | |
| Calcium | 2.3 | 2.9 | 2.7-2.9 | 1.6-1.8 | n/a | n/a | n/a | | 4.0 | |
| Magnesium | 1.5 | 1.9 | 1.6-2.1 | 1.8-1.9 | | | | | 1.3 | |
| Total sulphates | 1.2 | 1.0 | 0.5-0.9 | 0.4 | | | | | 0.4 | |
| Sodium | 2.2 | 1.3 | 1.2-1.5 | 1.2-1.5 | | | | | 1.3 | |
| Potassium | 3.9 | 2.6 | 1.1-4.0 | 4.0-4.1 | | | | | 2.6 | |
| Titanium | 1.3 | n/a | 0.4-1.1 | 0.9 | | | | | 1.2 | |
| Physical Properties | | | | | | | | | | |
| Relative density | 2.05 | 2.32 | 2.05-2.07 | 2.30 | | | 2.07 | | 2.16 | 2.1 |
| Loss on ignition (%) | 6.6 | 2.8 | 3.0-3.3 | 3.9 | n/a | n/a | n/a | n/a | 4.0 | 5.5 |
| Retained on | | | | | | | | | | |
| 45 μ sieve (%) | 26.0 | 4.9 | 15-20 | n/a | | | | | 24.6 | 17.8 |
| 150 μ sieve (%) | 10.0 | 0.3 | 1-3 | | | | | | n/a | n |

TABLE 53: MIX PROPORTIONS OF STANDARD MIXES

	C_f	C_w
a = 0.50
P = 0.44	0	1.13
	0.6	1.05
	0.8	0.93
a = 0.56
P = 0.41	0	1.60
	0.6	1.39
	0.8	1.24

TABLE 54: ESTIMATED DIRECT TENSILE STRENGTH OF STANDARD MIXES

		Tensile strength (MPa)		
	C_f	7 day	28 day	91 day
a = 0.50
P = 0.44	0	3.7	3.9	3.9
	0.6	2.8	3.5	4.0
	0.8	1.5	2.5	3.3
a = 0.56
P = 0.41	0	3.4	3.7	3.7
	0.6	1.9	2.7	3.5
	0.8	1.0	1.6	2.2

TABLE 55: ESTIMATED STATIC MODULUS IN TENSION OF STANDARD MIXES

		Tensile modulus (GPa)			
		C_f	7 day	28 day	91 day
$a = 0.50$
$P = 0.44$	0		46	46	46
	0.6		35	41	43
	0.8		29	34	37
$a = 0.56$
$P = 0.41$	0		44	44	44
	0.6		30	36	39
	0.8		24	28	31

TABLE 56: ESTIMATED TENSILE STRAIN CAPACITY OF STANDARD MIXES

		Tensile strain capacity (x 10 ⁻⁶)			
		C _F	7 day	28 day	91 day
a = 0.50
P = 0.44	0		80	89	89
	0.6		79	85	94
	0.8		52	73	90
a = 0.56
P = 0.41	0		77	84	84
	0.6		63	75	90
	0.8		41	57	71