Engineering Properties of Binary Cement Concretes and their Effect on Punching Shear of Flat Slabs

By

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Dedicated to

My late father Abdus Samad Khan

&

My late brother Shafqat Munir Khan

ABSTRACT

Concrete is the most important building material in the world due to the fact that it is versatile and gives architectural freedom. For sustainable construction solutions concrete is the material of choice if the embodied CO_2 content is considered. In concrete, cement is the main constituent and due to the limit on the availability of natural minerals used, the energy released and the CO_2 emissions produced during its manufacture, it can be partially replaced using industrial by-products e.g. Pulverised Fuel Ash (PFA), Ground Granulated Blast furnace Slag (GGBS) and silica fume. The effects of the partial replacement of cement with these industrial by products on fresh and hardened properties of concrete cured under summer and winter environments are established and compared with the Portland cement (PC) concrete.

Early age strength of concrete containing GGBS and PFA is less than the PC concrete, which would prevent its use in the in the post tensioned concrete and in fast-track construction, where early removal of formwork, or early application of load to the structure are the main requirements. For this reason, and due to the demand of high strength concrete in construction, for its improved durability properties, concrete containing GGBS and PFA was produced by keeping the water/cement ratio low and a superplasticiser was used to achieve the required workability. At low water/cement ratio, concrete containing GGBS up to 50 % and PFA up to 30 % can achieve the required early age strength for the removal of formwork if cured properly.

At the age of 28 days, the flexural strength and modulus of elasticity of concrete in which PC is partially replaced with GGBS and PFA are increased in comparison to where this is not carried out.

For practical applications of the sustainable concrete mixes in structural concrete production and due to the limited data availability on punching shear strength, concrete containing GGBS and PFA in flat slab specimens were tested for this property. In flat slabs without beams, the design criteria is often the resistance against punching shear failure at the column/slab connection. Punching failure is the separation of the portion of the slab surrounding the column from the rest and is a brittle failure.

Experimental punching shear results are compared to the estimates of BS 8110, BS EN1992-1-1 and ACI 318 and it was found that the estimates of ACI318 and BS 8110, ignoring the partial safety factors, are close to the experimental results and the estimates of BS EN 1992-1-1 are over conservative.

Based on the test results of punching shear resistance of flat slabs, and the materials used, it is concluded that concrete containing GGBS up to 50 % and PFA up to 30 % can be used in flat slabs without any special design requirements and the design rules given in different codes of practice can be used without modifications.

It is evaluated that for a concrete, designed for characteristic strength of 30 MPa, a reduction of 152 kg/m^3 of CO_2 and 0.65 GJ/m^{3}of energy consumption can be achieved by replacing PC with 50 % GGBS and a reduction of 62.5 kg/m^3 of CO_2 and 0.27 GJ/m^3 of energy can be achieved by using 30 % PFA concrete.

DECLARATION

I declare that this thesis and the work presented in this thesis is my own work which is generated from my own research carried out at Kingston University London. The source is always given where I have consulted the published work of others, and quoted from the work of others.

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF ABREVIATION

Abbreviation of Terms

BS	British Standard
BRE	Building Research Establishment
C&D	Construction and demolition
CCA	Climate Change Agreement
CIA	Concrete Industry Alliance
CSD	Commission on Sustainable Development
CTRL	Channel Tunnel Rail Link
EN	European Normative
EU	European Union
EC2	Eurocode 2
ECO_2	Embodied CO_2
EE	Embodied Energy
GGBS	Ground Granulated Blast furnace Slag
LCPD	Lafarge Combustion Plant Directive
LBA	London Building Act

LVDT	Linear Variable Differential Transformers
MR	Modulus of Rupture
NA	Natural Aggregate
PC	Portland Cement
PCA	Portland Cement Association
PFA	Pulverised Fuel Ash
RCC	Reinforced Cement Concrete
RH	Relative Humidity
SSD	Saturated Surface Dry
SCOSS	Standard Committee of Structural Safety
UNEP	United Nations Environment Programme
UNCED	United Nations Conference on Environment & Development
UKQAA	United Kingdom Quality Ash Association
WCED	World commission on environment and development

LIST OF SYMBOLS

A_c	Area of concrete
A_s	Area of steel
A_{st}	Area of tension steel bar, mm^2
d	effective depth of slab, mm (Overall depth 'h' minus the diameter of bar)
Ec	Modulus of elasticity of concrete, GPa
Es	Modulus of elasticity of steel, GPa
f_{cm}	Mean cylinder compressive strength
f_{ck}	Characteristic compressive cylinder strength
f_c	Characteristic compressive cube strength of concrete , MPa
f_y	Yield strength of steel, MPa
V_F	Experimental punching shear failure load
V_{col}	Ultimate load carrying capacity of slab based on maximum shear stress at the column face
V_{Rdc}	Punching shear strength EC2
V_{uo}	Punching shear strength ACI 318
V_c	Punching shear strength BS 8110
W_A	Water Absorption
ρ_{st}	Ratio of tensile reinforcement = As/bd

s	Spacing of bars in tension steel layer
u_o	Shear perimeter at the face of column (4b for a square column size "b")
u	First critical shear perimeter
w/c	Free water cement ratio of concrete mix
$\Delta\sigma$	Change in stress
$\Delta\varepsilon$	Change in strain
ρ_a	Aggregate Apparent Particle Density, Mg/m^3
ρ_b	Aggregate Loose Bulk Density, Mg/m^3
ρ_p	Aggregate oven-dried or pre-dried particle density, Mg/m^3
ρ_{ssd}	Aggregate Particle density on a saturated and surface-dried basis, Mg/m^3
ρ_{rd}	Aggregate Particle density on an oven-dried basis, Mg/m^3
ρ_w	Density of the water, Mg/m^3
$\mu\varepsilon$	micro strain
σ	Stress, MPa
δ	Deflection at the middle of the concrete beams, mm
ε	Strain
Ø	Diameter of steel bar, mm

1. INTRODUCTION

1.1. Background

According to the Brundtland commission (1987) Sustainable development is the development of the present generation for their needs without jeopardising those of future generations. The main aim of the thesis is to minimise the embodied CO_2 (ECO_2) of structural concrete. Concrete is an important structural material composed of cement, aggregate and water. During the last century concrete technology has gained importance, due to which researchers are developing methods for making concrete.

For sustainable construction solutions, concrete is the material of choice. In concrete, cement is the main constituent and due to the limit on the availability of natural minerals used for making cement and due to the emission of CO_2 produced in the manufacturing of cement, research is focussed on binders, where Portland cement in concrete is partially replaced using industrial by-products e.g. pulverised fuel Ash (PFA), ground granulated blast furnace slag (GGBS) and silica fume. Processing of these by-products into quality materials avoids the need to landfill.

The use of GGBS and PFA in concrete tends to slow down the early age strength which limits its use in the fast track construction and post tensioned concrete which are subjected to high early loads. Early age strength of concrete containing GGBS and PFA can be increased by reducing the water/cement ratio. Due to the lack of information on engineering properties of concrete containing GGBS and PFA with low water/cement ratio, it was chosen for this research.

Flat slabs have gained importance all around the world due to their ease of use in construction, flexibility, uninterrupted floor areas, with no downstand beams and more floor to ceiling height availability. In flat slabs without beams, the design criteria is often the resistance against punching shear failure at column/slab connection. Punching failure is the separation of the portion of the slab surrounding the column from the rest and is a brittle failure. For sustainable construction and due to the lack of sufficient knowledge available on punching shear strength of RCC (Reinforced Cement Concrete) flat slabs in which PC (Portland Cement) is partially replaced with GGBS and Fly ash, flat slab specimens were tested for punching shear resistance. As there is no published data available on punching shear resistance of flat slabs containing GGBS and PFA, punching shear resistance of flat slabs is determined.

1.2. Research Aims & Objectives

The main aim of this research is to assess the use of industrial by-products, as cement replacements for minimising the embodied CO_2 content of structural concrete without adversely affecting product performance.

The specific objectives of the work are to;

- Understand and review the concept of sustainable concrete construction and the published work on the subject matter.
- Investigate and compare key fundamental physical and chemical characteristics of cementitious contents; such as PFA, GGBS & silica fume, and assess their suitability for use to minimise the embodied carbon dioxide content of structural concrete.
- Establish strength development of a range of concrete mix compositions, produced using binary cements for low carbon concrete, and cured under simulated summer and winter conditions.
- Assess the influence of low carbon concrete mixes when produced using industrial products as cementitious constituents, on engineering properties (modulus of elasticity & flexural strength). Measure and compare the performance of traditional structural concrete with lower-embodied-CO_2 content concretes, designed to meet practical standards requirements.

- Examine the practical implications for use of low-carbon concrete mixes in structural concrete production, by testing a flat slab specimen, made up of low carbon concrete for punching shear resistance.

- Quantify the beneficial influence of GGBS and PFA in concrete with regard to the emission of CO_2 and energy consumption.

1.3. Scope of Work

The proposed research will contribute towards the sustainable concrete construction by providing a comparison of physical, engineering and durability-related properties between Portland cement (PC), concrete and those in which the Portland cement is partially replaced by cementitious constituents.

The low early age strength gain of concrete containing cementitious constituents, prevents its use in the post-tensioned and in fast-track construction, where early removal of formwork, or early application of load to the structure are the main requirements. For this reason, and due to the demand of high strength concrete in construction, for its improved durability properties, concrete containing GGBS and PFA was produced by keeping the water/cement ratio low and a superplasticiser was used to achieve the required workability. Concrete made was cured under stimulated summer ($20 \pm 2^{\circ}C$) and winter ($7 \pm 2^{\circ}C$) environments, to check its suitability in practical applications. The strength development of concrete containing cementitious constituents was recorded at 1,2,3,5,7,28 and 56 days under different curing regimes and the results were compared. Modulus of elasticity of concrete and Flexural strength was recorded at the age of 28 days.

As very limited data is available on the punching shear resistance of RCC flat slabs containing GGBS and PFA, this was used in the flat slab specimens and tested for punching shear resistance, which is often the design parameter for flat slabs. The results from the tests of the punching shear resistance are compared with the estimates of the punching shear resistance given by various codes of practice.

Beneficial influences of GGBS and PFA in concrete with regards to CO_2 emissions and energy released are determined for various concrete mixes.

1.4. Outline of Thesis

In Chapter 2 the concept of sustainability and sustainable concrete construction are explained. The importance of concrete in construction and its embodied CO_2 are discussed. A review of previous research on the use of GGBS, PFA and silica fume in concrete is presented. The properties of concrete made by partial replacement of cement with these materials are discussed. The effects of curing on the properties of concrete containing GGBS and PFA are reviewed. The punching shear resistance of flat slabs and testing methods for this are also considered in Chapter2.

In Chapter 3 the research methodology and the experimental programme followed are explained in detail.

In Chapter 4 the experimental results on fresh and hardened engineering properties of different low carbon emission concrete mixes are discussed. The effect of three different curing conditions on strength development is determined as it is very important for concrete to be used in structural applications like flat slabs and post tensioned concrete.

In Chapter 5 the experimental work and the results of the tests for punching shear resistance of low carbon emission concrete flat slabs are presented and compared with CEM I concrete flat slab. Experimental results are compared with BS8110, ACI 318 and Eurocode 2 estimates. Average mid-span deflection for different slab specimens are analysed and compared. Strains in reinforcement bars and the concrete surface under the application of load are calculated and analysed. The beneficial influence of GGBS and PFA in concrete is determined in Chapter 5.

In Chapter 6 the conclusions are drawn and recommendations for further research presented.

2. LITERATURE REVIEW

2.1. Introduction

In this chapter a review of previous research on the topic is presented. Published articles and information sheets on the subject matter are being collected from a range of sources; including KU learning resource centre, e-resources, professional institutions, government departments etc. Various journals and conference papers dealing with the sustainable concrete published in different times have been reviewed to date. As it was important to know about the research done on the topic in the past, no particular criteria regarding the year of publication was followed in the collection of literature review.

According to the UNEP (1983), the concept of sustainability has been known for a long time and there have been many conferences around the world in which governments and non government organisations have participated. In these conferences, they have agreed on promoting different policies of sustainability. Sustainability has three main components which are environment, social and economic. Despite the importance of all three components of sustainability, the environment component is focussed in this research and the methods of reducing the environmental effects of concrete are determined. Carbon dioxide (CO_2) is one of the greenhouse gases that cause global warming by trapping the Sun's energy in the atmosphere and this process is called the greenhouse effect. CO_2 contributes significantly to climate change. Sustainability is usually expressed, or assessed, in terms of either embodied energy or embodied carbon dioxide equivalents.

According to The Concrete centre (2010), the amount of embodied CO_2 (ECO_2) of concrete is a function of the cement content in the mix designs. The amount of CO_2 produced during the concrete manufacturing process is relatively small when compared with that of other building materials. The ECO_2 impact of concrete is around 100kg CO_2 per tonne.

According to The Concrete centre (2010), concrete is the most important building material in the world due to the fact that it is versatile and gives architectural freedom. For sustainable construction solutions, concrete really is the sustainable material of choice. For sustainable concrete construction, the embodied CO_2 content of concrete is considered. In concrete Portland cement is its main constituent. In the production of Portland cement a huge amount of CO_2 is produced. To reduce the embodied CO_2 of concrete, Portland cement can be partially replaced with GGBS and PFA.

GGBS is a by-product from the manufacture of iron in the blast furnace and PFA is a by-product from power stations, where furnaces are fired with bituminous coal. Both GGBS and PFA are available in large quantities and are mostly used in ready mix or precast concrete. As GGBS and PFA are by- products they reduce the embodied CO_2 of concrete, when added as partial replacement of cement in concrete. A review of the research on the properties of concrete containing GGBS and PFA is presented in this Chapter.

Concrete flat slabs are used for a wide range of buildings, including offices, warehouses, car parks and residential building blocks. Due to the importance and popularity of flat slabs and the amount of concrete used in these, it is necessary to reduce their embodied CO_2 content which can be done by replacing PC with GGBS and PFA. Due to the lack of research on concrete flat slabs in which PC is partially replaced by GGBS and PFA and the importance and popularity of flat slabs, flat slabs are chosen for this research. According to Muttoni (2008), Punching shear resistance is often the governing design requirement for flat slabs. A review of the research on punching shear resistance of flat slabs is also presented in this chapter.

2.2. Sustainability

The concept of sustainability has been known from 1960, but the implementation of the sustainable development concept started worldwide in the 1980's. According to the Brundtland Commission (1987) definition "Sustainable Development is development that meets the needs of the present without compromising the ability of future generations to meet their own needs". "Our Common Future" was published by the World Commission on Environment and Development in 1987. This report is also called the Brundtland report as it was led by Gro Harlem Brundtland of Norway and introduced the concept of 'Sustainable Development' internationally. The objective of this commission was to propose long- term environmental strategies and recommend ways between developing countries and countries which are at different stages of economical and social development to achieve a common objective that takes into account the interrelationship between people, resources, environment and development. Sustainable development was not defined as a fixed state of harmony, but rather a process of change in which the exploitation of resources, the direction of investments, the orientation of technological development, and institutional change are made consistent with future as well as present needs. It was highlighted that the most complex problem is global warming and the threat to the ozone layer. Environment and development are not separate challenges. They are closely linked together. Development cannot be done by deteriorating the environmental resources and the environment cannot be protected without development. Most of the development paths of the industrialized nations and the development decisions of these countries are unsustainable; because of their great economic and political power, they will have a profound effect upon the ability of all people to sustain human progress for generations to come.

According to the UN Conference on the Human Environment (UNEP) Stockholm (1972), 'The relationship between economic development and environmental degradation was introduced for the first time. After this Conference, Governments set up the United Nations Environment Programme (UNEP). The progress was very slow in the succeeding years to integrate environmental concerns into national economic planning and decision-making. Overall, the environment continued to deteriorate, and the problems of ozone depletion, global warming and water pollution grew more serious and the destruction of natural resources accelerated at an alarming rate'.

The UN (1983) set up the World Commission on Environment and Development. Environmental degradation was considered as a side effect of industrial wealth and with only a limited impact, was understood to be a matter of survival for developing nations.

According to Earth Summit (1992), after considering the 1987 Brundtland report, the UN General Assembly called for the UN Conference on Environment and Development (UNCED) in 1992 at Rio de Janeiro. The primary goals of the Summit were to establish ways for socio-economic development, prevent the continued deterioration of the environment, and to lay a foundation for a global partnership between the developing and the more industrialized countries, based on mutual needs and common interests, that would ensure a healthy future for the planet. The Commission on Sustainable Development (CSD) was formed to monitor and report the progress of the countries participating in the earth summit. A five year review of the earth Summit was planned in 1997 but unfortunately it was observed that the implementation of sustainable development plans was very slow.

Struble and Godfrey (2004) say that worldwide focus is on sustainable development due to the deterioration of our environment. An example of the environment deterioration is the climate changes resulting from the thinning of the ozone layer and is due to the human development. The economic component is often ignored in the developed countries of the world, but is very important to achieve the goal of sustainable development. Due to the global poverty and the availability of the resources, economic and environmental sustainability are closely linked. The social component is given less importance but is equally important because with broad social commitment a progress towards sustainability can be made.

The Earth Summit (2012) took place in Rio de Janeiro, Brazil, from 20th -22nd June, 2012. The main objective of this summit was to secure political commitment to sustainable development and assess progress towards internationally agreed commitments. The theme was the green economy in the context of poverty eradication and sustainable development and the institutional framework for sustainable development.

2.2.1. Embodied Energy

According to Cement and concrete institute (2011), "Embodied energy (EE) is the energy consumed for raw material extraction, transportation, manufacture, assembly, installation, disassembly and deconstruction for any product system over the duration of a product's life." Embodied energy and embodied carbon are linked together. Embodied carbon can be reported as embodied energy using the various emission factors. Embodied energy has two components. Direct energy, i.e. the energy used to transport building products to the site, and then to construct the building and indirect energy, i.e. the energy used to acquire, process, and manufacture the building materials, including any transportation related to these activities.

According to Cement and concrete institute (2011), typically, embodied energy is measured as a quantity of non-renewable energy per unit of building material, component or system. It is expressed as mega Joules (MJ) or giga Joules (GJ) per unit of weight (kg or tonne) or area (square metre).

2.2.2. Embodied CO_2

Embodied CO_2 is the total amount of CO_2 produced in the extraction and transportation of raw materials and their manufacture into the final product. It is often expressed as CO_2 per unit mass, or CO_2 per unit volume. (kgCO_2/tonne or kg CO_2/m^3).

Carbon dioxide (CO_2) is one the greenhouse gases that cause global warming by trapping the Sun's energy in the atmosphere and this process is called the greenhouse effect. CO_2 contributes significantly to the climate change. Sustainability is usually expressed or assessed in terms of either embodied energy or embodied carbon dioxide equivalents.

According to sustainable concrete (2011) ECO_2 is "the measure of the amount of CO_2 emissions generated from the energy needed for the raw material extraction, processing, transportation, assembling, installation, disassembly and deconstruction for any system over the duration of a product's life." It can be combined with in-use impacts when evaluating the whole life cycle impacts of buildings. ECO_2 can be measured from cradle-to-gate, cradle-to-site, cradle-to-end of construction, cradle-to-grave, or even cradle-to-cradle. Normally ECO_2 datasets are cradle-to-gate. ECO_2 is usually expressed in kilograms of CO_2 emissions per kilogram of product or material. Embodied energy and embodied carbon are linked. Embodied carbon dioxide can be reported as embodied energy using the various emission factors. There are CO_2 emissions throughout a building's life-cycle from the initial design to the refurbishment or eventual demolition of the building. These emissions can be quantified to produce a carbon life-cycle footprint for a building, which can then be used to plan an effective reduction strategy. According to Lemay (2008), the amount of ECO_2 of concrete is a function of the cement content in the mix proportions. The amount of ECO_2 produced during concrete manufacture is relatively small when compared with that of other building materials. The embodied CO_2 impact of concrete is around 100kg CO_2 per tonne.

According to United Kingdom Quality Ash Association (UKQAA) (2010), Indicative values of ECO_2 for the main cementitious constituents of reinforced concrete, which are derived from the calendar year 2010 data, are presented in Table 2.1.

These are 'cradle-to-factory-gate' values and they do not consider transport from place of manufacture to concrete plants. It is to be noted that GGBS and PFA are by- products obtained in the manufacture of iron and power plants respectively, so the impact of producing iron or electricity is not considered because these by-products were to be produced and land filled. For GGBS only the impact of processing the granulated slag to GGBS is considered. PFA is normally used without further processing and therefore its environmental burden is small.

Table 2-1 Embodied CO_2 after UKQAA (2010)

Materials	Embodied CO_2 kg/tonne
Portland cement CEMI	913
GGBS	67
PFA	4
Lime stone	75

2.3. Concrete

Concrete is an important structural material composed of cement, aggregate and water. During the last century concrete technology has gained its importance, due to which researchers are developing methods for making concrete. According to UNEP (2010), the social contribution of concrete to our civilization cannot be overestimated. Concrete is used second to water on a volume consumption basis and most of the infrastructure for modern civilization has been built using concrete in some form or other. Harrison (2003) stated that globally, concrete is used at a rate of over two tonnes per person per year. Concrete construction is robust and durable if mixed correctly, placed and cured. Concrete elements have good insulating and fire resisting properties. Concrete is a major contributor to greenhouse gas emissions and recycling of concrete is increasingly common in structures that have reached the end of their life. Structures made of concrete can have a long service life.

There are different opinions by various researchers in different timeframe for the use of concrete per person per year. According to Sustainable concrete (2011) "The current average worldwide consumption of concrete is about one tonne per year for every living human-being and due to this extensive use, concrete has a relatively large environmental footprint. Worldwide, the cement industry accounts for approximately 5 % of man-made CO_2 emissions. Approximately 40 % of this is from burning coal and 60 % from the calcination of limestone. According to sustainable concrete (2011) cement is one of the main constituents of concrete and is relatively costly to produce in both financial terms and in terms of embodied energy, concrete is a cost-effective material with low embodied energy. In the long-term, concrete's durability, low maintenance and re-usability have very positive economic effects, and concrete structures have optimal energy performance with associated positive effects on whole-life energy usage. Using concrete makes environmental sense. Properties such as thermal mass, fire resistance, water-tightness and economics add to the sustainability of concrete in our built environment and at the end of the usage phase, the concrete material can easily be recycled. Life-cycle cost analyses show that, because of concrete's durability, the whole life cost of many projects is lower when concrete is used as the major construction material."

According to Cemex (2010), the raw materials for cement are limestone and clay and for the production of 1 tonne of Portland cement, 1.5 tonnes of these raw materials are quarried. Limestone and clay are blended and heated together in a rotary kiln at a temperature of 1400 ^{o}C. The heating of the kiln emits CO_2, but more CO_2 is released when limestone ($CaCO_3$) is heated and decomposes to Calcium oxide (CaO).

It is well recognised by Khatib (2009) that concrete has beneficial effects on the energy performance and thermal comfort of a building, due to its high thermalmass. This effect of concrete has been a subject of Biasoli & Oberg (2007), Olssen & Hansen (2007) and Portland cement association (2008), which indicates that concrete is a sustainable material.

Concrete can be produced in-situ or precast. Nowadays ready-mixed concrete is widely used and is available in different proportions and strengths for a variety of jobs. Concrete products and elements can be constructed into any shape and can be cast in various ways including in-situ, precast, etc., which offers flexibility to the designer and architect.

Concrete finishes can be designed in a range of attractive colours and textures. Finish can be put into the concrete at the construction stage, rather than applying a final finish which saves the finishing activities such as painting, tiling, or coating, etc, and saves energy and materials required for finishing. In addition concrete finishes do not emit any toxic or volatile products into the environment and have no negative effects on this during their entire life cycle.

2.4. Sustainable concrete construction

It should be noted that despite the critical importance of all the three components, (environment, economic and social) the environmental component of sustainability is considered important for concrete construction. In concrete, cement is the main constituent and due to the limit on the availability of natural minerals used for making cement and due to the energy released and emission of CO_2 produced in the manufacturing of cement, it can be partially replaced using industrial by-products e.g. PFA, GGBS and silica fume. Processing of these by-products into quality products avoids the need to landfill.

According to Cement and concrete institute (2011), Portland cement is manufactured by heating a mixture of limestone and shale in a kiln to a high temperature (approximately 1500 °C) and then grinding the resulting clinker with gypsum to form a fine powder. The fuel used in the kiln and the electricity in the grinding mills produces CO_2 and so the Portland cement has a high embodied energy. Nowadays concrete is usually manufactured in large batches in a ready-mix concrete plant and transporting the mixture to the construction site in a truck. The process of mixing, moving materials and transportation of concrete requires energy and produces small amounts of waste. In structural applications, concrete is reinforced with steel. In some applications the reinforcing steel is pre-stressed and is manufactured by precasting at a plant. The environmental impact of in-situ concrete is similar to the impact of manufacturing concrete in a ready-mixed concrete plant. The transportation of concrete, pouring it into the formwork, finishing and removal of formwork after it has gained sufficient strength are all low energy operations. At the end of the service life of a concrete structure it is demolished. The demolition process also needs energy. The waste produced by demolition of a concrete structure includes dust, powder, and fragments of concrete. These are typically land filled.

Struble & Godfrey (2004) proved that concrete is the most suitable material for construction having lower impact on the environment compared to steel. For the production of a reinforced concrete beam and steel I beam (both having the same moment capacity) the Environmental impact was compared. A computer programme designed for estimating the environmental impact of the production of these two beams was used and it was proved that the production of a concrete beam required less energy and had lower environmental impact compared to the steel beam of same moment capacity.

According to The Concrete Centre (2010), in 2008, the UK concrete industry agreed a Concrete Industry Sustainable Construction Strategy. The vision of this sustainable concrete forum was that, "By 2012, the UK concrete industry will be recognised as the leader in sustainable construction, by taking a dynamic role in delivering a sustainable built environment in a manner that is profitable, socially responsible and functions within environmental limits".

According to Nicholson (2011), responsible sourcing is referred to supply chain responsibility and is a voluntarily commitment by companies to take into account social and environmental considerations when managing their relationship with their supplier. Codes are used to show that an industry accepts a broader responsibility for its licence to operate beyond profit maximising activities to reduce risk to corporate reputation by improving procurement policy, labour practices and management and environmental impacts.

According to Responsible sourcing (2012) "A responsible sourcing scheme provides qualitative information which identifies and promotes responsible practices throughout the concrete supply chain and address social and environmental impacts of the business. It is based on a set of agreed principles of sustainability, the precise scope of which is determined by stakeholder engagement. Producers can get third-party verification against the requirements of BES 6001 to substantiate their claims for responsible practices".

According to The Concrete Centre (2010) "The sustainable concrete form has achieved the government target on responsible sourcing and 88 % of concrete production is responsibly sourced to BS 6001(1999).This makes concrete the leading construction material for responsible sourcing. The concrete industry supports BS 6001 responsible sourcing standard as currently it is the most comprehensive standard available Environmental Management Systems (EMS) is the cornerstone of a sustainability strategy and a key element in the certification to responsible sourcing schemes. EMS helps to deliver performance improvements at each individual site and is a powerful, rigorous tool for driving the ongoing performance of the industry."In Figure 2.1 the energy efficiency of concrete is shown at various times. Some of the important outcomes of the Concrete Industry Sustainability Performance report No4 (2010) are summarised below.

Energy used as a proportion of production output of concrete increased from 2008 to 2010 by 3 % although it decreased in the first year, but the total energy used by the industry in 2010 has fallen significantly. This is because a fixed energy has been allocated to a lower volume of concrete at production sites due to the continued reduction in production volume over 2009 and 2010.

The CO_2 emissions per tonne of concrete production for rolling concrete mixes have increased by 7 % in 2010 compared to 2009 but are lower than the 1990 base line. This increase is the result of higher average cement contents in the rolling concrete mix. The CO_2 emissions for the production of rolling concrete mixes for three years are given in Figure 2.2.

The CO_2 emissions per tonne of concrete for a standardised concrete mix are reduced by 16.3 % compared to the 1990 baseline and are on track to meet the 2012 target and are given in Figure 2.3. For a standardised concrete mix, the CO_2 emissions have not changed significantly between 2009 and 2010 and are 16.3 % under the 1990 baseline in 2010.

The 'rolling mix' is calculated on the CO_2 emissions from the annual reported use of the concrete components. The standardised mix is calculated from the annual concrete production using the ratio of components for the average concrete established in 2008. The 'rolling mix' indicates the net actual changes in CO_2 emissions which is a combination of production efficiencies and changes of product mix in the market. The standardised mix' indicates improvements in production efficiency on an annual basis.

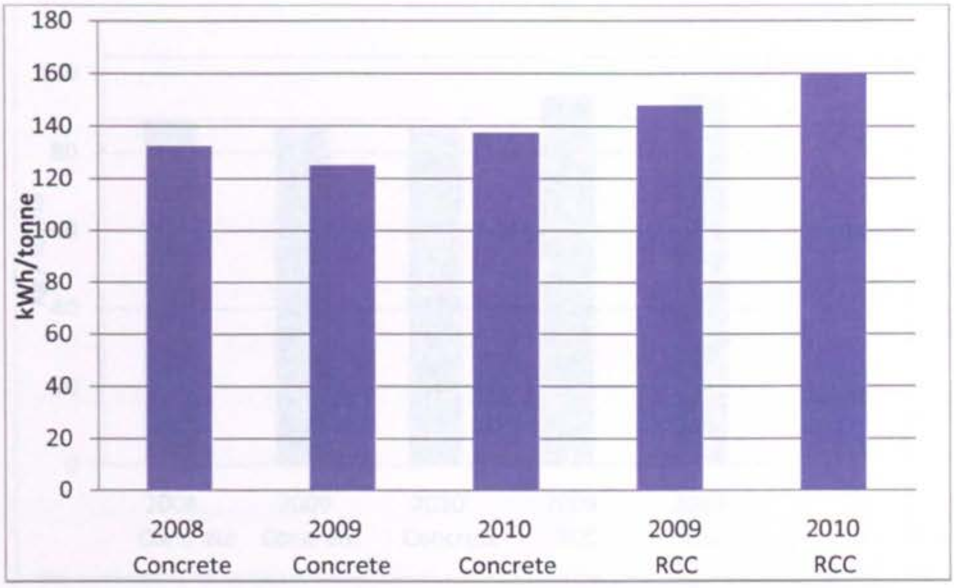

Figure 2-1 Energy efficiency of concrete After (The Concrete Centre 2010)

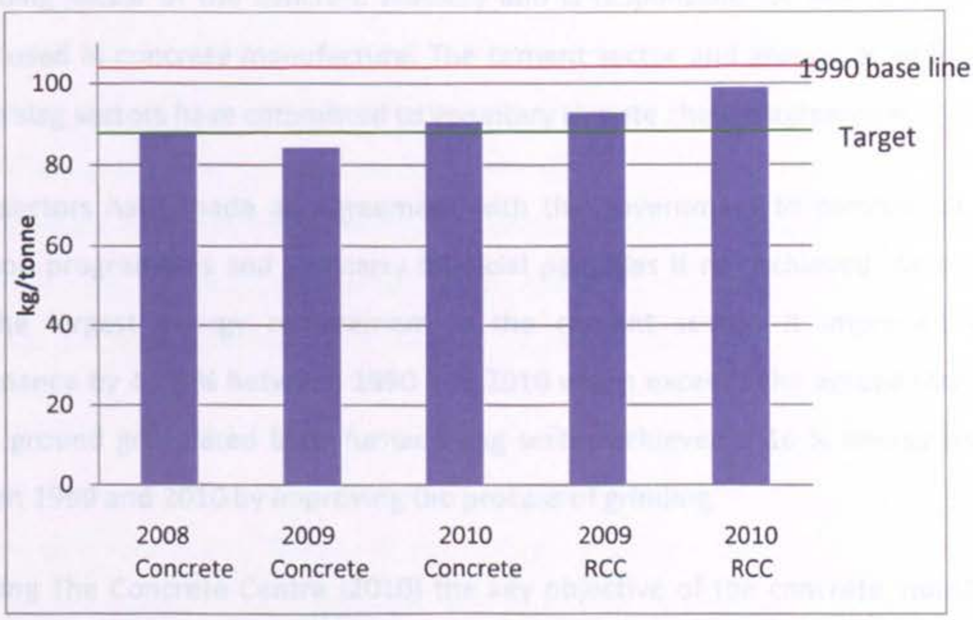

Figure 2-2 CO_2 emissions for production of concrete After (The Concrete Centre 2010)

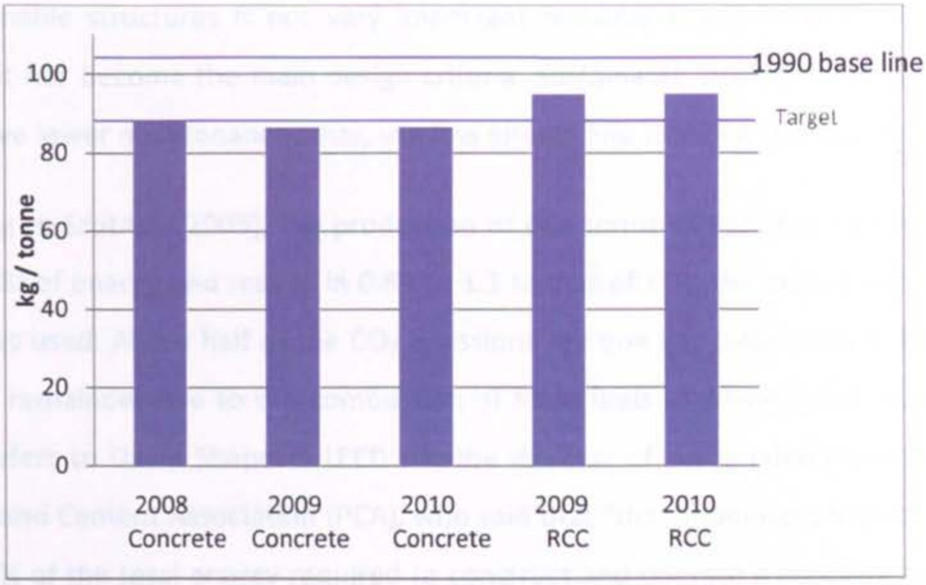

Figure 2-3 CO_2 emissions for standardised concrete mix (CISPR 2010)

According to The Concrete Centre (2010) Portland cement is the largest energy demanding sector of the concrete industry and is responsible for almost 74 % of the energy used in concrete manufacture. The cement sector and ground granulated blast furnace slag sectors have committed to voluntary climate change agreements (CCA).

These sectors have made an agreement with the government to commit for energy reduction programmes and will carry financial penalties if not achieved. As the sector with the largest energy requirement is the cement sector, it improved its CCA performance by 44.8 % between 1990 and 2010 which exceeds the agreed target of 30 %. The ground granulated blast furnace slag sector achieved a 16 % energy reduction between 1999 and 2010 by improving the process of grinding.

According The Concrete Centre (2010) the key objective of the concrete industry is to improve the energy efficiency to reduce the embodied energy of its products. Concrete products play a significant role in optimising the operational energy efficiency over the entire lifetime of a building. The average distance for the delivery of ready mix concrete to the construction site in 2010 was 10.5 km and 119 km for precast concrete products.

The average delivery distance for all concrete is 36 km. Common thinking is that the idea of sustainable structures is not very important because it will increase the cost but actually it has become the main design criteria. Sustainable structures can cost less to build, have lower maintenance costs, use less energy and produce less CO_2 emissions.

According to ScotAsh (2005), the production of one tonne of Portland cement requires about 4 GJ of energy and results in 0.89 to 1.1 tonnes of CO_2, depending upon the type of process used. About half of the CO_2 emissions are due the calcination of lime-stone, with the remainder due to the combustion of fossil fuels in energy production. Nasvik (2009) refers to David Sheperd, LEED AP, the director of sustainable development for the Portland Cement Association (PCA), who said that "the embodied concrete energy is about 3 % of the total energy required to construct and operate a concrete house with more than a 100-year expected life. For commercial buildings, it is 5 % to 15 %. Sustainable building causes will push the owners of the building to think long-term, rather than worrying about the construction cost."

Novak (2009) further comments that concrete is the most adaptable of all construction materials. It can be adapted to meet a large number of specific needs. The cost of concrete in a structure can be reduced by specifying higher performance mixes to reduce the size of building elements. Sustainability relates to the life-cycle of a structure which depends on concrete durability. Replacing a building because of fire, extreme wind events, blast, or seismic events is costly and by considering robustness, engineers can design reinforced concrete structures to resist extreme events.

Desai (1998) pointed out that the concrete industry should be economical, efficient and high tech and it should seek to make optimum use of materials and minerals. Durability is the key to sustainability. The life of a structure can be extended by using concrete and is a great achievement for sustainability. This is the direction for the next era.

Anonymous (2001) says that proper environmental assessments are required to reduce the environmental impacts of buildings and structures. The construction industry is playing its part in minimising the impact of concrete, as a primary construction material, on the environment.

2.5. Use of binary cements in concrete

According to ScotAsh (2005), Portland cement is the main constituent of concrete and the production of one tonne of it requires about 4 GJ of energy and results in 0.89 to 1.1 tonnes of CO_2, depending upon the type of process used. About half of the CO_2 emissions are due the calcination of limestone, with the remainder due to the combustion of fossil fuels by energy producers. Due to the high demand of energy required and the release of embodied CO_2 in the production of Portland cement the research is focussed on binders, where Portland cement in concrete is partially replaced using industrial by-products e.g. pulverised Fuel Ash (PFA), Ground Granulated Blast furnace Slag (GGBS) as cementitious constituent.

According to Higgins (2006) UK uses about two million tonnes per year of GGBS as cement and in UK 500,000 tonnes of Fly ash is used as cement addition per year and uses 100,000 tonnes of Fly ash as a component of blended cement.

Fib (2004) estimated that the production of 1 m^3 ready-mix concrete requires about 100 MJ of energy on average and sometimes this may be doubled, depending on the mix proportions.

It is well recognised by The Concrete Centre (2007) that the hydration and strength of concrete is reduced if cured at lower temperatures than 20 ^{o}C and concrete with GGBS and PFA is more sensitive to these. It is beneficial to maintain fresh concrete at higher temperatures, typically at 10 ^{o}C or more in cold weather for curing so that it can gain enough strength.

2.6. Ground Granulated Blast furnace Slag (GGBS)

According to Production and use of GGBS (2007), GGBS is a by-product obtained during the manufacture of iron in the blast furnace. GGBS is economically available in large quantities, requiring storage facilities and therefore, it is suitable for use in ready-mix concrete, in the production of large quantities of site batched concrete and in precast product manufacturing. Blast furnaces are fed carefully with controlled mixtures of iron-ore, coke and limestone, at a temperature of about 2000 ^{o}C. The iron ore is reduced to iron and sinks to the bottom of the furnace. The remaining material that floats on top is the slag. The slag is rapidly quenched in large volumes of water. This process of quenching optimises the cementitious properties and produces granules similar to coarse sand particles. The granulated slag is dried and ground to a fine powder that is called GGBS. It is off-white in colour and has a bulk density of 1200 kg/m^3.

According to Jones (2011) the first commercial blended blast furnace slag cement was produced in Germany in 1865 and currently over 200 million tonnes per annum of blast furnace slag cement is used around the world.

2.6.1. Standards and Specifications

According to BS EN 197-1(2011) "Granulated blast furnace slag is made by rapid cooling of a slag melt at suitable composition, as obtained smelting iron ore in a blast furnace and contains at least two-thirds by mass of glassy slag and possesses hydraulic properties when suitably activated. Granulated furnace slag shall consist of at least two-thirds by mass of the sum of calcium oxide (CaO), magnesium oxide (MgO) and silicon dioxide (SiO_2). The remainder contains aluminium oxide (Al_2O_3), together with small amounts of other compounds. The ratio by mass $(CaO+MgO)/SiO_2$ shall exceed one. GGBS is obtained by finely grinding granulated blast furnace slag."

According to BS EN 197-1(2011), different types and composition of blast furnace slag cement are presented in Table 2.2.

Table 2-2 Types and composition (% by mass) of Blast furnace slag cement after BSEN 197-1(2011)

Designation	Notation	Clinker	Blast furnace slag	Minor additional constituents
Blast Furnace Slag	CEMIII/A	35-64	36-65	0.5
	CEMIII/B	20-34	66-80	0.5
	CEMIII/C	5-19	81-95	0.5

According to BS EN 15167-1(2006), the chemical requirements of GGBS shall conform to those given in Table 2.3. According to clause 5.3.1 the fineness of GGBS shall not be less than 275 m^2/kg. These chemical requirements of GGBS are very important for it to be used in concrete.

Table 2-3 Chemical requirements of GGBS as characteristic values after (Table 1 BS EN 15167-1:2006)

Property	Test reference	Requirements[a]
Magnesium Oxide	EN196-2	≤ 18%
Sulphide	EN196-2	≤2.0%
Sulphate	EN196-2	≤2.5%
Loss on ignition, corrected for oxidation of sulphide	EN196-2	≤3%
Chloride	EN196-2	≤0.1%
Moisture Content	EN196-2	≤1.0%

[a] Requirements are given by mass of ground granulated blast furnace slag

2.6.2. Properties of GGBS concrete

2.6.2.1. Chemical Properties

According to Hanson (2010), for a typical GGBS produced in the UK, the chemical constituents are given in Table 2.4. According to the Slag Cement Association, SCA (2003), concrete made with slag cement has higher long age compressive and Flexure strengths compared to PC concrete and varies for different curing conditions, mix proportions and age of testing. When PC reacts with water it forms calcium silicate hydrate (CSH) and calcium hydroxide $(CaOH)_2$. CSH is a glue that provide strength to the concrete and holds it while $Ca(OH)_2$ is a by- product and does not contribute to the strength of concrete. When slag is used as part of the cementitious constituent in concrete, it reacts with water and Ca $(OH)_2$ to form more CSH gel and increases the strength.

Table 2-4 Typical constituents of GGBS after Hanson (2012)

Constituents	Percentage in GGBS
Calcium oxide (CaO)	40%
Silica (SiO2)	35%
Alumina (Al2O3)	16%
Magnesia (MgO)	6%
Other - Fe2O3, etc.	3%

Oner and Akyuz (2007) found that the compressive strength of concrete mixtures containing GGBS is increased as the level of GGBS is increased but after an optimum point, which is around 55 % of the total binder content, further addition of GGBS did not improve the compressive strength of concrete. The strength gain is slow in concrete containing GGBS because the pozzolanic reaction is slow and depends on the calcium hydroxide availability.

According to MPA (2012), the hydration mechanism of a combination of GGBS and PC involves the activation of the GGBS by alkalis and sulfates to form its own hydration products. Some of these combine with the PC products to form further hydrates which have a pore blocking effect. The result is a hardened cement paste, but the rate of strength development is slower.

2.6.2.2. Fresh Properties of GGBS

Hooton (2000) found that the slump of GGBS concrete is unaffected compared to PC concrete but slag concrete is much easier to compact by vibration and is therefore considered to be more workable. Due to the improved workability of slag concrete, the entrapped air content is lowered. The GGBS concretes are easy to finish because of the higher fines content but at higher replacement levels and ambient temperatures (< 15 ^{o}C) setting times can be extended up to one or two hours. GGBS concrete with higher replacement (50 % and above) or placed at lower temperature needs extra curing, if bleeding and finishing times are extended significantly.

Hooton (2000) concluded that the setting time of GGBS mixtures can be extended up to one or two hours at high replacement levels and low ambient temperatures (<15 ^{o}C). In hot weather at a temperature above 20 ^{o}C, the finishing time can be extended by only a few minutes. For all concretes curing against the loss of moisture is essential and for GGBS concrete placed at low temperature, extra curing is required.

GGBS has to be handled very carefully. GGBS and water solution is highly alkaline and can damage the skin severely. Activation of GGBS by alkalis and sulfates results in hydration products which combine with the Portland cement hydration products to form further hydrates that have the pore-blocking effect.

2.6.2.3. Hardened Properties of GGBS

Dhir et al (2005), indicated that the influence of the concrete on compressive strength is in proportion to the effect on other engineering properties and are in line with the current design assumptions for concrete. It was also found that the Eurocode 2 equations for predicting the shear strength of reinforced concrete beams, based on compressive strength are appropriate for the range of concrete mixes considered which included PC/PFA and PC/GGBS mixes. It was suggested that there is no need for further reviewing the design procedures relating to flexure or shear for using concrete mixes containing GGBS and PFA with regard to the relationship between compressive strength and other engineering properties. It was concluded that the developing concrete technology solutions that improve the concrete durability and achieve sustainable concrete construction, can be used effectively within the framework of present design procedures. In their research, it was shown that the compressive strength in the equation for calculating the shear strength of reinforced concrete beams given in BS EN 1992-1-1 is valid for all types of concrete and constituents including GGBS and PFA.

In Figure 2.4 the comparison of compressive strength of concrete containing slag at 7 and 28 days with PC concrete is shown by the Slag cement association SCA (2003). In

Figure 2.5 the comparison of flexural strength for Slag cement concrete and PC concrete has been presented.

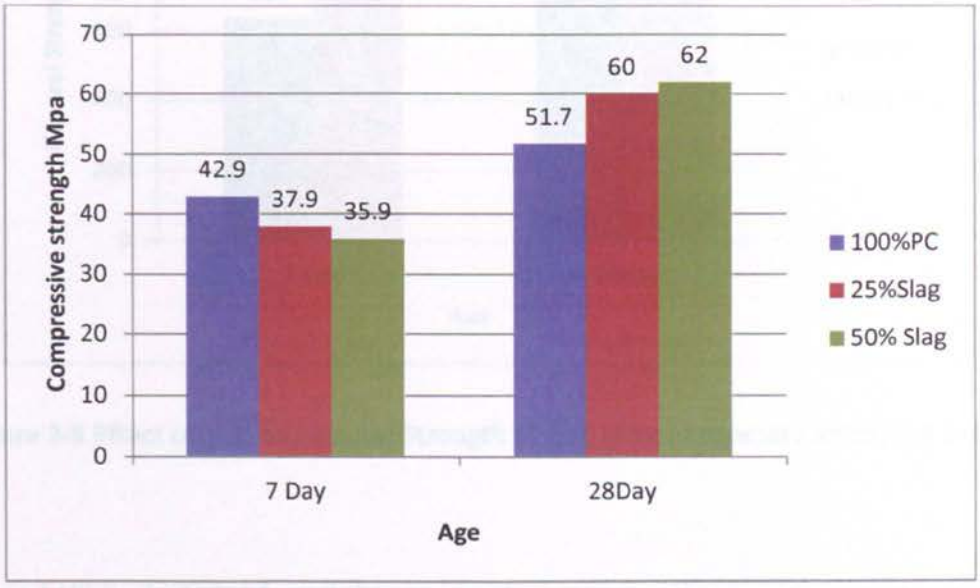

Figure 2-4 Effect of Slag on Compressive strength of slag cement concrete after (SCA 2003)

Johari et al (2011) found that at the age of one day, the relative strength of GGBS with 20 %, 40 % and 60 % replacement was 72 %, 45 % and 4.6 % respectively of the Portland cement concrete. The lower early age strength was due to the slower reactivity of GGBS and due to its dilusion effect.

Figure 2-5 Effect of Slag on Flexural Strength of slag cement concrete after (SCA 2003)

Khatib and Hibbert (2005) found that the early age strength gain of concrete containing GGBS decreases with increasing percentage of GGBS in concrete but the strength between 28 days and up to 90 days increased compared to the PC concrete. The replacement level of GGBS up to 60 % was beneficial and beyond that the strengths were very low. The comparison of compressive strength at different ages between GGBS and the PC concretes, by Khatib and Hibbert (2005) is given in Figure 2.6. Concrete was casted for equal water/binder ratio of 0.5 and the specimens were cured in water at 20 ^{o}C. It was concluded that 60% GGBS replacement of PC in concrete causes an increase in compressive strength.

The 90 days flexural strength values of different mixes prepared by Khatib and Hibbert (2005) are given in the Figure 2.7. The flexural strength of concrete containing 60 % GGBS was higher than the control mix. The flexural strength of concrete containing 40 % GGBS was reduced while that 80 % GGBS reduced considerably.

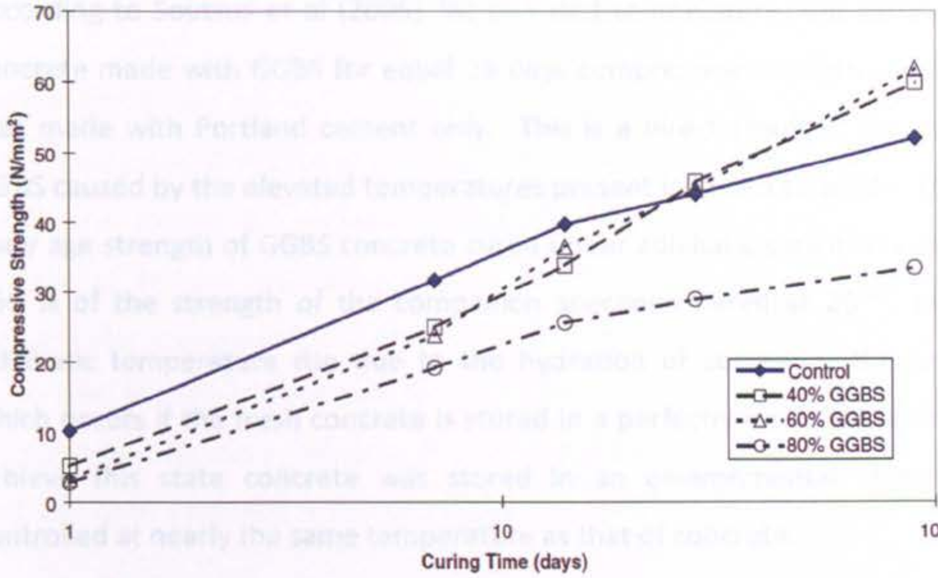

Figure 2-6 Effect of GGBS on Strength Development (Khatib & Hibbert 2005)

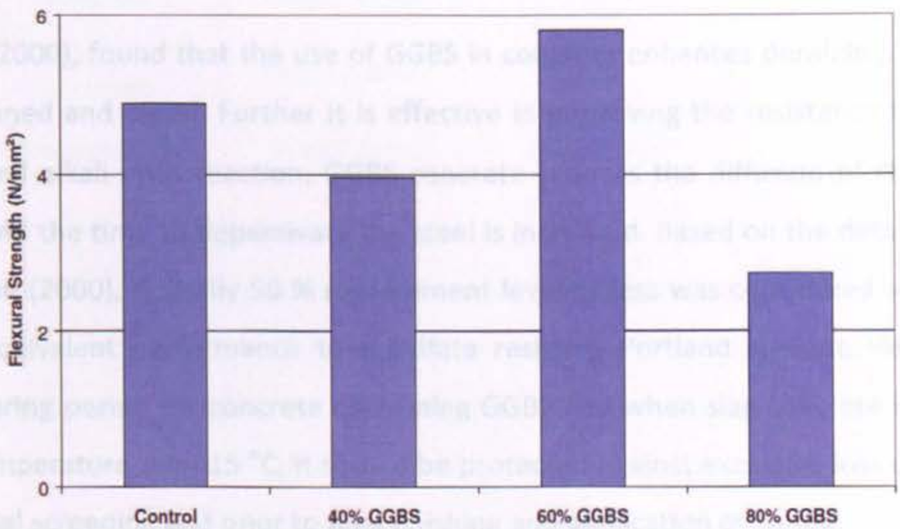

Figure 2-7 Effect of GGBS on flexural strength(Khatib & Hibbert 2005)

According to Soutsos et al (2005) "At elevated temperature, the early age strength of concrete made with GGBS for equal 28 days compressive strength can be greater than that made with Portland cement only. This is a direct result of the activation of the GGBS caused by the elevated temperatures present in in-situ concrete. They proved that early age strength of GGBS concrete cured under adiabatic conditions can be as high as 250 % of the strength of the companion specimen cured at 20 °C in a water tank. Adiabatic temperature rise due to the hydration of cement is the temperature rise which occurs if the fresh concrete is stored in a perfectly insulated environment and to achieve this state concrete was stored in an environmental chamber which was controlled at nearly the same temperature as that of concrete.

Chu (2007) concluded that from a structural point of view, GGBS replacement reduces heat of hydration, enhances durability, including higher resistance to sulfate and chloride attack, when compared with normal concrete. On the other hand, it also contributes to environmental protection because it minimizes the use of cement during the production of concrete.

Hooton (2000), found that the use of GGBS in concrete enhances durability, if properly proportioned and cured. Further it is effective in improving the resistance to chloride, sulfate and alkali-silica reaction. GGBS concrete reduces the diffusion of chloride and oxygen and the time to depassivate the steel is increased. Based on the data developed by Hooton (2000), typically 50 % replacement level or less was considered sufficient to obtain equivalent performance to a sulfate resisting Portland cement. He proposed longer curing period for concrete containing GGBS and when slag concrete is placed at lower temperature than 15 °C, it should be protected against excessive loss of moisture after initial screeding and prior to final finishing and application of curing.

Clear (1995) says that the form striking time is not increased if the replacement of GGBS in concrete is limited to 50 %. The principle of equivalent age was applied to estimate the early age in-situ strength of concrete containing up to 70 % GGBS. Early age is equivalent to 7 days which is achieved in reality between 4 and 6.5 days for 70 % GGBS concrete cast in 1m thick sections under cold winter conditions and is achieved in an equivalent of 3 days by PC concrete.

2.6.2.4. Environmental effect of GGBS concrete

Higgins (2006) has shown the environmental profile for the production of one tonne of GGBS, compared with typical values of PC presented in Table 2.5. For the production of GGBS, the impact for processing the granulated slag to produce GGBS has been considered and no impact has been taken into account for the production of iron because slag is produced as a by-product in the production of iron and if not utilised will go to land fill. The replacement level and the need for extra cementitious content are the important factors in selecting the most sustainable material for concrete production. GGBS is highly cementitious and can usually replace Portland cement by 50 % or more.

Table 2-5 Environmental Burden for the manufacture of GGBS after Higgins (2006)

Source	Measured as	Impact	
		Manufacture of 1 tonne of GGBS	Manufacture of 1 tonne of PC
Climate change	CO_2 equivalent	0.05 tonne	0.95 tonne
Energy use	Primary energy	1300MJ	5000MJ
Mineral extraction	Weight quarried	0	1.5 tonnes
Waste disposal	Weight to tip	1 tonne saved	0.02 tonnes

In Table 2.1, the value of embodied CO_2 for GGBS is 67 kg/tonne which is slightly more than the value of 0.05 tonne/tonne given by Higgins (2006) in Table 2.5. Similarly the embodied CO_2 value for PC is 913 kg/tonne production in Table 2.1, which is slightly less than 0.95 tonne/tonne, the value given by Higgins (2006) in Table 2.5. Improvement in design and efficiency of PC production is reducing its environmental burden.

The environmental impacts benefits of using GGBS and PFA in concrete studied by the UK Concrete Industry Alliance project were tabulated by Higgins (2006) and are given in Table 2.6. The environmental impacts are per tonne production of a C30 concrete. As shown in Table 2.6, replacement of 50 % Portland cement with GGBS saves 40 % CO_2 emissions in concrete. It has a negligible effect on mineral extraction which is 8 %. GGBS and PFA are widely available in the UK and transportation distance between the point of production and the point of use is comparable with those of Portland cement. Higgins (2006) concluded that in 2005 the use of GGBS and PFA saved the UK 2.5 million tonnes of carbon dioxide emissions, 2 million megawatt hours of energy, 4 million tonnes of mineral extraction and 2.5 million tonnes of material sent to landfill. The CO_2 emissions are compared in Figure 2.8.

Table 2-6 Calculated environmental impacts for 1 tonne of concrete After Higgins (2006)

Impact	100% PC	50% GGBS	30%PFA
Greenhouse gas (CO_2)	142kg (100%)	85.4 kg (60%)	118kg (83 %)
Primary energy use	1070 MJ (100%)	760 MJ (71 %)	925 MJ(86%)
Mineral Extraction	1048kg (100%)	965 kg (92 %)	1007kg (96%)

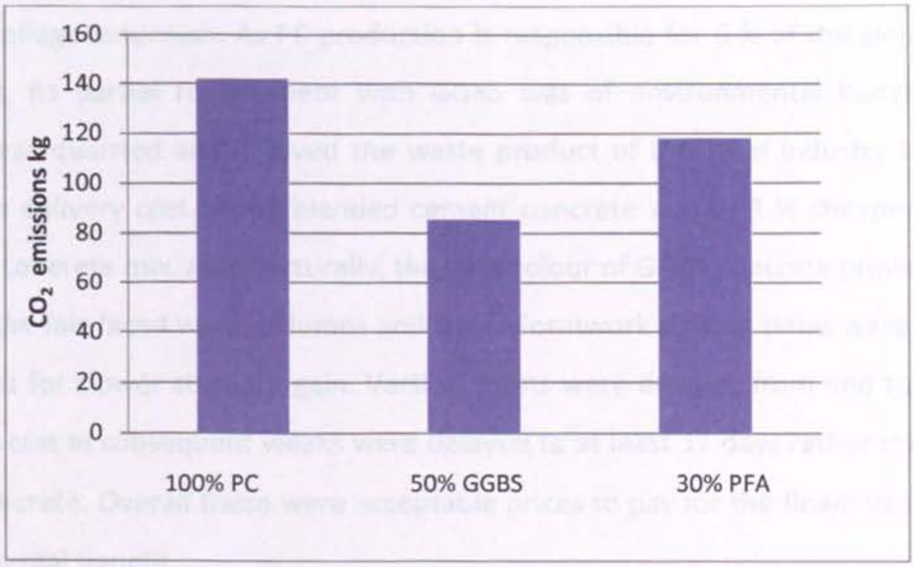

Figure 2-8 CO_2 Emissions After Higgins (2006)

According to Chen (2005) the data published by the Building Material Research Centre of the Aachen University of Technology in Germany, using industrial by products in cement can result in significant savings in energy and reductions in CO_2 emissions. By using 60 % blast furnace slag in blended cement, reductions in energy consumption of around 43 % and in CO_2 emissions of about 50 % in the production of 1 m^3 of concrete of strength class C25/30 can be achieved (Taking account of the transportation of the aggregate over a distance of 40 km and cement over 80 km).

According to Jasen et al (2006), over 800 m^3 of concrete with 70 % GGBS was poured in the raft foundation of Milharbour in London Docklands. Milharbour is the Europe's tallest residential development with over 700 apartments located in two interlinking towers rising up to 50 storeys. The concrete Industry Alliance within a DETR-supported project (1999) calculated the environmental impact of GGBS and found 50 % reductions in green house gases for the Milharbour project by using 70 % GGBS. It has been determined that with concrete volume of over 800 m^3, reductions in green house gases in this project were equal to around 60,000 journeys around the M25 in an average family diesel saloon car.

According to Thomas (2009) 35 % GGBS was used in the flat slab structure of West Thames college extension. As PC production is responsible for 6 % of the global carbon emissions, its partial replacement with GGBS was of environmental benefit as less cement was quarried and it saved the waste product of the steel industry to be land filled. The delivery cost of the blended cement concrete was 2- 3 % cheaper than the standard concrete mix. Architecturally, the light colour of GGBS concrete provided a nice finish to the fair-faced walls, columns and slabs. Formwork striking times were extended to account for slower strength gain. Vertical forms were delayed from one to two days and slabs cast in subsequent weeks were delayed to at least 11 days rather than a week for PC concrete. Overall these were acceptable prices to pay for the financial saving and environmental benefit.

According to Parker (2012), the Shard is the tallest building in the European Union. The Shard is 310 m high and has 95 floors including plant floors with 72 habitable floors. The facade of the Shard is made of angled glass panes which reflect the sun light so that the appearance of the building will change according to the weather. The Shard is an unusual mixture of concrete and steel, and has a concrete basement. There is a structural steel from ground to level 40, concrete from levels 41 to 69, and steel again from there to the top at level 95. Stability of the structure is achieved by a massive concrete core that is placed in the middle of the building. In the Shard, 75 % GGBS was used in the base slab. GGBS was used not only to reduce the propensity for early-age cracking but also to reduce embodied CO_2. An innovative approach was used on this project to allow construction above and below ground to start simultaneously. High replacement of cement with GGBS has the potential disadvantage of low early age strength so the concrete was developed that it could achieve sufficient strength gain to meet initial structural requirements within 14 days, with the full strength being achieved at 56 days. According to the Concrete Centre, the core had already reached 21 storeys high by the time that 700 truckloads of concrete were poured into the basement to form the 3 m deep raft foundation upon which the tower had to sit.

Hanson cement (2010) considered that the production of 100 m^3 concrete used 32 tonnes of cement. Replacing 50 % cement with GGBS saves 12.96 tonnes of CO_2 and this is equal to taking 42 cars off the road for one year or equal to 41 years of electricity usage in the average home. A comparison of the CO_2 emissions of Portland cement and Regen (GGBS) is given in Figure 2.9.

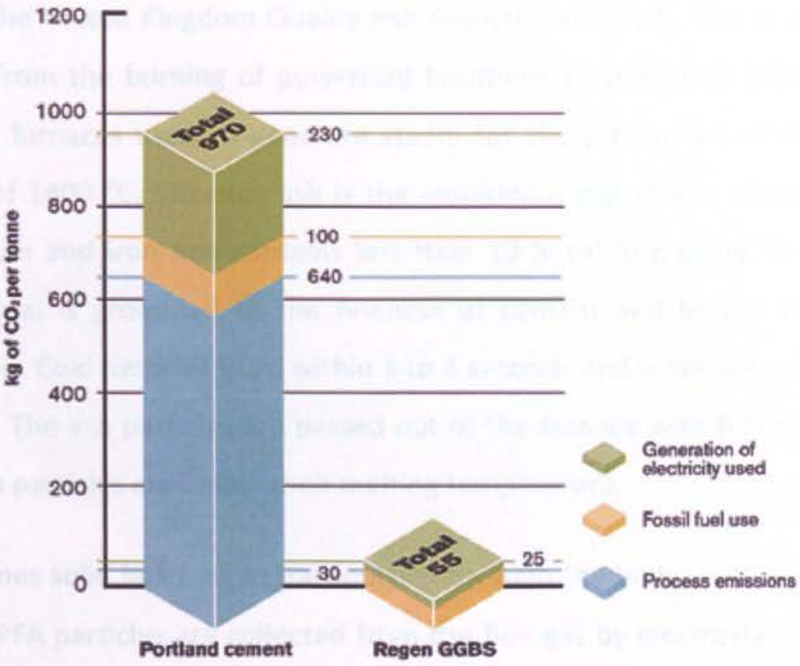

Figure 2-9 Typical CO_2 emissions for Portland cement and GGBS (Hanson 2010)

2.7. Pulverised Fuel Ash (PFA)

According to the information given in the United Kingdom Quality Ash Association (UKQAA) (2004), PFA is a by-product obtained at power stations and is a solid material extracted by electrostatic and mechanical means from flue gases of furnaces fired with pulverised bituminous coal. It is carried by the exhaust gases and recovered as fly ash with fine particles.

2.7.1. Production & Applications

PFA has been used in the UK since 1950 for various applications. According to Thomas (2010) the use of fly ash as supplementary cementing material in concrete has been known from the start of last century but the first research in fly ash was conducted at the university of California by Davis et al (1937) and the first significant utilization of fly ash in concrete began with the construction of the Hungry Horse Dam in Montana in 1948. The production of the material has been changed to reduce the gaseous emissions in recent years but has not affected the nature of PFA except it has increased the loss on ignition (LOI).

According to the United Kingdom Quality Ash Association (2002), PFA is defined as the ash resulting from the burning of pulverised bituminous, hard coals (80 % Carbon) in power station furnaces used to generate steam for the production of electricity at a temperature of 1400 ^{o}C. Siliceous ash is the resulting material and contains oxides of silica, aluminium and iron and contains less than 10 % calcium oxide. In modern coal fired boilers coal is grounded to the fineness of cement and blown into the boiler furnace with air. Coal particles burn within 3 to 4 seconds and leave the ash as spherical molten beads. The ash particles are passed out of the furnace with flue gas and cooled so that the ash particles are below their melting temperature.

The ash becomes solid bead while transporting through the boiler with the combustion gases. Finally PFA particles are collected from the flue gas by electrostatic precipitators and are held in hoppers beneath the precipitators. PFA can be used as a dry product for direct use in cement and concrete or it can be conditioned by mixing with approximately 18 % water. PFA may be transported as slurry to lagoons for storage. Normally PFA is 85 % of ash production and the remaining 15 % is called Furnace bottom Ash (FBA).

According to Sear (2011) in "Future trends for PFA in cementitious systems" the ash production in the UK is falling overall. The UK produced about 5,300,000 tonnes of PFA which is fairly consistent for a number of years but has significantly reduced now. The Lafarge Combustion Plant Directive (LCPD) means that a number of stations are due to close by 2015. The coal fired power station products sold in the UK in 2009 are shown in Figure 2.10 by Sear (2011).

PFA can be used in a vast range of structural and civil applications. PFA is used for grouting and stabilising mine works, in sub- base of road construction, as a binder, as an aggregate in concrete and in the pre cast block production. In Table 2.7 the typical uses of PFA annual production in the UK are presented by Sear (2011). The data used in Figure 2.10 and Table 2.7 are from the same source. PFA is used in a wide range of applications in the UK which includes the pavement concrete for taxi ways of airplanes at Heathrow Airport terminal 5 London, Channel Tunnel Rail Link (CTRL), Canary Wharf, M6 Toll road and various fish and wind farms.

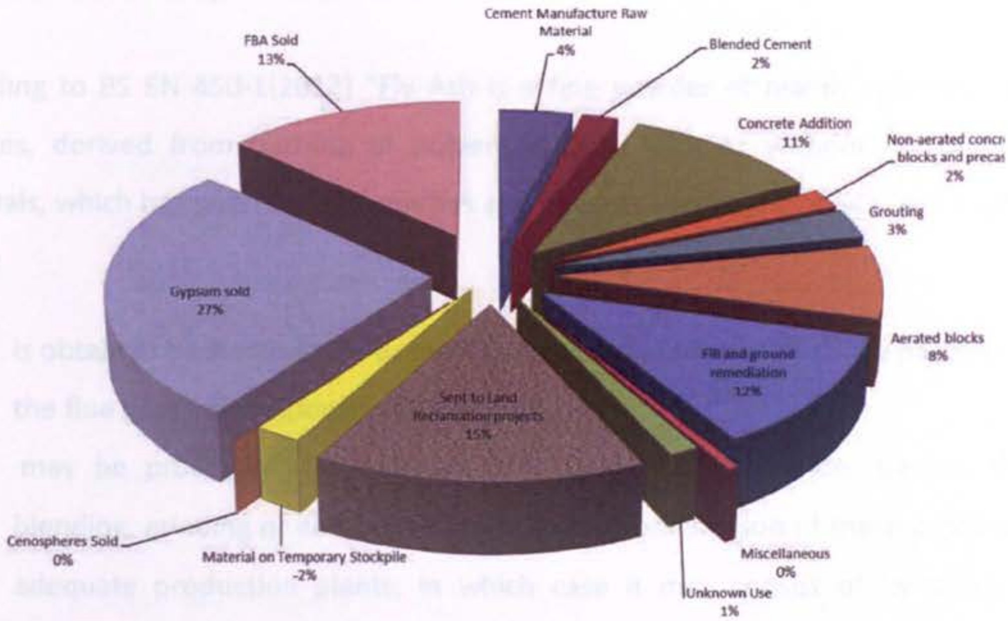

Figure 2-10 Coal fired power stations products sold in the UK during 2009 Sear (2011)

Table 2-7 Applications for PFA annual PFA production for UK after Sear (2011)

Application for PFA annual PFA production	**Tonnes**
Cement raw material	200,000
Blended cement	100,000
Non aerated concrete blocks and precast	100,000
Grouting	150,000
Aerated blocks	400,000
Fill, ground remediation and other uses	650,000
Sent to land reclamation projects	750,000
Land filled material	1,600,000
Total	4.400,000

2.7.2. Standards & Specifications for PFA

According to BS EN 450-1(2012) "Fly Ash is a fine powder of mainly spherical, glassy particles, derived from burning of pulverised coal, with or without co-combustion materials, which has pozzolanic properties and consists essentially of SiO_2 and Al_2O_3 and which:

- is obtained by electrostatic or mechanical precipitation of dust-like particles from the flue gases of the power stations; and
- may be processed, for example by classification, selection, sieving, drying, blending, grinding or carbon reduction, or by combination of these processes, in adequate production plants, in which case it may consist of fly ashes from different sources, each conforming to the definition given in this clause."

According to BS EN 197-1(2011) different types and composition of Fly Ash are presented in Table 2.8.

Table 2-8 Types and composition (% by mass) of fly ash after BSEN 197-1(2011)

Designation	Notation	Clinker	Fly Ash	Minor additional constituents
Fly Ash	CEMIV/A	65-89	11-35	0.5
	CEMIV/B	45-64	36-55	0.5

According to BS EN 450-1(2012) fly Ash shall have the following chemical properties given in Table 2.9. The values given in the Table 2.9 are the optimum values and beyond these limits, the performance of concrete will be affected.

Table 2-9 Chemical requirements of Fly Ash as Characteristic values

Property	Test reference	Requirements[a]
Magnesium Oxide	EN196-2	≤ 4.0%
Phosphate (P_2O_5)	ISO 29581-2	≤5.0%
Sulphate (SO_3)	EN196-2	≤3.0%
Loss on ignition for Cat A	EN196-2	≤5.0%
Chloride	EN196-2	≤0.10%
Free Calcium Oxide	EN 451-1	≤1.5%
Reactive Calcium Oxide	EN 197-1:2011	≤10.0%
Reactive Silicon Dioxide	EN 197-1	≤25.0%

[a] Requirements are given by mass of Fly Ash

2.7.3. Properties of PFA concrete

2.7.3.1. Chemical properties of PFA

According to Ash utilisation division (2007), "Portland Cement (PC) is a product of four principal mineralogical phases. These phases are Tricalcium Silicate- C_3S ($3CaO.SiO_2$), Dicalcium Silicate- C_2S ($2CaO.SiO_2$), Tricalcium Aluminate- C_3A ($3CaO.Al_2O_3$) and Tetracalcium alumino-ferrite- C_4AF ($4CaO.\ Al_2O_3.\ Fe_2O_3$). The setting and hardening of the PC takes place as a result of reaction between these principal compounds and water as follows".

$$2\,C_3S + 6\,H \longrightarrow C_3S_2H_3 + 3\,CH$$

$$2\,C_2S + 4\,H \longrightarrow C_3S_2H_3 + CH$$

The hydration products from C_3S and C_2S are similar but quantity of calcium hydroxide produced is different.

The reaction of C_3A with water takes place in the presence of sulfate ions supplied by the dissolution of gypsum present in the PC. This reaction is very fast and is shown as follows.

$$C_3A + 3\,(CSH_2) + 26\,H \longrightarrow C_3A(CS)_3H_{32}$$

$$C_3A + CSH_2 + 10\,H \longrightarrow C_3A\,CSH_{12}$$

These reactions indicate that during the hydration process of cement, lime is released and remains as surplus in the hydrated cement. This surplus lime makes the concrete porous and gives chance to the development of micro-cracks which weakens the bond with aggregates and thus affects the durability of concrete.

If fly ash is available in the mix, this surplus lime becomes the source for pozzolanic reaction with fly ash and forms additional C-S-H gel having similar binding properties in the concrete as those produced by hydration of cement paste. The reaction of fly ash with surplus lime continues as long as lime is present in the pores of liquid cement paste. Pozzolanic reaction is given as follows by Dunstan (2011).

$$3\,Ca(OH)_2 + 2\,SiO_2 \longrightarrow 3\,CaO\,2SiO_2\,3\,H_2O$$

According to the United Kingdom Quality Ash Association (2004), PFA has both pozzolanic and physical properties that enhance the performance of concrete. When Portland cement hydrates it produces alkali calcium hydroxide (lime). PFA reacts with this lime to form more stable calcium silicate and aluminium hydrates which fill the voids in the concrete. This process improves strength, durability, chloride resistance and sulfate resistance of concrete.

2.7.3.2. Fresh Properties of PFA

It is widely recognised that PFA in concrete enhances the fresh and hardened characteristics. The most important physical property of PFA is the fineness and the chemical property is the Carbon Content. According to Joseph & Ramamurthy (2009), fresh density of concrete is reduced with increase in replacement level of cement by fly Ash and its workability is increased.

According to Ash utilisation division (2007) "Fly Ash particles are usually spherical in shape and reduce the water demand for a given value of slump. The spherical shape of fly ash reduces the friction between the aggregates and between concrete and the pump line and thus increase the workability and pumpability of concrete. Fly ash in concrete increases the fine volume and decreases the water content and so reduces the bleeding of concrete".

2.7.3.3. Hardened Properties of PFA

Dhir et al (1984) found that the there is a close correspondence between the strength of PC concrete and PC/PFA concrete at the age of 28 days. They designed the concrete for equal workability and 28-day strength 15-60 N/mm^2. PFA concrete was designed using known cementing efficiency factor and all the specimens were cured under standard curing conditions. Early age strength of PFA concrete is slower than PC concrete because the pozolanic reactions are slower than the hydration reactions and they start after about five days. At low water curing temperature of 5 $^{\circ}$C the strength of PC concrete and PC/PFA concrete are very similar because the lower temperature are not able to slow down the pozzolanic reactions more than those of hydration reactions. At ages between 28 to 180 days, at low temperature there is a decrease in strength of PC/PFA concrete. At 20 $^{\circ}$C curing condition there is a significant increase in compressive strength of PC/PFA concrete at 365 days strength compared to PC concrete. For standard curing temperature of 20 $^{\circ}$C there was a close correspondence between the strength of PC/PFA concrete and the PC concrete. The concept of equal strength for equal maturity irrespective of the route to maturity for PC concrete does not apply to PC/PFA concrete and is dependent on the curing temperature.

Dhir et al (1998) found that PFA fineness affects the strength of concrete and the strength of PFA concrete is reduced by using coarser PFA. In order to take care of the effect of PFA fineness on strength they developed a simple procedure of varying the water content, cement content or both. The acceptable variation of PFA fineness in BS-EN 450 (2012) has an effect on the concrete production and it can be adjusted by modifying the margin between the characteristic and mean strength. It was suggested that the 'k' factor approach to mix design is not suitable for BS EN 450 PFA and simple adjustment to the mix proportions is appropriate. For durability tests, which include chloride diffusion, carbonation, sulfate Resistance, alkali silica reactivity, freeze thaw and abrasion similar performance was achieved by BS-EN 450 PFA and BS 3892 part 1 PFA provided equivalent design strength was achieved.

Safan & Kohoutkova (2001) reported that continuously water-cured concrete specimens provided a better rate of strength development compared to other curing conditions and the compressive strength affected by drying condition varies at different ages.

Solanki and Pitroda (2013) performed flexural strength test on mini beams of size 100 mm x 100 mm x 500 mm. A concrete mix M20 grade was designed as per IS10262:2009 method. The water/cement ratio was 0.48 for all the mixes. Flexural strength test was performed at the age of twenty eight days. It was concluded that the twenty eight days flexural strength of concrete is increased up to 11.1 % with 20 % replacement level of PC by fly ash. The flexural strength test results are tabulated in Table 2.10.

Table 2-10 twenty eight days Flexural Strength after VinodsinhSolanki and Pitroda (2013)

Fly ash replacement level	Flexural Strength N/mm^2	% change in Flexural Strength
0%	5.05	0
10%	4.48	-11.28
20%	5.61	11.08
30%	4.46	-11.68

Kayali and Ahmed (2013) prepared concrete mixes by replacing PC with different percentages of Fly ash. The water/cement ratio was 0.38 for all the concrete mixes and the total amount of cementitious material content was kept constant for all the mixes and was equal to 450 kg/m^3. The concrete samples were cured with fog for seven days and then they were air dried till the age of 28 days for testing. Kayali and Ahmed (2013) found that there was a decrease in the compressive strength of concrete made with fly ash and this decrease was increased with the replacement level of fly ash. The decrease in compressive strength with different replacement levels of fly ash is shown in Figure 2.11. They also found out that there was a decrease in the modulus of elasticity of fly ash concrete compared with the PC only concrete and this decrease was increased with the increase in replacement level of Fly ash. The effect of Fly ash on modulus of elasticity is shown in Figure 2.12. They emphasised that these trends were for total cementitious content of 450 kg/m^3 and these trends might be significantly different for different total cementitious contents.

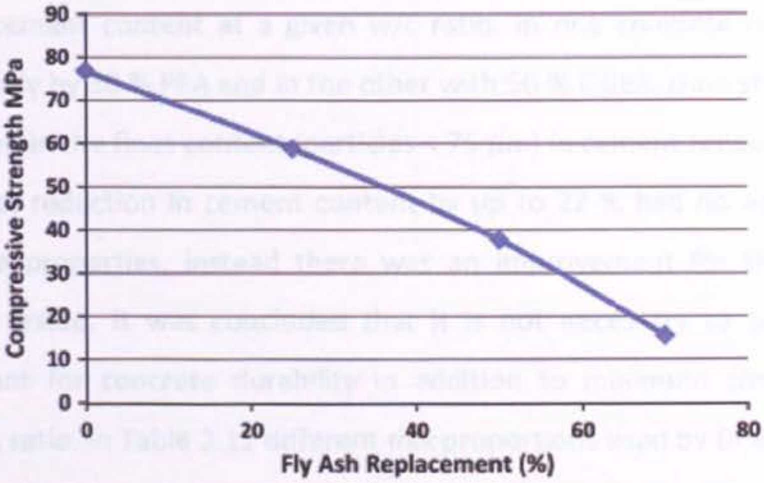

Figure 2-11 Trend of compressive strength of Fly ash replacement (Kayali and Ahmed2013)

Figure 2-12 Trend of elastic modulus variation for Fly ash concrete (Kayali and Ahmed 2013)

According to Concrete Society (1991) "The elastic modulus of PFA concrete is generally equal to or slightly in excess of that shown by PC concrete of the same grade". This statement is in contradiction with the findings of Kayali and Ahmed (2013) and it depends on the total cementitious content, curing and water/cement ratio.

According to Dhir (1986), the normal methods for estimating modulus of elasticity, based on standard cured cube strengths can generally be applied to concrete with or without PFA with the same degree of prediction accuracy. Concrete with PFA gains higher strength with time than PC concrete and so does the modulus of elasticity.

Dhir et al (2004) concluded that most engineering properties were improved with reduction in cement content at a given w/c ratio. In one concrete mix, cement was replaced partially by 30 % PFA and in the other with 50 % GGBS. Lime stone was used as a filler to maintain the fines content (particles < 75 μm) in cement reduced concrete. For fixed w/c ratio, reduction in cement content by up to 22 % had no adverse effect on most concrete properties, instead there was an improvement for the cements and combinations tested. It was concluded that it is not necessary to specify minimum cement content for concrete durability in addition to minimum strength class and maximum w/c ratio. In Table 2.11 different mix proportions used by Dhir et al (2004) are reproduced and in Figure 2.13 the influence of cement content on selected properties of concrete are compared.

Table 2-11 Mix proportions Dhir et al (2004)

Mix	PC Kg/m^3	Free water l/m^3	Natural sand Kg/m^3	Natural Gravel 5-10mm	Natural Gravel 10-20mm
M1	320	175	565	440	880
M2	280	155	590	460	920
M3	245	135	615	480	955
M5	320	175	660	410	815
M6	355	195	630	390	780

The 28 day cube strength results of Dhir et al (2004) for PC concrete at various w/c ratios are shown in Figure 2.14 (a) and in Fig.2.14 (b), the strength development to 180 days at 0.55 w/c ratio are shown. It can be seen that the 28 day strengths increased with cement reduction, compared to the reference mix, M1, ranging from about 5.0 N/mm^2 for M3 at 0.45 w/c ratio to 8.0 N/mm^2 at 0.65 w/c. With increased cement content of M6 there was a reduction in 28 day strength compared to M1.

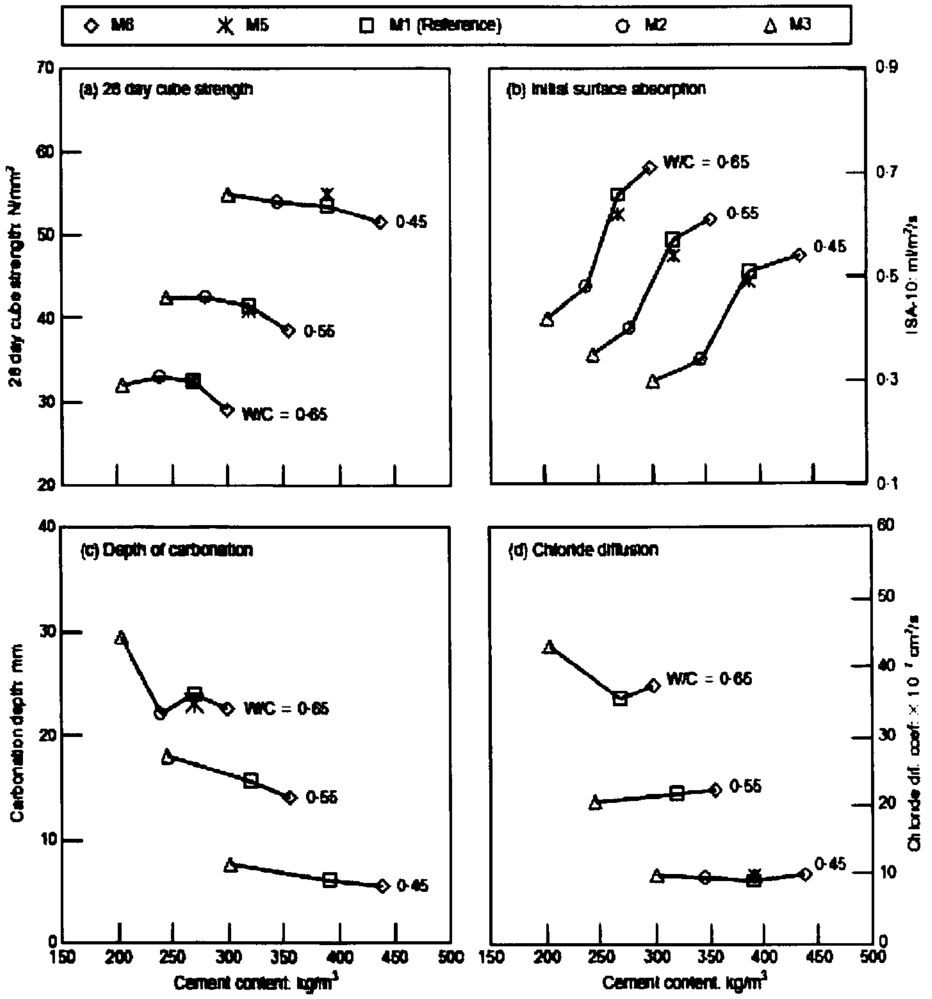

Figure 2-13 Influence of cement content on properties of PC concrete Dhir et al (2004)

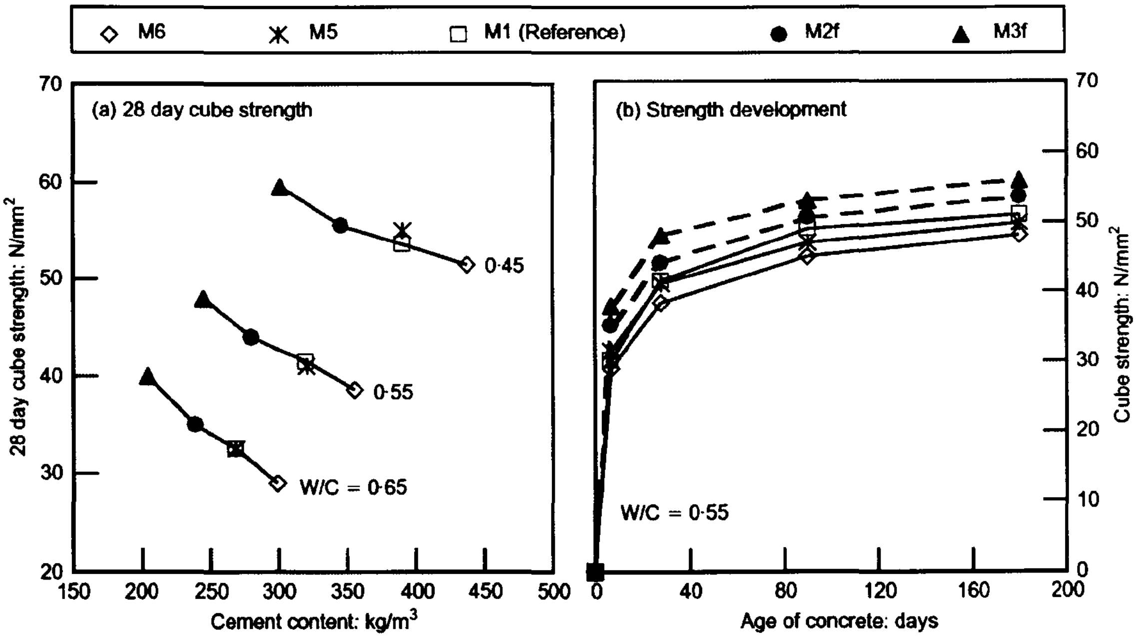

Figure 2-14 Influence of cement content on: (a) 28 day cube strength; (b) strength development of PC concrete Dhir et al (2004)

Dias et al (2003) tested grade 25 and 35 concretes, where Portland cement was replaced with 15 % and 25 % class F PFA with very high fineness (i.e. percentage passing through 45 micron sieve is 85 %). PC content was 345 kg/m^3 and 444 kg/m^3 for grade 25 and grade 35 concrete respectively. Water content for both grades was 200 kg/m^3 which made the water/cement ratio of 0.58 for grade 25 and 0.45 for grade 35 concrete. For PFA concrete mixes the water content was reduced to 190 and 185 kg/m^3 for 15 and 25 % PFA mixes respectively. He found that the best performance of PFA mixes was in the reduction of charge passed in the indirect rapid chloride ion transport test and in the reduction of heat of hydration and temperature rise. In general, the results of PFA mixes were similar or better than the PC control mix. The strength ratio between PFA and PC mixes was slightly higher at 3 days than at 7 days after which the ratio increased again.

According to the findings of Jia et al (2004), to achieve an excellent carbonation resistance of concrete, good curing is very important. Specimens were stored in a moist curing room maintained at 20 ± 2 °C for 1, 3, 7 and 28 days before they were exposed to the natural and accelerated carbonation environment. It was noted in the study that by prolonging the initial curing time causes decrease in the carbonation depth of concrete. The linear relationship between compressive strength and carbonation depth is not suitable when cement is replaced by mineral admixtures in 20 % or 35 % proportions and the water–cement ratio is reduced to 0.40 or 0.37 because of the pozzolanic reaction.

Sear (2010) says that fly ash has been used for many years and can improve the sulfate resistance, reduce chloride diffusion, prevent alkali-silica reaction and reduce heat generation. These beneficial properties of fly ash have been researched by many people with more than a thousand papers. It has been increasingly recognised in recent years that using fly Ash as a cementitious binder reduces green house gas emissions, enhances durability and extends the structures life.

2.7.3.4. Environmental Properties of PFA

According to ScotAsh (2005), each tonne of PFA re-used in cement products saves an average 900 kg of CO_2 emission. In addition using processed PFA as an addition to concrete lowers the water demand, which in turn saves energy.

According to the United Kingdom Quality Ash Association (2010), about 400,000 tonnes of fly Ash are used in concrete production per annum and saves about 250,000 tonnes of CO_2 to the environment. Fly ash is also used by the cement industry in the production of blended fly ash cement so the overall benefit of the UK coal fired power stations fly ash is the reduction of about 600,000 tonnes of CO_2 per annum. This figure could be doubled because only about half of PFA produced is currently utilised.

According to ScotAsh (2009), cement containing PFA typically cost 5 % less than conventional blends. Cement contents are typically about 10 % higher for 30 % PFA mixes than CEMI mixes. For a typical mix of 300 kg/m^3 PC, contains 330kg of PFA (30 %) brings the price down by £ 1/m^3.

In the Angel Building, London, self compacting (36 % fly ash) concrete was used to eliminate the need for conventional methods of compaction, such as vibrating pokers.

By using PFA workability of concrete was improved around difficult interfaces and the light grey colour of PFA concrete added towards the aesthetic of the building.

2.8. Silica Fume

Silica fume is a by-product of producing silicon metal or ferrosilicon alloys. Production of silica fume involves the reduction of high purity quartz (SiO_2) in electric arc furnaces at a temperature of more than 2000 ^{o}C. The smoke that results from furnace operation is collected and converted to silica fume. Silica fume is a very fine powder consisting mainly of spherical particles of diameter 0.15 micron. The average size of particle of silica fume is about 100 times smaller than an average cement particle. Silica fume is generally dark grey to black or off white in colour and can be supplied as a dense powder or slurry depending on the application and the handling facilities available. In the UK silica fume is supplied as slurry consisting of 50 % silica fume and 50 % water. Global consumption of silica fume exceeds one million tonnes per annum.

According to Norchem (2012), in cementitious compounds, the chemical reaction of silica fume is called "pozzolanic" reaction. The hydration of Portland cement produces many compounds, including calcium silicate hydrates (CSH) and calcium hydroxide (CH). The CSH gel is the source of strength in concrete. When silica fume is added to the fresh concrete, it chemically reacts with the CH to produces additional CSH. The benefit of this reaction is increase in the compressive strength and chemical resistance. The bond between the concrete paste and the coarse aggregate, in the crucial interfacial zone, is greatly increased, which results in increase in compressive strength. The additional CSH produced by silica fume is more resistant to attack from aggressive chemicals then the weaker CH. The second function of silica fume in cementitious compounds is a physical one. As silica fume is 100 to 150 times smaller than a cement particle it can fill the voids created by free water in the matrix. This function is called particle packing and it refines the microstructure of concrete, creating a much denser pore structure. Impermeability is increased with the addition of silica fume in concrete.

According to The Concrete Society (2011), the average quantity of silica fume used in concrete is typically less than 8 % of cement content but can be used up to 12.5 % or more. BS EN 13263-1 (2005) and BSEN 13263-2(2005) applies for Silica fume in concrete. The water demand of concrete increases with the use of silica fume and is due to the high surface area of the material. To achieve maximum strength and durability, high-range water reducing admixtures are used in concrete containing silica fume. Concrete containing silica fume is more cohesive and reduces the risk of segregation. The slump of silica fume concrete is more than the PC concrete. Silica fume is a very reactive pozzolan because of its chemical and physical properties. Concrete containing silica fume can have a very high strength and can be very durable. Using silica fume, high early strength concrete can be achieved and has been used in tall buildings in USA and Asia. Time of striking formwork is not affected by the use of silica fume in concrete.

According to Wolsiefer (1991), silica fume lowers the concrete permeability and prevents chloride ingress to the reinforcement, while simultaneously increasing the electrical resistance of concrete to corrosion. Silica fume addition prevents the salt induced corrosion of steel bars. It was concluded that silica fume concrete has less creep than that of PC concrete at equal strength. Shrinkage of concrete is reduced by using silica fume. GGBS, PFA and silica fume reduce the environmental impacts significantly because their manufacturing does not require any quarrying of virgin minerals, they use much less energy in their manufacture as compared to PC and their use in concrete avoids them being land filled. There use in concrete structures enhances the durability and can lead to longer service life. Due to the higher strength achievable by silica fume, overall volume of concrete can be reduced.

2.9. Creep

Creep of concrete can be defined as the increase in strain under a sustained stress. According to Bamforth et al (2008), "Creep is time-dependent deformation (strain) under sustained loading, excluding non-load induced deformations such as shrinkage, swelling and thermal strain, etc." According to Bamforth et al (2008), creep of concrete is determined for specimens stored in such conditions where no shrinkage or swelling occurs. If the specimen is drying while under load it is assumed that creep and shrinkage are additive and creep is calculated by the difference of total time deformation of the loaded specimen and the shrinkage of a similar unloaded specimen stored under the same conditions through the same period. Actually the effect of shrinkage on creep is to increase the magnitude of creep. In actual structures creep and shrinkage occurs simultaneously and the treatment of them together is important. The majority of the data available on creep considers creep in addition to shrinkage.

According to Neville (1995), creep can lead to huge deflection of structural members and cause serviceability problems, especially in high rise buildings and long bridges. creep affects strains and deflections and sometimes stress distribution but its effect vary with the type of structure. Creep of concrete depends on various factors like type and volume of aggregate, strength of concrete, stress on concrete, age at loading, type of cementitious material and the humidity and temperature of the environment while the concrete tested for creep is under load. Creep is a function of volumetric content of cement paste in concrete. Creep is inversely proportion to the strength of concrete at the time of application of load and therefore the stress/strength ratio is more practical than considering the type of cement, water/ cement ratio and the age of concrete.

According to Lyse (1995) "Creep of concrete may be considered as directly proportional to the amount of cement paste in the concrete, regardless of the composition of the paste, provided that the sustained stress is based upon the ultimate strength of the concrete at the time of application of load. The age at loading is not very important and creep is directly proportional to the sustained stress. There is no creep at all only when there is no sustained stress." The type of cement affects creep because of the influence of the strength development of concrete at the age of loading. Various cementitious materials have different rates of hydration and gains different strengths while the concrete is under load. The influence of strength of concrete at the time of application of load applies to different cementitious materials used.

According to Neville (1995), creep of GGBS and PFA concrete is lower than the CEM-I concrete if cured in an environment where there is no moisture loss. This is because of the greater strength gain of the GGBS and PFA concretes during the period under load. Under conditions of no moisture loss creep of concrete containing 50 % GGBS will be 40

% lower than the PC concrete of same 28 days strength and subjected to the same stress which is due to the greater strength increase of GGBS concrete at later ages. According to Neville (1995), in practical situations (beams, columns and slabs) there is a significant long term drying and the creep of GGBS concrete is similar to the PC concrete. If the load is applied at an age of less than 28 days on GGBS and PFA concretes then they will have higher creep values.

According to Dhir et al (1986), for similar stress /strength ratio creep of concrete containing PFA is less than PC concrete. The decrease in creep of PFA can be up to 50 % compared to PC concrete and is dependent on the availability of sufficient moisture. Decrease in creep of PFA concrete is believed to be from the greater strength gain compared to the PC concrete but recent data indicate a similar creep reduction under drying conditions with little change in strength during loading. Decrease in creep of PFA concrete has been reported and is due to the particular storage under water or high humidity.

According to Dhir et al (1986), when the sustained load is removed the strain decreases by an amount equal to elastic strain at the given age and is followed by instantaneous recovery called creep recovery. Advantage of PFA concrete is that creep recovery of PFA concrete is more than PC concrete and increases as the strength of concrete increases.

According to The Concrete Society (2011), creep of silica fume concrete is not higher than the PC concrete. Creep of high strength concrete containing 10 % silica fume was 20 % less than the theoretical predictions.

According to Wolsiefer (1991), concretes containing silica fume have less creep than PC concrete of equivalent strength.

2.10. Flat Slabs

Jones (2004) describes the history of flat slabs as being over a century old. The flat slab system was invented by Turner in 1905. In Europe the founder of flat slabs was Robert Maillart who was a design and-build contractor. In 1908 Maillart performed full-scale tests on slabs. Then in January 1909 he applied for a patent advertised as 'An Important Advance in Reinforced Concrete Construction called the Beamless Deck System'. Design and construct was the normal practice at that time.

Lord (1911) obtained approval to instrument and load test a seven-storey flat slab building in Chicago. All these works contributed to the development of a design code in 1930, which became the London Building Act. (1930).

The striking of flat slabs depends on the compressive strength of concrete and many factors are involved in it like water/cement ratio and the addition of PFA and GGBS, which will tend to slow down the early strength gain. There are different guides and reports available that can be used to calculate the striking time.

According to Muttoni (2008), in flat slabs without beams, the design criteria is often the design against punching shear failure at the column/slab connection.

2.11. Punching Shear Resistance

According to Desai (2000), punching shear resistance is often considered as critical design requirement for flat slabs. Practically punching shear failures should be considered for three situations.

1) Transfer of load from slabs to columns.

2) Concentrated or moving loads on slabs e.g. bridge decks.

3) Transfer of loads from columns to foundations.

Punching shear resistance of a flat slab without shear reinforcement depends on strength of concrete, depth of slab, tension reinforcement and the column size.

Regan (1974) noted that flexure has significant effects particularly when moments are transferred between slabs and columns but the failure at the column head will always be a punching failure.

According to Alexandros (2006), a typical punching shear failure starts by a diagonal crack from the bottom of the slab which propagates to the top at an angle of 20-45 $^{\circ}$ to the horizontal and finally there is a separation of the portion of the slab surrounding the column in a truncated pyramid shape.

Chana (1991) describes punching as a form of shear failure around a column or concentrated load on a slab due to which a portion of slab surrounding the column is separated in the shape of a truncated cone. Generally slab specimens are supported at the nominal line of contraflexure, which is at a distance of 0.2 L from the centre of the column as shown in Figure 2.15.

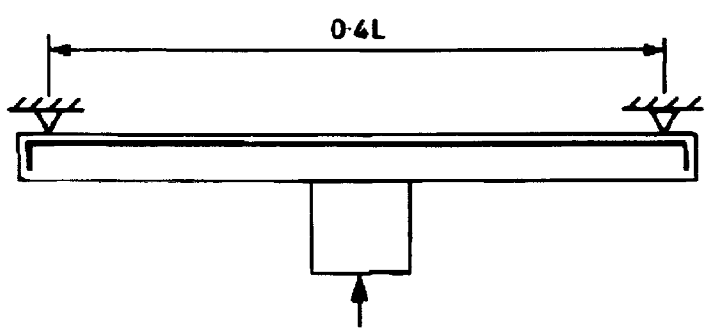

Figure 2-15 Conventional Punching shear specimen Chana (1991)

In Figure 2.16 the crack patterns and the actual mode of failure of a slab and internal column is shown. The first cracks are flexural around the column perimeter and they grow radially from the column extending to the edge of the slab under the increase of load. At about 60 % to 80 % of the ultimate load, a series of circumferential cracks are formed and the shear force is transferred through dowel action, aggregate interlock and the compression zone of the slab. Punching shear failure occurs suddenly with a very low residual load. Chana (1991) further explains the most significant difference between slab and beam behaviour under shear forces as the presence of in-plane restraining forces due to compressive membrane action generated by supports, or the portion of the slab outside the failure zone. Conventional column/slab test specimens are supported at the nominal line of contraflexure and the influence of membrane action from the portion of the surrounding slab beyond the line of contraflexure is ignored.

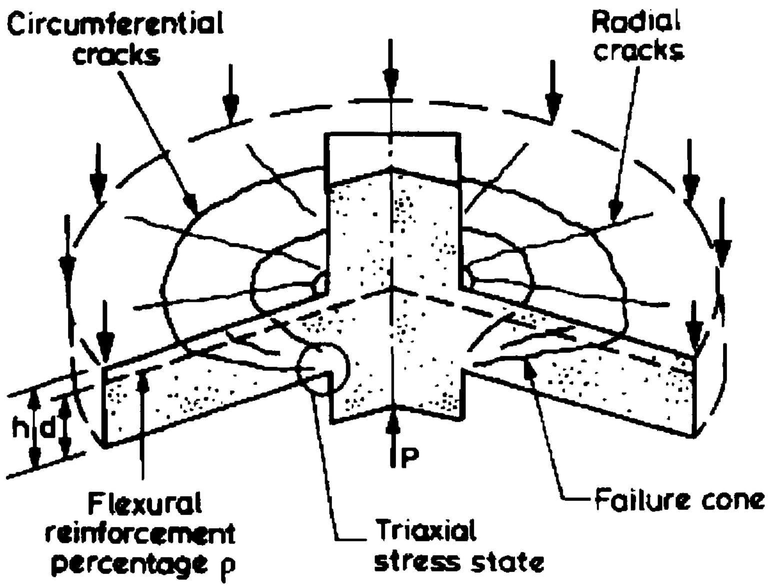

Figure 2-16 Crack Pattern Chana (1991)

Chana and Desai (1992) did some punching shear tests on normal PC concrete slab specimens 9 m x 9 m in plan and 250 mm thick and represented an area of slab surrounding an internal column of a continuous flat slab with columns at 6 m centres each way, with the point of contraflexure 1.2 m away from the centre of the column. Distance of 4.5 m was chosen as the span between the central column of the specimen and the simply supported outer edges. The aim of the testing was to find the effect of membrane action on the punching shear resistance of a flat slab. A schematic of their test arrangement is shown in the Figure 2.17. The slab was simply supported on a slip membrane and blockwork from four sides. The slab was loaded through hydraulic jack at eight locations in the middle of the slab and in the circumference of a circle of diameter 2.4 m. The deflection was measured at nine points equally spaced on the central line of the slab specimen. The full size specimen and the conventional specimen used earlier in the tests are shown in Figure 2.18.

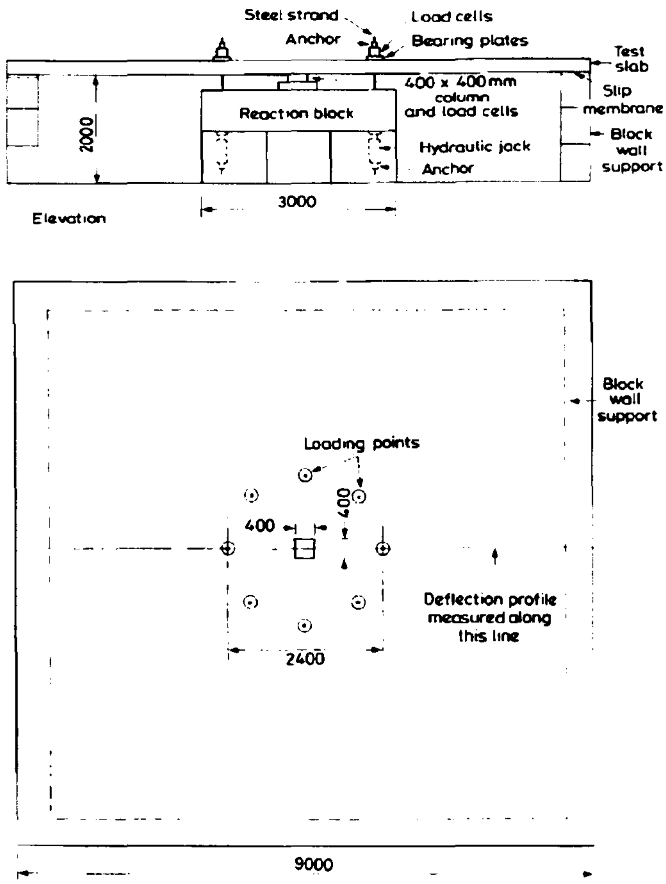

Figure 2-17 Schematic of Test arrangements (Chana & Desai, 1992)

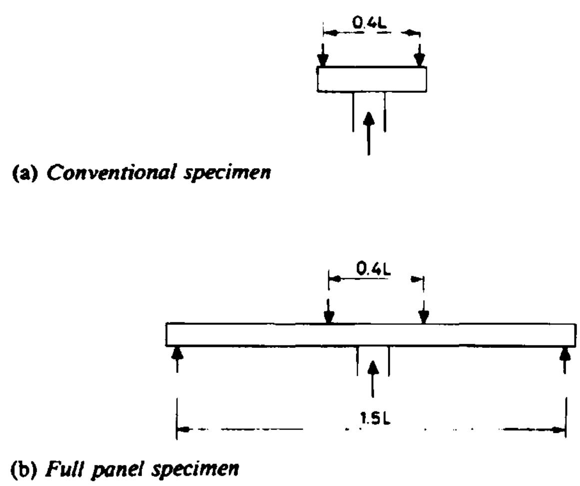

Figure 2-18 Comparison of slab test specimens, Chana & Desai (1992)

All concrete slabs failed in a brittle manner as expected in a punching shear failure mode and the typical crack pattern at failure is shown in the Figure 2.19.

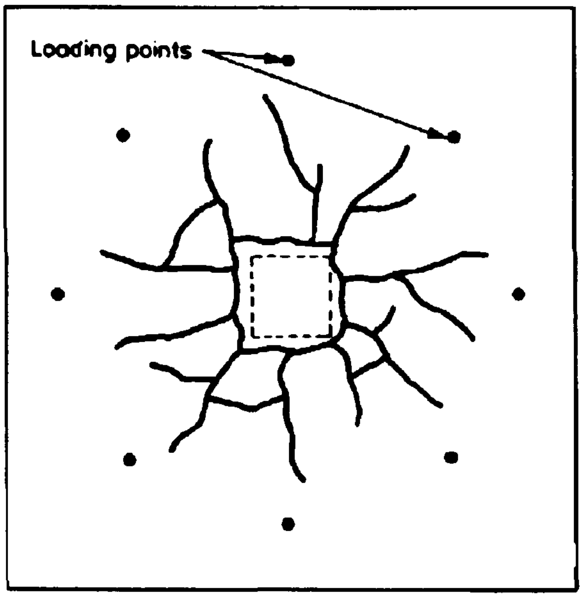

Figure 2-19 Typical Crack pattern at failure Chana & Desai (1992)

Radial cracking developed from the centre of the loaded column and the crack width increased with the increase of load. The maximum crack width in this experiment was 0.15 mm compared to 0.3 mm cracks the author observed in his previous conventional 3m flat slab specimen test. So Chana and Desai (1992) demonstrated the benefit of compressive membrane action on the control of crack width propagation.

The deflection of the full panel specimen was significantly less than the conventional 3 m x 3 m test specimen. The failure mode of the slab is shown in the Figure 2.20 and is similar to the conventional 3 m x 3 m test specimen.

Figure 2-20 Typical Failure Mode Chana & Desai (1992)

The experimental failure loads were compared to the punching shear design strength of BS 8110 and an increase of 40 % in the punching shear resistance of slabs without shear reinforcement was proposed due to the in-plane restraining forces developed from the membrane action of the surrounding slab. It was proposed that the maximum shear capacity of a slab where membrane action is present is limited to 2.4 V_C (resistance of the concrete cross-section) in comparison to 2 V_C for slabs where no membrane action is present. In the authors view, the restriction of the limit on the column face shear in BS 8110 (0.8 $\sqrt{f_{cu}}$) is unnecessary because there were no signs of distress on the slab soffit at the column face in any of the tests.

Gardner (2011) concluded that most experimental tests are on thin slabs and suggested that the code equations should incorporate a size effect term. It was concluded that BS 8110-97, CEB-FIP MC90 /EN 1992-1 have the size effect term, reinforcement ratio term and use the control perimeter at a distance of 1.5 d and 2.0 d respectively from the face of the column and have smaller (14 % to 15 %) coefficients of variation than ACI 318-08 for an interior column slab connection.

Desai (2000) tested flat slab specimens of 1.5 m x 1.5 m square in size and 250 mm thick for punching shear resistance. Each slab had eight holes of 20 mm for loading cables which were equally spaced along the circumference of a circle of 1.125 m diameter. Comparison of BS 8110 (1997) estimates and the test results of Desai (2000) are given in Table 3.8. For the purpose of this exercise, the author ignored the upper limit of 40 kN/mm^2 for f_{cu} and suggests that this limit can be safely increased to 60 N/mm^2. The author confirmed the limit of BS 8110 for ultimate load carrying capacity of slab ≤ $2V_{cu}$ was satisfactory by the results given in the Table 2.12. It was found that the EC 2 rule give estimates for the punching shear capacity of slabs without links 10 % lower than BS8110 (1997) estimates.

V_{cu} is the BS 8110 ultimate punching shear capacity of a slab without shear reinforcement.

V_{col} is the BS 8110 ultimate load carrying capacity of the slab based on maximum shear stress at the column face.

Table 2-12 Comparison of BS 8110 estimates with test results after (Desai 2000)

Slab No	Compressive strength of concrete f_{cu}	Experimental failure load v_F	$2\ V_{cu}$	V_{col}	$V_F/2\ V_{cu}$	$V_F/\ V_{col}$
	N/mm^2	kN	kN	kN		
1	22	1102	1196	980	0.92	1.12
2	22	1044	1196	980	0.87	1.07
3	22	1069	1196	980	0.89	1.04
4	40	1379	1459	1322	0.95	1.03
5	40	1367	1459	1322	0.94	1.06
6	40	1404	1459	1322	0.96	1.05

Desai (1997) concluded that the provision of central mesh reinforcement enhance the punching shear capacity of the flat slab. It was realised that there could be a more effective dowel action of the central mesh resisting the punching shear failure for deeper slabs having better bonding of the mesh with the surrounding concrete. A revised rule for the contribution of central bars (V_{bu}) resisting the ultimate shear resistance of a section was suggested, as given in the following equation.

$V_{bu} = 0.4\ \rho_{bu}\ V_{cu}$

$V_{bu} \leq 0.6\ V_{cu}$

Or $V_{bu} \leq 0.003\ d\ v_{cu}$ (d is the effective depth) whichever is the lesser.

$\rho_{bu} = \rho_b\,(u+u_o)\ /\ u$

$V_{cu} = 0.27\rho^{1/3} f_{cu}^{1/3} (400/d)^{1/4}\ ud\ /1000$

ρ_{bu} is the horizontal steel factor taken as percentage area of central bars (ρ_b), shear perimeter (u) and the perimeter of column or loaded area (u_o)

Desai (1997) believed that the provision of the central mesh is the solution to the difficulty of the effectiveness of the shear reinforcement in slabs less than 200mm.

Kotsovos & Kotsovos (2009) proposed a simple expression for defining the width of the slab strips which enables the criteria for the shear failure of reinforced concrete beams to be extended to the case of punching of flat slabs. The assessment of punching shear capacity with this criteria gives values which are very close to those proposed by ACI and EC 2 and provide a more realistic description of the cause and location of the punching failure. The proposed criteria is based on the assumption that the resistance of flat slabs to punching are provided by two slab strips extending along the flexural reinforcement and intersecting each other at the head of the supporting column.

In the study conducted by Kotsovos & Kotsovos (2010), they suggested that the cause of punching shear relates to the development of transverse tensile stresses in the compressive zone of the slab portion. These stresses develop either due to the loss of bond between concrete and the flexural reinforcement, or due to the sudden change in the direction of the compressive stress trajectories.

Papanikolaou et al (2005) tested slab specimens of 750 mm x 750 mm with the diameter of the central stub column 150 mm for punching shear. The punching shear test results were compared to the two major code (ACI 318-99 and Eurocode 2) predictions. It was found that the predictions of these two codes were conservative for slabs without shear reinforcement and were less conservative for those with shear reinforcement. The mode of failure for slabs with high flexural reinforcement ratio (1 %) was punching shear failure mode and for the slabs with low flexural reinforcement ratio (0.5 %) was a mix of flexural and punching mode.

Regan (1986) performed test on reinforced concrete flat slabs and concluded that if the definition of ratio of reinforcement is improved in the calculation of punching shear capacity using the method in BS 8110 (1985), the intended level of safety can be achieved by 10 % reduction in either the basic shear stress or the size factor.

Regan (2003) proposed that a control perimeter 0.25 d from the load should be considered for highly concentrated loads and a limiting shear stress should be equal to 3.0 times the value at the normal perimeter. In Figure 2.21, Regan (2003) compared the estimates of ACI 318-02 and CEB-FIP MC90 for punching shear strength. The ACI code has no influence from either the main steel ratio or the slab effective depth in limiting its shear values. The strain % in the main reinforcement at different increment of loads applied for different slab specimens tested by Regan (2003) are given in Figure 2.22. It can be seen from the load strain relationship that the steel has exceeded its yield strength of 0.25 % prior to the punching shear failure.

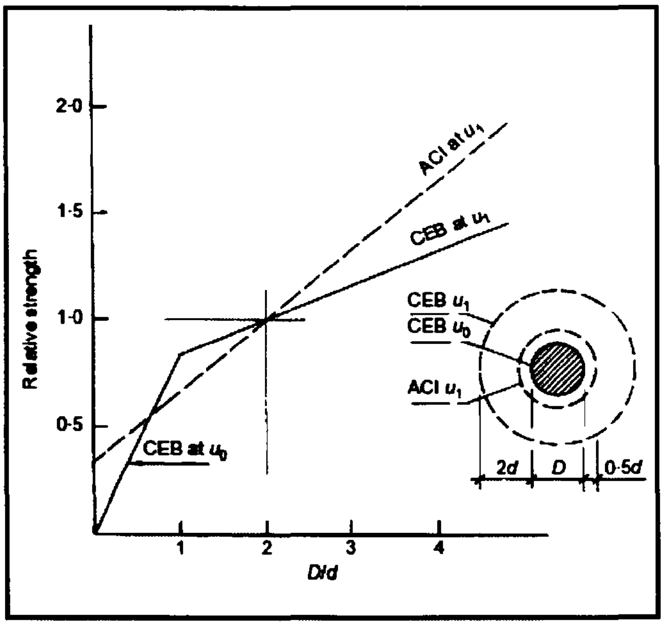

Figure 2-21 Effects of column size on punching shear strength (Regan 2003)

According to Park et al (2011) model for direct punching shear strength of interior column, the compression zone of the critical section which is subjected to combined shear and compressive stresses fails due to either compression crushing or tensile cracking. The punching shear strength is determined by using the shear capacity controlled by tension and is controlled by the tensile strength of concrete and the depth of the compressive zone. Punching shear strength is increased with the increase in compressive strength of concrete because the tensile strength of concrete is increased. For high compressive strength concrete, the depth of the compression zone is reduced and the punching shear strength does not increase proportionally to concrete strength. With the increase of flexural reinforcement ratio, the depth of the compression zone is increased and thus the punching shear strength is increased.

Fraser and Jones (2009) say that punching shear failure and flexural failure may combine into a single failure mode in reality, but this is not checked for in current codes of practice.

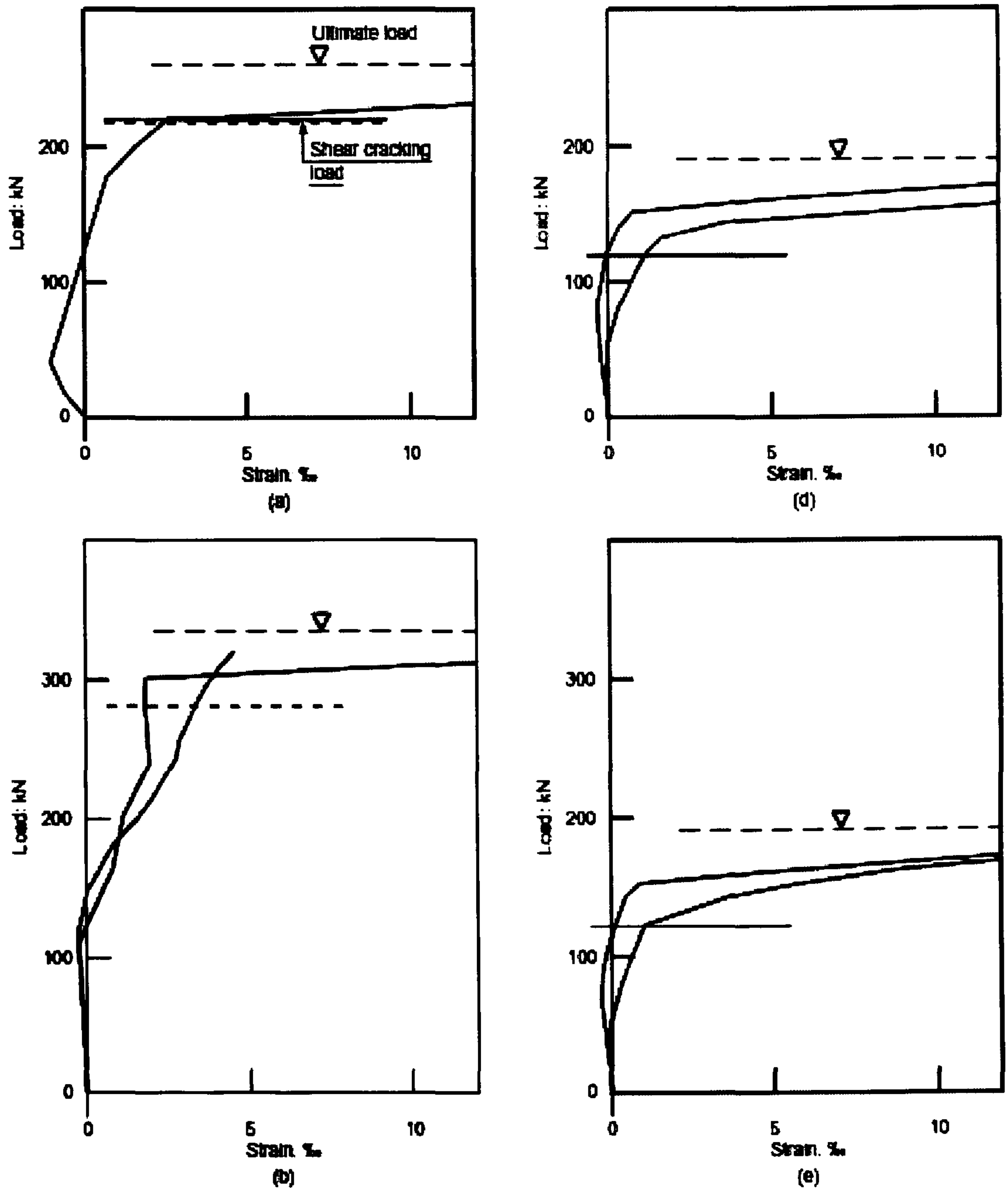

Figure 2-22 Reinforcement strain Regan (2003)

According to Wood (2002) Pipers Row car park in Wolverhampton collapsed because its top slab should have been designed and maintained for both dead and live loads but it contained the flaws in the design requirements of CP-114 (1969). The design, construction and progressive deterioration contributed to the punching failure of the car park under dead load. The car park collapsed because of the deterioration of the top slab weighing 120 tonnes and measuring 15 m x 15 m at night due to the dead load only.

The standard Committee of structural safety (SCOSS) (1994) had warned about the risk of the deterioration of the Wolverhampton car park in advance.

Clarke (1987) tested 12 high strength concrete beams with compressive cube strengths between 83 and 93 N/mm^2, shear span ratio of 3 and tension steel of 1.8 % and 2.6 %. He concluded that the BS8110 rule for V_c could allow for f_{cu} in excess of 40 N/mm^2. Clarke believed that this limit should be increased to 60 N/mm^2, to correspond to the EC2 limit of f_{ck} < 50 N/mm^2 and this change should correspond to the limit on the maximum value of applied shear (V). This limit should be as follows.

$V < 0.8\, f_{cu}^{0.5}$ or 6.2 N/mm^2, whichever is the lesser.

The ratios V_F/V_{RD1} were between 1.11 to 1.26 for the Eurocode BS EN 1992-1 estimates for the punching shear strength.

2.11.1. Punching Shear Resistance of Flat slabs Test methods

The test methods used by different researchers for Punching shear resistance of flat slabs were reviewed for the purpose of designing the testing rig. The experimental results of these researchers were not directly related to the current research so they are not discussed.

Ramdane (1996) tested circular slabs of 125 mm thick and 1700 mm diameter. The slabs were supported by column stubs or loading plates. The punching load was applied upward by a 550 kN hydraulic jack through a thick steel disc with a diameter of 150 mm situated in the centre beneath the slab. The reactions were provided by 12 high tensile steel rods equally spaced around a circle with a diameter of 1372 mm.

Hallgren and Kinnunen (1996) tested circular slabs, supported on circular concrete column stubs. The diameter of the slab was 2540 mm and the diameter of the circle along which the load was uniformly distributed was 2400 mm. All slabs were provided with two way flexural reinforcement.

Tomaszewicz (1993) tested square flat slabs with orthogonal equally spaced flexural reinforcement and without shear reinforcement. Slabs were supported along the edges and loaded at mid-span by a concentrated load to failure in punching. Variables in the tests were concrete strength (64-112 MPa), slab thickness (120,240,320 mm) and reinforcement ratio.

Nguyen et al (2010) tested flat slabs of different dimensions and 125 mm depth. He used three linear variable differential transformers (LVDTs) to determine deflection at mid span and at a quarter of the span of slabs. One pair of electrical strain gauges bonded on two tensile re-bars to measure strain and one pair on top of the slab, near the column face to measure concrete strain were used. Slabs were loaded by 1000 kN capacity machine under load control in increments of 10 kN up to failure. The loading rate was 15 kN/m. At each load level, deflection, concrete and rebar strains and crack development were recorded. Laboratory arrangements of the test are shown in Figure 2.23.

Kruger, Burder and Favre (1998) tested square flat slabs (3 m x 3 m x 0.15 m) supported on a (0.3 m x 0.3 m) square concrete column in the centre. The slab was simply supported on knife edges fixed on steel beams so that the edge was free to lift. He tested 6 slabs with varying eccentricities of loading: zero, approximately one half and one times the column dimension (0, 0.16, 0.32 m). The test was performed with a deformation controlled hydraulic jack which gave a constant loading rate of 4 kN per minute. During every test, the load was applied in steps of 40 kN .Between load steps, deformation was kept constant for 10-15 minutes for inspection and measurement. After the peak load was reached, the deformation was further increased to record the post- punching behaviour of the slabs. The test ended when the column had penetrated in the slab, or when the rotation of the column exceeded 5 %. Automatic data acquisition devices recorded every minute on a computer. The crack pattern was inspected and the manual measurement of radial and tangential deformation at the bottom surface of the slab (tension) was performed. The test set up is shown in Figure 2.24.

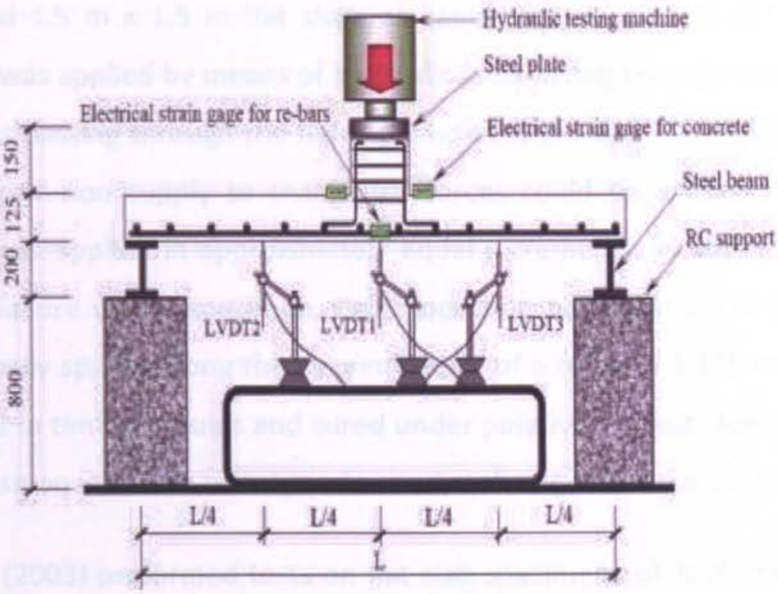

Figure 2-23 Arrangements for testing a Slab Specimen (Nguyen et al 2010)

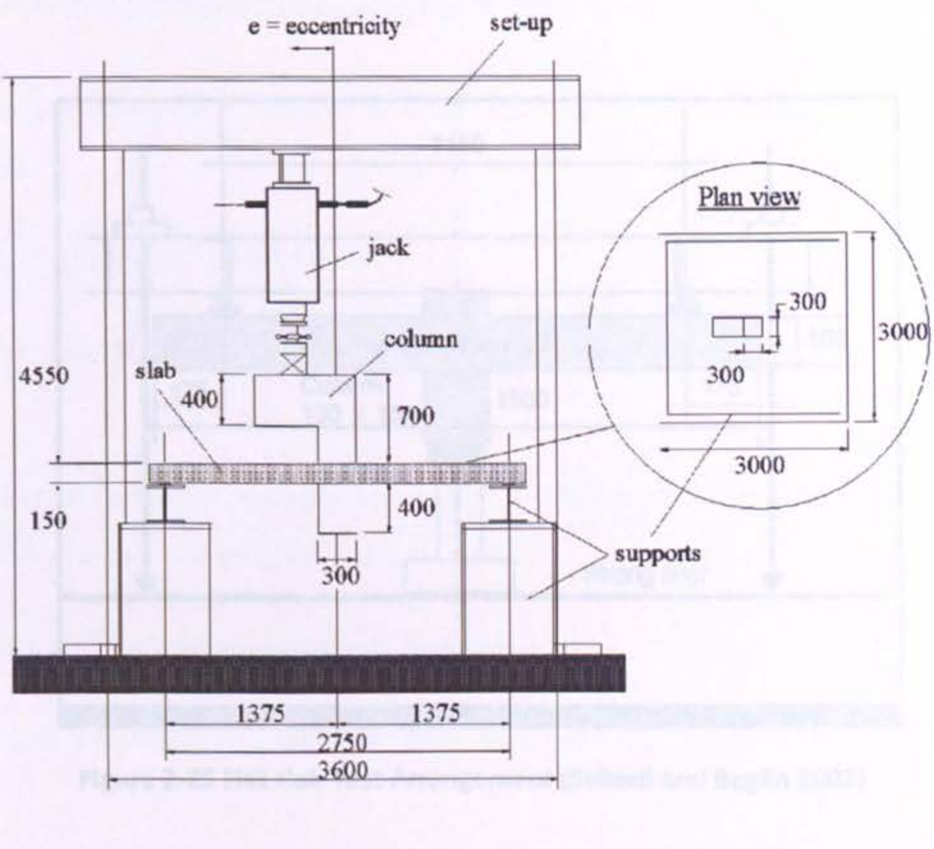

Figure 2-24 Testing a concrete flat slab specimen (Kruger, Burder & Favre 1998)

Desai (1995) tested 1.5 m x 1.5 m flat slabs as cantilevers supported over the stub columns. The load was applied by means of hydraulic jacks acting through load cells and eight loading cables passing through the holes provided in the slab. The hydraulic jacks were linked to a common supply so that equal forces could be applied through all cables. The load was applied in approximately equal increments, in about ten stages from start to the failure of the specimen. Each specimen had eight 20 mm holes for loading cables, equally spaced along the circumference of a circle of 1.125 m diameter. The slabs were cast in timber moulds and cured under polythene sheets for seven days. They were then positioned on the test rig and tested at the age of about 28 days.

Subedi and Baglin (2003) performed tests on flat slab specimens of 1500 mm^2 size for punching shear. The load was applied to the column from the underside of the slab as shown in Figure 2.25. The slab was supported on roller supports at a square perimeter of side 1150 mm, by means of rigid beams.

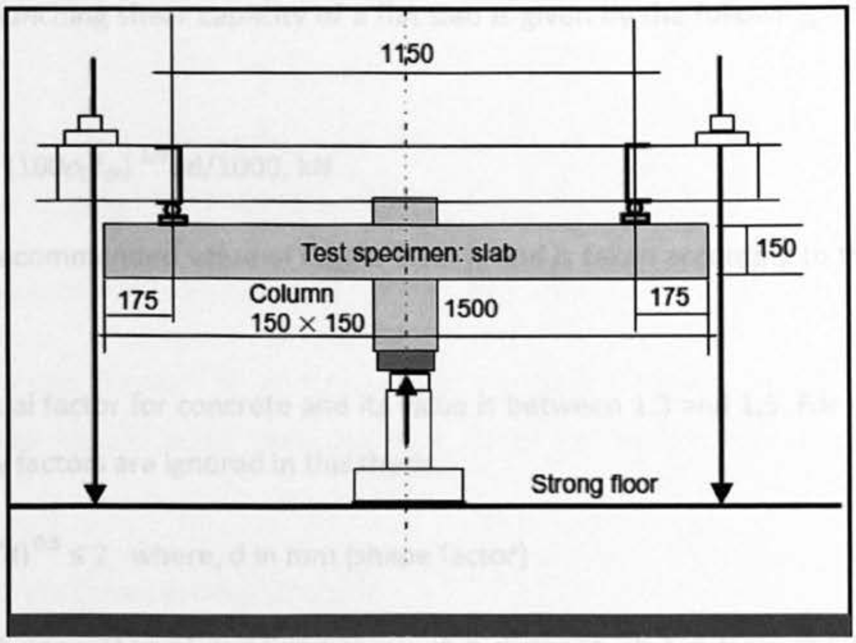

Figure 2-25 Flat slab Test Arrangement (Subedi and Baglin 2002)

Regan (2003) performed punching shear strength tests on slab specimens 2 m x 2 m square and 160 mm thick as shown in the Figure 2.26. Slabs were loaded upward at their centres and reaction was provided by 1.83 m x 1.83 m square on the sides. The slabs were cast with the main steel at the bottom and were inverted for testing. The diameter of the steel disc through which load was applied varied from 25 mm to 170 mm. The flexural reinforcement ratio for all slabs was 0.93 except one which was 1.71. The test arrangement of Regan (2003) is shown in Figure 2.26.

All these test methods for punching shear have different advantages. After reviewing the test methods for punching shear resistance of flat slabs and considering the available laboratory facilities, it was decided to design a testing rig that will support the slab specimen at the nominal line of contraflexure (0.2 L from the centre of the column) and the load be applied with a hydraulic jack in equal increments of 10 kN, vertically upwards at the central stub column.

2.11.2. Punching Shear according to BSEN 1992-1-1(2004)

The design punching shear capacity of a flat slab is given by the following expression in Eurocode2.

$V_{Rd,c} = C_{Rd,c}\, k\, (100\rho_l\, f_{ck})^{1/3} ud/1000$, kN

Where the recommended value of $C_{Rd,c}$ is $0.18/\gamma_c$ and is taken according to the National Annex.

γ_c is the partial factor for concrete and its value is between 1.3 and 1.5. For comparison partial safety factors are ignored in this thesis.

$K = 1 + (200/d)^{0.5} \leq 2$ where, d in mm (shape factor)

u is the first perimeter of punching shear at a distance of 2 d from the face of the column.

d is the effective depth

f_{ck} is the cylinder crushing strength

$\rho_l = (\rho_{ly} \cdot \rho_{lz})^{0.5}$ (For equal steel in both direction, this should be the same as $\rho_{st} = 0.0082$)

ρ_{ly} and ρ_{lz} relate to the bonded tension reinforcement in each direction. These should be calculated as a mean value, if the steel is not uniformly provided over a slab width of "3d" from each face of the column plus the column dimension (b), i.e. (b + 6d) for a square column.

The basic control perimeter is taken at a distance of 2.0 d from the loaded area as given in Figure 2.27, taken from Eurocode 2 BS EN 1992-1-1(2004).

2.11.3. Punching Shear according to BS 8110 (1997)

Although BS 8110 (1997) was withdrawn during the research yet it is very important for comparison. According to BS 8110 (1997), the ultimate punching shear capacity of slab without shear reinforcement is calculated from the following expression.

$V_c = 0.79\ [100\ A_s / (b_v\ d)]^{1/3}\ (400/d)^{1/4} / \gamma_m\ (u\ d/1000)$, kN

γ_m is the partial safety factor and its value is 1.25 but is ignored for the purpose of this exercise.

Where, $(400/d)^{1/4} \geq 1$ (shape factor) for members with shear reinforcement providing a design shear resistance of ≥ 0.4 N/mm^2.

$(400/d)^{1/4}$ should not be less than 0.67 for members without shear reinforcement.

For characteristic strength of concrete greater than 25 N/mm^2, the values of the punching shear resistance are multiplied by $(f_{cu}/25)^{1/3}$. BS 8110 limits the value of f_{cu} to 40 N/mm^2.

f_{cu} is the concrete cube crushing strength.

$u = 4 \times 3d + u_o$

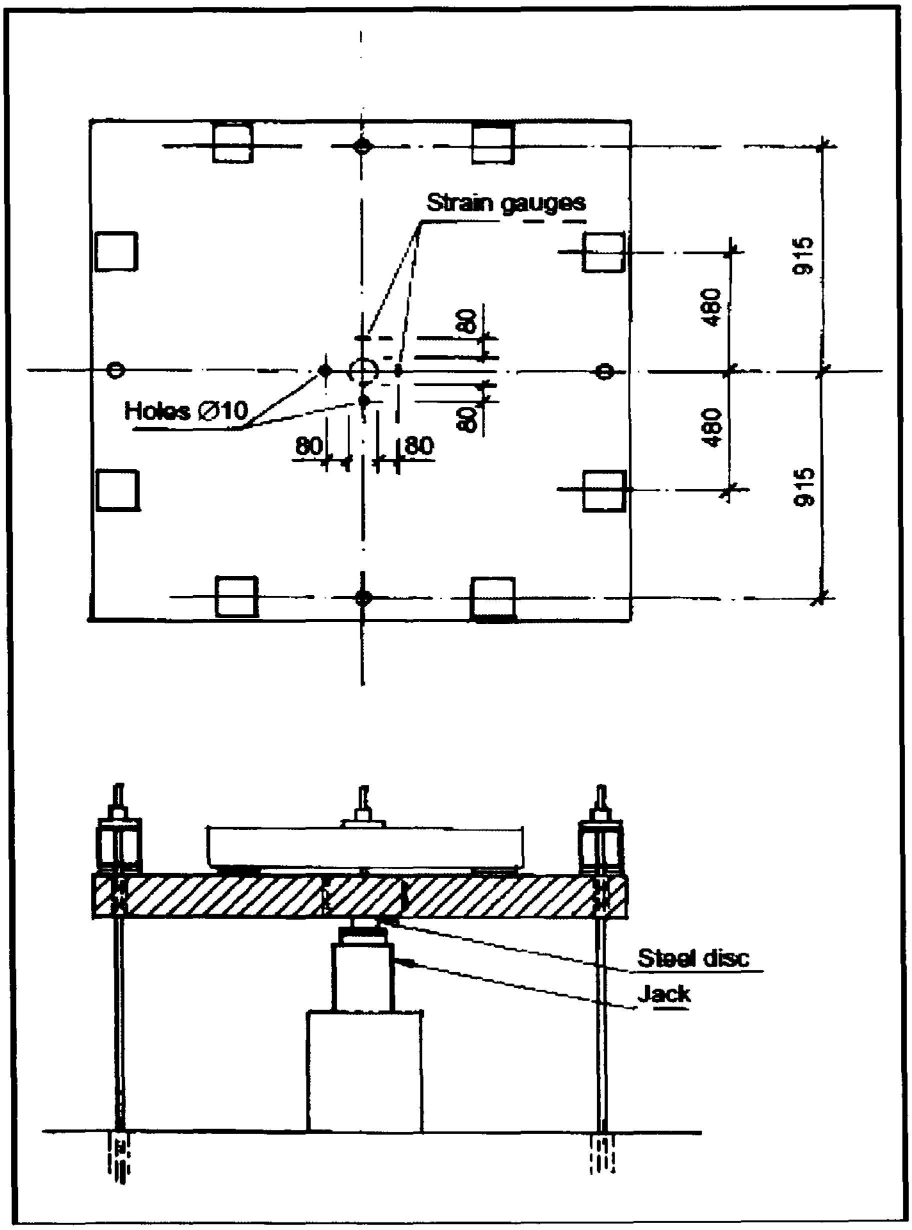

Figure 2-26 Punching shear test Arrangement (Regan 2003)

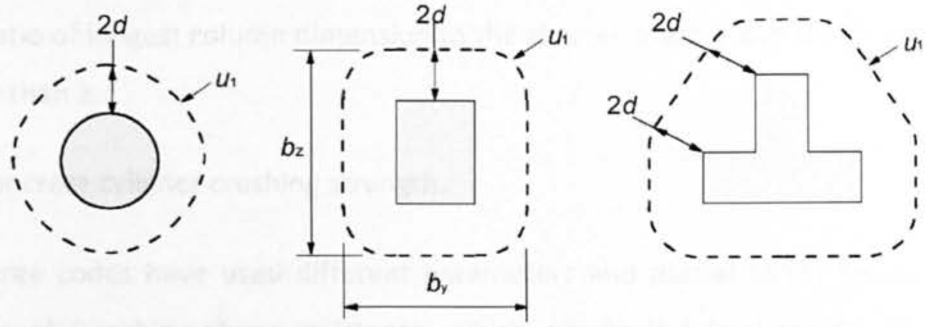

Figure 2-27 Typical basic control perimeters around loaded areas

(Figure 6.13 EN 1992-1-1 :2004)

2.11.4. Punching Shear According to ACI 318 (2008)

According to ACI 318 (2008) the ultimate shear strength of slabs without pre stress is given by the following equation.

$V_{uo} = ud\ (V_n)$, kN

Where, u is the length of the critical perimeter taken at a distance of d/2 from the face of the loaded area.

d is the effective depth in mm

V_n is the punching shear strength in MPa and shall be the smallest of:

$V_n = \phi\ (1+2/\beta_c)\ \sqrt{f_c}/6$

$V_n = \phi\ (\alpha_s d/u+2)\ \sqrt{f_c}/12$

$V_n = \phi\ \sqrt{f_c}/3$

ϕ is the partial safety factor and its value is 0.75 but is ignored for the calculation of Punching shear resistance in this exercise.

Where α_s, is 40 for interior column, 30 for edge and 20 for corner column.

β_c is the ratio of longest column dimension to the shorter column and should be equal to or greater than 2.

f_c is the concrete cylinder crushing strength.

All the three codes have used different parameters and partial safety factors for the calculation of Punching shear resistance, which affects the final results. The partial safety factors of 1.5, 1/1.25 and 0.75 are used by BS-EN 1992-1-1, BS8110 and ACI318 respectively for the calculation of punching shear strength.

The method for calculating critical perimeter is different for each of the codes discussed. A square perimeter is taken at a distance of 1.5 d from the face of the column in BS 8110 and a distance of 0.5 d in ACI 318. In BS EN 1992-1-1 a circular perimeter at a distance of 2.0 d from the face of column is considered in the calculation of punching shear strength. For the calculation of punching shear strength, there is no consideration of the effect of flexural reinforcement in ACI 318, whereas the other two codes consider the affect of flexural reinforcement.

2.12. Summary

Sustainability is the development that meets the needs of the present without compromising the ability of future generations to meet their needs. (Brundtland (1987)).The concept of sustainability was introduced for first time in 1960. Since then there have been several conferences on sustainability at different times throughout the world. Some of the main conferences on sustainability held throughout the world are Earth Summit Stockholm held in 1972, Our Common Future held in 1987, Earth Summit Rio held in 1992, World Summit on Sustainable Development Johannesburg held in 2002 and Earth Summit held in 2012. The common aim of these summits was based on the three pillars of sustainability, economic development, social equity and environmental protection. The main objective of these summits was to make political commitment to sustainable development and to propose long term environmental strategies and recommend ways between the developed and developing countries which are at different stages of economic and social development to achieve a common objective.

Concrete has been used for over 5000 years with early forms adopted by the Egyptians in making pyramid. Concrete technology has gained importance over the last century and the researchers are developing methods for making concrete. Concrete is used second to water on a volume consumption basis. Globally, concrete is used at a rate of over two tonnes per person per year.

Sustainability means to have no negative impact on the environment. For sustainable construction solutions, concrete really is the sustainable material of choice, because it has less environmental impact than steel when compared in structures designed for the same engineering function. The embodied concrete energy is about 3 % of the total energy required to construct and operate a concrete house with more than a 100-year expected life. For commercial buildings, it is 5 % to 15 %.

In concrete cement is the main constituent and production of one tonne of Portland cement requires about 4GJ of energy and results in 0.89 to 1.1 tonnes of CO_2. Due to the limit on the availability of natural minerals used for making cement and due to the energy released and emission of CO_2 produced in the manufacturing of cement, it can be partially replaced using industrial by-products e.g. pulverised Fuel Ash (PFA), Ground Granulated Blast furnace Slag (GGBS) and silica fume. Concrete construction is robust and durable, provided that the concrete is correctly mixed, placed and cured.

In the UK concrete industry agreed a Concrete Industry Sustainable Construction Strategy and its vision is that, "By 2012, the UK concrete industry will be recognised as the leader in sustainable construction, by taking a dynamic role in delivering a sustainable built environment in a manner that is profitable, socially responsible and functions within environmental limits".

According The Concrete Centre (2010), the cement sector has improved its Climate Change Agreement (CCA) performance by 44.8 % between 1990 and 2010 which exceeds the agreed target of 30 %. The ground granulated blast furnace slag sector achieved a 16 % energy reduction between 1999 and 2010. Common thinking is that the idea of sustainable structures is not very important because it will increase the cost but actually it has become the main design criteria. Sustainable structures can cost less to build, less maintenance cost, use less energy and produce less CO_2 emissions. The concrete industry should be economical, efficient and high tech and it should seek to make optimum use of materials and minerals. Durability is the key to sustainability. The life of a structure can be extended by using concrete and is a great achievement for sustainability.

GGBS is a by-product from the manufacture of iron in blast furnace slag. The first commercial use of slag was slag-lime cement in Germany in 1865 and the first blast-furnace slag cement company was opened in Germany in 1888. GGBS has to be handled very carefully because it is highly alkaline in solution and can damage the skin severely. From structural point of view, GGBS replacement gives lower heat of hydration, enhanced durability including resistance to sulfate and chloride attack when compared with normal concrete. On the other hand, it also contributes to environmental protection because it minimizes the use of cement during the production of concrete. Concrete made with slag cement has higher compressive and flexure strength compared to the PC concrete. The strength gain is slow in the concrete containing GGBS but the long term strength of GGBS concrete is more compared to the PC concrete. All these beneficial effects of GGBS concrete depends on concrete mix proportions and curing conditions. A maximum replacement level of 50 % is recommended for GGBS and the curing temperature of at least 20 °C is beneficial.

The slump of the GGBS concrete is unaffected compared to PC concrete and is much easier to compact. Due to the improved workability of slag concrete, the entrapped air content is lowered.

The influence of concrete on compressive strength is in proportions to the effect on other engineering properties and is in line with the current design assumptions for concrete. Literature review suggests that there is no need to review the design procedures relating to flexure or shear for using concrete mixes containing GGBS and

PFA with regard to relationship between compressive strength and other engineering properties.

PFA is a by-product obtained at power stations where Pulverised bituminous coal is fired in the furnaces and is a solid material. It is carried by the exhaust gases and recovered as fine particles. In the UK, PFA has been used since 1950's for various applications. It is widely recognised that PFA in concrete enhances the basic characteristics of fresh and hardened concrete. The most important physical property of PFA is the fineness and the chemical property is the Carbon Content. Early age strength of PFA concrete is slower than the PC concrete because the pozolanic reactions are slower than the hydration reactions and they start after about five days.

At low water curing temperatures of 5 °C, the strength of the PC concrete and the PC/PFA concretes are very similar because the lower temperature are not able to slow down the pozzolanic reactions more than the hydration reactions.

For water/cement ratio of 0.48 and with 20 % replacement of PC with PFA, the flexural strength is increased by 11.0 % compared to the PC only concrete. According to the findings of Kayali and Ahmed (2013), if the total cementitious content of concrete is kept constant and it is cured for a limited time, there is a decrease in the compressive strength and modulus of elasticity of concrete containing PFA, compared to the PC only concrete and this decrease in strength is increased with the replacement level. According to Concrete Society (1991), "The elastic modulus of PFA concrete is generally equal to or slightly in excess of that shown by PC concrete of the same grade". This statement is in contradiction with the findings of Kayali and Ahmed (2013) and it depends on the total cementitious content, curing and water/cement ratio.

Fly ash has been used for many years and is used to improve the sulfate resistance, reduce chloride diffusion, prevent alkali silica reaction and reduce heat generation. Fly ash is used as a cementitious binder to reduce green house gas emissions, enhance durability and extend structure life. Each tonne of PFA re-used in cement products saves an average 900 kg of CO_2 emission. In UK the use of Fly Ash in concrete production per annum saves about 250,000 tonnes of CO_2 to the environment.

Flat slabs are most popular structures around the world because of their easy and fast construction, good aesthetics without downstand beams and more floor to ceiling height. Striking of flat slabs depends on the compressive strength of concrete and many other factors are involved in it like water/cement ratio and the addition of PFA and GGBS which will tend to slow down the early strength gain.

A critical design criterion for most of the flat slabs is the design for punching shear failure. Punching shear is the separation of the portion of the flat slab surrounding the column from the rest. A typical punching shear failure starts by the propagation of a radial crack from the loaded column that grows towards the slab and increases in width with increase of load. After at about 60 % to 80 % of the ultimate load a series of circumferential cracks are formed and the shear is transferred from the slab through dowel action, aggregate interlock and the compressive zone of the slab.

Punching shear resistance of a flat slab without shear reinforcement depends on the strength of concrete, depth of slab, tension reinforcement and the column size. Punching shear relates to the development of transverse tensile stresses in the compressive zone of the slab portion. The punching shear strength can be determined by using the shear capacity controlled by the tensile strength of concrete and the depth of compressive zone.

Normally the slab test specimen for punching shear strength is supported at the nominal line point of contraflexure. The punching shear strength test of flat slabs has been performed by different researchers at various times and the experimental results compared with the estimates according to BS 8110, BS EN19921-1 and ACI codes. It was concluded that the predictions of ACI 318-99 and Eurocode 2 were conservative for slabs without shear reinforcement and were less conservative for the slabs with shear reinforcement.

An increase of 40 % in the punching shear resistance of slabs without shear reinforcement was proposed due to the in plane restraining forces developed from the membrane action of the surrounding slab.

It is clear from the literature review that partial replacement of PC with GGBS and PFA reduces the embodied CO_2 of concrete. The properties of concrete containing GGBS and PFA, like strength development, flexural strength and modulus of elasticity are significantly affected by water/cement ratio, total amount of cementitious material, replacement level and the curing conditions. Different researchers have concluded different results for these properties because the concrete mix design method was different or the curing condition was different so there is a need of designing a concrete mix and cured under an environment, that suits the practical requirements. Due to the slow rate of gain in strength of concrete containing GGBS and PFA there is a need to design and cure the concrete containing GGBS and PFA in a way, so that it can suit the practical requirements to be used in the precast concrete and fast track construction where structural elements are often subjected to high early loads.

As there is no data available on Punching shear strength of RCC flat slabs containing GGBS and PFA, there is a need to test the RCC flat slab specimens containing GGBS and PFA for punching shear strength and compare the results with the estimates of different code of practices, which will provide guidance and confidence to the designers for using these industrial by-products as partial replacement of PC in flat slabs.

3. EXPERIMENTAL PROGRAMME

3.1. Introduction

A comprehensive research programme was devised to achieve the main aim of the study. This chapter describes the methodology and the experimental programme used for reducing the embodied CO_2 content of structural concrete, without adversely affecting the product performance by partial replacement of cement with GGBS and PFA.

3.2. Experimental Programme

A brief summary of the research programme is shown in Figure 3.1. After identification of the main research objective, literature about sustainable concrete practices and sustainable concrete constituents was reviewed. To achieve the main aim and the specific objectives of the research the experimental programme was planned.

3.2.1. Concrete mix proportions

To achieve the main aim and specific objectives of the research, different concrete mix proportions were devised to suit the requirements of practical applications. To reduce the embodied CO_2 content of structural concrete Portland cement was partially replaced with GGBS and PFA in different proportions. Initially, twelve different concrete mixes were designed and are referred as Trial 1 mixes. Trial 1 mixes were designed to minimise the embodied carbon dioxide content of post-tensioned concrete without adversely affecting the construction programme. This section of the research was a part of a practical project by ARUP Consulting Engineers for the design of post tensioned concrete beams. The water/cement ratio was adjusted to achieve different strength concrete to meet the practical applications and to achieve the required workability, a superplasticiser was used. These mixes were designed to achieve the strength of 10 MPa after 16 hours and 25 MPa after 38 hours which was the requirement of Arup Consulting Engineers for the design of post tensioned concrete beams. The strength development of these concrete mixes was determined under simulated summer and winter conditions.

Seven concrete mixes from Trial 1 were further modified as Trial 2 mixes and were tested for further engineering properties. Concrete was prepared with different percentage of Portland cement with GGBS and PFA and its performance was checked. Concrete was prepared in the laboratory by using a concrete mixer and following the described in BS 1881-125 (1986).

3.3. Material Types & Properties

The following materials were used in the concrete mix.

3.3.1. Portland cement

The 52.5 N Portland cement (PC) used conformed to BS EN 197-1(2011) and was classified as CEM-I. The Portland cement was stored in the laboratory to avoid exposure to humidity and was purchased in the quantities required to avoid long storage and minimize the uptake of moisture. The typical physical and chemical properties of Portland cement given by the supplier are presented in Table 3.1 and Table 3.2 respectively.

Table 3-1 Physical properties of Portland cement

Surface Area	300-450 m^2/kg
Setting time initial	80-200 minutes
Apparant particle Density	3080-3180 kg/m^3
Bulk Density Aerated	100-1300 kg/m^3
Settled	1300-1450 kg/m^3

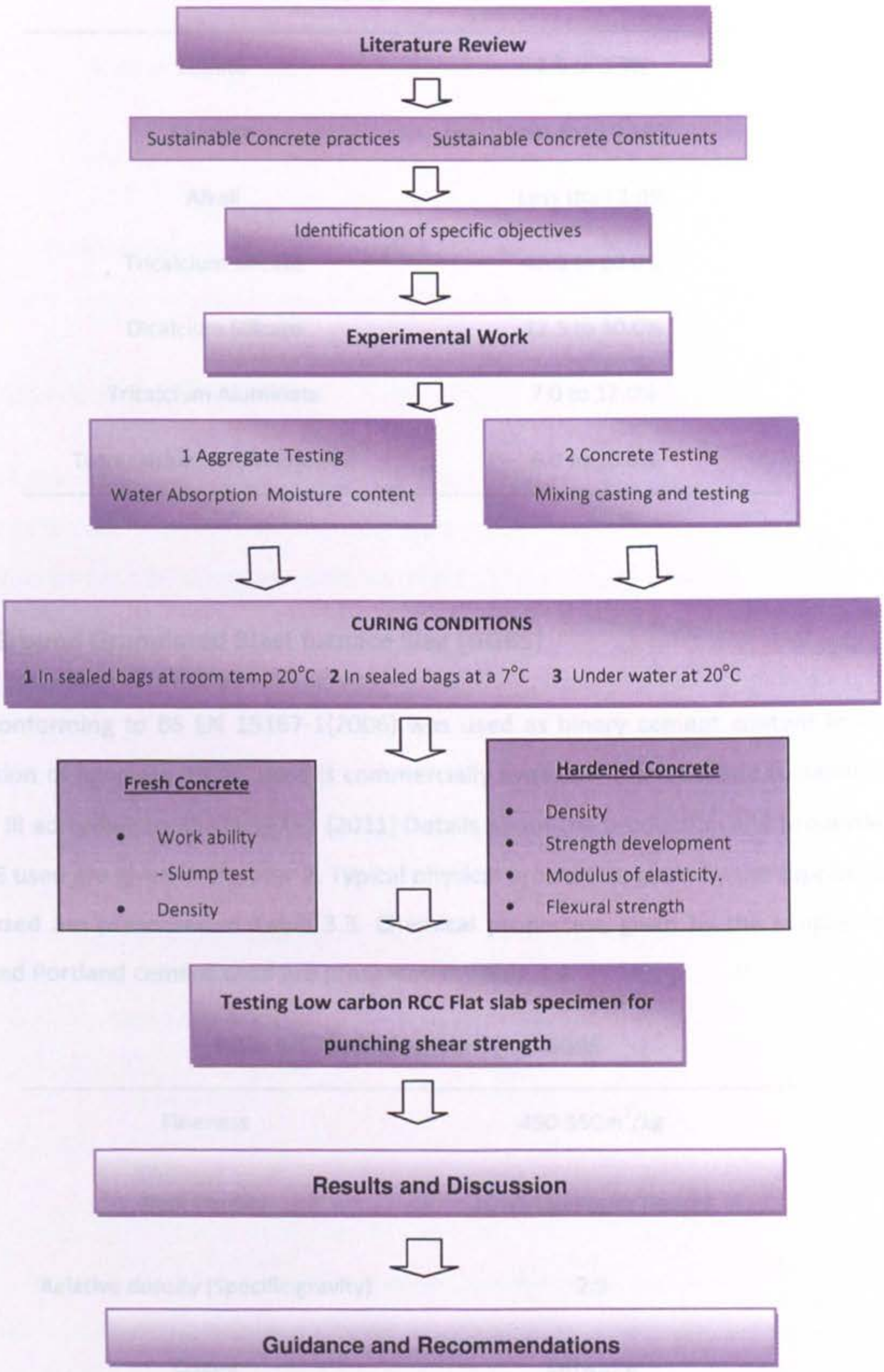

Figure 3-1 Flow chart showing Research programme

Table 3-2 Chemical properties of Portland cement

Sulfate	2.5 to 3.5%
Chloride	Less than 0.10%
Alkali	Less than 1.0%
Tricalcium silicate	40.0 to 60.0%
Dicalcium Silicate	12.5 to 30.0%
Tricalcium Aluminate	7.0 to 12.0%
Tetracalcium Aluminoferrite	6.0 to 10.0%

3.3.2. Ground Granulated Blast furnace Slag (GGBS)

GGBS conforming to BS EN 15167-1(2006) was used as binary cement content in the production of concrete. GGBS used is commercially available in the UK and is classified as CEM III according to BS EN 197 -1 (2011) Details about the production and properties of GGBS used are given in Chapter 2. Typical physical properties, given by the supplier of GGBS used are presented in Table 3.3. Chemical properties, given by the supplier of GGBS and Portland cement used are presented in Table 3.4.

Table 3-3 Physical Properties of GGBS

Fineness	450-550m^2/kg
Bulk Density	1000-1100kg/m^3(loose)
Relative density (Specific gravity)	2.9
Colour	Offwhite

Table 3-4 Chemical properties of GGBS & PC

Product	CaO	SiO_2	Al_2O_3	MgO	Fe_2O_3
GGBS	40%	35%	12%	10%	0.2%
Portland Cement	65%	20%	5%	1%	2%

3.3.3. Pulverised Fuel Ash (PFA)

PFA conforming to BS-EN 450-1(2012) was used as binary cement component in the production of concrete. PFA used in the concrete is commercially available in the UK and is classified as CEM IV according to BS EN 197-1 (2011). Typical physical and chemical properties, provided by the supplier of PFA used, are presented in Table 3.5 and Table 3.6 respectively.

Table 3-5 Physical properties of PFA

Odour	Virtually None
Particle Density (Specific Gravity)	1.8 to 2.4
Solubility in water	Less than 2%
Bulk density	1.1 to 1.7 g/cm^3
Alkalinity - pH	9 to 12 when damp
Dielectric Constant	1.9-2.6

Table 3-6 Chemical properties of PFA

Component	Typical % by weight
SiO_2	45 to 51
Al_2O_3	27 to 32
Fe_2O_3	7 to 11
CaO	1to 5
MgO	1 to 4
K_2O	1 to 5
Na_2O	0.8 to 1.7
TiO_2	0.8 to 1.1
SO_3	0.3 to 1.3
Cl	0.05 to 0.15

3.3.4. Water

The tap water was used for the production of concrete mixes.

3.3.5. Superplasticiser (SP)

ADVA 655 supplied by Grace Construction Ltd was used as a superplasticiser to achieve the required workability. The data sheet of the superplasticiser used in the experimental programme is given in the Appendix A.

3.3.6. Aggregates

Graded natural sand with a maximum particle size of 5 mm and complying with the requirements of BS EN 12620-1 (2009) was used as fine aggregate in the concrete mixes. Thames valley natural aggregate was used as coarse aggregate in the concrete mixes. The maximum size of the aggregate used was 20 mm.

3.4. Properties of Aggregate

3.4.1. Aggregate Particle density and water absorption

The particle density and water absorption of the aggregates was determined according to the pyknometer method described in BS EN 1097-6(2000). Sampling for the test was done according to BS EN 932-1(1997) and reduction was done according to BS EN 932-2(1999).

Apparent particle density ρ_a , particle density on an oven dried basis ρ_{rd}, particle density on a saturated and surface dried bases ρ_{ssd} and the water absorption were calculated in Mg/m^3 according to the expressions given in the following equations.

$\rho_a = \rho_w$ M4 /M4 – (M2 –M3)

ρ_{rd} = M4 / M1 – (M2 – M3)

ρ_{ssd} = M1 / M1 – (M2 – M3)

W_{A24} = 100*(M1 – M4)/M4

Where ρ_w, is the density of water at the test temperature in Mg/m^3

M1 is the mass of the saturated and surface dried aggregate in air in grams.

M2 is the mass of the pyknometer containing the sample of saturated aggregate and water in grams.

M3 is the mass of the pyknometer filled with water only in grams.

M4 is the oven dried mass of the aggregate in grams.

3.4.1.1. Coarse aggregates

The test portion was sieved and washed on 31.5 mm and 4 mm sieves to remove the finer and those coarser particles.

The prepared test sample was immersed in water at 22 ± 3 °C in a pyknometer and the entrapped air was removed by gently rolling and jolting the pyknometer in the tipped position.

The pyknometer was kept in the water bath at a temperature 22 ± 3 °C for 24 hours.

After 24 hours the pyknometer was removed from the water bath and the entrapped air was removed again and the pyknometer was over filled with water. The pyknometer was dried from the outside and was weighed (M2). The temperature of the water was recorded.

Aggregates were removed from the water and allowed to drain for a few minutes. The pyknometer was filled with water and weighed for M3. The temperature of water was recorded. The difference in temperature of water in the pyknometer during M2 and M3 weighing should not be more than 2 °C.

Drained aggregates were transferred onto a dry cloth in a tray and were surfaced dried. The aggregates were spread not more than one stone deep and exposed to the atmosphere away from sun light until all visible films of water were removed but the aggregates still had a damp appearance. The saturated and surface dry aggregates were weighed for M1. The aggregates were than oven dried in a ventilated oven at a temperature of 110 ± 5 °C until it reached a constant mass of M4.

The apparent particle density ρ_a, particle density on an oven dried basis ρ_{rd}, particle density on a saturated and surface dried bases ρ_{ssd} and the water absorption were calculated in Mg/m^3 according to the expressions given before. The aggregate water absorption test is shown in Figure 3.2.

Figure 3-2 Aggregate Water absorption test

3.4.1.2. Fine aggregates

The test portion was washed on a 4 mm and 0.063 mm sieves to remove the fine particles and those coarser than 4mm particles were discarded.

The test procedure for coarse aggregates was followed until the calculation of M3 and then the drying process, slightly different from the coarse aggregate.

After the calculation of M3 the soaked test sample was spread in a uniform layer in a tray. The aggregate was kept under gentle warm air to evaporate the surface moisture. Aggregates were stirred frequently to ensure uniform drying until no surface moisture can be seen and the aggregate particles do not adhere to each other. Aggregates were stirred and allowed to cool to room temperature. To check whether the surface dry state has been achieved the metal cone was placed with its larger diameter facing downwards in a tray. The cone was filled loosely with part of the sample and the tamper was used 25 times to lightly tamp the surface from the top. The mould was gently lifted and if the aggregate cone did not collapse, the drying procedure was continued until the collapse of the aggregate cone was achieved .The saturated and surface dry portion was weighed for M1 and then the aggregate portion was oven dried at a temperature of 105

± 5 °C until it has reached a constant mass of M4. The apparent particle density ρ_a , particle density on an oven dried basis ρ_{rd}, particle density on a saturated and surface dried bases ρ_{ssd} and the water absorption were calculated in Mg/m^3 according to the expressions for coarse aggregates given before. Aggregates drying for water absorption test are shown in Figure 3.3.

Figure 3-3 Aggregate drying in water absorption test

3.4.2. Water content of aggregates

As the aggregates were stored in the laboratory, the water contents of coarse and fine aggregates were determined before each mix to adjust the amount of water in the concrete accordingly. To determine the water content a sample of known weight of aggregates was taken in a tray and kept in the oven at 105 °C until it achieved a constant mass. From the difference between the two weights, the water content was determined. If the water content of the aggregate was more than the water absorption then the amount of water was reduced from the concrete mix equal to the difference between the water content and the water absorption and vice versa.

3.5. Concrete Mixing Procedure

Aggregates were stored in the laboratory at 20 °C and 35 %-55 % relative humidity. Adjustment was made to the concrete mix proportions to allow for water absorption of the aggregates.

A horizontal WinGET Crocker type concrete mixer, with a capacity of 198 kg was used for mixing the concrete. The concrete mixer used for making concrete is shown in Figure 3.4. The mixing procedure of concrete given in BS 1881- 125 (1986) was followed as outlined below.

- The mixer pan and paddles were lightly dampened.
- Coarse aggregate and fine aggregate were added and mixed for 30 seconds.
- Approximately half of the mixing water was added and mixed for 1 minute and then thoroughly by hand.
- The mix was left for 8 minutes to allow water absorption by aggregates.
- Cement was spread evenly over the aggregates and mixed for 1 minute. (In the silica fume mix, it was mixed separately with some of the mix water for 1 minute, prior to addition of cement to the concrete mixer).
- The paddles and mix material were cleaned thoroughly by hand.
- The remaining water was added (containing admixtures when used) and mixed for a further 2 minutes. The concrete was then mixed by hand to ensure homogeneity
- The slump test was carried out.
- After this concrete was put in the mixer and remixed for 30 seconds.
- The specimens were then casted.

- The mixer was cleaned thoroughly, ensuring all concrete was removed from paddles.

Figure 3-4 Concrete mixer

3.6. Curing Environments

Engineering performance of concrete cured under three different regimes was recorded. The following three methods were chosen for curing the concrete which have a close resemblance with the onsite curing environment were chosen.

3.6.1. Summer Curing Environment (C1)

After casting concrete in the moulds, it was stored for 24 hours at a laboratory temperature of about 20 ± 2 °C and covered with plastic sheets to minimize the loss of moisture. After 24 hours concrete was demoulded and sealed in air-tight plastic bags so that there is no loss of moisture and stored at a laboratory temperature of 20 °C. Concrete cubes cured under the C1 curing environment are shown in Figure 3.5.

Figure 3-5 Summer Curing Environment (C1)

3.6.2. Winter Curing Environment (C2)

After casting concrete, it was stored for 24 hours with in the moulds in the environmental chamber controlled at a temperature of 7 °C and 55 % relative humidity which resembles the normal winter temperature in the UK. Moulds were covered with plastic sheets to minimize the loss of moisture. After 24 hours, concrete was demoulded and sealed in the air-tight plastic bags to avoid any loss of moisture and stored in the environmental chamber controlled at 7 °C. Concrete cubes cured under the C2 curing environment as shown in Figure 3.6.

3.6.3. Water Curing Environment (C3)

After casting concrete in the moulds, it was stored at a laboratory temperature of 20 °C and was covered with plastic sheets. After 24 hours, concrete was demoulded and was immersed in the water chamber controlled at a temperature of 20 ± 2 °C. Concrete stored under curing environment C3 is shown in Figure 3.7.

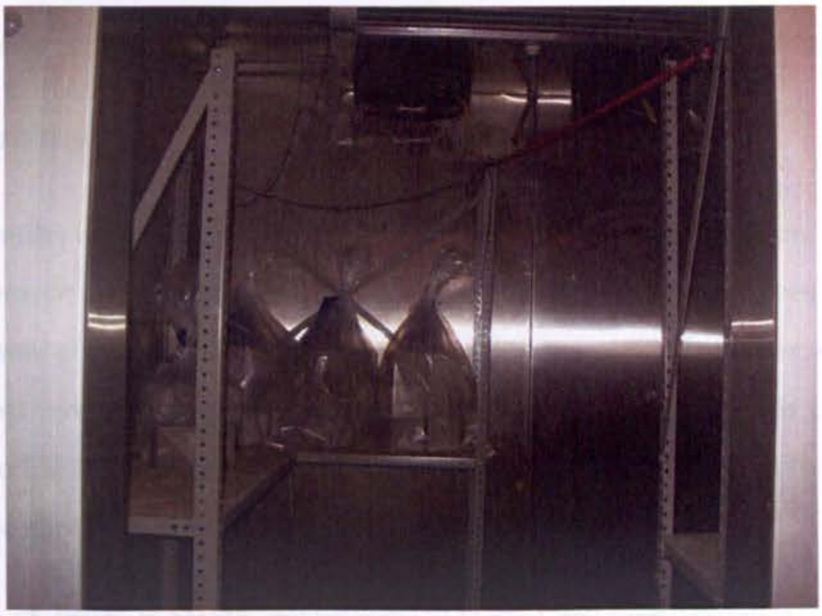

Figure 3-6 Winter Curing Environment (C2)

Figure 3-7 Water curing environment (C3)

3.7. Concrete Fresh Properties

The slump test was carried out on fresh concrete to provide a measure of its workability.

3.7.1. Workability

All concrete mixes were designed to achieve a maximum slump of 200 mm. The effect of the binary cement content on workability was established. The slump test was carried out immediately after the mixing process and additional superplasticiser was added as required to achieve the required slump of 200 mm. The slump test was carried out in accordance to EN 12350-2 (2009) and is shown in Figure 3.8. The aim of this test was to determine the consistency of fresh concrete.

The stability of fresh concrete mixes was observed through visual inspections. The Cohesiveness, handling and finishing properties of the concrete mixes were observed through visual inspections. The cohesiveness of fresh concrete was carried out on the resulting cone from the slump test. Slump cone was disturbed by tamping up to five times on one side with the help of a 16mm diameter tamping rod. The behaviour of the concrete was observed. Concrete surface was manually finished after one hour of placing the concrete into the form.

3.7.2. Fresh Density

The fresh density of concrete was determined just after mixing concrete for quality control purposes. Empty moulds were weighed before pouring concrete in to them. After pouring and finishing the concrete in the mould, it was weighed again. The difference between the two weights was divided by the volume of the cube mould to get the density.

Figure 3-8 Slump test

3.8. Concrete Hard Properties

3.8.1. Compressive strength development

The compressive strength of concrete was determined using 100mm concrete cubes and cylinder specimens of 150 mm diameter and 300 mm height. The specimens were loaded at a rate of 0.4 N/s until failure, following the method described in EN 12390-3 (2009). The strength development of the low carbon concrete made was tested at different ages and compared with Portland cement concrete to check the suitability of this in various practical applications. Concrete specimens were cured under the three different conditions explained in section 3.6. The compressive Cube Strength was tested on the 1st, 2nd, 3rd, 5th, 7th, 14th, 28th and 56th day. Cylinders were tested at the age of 28 days.

Two cube specimens of each mix and each curing regime were tested for compressive strength using an Avery Denison 2500 kN machine as shown in Figure 3.9. In the case of more than 10 % difference in two results a third specimen was also tested. The concrete samples cured under the regime C3 were dried at room temperature for three hours before testing.

Figure 3-9 Compressive strength Test (Cube and cylinder)

3.8.2. Flexural Strength

Although concrete is normally assessed for its compressive strength capacity, it is important to know the other strength aspects such as tensile strength. Tensile strength of concrete is usually between 10 % to 20 % of the compressive strength. Measurement of flexural strength is a way of establishing the tensile strength of concrete indirectly which is also expressed as its modulus of rupture (MR) in N/mm^2. Flexural strength is useful in assessing the quality of concrete pavements but is less important for structural concrete, where tensile capacity is mostly provided by steel reinforcement. Split cylinder test is also used for measuring the tensile strength of concrete but flexural strength test is more frequently used by the designers for establishing the tensile strength of concrete.

The flexural strength of the low carbon concrete was tested at the age of 28 and 56 days and compared with the Portland cement concrete. The flexural Test was carried out on 100 mm x 100 mm x 500 mm beams after curing for 28 days using the three different conditions explained in section 3.6. Nine samples were casted for each mix to be tested for flexural strength. Two specimens were tested of each concrete mix and each curing regime. In case of more than 10 % difference in the two results a third specimen was also tested. The flexural strength was calculated following the method described in EN 12390-5 (2009) using simple bending theory. The flexural strength test is shown in Figure 3.10.

Figure 3-10 Flexural strength test

3.8.3. Modulus of Elasticity

The static modulus of elasticity of concrete specimens was determined following the procedure described in BS 1881-121(1983). A step loading was applied on a standard cylinder up to a maximum of one third of the failure load. The change in length of the test specimen was recorded at each load step, and converted to strain (ε). The load at each step was converted to stress (δ). The stress-strain relationship was established and its slope gave the static modulus of elasticity expressed in kN/mm^2. A RUBICON-MAYES testing machine was used for determining the modulus of elasticity and it was operated by a computer programme to apply the load cycles at a precise rate. The testing machine setup with the specimen during testing is shown in Figure 3.11. The modulus of elasticity test was carried out on 150 mm diameter and 300 mm high cylinders specimens cured for 28 days under the conditions described in section 3.6. Two specimens were tested of each concrete mix and each curing regime. In the case of more than 10 % difference in two results a third specimen was also tested.

Figure 3-11 Modulus of elasticity test

3.9. Punching shear resistance test for flat slabs

For the practical application of low carbon concrete flat slabs were chosen because there was not sufficient literature available on the use of this concrete in these structural elements. As design against punching shear failure is often the main design criteria for flat slabs, slab specimens made with low carbon concrete were tested for punching shear strength in the laboratory. Details of the concrete mixes used for flat slab specimens and the experimental results are given in Chapter 5.

The literature on punching shear strength tests of flat slabs was reviewed and is presented in Chapter 2. As there was no facility available for testing the slabs at Kingston University, there was a need to assemble a testing rig for punching shear strength from scratch. The slab testing rig was designed in a way that was easily manageable for the laboratory facilities available.

Various factors were considered for the design of the test which included the size, casting, handling, curing and disposal of the slab specimen. Health and safety was one of the factors considered in the design of the testing rig.

After considering all factors, finally a sample size of 1.15 m x 1.15 m square and 120 mm thick with a central stub column of 200 mm x 200 mm was chosen and a testing rig was designed accordingly.

3.9.1. Testing rig set up

For setting up the rig, a suitable location in the research lab was chosen which had a strong RCC floor. Four holes of diameter 150 mm were drilled in the floor. For drilling the holes, a diamond-head core drill was hired to cut through the steel bars inside the floor. Final excavation of the holes was done, using a chisel and hammer to make a mushroom shape hole for achieving extra strength. The drilling of holes for testing rig is shown in Figure 3.12.

Figure 3-12 Drilling holes for the slab testing rig

Five steel bars of 12 mm diameter were inserted vertically in each hole to provide strength in shear. The inserts for studding of 25 mm were placed in each hole by using a specially designed steel frame shown in Figure 3.13. The holes were filled with high strength concrete (70MPa) and cured for 28 days before using the testing rig.

Figure 3-13 Steel frame for placing inserts in the holes

The slab test arrangement is shown in Figure 3.14. The slab specimen was loaded on four 25 mm diameter steel studs and the reaction was provided over a square area of 900 mm^2 by steel beams placed on top of the slab. Load was applied upward by means of a hydraulic jack in equal increments of 10 kN.

3.9.2. Preparation of Reinforcement Cage

The cage of reinforcement was prepared manually in the laboratory. The Nominal bars of 6 mm diameter were used as bottom reinforcement. These bars were bent from the sides so that the main tension reinforcement bars could be placed on top of them and tied together with binding wires. Twelve high yield 10 mm ribbed bars were used as tension reinforcement in both directions at 100 mm centres. The typical final cage for the slat slab specimen is given in Figure 3.15. Column bars were tied with the main tension reinforcement and three links of diameter 8 mm were used.

According to the clause 9.2.1.1 of BS-EN 1992-1-1 (1992), the minimum and maximum percentages of steel permitted in slabs is given as.

$A_{s,min} = 0.26\ f_{ctm}/f_{yk}\ x\ b_t\ d$

The value of f_{ctm} for the strength class used is calculated from Table 3.1 of BS-EN 1992-1-1(2004) and is equal to 2.8

So $A_{s,min} = 0.26 \times 2.8/460 \times 1000 \times 95 = 150\ mm^2/m$

$A_{s,max} = 0.04\ A_c = 0.04 \times 1000 \times 120 = 4800\ mm^2/m$

Area of steel provided (A_s) for 10 mm bars @ 100 mm c/c = 785 mm^2/m is OK

Figure 3-14 Set up for punching shear strength test of flat slabs

Figure 3-15 Reinforcement cage for slab specimen

3.9.3. Installation of Steel strain gauges

Micro measurements CEA series linear steel strain gauges supplied by Vishay Micro measurements were used to measure the strains in steel bars. Four strain gauges were installed on the two tension bars, two in each direction for measuring the strains in these. The location of the strain gauges on the steel bars was chosen in the maximum moment area which is around the perimeter of the column area. For the installation of steel strain gauges the procedure described by Vishay Micro measurements in the Application Note TT-611 (2007) was followed. The location on the steel bar where the strain gauge was to be installed was filed to achieve a smooth surface by using two different fineness files and sand paper. The fine surface was cleaned with the application of liquid provided by Vishay Micro measurements for installing the strain gauges and finally the strain gauge was bonded on the steel bar by means of M-bond 200 adhesive solution. After installing the gauge it was coated with M-coat A air drying polyurethane coating. The wires were soldered to the strain gauges using a soldering rod and the connectivity of the wires was checked with an ohm meter. Finally the strain gauge was covered with bitumen for protection. A strain gauge installed on the reinforcement bar is shown in Figure 3.16. Grid resistance of the strain gauges was 120 ± 0.3 % and the gauge factor was 1.2 ± 0.2.

Figure 3-16 Steel Strain Gauges

3.9.4. Installation of concrete strain gauges

KYOWA strain gauges were installed on the bottom concrete surface to measure the strain in concrete. The gauge Factor of the strain gauges was 2.13 ± 1 % and the gauge resistance at 24 °C and 50 % RH was 120.2 ± 0.2 ohms. Two strain gauges were installed on the concrete surface for calculating the strains in concrete.

The concrete strain gauges were installed after the slab specimen was cured, a day before the test day. Concrete strain gauges were installed at the points of maximum moment, around the perimeter of the column. Two strain gauges were installed on the perimeter of the column, perpendicular to the slab edges in each direction as shown in Figure 3.17. As the strain gauges were to be installed at the bottom surface of the slab, the slab was turned over. After this the preparation of the base for the strain gauges was made according to the procedure given in the Application Note TT-611 (2007) by Vishay Micro measurements, which included decreasing, conditioning, and making a smooth base for the strain gauges by filling the pores with M-bond adhesive resin type AE. The base for the strain gauges was cured for 24 hours at room temperature and then it was rubbed with sand paper until the base material was exposed. The base was cleaned and conditioned using the liquids provided. Finally the strain gauges were installed on the base in the required position by using M bond. M bond was cured for a couple of hours before the wires were attached to the strain gauges by soldering. After the wires were attached, a strong insulation tape was used to secure them while placing the slab on the testing rig.

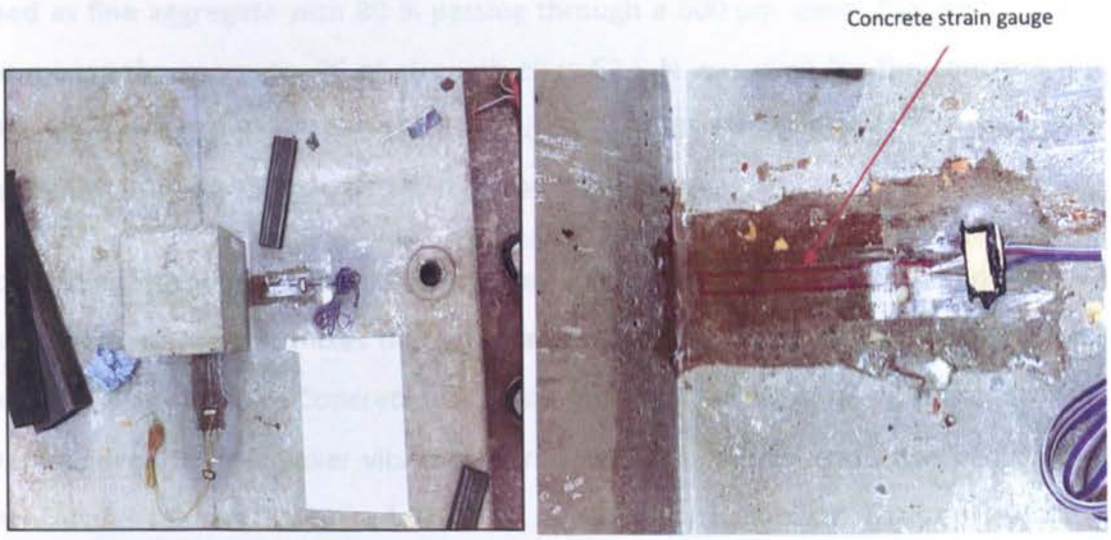

Figure 3-17 Concrete strain gauges

3.9.5. Preparation of Mould for slab specimen

A wooden mould was used for the slab specimen. The sides of the moulds were strengthened by steel sections in parts which were joined together with bolts and nuts. The mould was designed to have sufficient strength for holding the wet concrete and to provide ease in demoulding. The bottom of the mould was made with wooden board screwed to the sides. The bottom of the mould was replaced for each slab sample. Four inserts were placed in the correct locations for making the holes in the slab. These inserts were designed with a 10 mm thread at the bottom and 20 mm at the top. The bottom thread was made for holding the inserts in the correct position and the top thread so that the slab could be lifted up by making a suitable arrangement. The insert for making holes in the slab is shown in Figure 3.18. The mould prepared for the slab specimens served its purpose successfully without any leakage or breakage and is shown in Figure 3.19.

3.9.6. Casting of slab specimen

Concretes for the slab specimens were cast in the laboratory using the concrete mixer and were proportioned following the method specified in the BRE guide "Design of normal concrete mixes second edition". Thames valley natural aggregate as coarse aggregate with a maximum size of 20 mm was used. Thames valley natural sand was used as fine aggregate with 80 % passing through a 600 μm sieve. Tap water was used for mixing the concrete. PC of strength class 52.5 N was used for the concrete mixes. PFA conforming to BS-EN 450-1(2012) and GGBS to BS EN 15167-1 (2006) were used as cementitious materials to partially replace PC in concrete.

For casting the slab specimen the concrete mixer had to be used twice. To reduce the time between the two mixes the aggregates, cement and water were already weighed before starting the mix. Concrete was poured manually using small shovels and vibration was achieved using a poker vibrator. During the time the concrete was being vibrated the second mix was prepared and the mould was then filled. The top surface was levelled using a float and then the specimen was covered with a plastic sheet.

After casting the slab specimen, a third mix was prepared using the same proportions and cubes, cylinders and mini beams were casted. Cubes were tested at the 1st, 2nd, 3rd, 5th, 7th, 14th, 28th and 56th day to check the gain in compressive strength of concrete. Cylinders were tested on the 28th day for compressive strength. Mini beams were cast for determining the flexural strength. At least three persons were required on the casting day. In Figure 3.20, casting of the slab specimen is shown. With each slab specimen twenty four 100 mm cubes and three cylinders of 150 mm diameter and 300 mm height, were cast and tested to check for the rate of gain in compressive strength and to determine the strength of concrete at the day of the slab test. The rate of gain in compressive strength is very important for the use of concrete in structural applications which are subjected to high early loads or where early removal of formwork is required.

Figure 3-18 Inserts for making hole in the slab

Figure 3-19 Mould for the concrete slab specimen

Figure 3-20 casting of slab specimen

3.9.7. Curing of Slab Specimen

The curing process was kept simple to give close resemblance to real curing procedures and was kept the same for all slab specimens. Slab specimens, cubes, mini beams and cylinders were cured for at least seven days by manually sprinkling water on them once every day. The specimens were covered under plastic sheets at room temperature of about 20 °C. Water was sprinkled manually on both sides of the slab by using a soft water pipe with a sprinkling nozzle fitted to it for at least ten minutes each day.

3.9.8. Demoulding of slab specimen

Demoulding of the slab specimens was normally done on the 5th day after casting. First the sides of the moulds were unscrewed and the box of the column was unscrewed. Thereafter the mould was lifted up with the fork lift truck to remove its base. Finally the inserts for the holes were removed by striking them with a hammer. The demoulding process is shown in the Figure 3.21.

Figure 3-21 Demoulding of slab specimen

3.9.9. Disposal of slab specimen

After testing the slab the specimen was lifted by the fork lift truck and put in a skip for disposal. The bottom surface of slab specimen after failure is shown in Figure 3.22.

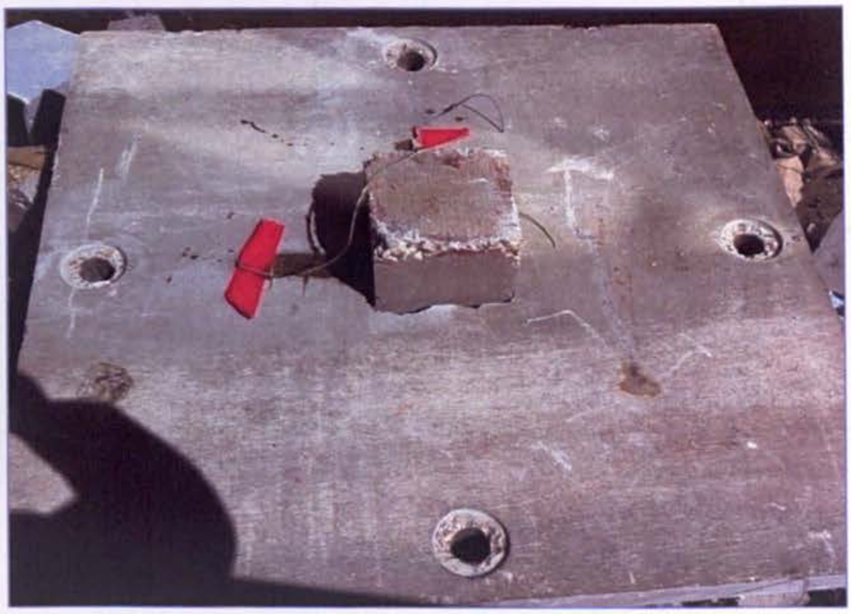

Figure 3-22 Bottom surface of slab after failure

3.9.10. Testing Rig Assembling and Test Procedure

The test arrangement is shown in Figure 3.23 with the slab specimen loaded for punching shear strength. Load was applied upwards by means of a hydraulic jack, in equal increments of 10 kN. At each load level, deflection and the cracking pattern was recorded. Before assembling the testing rig, the slab was lifted with the fork lift and its central stub column was put centrally on a wheelie trolley, which can move in every direction and then the final position of the slab was adjusted over the holes in the floor made for the rig. The steel rods were screwed in the insets of the holes in the floor after passing through the slab holes. After this, the slab was lifted on four jacks on the sides to position the main jack under the column.

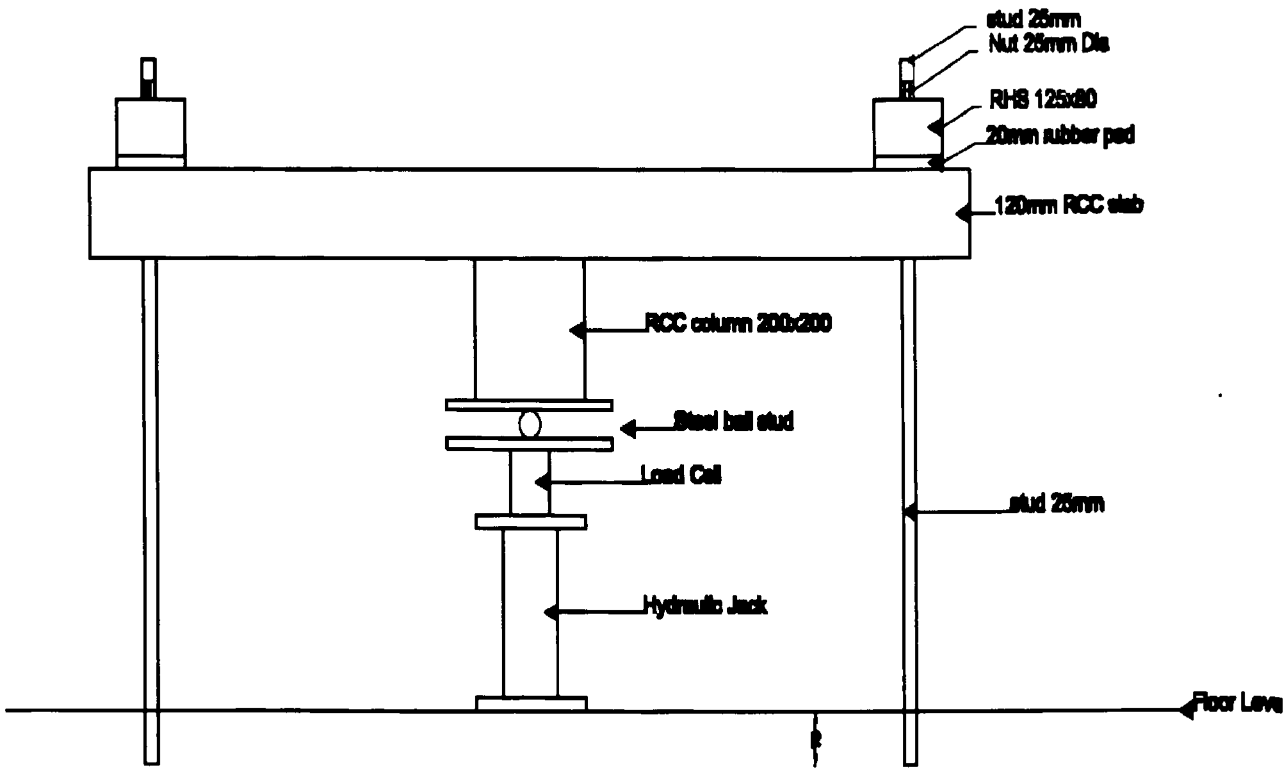

Figure 3-23 Testing rig arrangement for punching shear strength

On top of the hydraulic jack a load cell was placed to measure the load applied. On top of the load cell a steel ball stud was placed to minimise eccentricity and transfer the load to the column without any bending moment. It was made from two 200 mm square steel plates with grooves inside them for the steel ball.

Each slab specimen had holes drilled for rods, which were connected on to the steel sections placed over the slab to hold it down. This was meant to offer reactions on the sides of a square 990 mm x 990 mm, corresponding to the nominal lines of contraflexure at 495 mm from the centre of the column (0.2 L of the span). An additional steel plate was provided between the nut and the reaction steel sections to avoid punching of the nut into the steel section.

The slab was levelled by the adjustment of the screws of the steel rods on top of the slab and the level was checked with a spirit level.

Four linear variable differential transformers (LVDTs) were used to determine deflections near the column and the two supports in one direction for the calculation of average mid-span deflection as shown in Figure 3.24.

In the trial specimen the manual deflection gauges were used as well to check the accuracy of the LVDT's. The results of the LVDT's were similar to the manual deflection gauges so only LVDT's were used for the other samples.

Wires of the strain gauges and the LVDT's were connected to the strain gauge recording device as shown in Figure 3.25.

Finally the load was applied on the central jack with a pump in increments of 10 kN as shown in Figure 3.14. The readings for deflection and strain gauges were recorded manually until failure. Any appearances of cracks on the slab were marked and photographs were taken.

After testing, the slab the specimen was lifted by the fork lift truck and put in the skip for disposal.

Figure 3-24 Deflection gauges

Figure 3-25 Data logger for strain gauges and LVDT's

4. ENGINEERING PERFORMANCE OF BINARY CEMENT CONCRETE

4.1. Introduction

Engineering performance of binary cement concrete, where PC is partially replaced with various percentages of GGBS or PFA has been evaluated in this chapter and the results are compared with the engineering performance of PC concrete. Due to low early age strength gain of concrete containing GGBS and PFA, their use in the fast track construction and post tensioned concrete where they are exposed to high early age loads is limited. To overcome this problem, the water/cement ratio of the concrete produced was kept low to achieve higher early age and ultimate strength which also adds to the durability properties of concrete. To achieve the maximum workability, superplasticiser was used. Slump and fresh density of concrete were recorded for each mix and the results tabulated. Concrete made was cured under three different curing regimes described in Chapter 3, to check its suitability in practical applications. Strength development of concrete containing GGBS and PFA was recorded at 1,2,3,5,7,28 and 56 days under the different curing regimes. The Modulus of elasticity of concrete and Flexural strength was recorded at the age of 28 days.

4.2. Concrete Mix Proportions

To minimise the embodied CO_2 content of post tensioned concrete without adversely affecting the construction programme, the strength development of thirteen different mixes was determined under simulated summer and winter conditions in the first trial. This section of the research was a part of a practical project by ARUP Consulting Engineers for the design of post tensioned concrete beams. These trial concrete mixes were designed to achieve an equal 28 days compressive strength of 40 MPa and the strengths of 10 MPa after 16 hours and 25 MPa after 38 hours to meet the practical requirement of post tensioned concrete beams given by ARUP consulting Engineers. In these trial concrete mixes overall maximum water/cement ratio was kept 0.45 and overall minimum cement content 340 kg/m^3. The Free water content was kept between 150-170 litres/m^3. To achieve a practical level of workability and cohesion, suitable for pumping, concrete was designed for a target slump of 200 mm. A superplasticiser was used to minimise water and cement contents and achieve low free w/c ratio. Mix proportions and details of the trial mixes are presented in Table 4.1. Two batches of concrete were made for each concrete mix, to cast samples. Sixty 100 mm x 100 mm cubes were cast for each mix to measure the compressive strength development according to the British standard test method (BS EN 12390) at the age of 1,2,3,5,7,14,28 and 56 days cured under different regimes. Ten mini beams and cylinders were cast for each concrete mix, and tested for flexural strength and modulus of elasticity at the age of 28 days after curing under the different regimes.

From the compressive cube strength results of Trial 1 concrete mixes it was concluded that the requirement of 25 MPa after 38 hours was not achievable so the concrete mixes were further modified and Trial 2 concrete mixes were developed. In Trial 2 concrete mixes w/c ratio was further reduced to achieve the required strength of 25 MPa after 38 hours and it helped in the reduction of PC content for PFA concrete mixes. In Trial 2 concrete mixes twenty eight days compressive strength was not the requirement and the concrete mix design for PFA was designed on the basis of fixed free water content of 150 kg/m^3 for equal 28 days compressive strength. Mix proportions of the Trial 2 concrete mixes are presented in Table 4.2. It can be seen from the mix proportions that by using 30 % PFA there was a potential saving of 130 kg/m^3 of PC compared to the PC only concrete. By using 50 % GGBS there was a saving of 229 kg/m^3 of PC.

Table 4-1 Concrete Mix proportions (Trial1)

Mix	Free Water	Constituent Materials, Kg/m³ – Cement Constituents				Aggregates		w/c Ratio	Super plasticiser	Calculated Density
	Litres	PC	PFA	GGBS	SF	Coarse	Fine		ml/100kg cement	kg/m³
CEM I (100PC) @ w/c 0.45	170	380	-	-	-	1285	550	0.45	750	2385
CEM I (100PC), PC 375 kg/m3 @w/c 0.40	150	375	-	-	-	1370	520	0.40	500	2415
90PC/10PFA@w/c0.4	154	345	40	-	-	1360	515	0.40	525	2410
80PC/20PFA@w/c0.375	150	345	85	-	-	1230	635	0.375	925	2410
70PC/30PFA@w/c0.35	135	340	145	-	-	1180	635	0.35	1300	2425
80PC/20GGBS @ w/c 0.45	170	305	-	75	-	1285	550	0.45	500	2385
65PC/35GGBA @w/c0.45	160	245	-	130	-	1325	540	0.425	1100	2400
50PC/50GGBS@ w/c 0.4	160	200	-	200	-	1325	515	0.40	1300	2400
90PC/10PFA, PC 495 kg/m³ @ w/c0.37	180	445	50	-	-	1280	485	0.37	500	2375
70PC/30PFA, PC 550 kg/m³ @w/c 0.33	180	385	165	-	-	1230	445	0.33	1200	2375
90PC/10 Silica fume	170	345	-	-	35	1285	550	0.45	500	2385
CEM II/A- L (42.5)@w/c 0.40	152	380	-	-	-	1370	520	0.40	600	2415

Table 4-2 Concrete Mix Proportions (Trial2)

	CONSTITUENT MATERIAL kg/m^3								
Mix	Free water	Cement constituents			Aggregates		w/c Ratio	Super plasticiser	Density
	Litres	PC	PFA	GGBS	Coarse	Fine		ml/100kgPC	kg/m^3
90PC/10PFA	150	360	40	-	1325	540	0.375	1200	2415
80PC/20PFA	150	370	92	-	1310	495	0.325	1200	2415
70PC/30PFA	150	324	138	-	1310	495	0.325	1200	2415
70PC/30GGBS	160	320	-	137	1285	500	0.35	1200	2400
60PC/40GGBS	160	274	-	183	1285	500	0.35	1200	2400
50PC/50GGBS	160	229	-	228	1285	500	0.35	1200	2400
100PC-Control	160	457	-	-	1285	500	0.35	1200	2400

4.3. Workability/Slump

According to Neville (1995) "A concrete which can be readily compacted and can be easily placed without segregation is called workable concrete."

The slump test is used extensively around the world but it does not measure the workability directly. According to ACI 116R-90 (1990), it is a measure of consistency and is very useful in detecting variations in the uniformity of a mix of given nominal proportions. The slump test was performed according to the method described in BS EN 12350-2(2009) and is explained in Chapter3.

Concrete was designed for slump class S5 with a tolerance of +60 and -50 mm according to BS 8500-1(2006). Concrete was designed for the maximum workability and slump value of 200 mm. Due to the low water/cement ratio, the concrete loses its workability and to achieve the maximum workability a superplasticiser called ADVA 655 supplied by Grace Construction Ltd was used. The slump values recorded for different concrete mixes are presented in Table 4.3 and Table 4.4. It was observed that the required slump value of concrete can be achieved by adding the correct dose of superplasticiser. It can be observed from the results that the slump values of GGBS concrete increased with the addition level of GGBS content and are in line with the literature reviewed in Chapter3. Using the same water/cement ratio and similar quantities of superplasticiser the slump value of GGBS concrete increased with the GGBS level.

According to ACI 233R-95 (1996) the improved workability of GGBS concrete is attributed to the better cementitious particle dispersion and the surface characteristics of GGBS particles which are smooth and dense, and thus absorb little water during mixing.

As with GGBS concrete, slump values of PFA concrete increased with the addition level of PFA content. It is concluded that there is a significant increase in workability of concrete and it increases with the increase in level of PFA added to the concrete and it reduces the water/cement ratio. The plasticising effect of concrete containing PFA is increased and as concluded by Joshi et al (1997) is due to the spherical shape, smooth glassy texture and finer particle size distribution of PFA.

All the concrete mixes were found to be cohesive after stability observations and with the addition of GGBS and PFA, they did not require any additional water because all the slump results met the target values. Observations showed that the GGBS and PFA concrete mixes did not bleed and produced a workable concrete with no deleterious effect on finishing compared to the 100PC concrete. During the compaction of concrete, it was observed that there was no indication of segregation and there was no evidence of concrete bleeding. Handling involved concrete being transferred from the mixer using scoops into moulds and compacted. No special care was necessary for GGBS and PFA concrete mixes.

According to ACI 233R-95(1996) the improved workability of GGBS concrete is attributed towards the better cementitious particle dispersion and the surface characteristics of GGBS particles which are smooth and dense, and thus absorb little water during mixing.

4.4. Fresh Concrete Density

Fresh densities of each concrete mix were determined for quality control purposes and are tabulated in Table 4.3. Empty moulds were weighed before and after moulding concrete. The difference between the two weights was divided by the volume of the cubes to get the density of the wet concrete and in the same way densities of the specimens were calculated before testing for compressive strength at different ages.

The fresh density of Trial 1 concrete mixes with corresponding slump values are presented in Table 4.3 and for Trial 2 concrete mixes in Table 4.4. Fresh density results confirm the concrete produced as normal weight concrete which is in the range of 2375 kg/m^3 to 2420 kg/m^3.

Table 4-3 Fresh Density & Slump of concrete mixes (Trial1)

Mix	Fresh Density kg/m^3	Slump mm
CEM I(100PC)@w/c 0.45	2375	200
CEM I(100PC), PC 375 kg/m^3@w/c 0.40,	2405	205
90PC/10PFA@w/c 0.40,	2410	200
80PC/20PFA@w/c 0.35	2400	175
70PC/30PFA@w/c 0.28	2420	185
80PC/20GGBS@w/c 0.45	2380	225
65PC/35GGBS@w/c 0.425	2395	200
50PC/50GGBS@w/c 0.40	2400	200
90PC/10PFA, PC 495 kg/m^3@ w/c 0.37	2375	200
70PC/30PFA, PC 550 kg/m^3@ w/c 0.33	2375	180
90PC/10 silica fume @w/c 0.45	2380	210
CEM II/A- L (42.5)@w/c 0.40	2410	200

Table 4-4 Fresh Density & Slump of concrete mixes (Trial2)

Mix	Fresh Density kg/m³	Slump mm
90PC/10PFA	2410	185
80PC/20PFA	2410	190
70PC/30PFA	2415	200
70PC/30GGBS	2390	190
60PC/40GGBS	2395	195
50PC/50GGBS	2395	200
100PC-Control	2400	150

4.5. Hardened Concrete Density

The densities of concrete mixes were determined at different ages for quality assurance purpose. The density of concrete varies, depending on the amount and density of the aggregate, the amount of entrained air and the water and cement content. Density of a normal weight concrete is in the range of 2200 to 2400 kg/m^3. Hard densities of Trial 1 concrete mixes at different ages are given in Table 4.5 and hard densities of Trial 2 concrete mixes are presented in Table 4.6. Although concrete was cured in an environment where the loss of moisture was minimised, there is a slight reduction in the densities of concrete with the age which shows the loss of moisture from concrete. From the density results it can be seen that all the concrete mixes have densities in the range of 2310 kg/m^3 to 22360 kg/m^3 and represents the normal weight concrete.

Table 4-5 Hard Concrete Density of concrete mixes (Trial1)

	Density kg/m^3														
	Age at Test, days														
	1	2		3		5		7		14		28		56	
	Curing Condition														
	after demoulding	C1*	C2$^\pm$	C1*	C2$^\pm$	C1*	C2$^\pm$	C1*	C2$^\pm$	C1*	C2$^\pm$	C1*	C2$^\pm$	C1*	C2$^\pm$
CEM I(100PC)@w/c 0.45	2370	2365	2370	2365	2365	2350	2360	2340	2350	2330	2340	2310	2330	2300	2320
CEM I(100PC), 375 kg/m^3@w/c 0.40	2400	2390	2400	2380	2390	2370	2390	2365	2380	2355	2365	2340	2355	2330	2340
90PC/10PFA@ w/c0.4	2400	2390	2405	2385	2390	2380	2380	2360	2370	2360	2360	2355	2355	2340	2350
80PC/20PFA@ w/c 0.375	2390	2380	2390	-	-	2365	2375	2360	2365	2350	2360	2340	2355	2330	2345
70PC/30PFA@ w/c 0.375	2415	2405	2410	2390	2400	2380	2390	2370	2380	2365	2375	2360	2370	2350	2365
100PC/20GGBS@ w/c 0.45	2370	2360	2370	2350	2355	2345	2345	2340	2340	2320	2330	2310	2320	2300	2310
65PC/35GGBS@ w/c 0.425	2390	2380	2390	-	-	2360	2370	2350	2360	2340	2350	2330	2330	2300	2310
50PC/50GGBS@ w/c 0.4	2390	2380	2385	2370	2380	2360	2365	2360	2360	2350	2355	2330	2335	2310	2315
90PC/10PFA, 495 kg/m^3@ w/c 0.4	2370	2360	2370	2350	2365	2340	2350	2330	2335	2320	2325	2310	2315	2300	2305
70PC/30PFA 550 kg/m^3@ w/c 0.33	2360	2350	2340	2340	2335	-	-	2330	2330	2320	2320	2320	2315	2320	2310
90PC/10 silica fumes @w/c 0.45	2370	2365	2375	2365	2370	2360	2370	2350	2360	2340	2355	2335	2350	2325	2345
CEM II A-L 42.5,@w/c 0.4	2405	2390	2400	2375	2390	2365	2370	2350	2360	2340	2350	2320	2330	2300	2315

* C1 = Sealed room temperature curing (@ 20° C & 55% RH)

$^\pm$ C2 = Sealed winter curing (@ 7°C)

Table 4-6 Hard density of concrete mixes (Trial2)

Mix	Density, kg/m³													
	Age at Test, days													
	1		2		3		5		7		28		56	
	Curing Condition													
	C1*	C2±	C1*	C2±	C1*	C2±	C1*	C2±	C1*	C2±	C1*	C2±	C1*	C2±
90PC/10PFA	2390	2380	2380	2375	-	-	2370	2370	2360	2365	2355	2360	2345	2350
80PC/20PFA	2400	2410	2380	2385	2370	2375	2360	2365	2350	2355	2345	2340	2340	2350
70PC/30PFA	2400	2410	2370	2390	2350	2370	2340	2360	2335	2350	2320	2335	2310	2320
70PC/30GGBS	2380	2370	2370	2365	-	-	-	-	2360	2360	2355	2355	2340	2345
60PC/40GGBS	2380	2375	2370	2360	-	-	-	-	2350	2350	2340	2340	2330	2330
50PC/50GGBS	2390	2390	2380	2380	-	-	2370	2370	2360	2365	2355	2360	2345	2350
100PC-Control	2370	2360	2365	2360	2360	2355	2355	2350	2350	2350	2345	2340	2340	2335

* C1 = Sealed room temperature curing (@ 20° C & 55% RH)

± C2 = Sealed winter curing (@ 7°C)

4.6. Compressive Strength Development

Compressive Strength development of the concrete mixes cured under the three different regimes was determined on 100 mm x 100 mm cubes at 1,2,3 ,5,7,14,28 and 56 days. Compressive strength development of Trial 1 mixes is presented in Table 4.7 for curing environments C1 and C2. Compressive strength results of Trial 1 mixes at the age of 28 and 56 days for curing environment C3 are presented in Table 4.8.

In Trial 1 concrete mixes, the early age strength gain of concrete containing GGBS and PFA was slow compared to the PC concrete mixes and this decrease in strength increased with the replacement level, which is according to the literature reviewed in Chapter 2.

In mixes M6, M7 and M8 PC was replaced by 20 %, 35 % and 50 % respectively and the water/cement ratio was 0.425, 0.425 and 0.4 respectively. The twenty eight days and fifty six days compressive strength of these mixes were less than the PC concrete mixes, M1 and M2.

For M3, M4 and M5 PC was replaced by 10 %, 20 % and 30 % PFA respectively. The water/cement ratios were 0.4, 0.35 and 0.28 for M3, M4 and M5 respectively. The twenty eight days compressive strengths of M3 and M4 were less than the M2 (100PC) concrete mix but the twenty eight days compressive strength for M4 was slightly more than the M2 concrete mix and it gained more strength at the age of fifty six days. In M9 and M10 PC was replaced by 10 % PFA and 30 % PFA respectively. The total amount of cementitious content for M9 and M10 was 495 kg/m^3 and 550 kg/m^3 respectively. The requirement of compressive strength of 10 MPa after 16 hours and 25 MPa after 38 hours was satisfied by M9 and M10 under curing regime C1 and C3 but they could not satisfy this requirement under curing regime C2. Both M9 and M10 gained more strength than the M2 (100PC) concrete mix at the age of 28 days. By increasing the overall cementitious content the compressive strength of PFA mixes was increased which is according to the literature reviewed in Chapter 2. M11 and M12 could not satisfy the required strength requirements.

Curing environments have affected the strength development of all the mixes. The compressive strengths of concrete mixes cured under regime C2 (winter temperature at 7 °C) were reduced significantly compared to the regime C1 (summer temperature at 20 °C) and regime C3 (cured under water at 20 °C). Concrete specimens cured under regime C3 gained slightly more strength than the concrete specimens cured under regime C1 which is according to the literature reviewed in Chapter 2.

From the compressive cube strength results of Trial 1 concrete mixes it was concluded that the requirement of 25 MPa after 38 hours was not achieved for majority of concrete mixes, so the concrete mixes were further modified and Trial 2 concrete mixes were developed. The Trial 2 concrete mixes were designed for equal 28 days compressive strength of 60 MPa.

Compressive strength development of Trial 2 mixes is presented in Table 4.9 for the curing environments C1 and C2. The compressive strength results of Trial 2, at the age of 28 and 56 days for the curing environment C3 are presented in Table 4.10. It can be seen that all the concrete mixes of the Trial 2 cured under regime C1, except 50PC/50GGBS have satisfied the requirement of 25 MPa, compressive strength after 38 hours.

Table 4-7 Compressive cube strength development (Trial 1) for C1 and C2

Mix	Compressive Cube Strength, MPa														
	Age at Test, days														
	1	2		3		5		7		14		28		56	
	Curing Condition														
	after demoulding	C1*	C2±	C1*	C2±	C1*	C2±	C1*	C2±	C1*	C2±	C1*	C2±	C1*	C2±
CEM I(100PC), 380 kg/m³@w/c 0.45	14.0	26.0	21.0	31.0	25.0	39.5	35.0	41.5	36.0	44.5	39.5	50.5	43.5	54.5	45.0
CEM I,(100PC)375 kg/m³@ w/c 0.40	17.0	25.0	24.0	33.0	27.0	41.5	35.0	45.5	36.5	46.5	39.0	51.5	46.0	57.5	51.5
90PC/10PFA@ w/c 0.40	13.0	22.0	19.5	25.5	21.0	26.5	23.0	29.0	26.0	31.5	29.0	41.0	31.0	44.5	40.5
80PC/20PFA@ w/c 0.35	16.5	24.0	19.5	25.5	21.0	28.0	25.5	30.5	27.0	38.5	31.5	42.5	34.0	46.0	41.5
70PC/30PFA@ w/c 0.28	20.0	-	-	34.0	28.5	38.5	30.5	41.0	34.0	46.0	40.5	55.0	44.0	65.0	58.0
80PC/20GGBS@ w/c 0.45	11.5	19.5	14.0	23.0	17.0	28.0	19.0	30.0	22.5	35.5	26.5	39.5	33.5	43.0	39.5
65PC/35GGBS@ w/c 0.425	7.5	14.5	10.5	19.0	14.0	21.0	15.5	23.0	17.5	29.5	22.0	35.0	32.0	40.5	37.0
50PC/50GGBS@w/c 0.40	8.0	16.0	11.0	23.5	13.0	34.0	18.0	34.5	23.0	43.0	32.0	46.0	43.0	51.5	46.5
90PC/10PFA, PC 445 kg/m³@ w/c 0.37	18.0	29.5	25.0	37.5	29.0	39.5	32.0	42.0	35.0	45.5	42.0	52.0	49.0	57.0	54.5
70PC/30PFA, PC 440 kg/m³@ w/c 0.33	23.5	38.5	32.0	46.5	38.0	48.0	42.0	52.0	48.0	63.0	57.0	71.0	66.5	74.5	71.0
90PC/10silica fume@w/c 0.45	9.0	19.0	16.0	22.5	18.0	25.5	19.0	27.0	20.5	29.5	25.5	34.5	33.5	42.0	36.0
CEM II/A- L (42.5)@w/c 0.40	13.0	18.5	15.5	22.5	19.5	27.0	20.0	28.5	26.0	33.5	30.0	35.0	33.0	40.5	36.5

Table 4-8 Compressive cube strength for C3 curing regime (Trial 1)

MIX	Compressive cube strength MPa	
	Age Days	
	28	**56**
CEM I,(100PC)380 kg/m^3@w/c 0.45	51.5	56.0
CEM I(100PC) 375 kg/m^3@w/c 0.40	53.5	58.0
90PC/10PFA@ w/c 0.40	43.0	44.0
80PC/20PFA@ w/c 0.35	42.5	44.0
70PC/30PFA@ w/c 0.28	56.5	62.0
80PC/20GGBS@ w/c 0.45	41.5	46.5
65PC/35GGBS@ w/c 0.425	38.0	42.0
50PC/50GGBS@ w/c 0.40	51.0	54.0
90PC/10PFA, PC 490 kg/m^3@ w/c 0.37	52.5	58.0
70PC/30PFA, PC 550 kg/m^3@ w/c 0.33	56.0	61.0
90PC/10silica fume @w/c 0.45	37.0	42.5
CEM II/A- L (42.5)@w/c 0.40	34.5	40.0

Table 4-9 Compressive cube strength development (Trial 2) at C1 and C2

	Compressive Cube Strength, MPa															
	Age at Test, days															
	1		38 hours		2		3		5		7		28		56	
	Curing Condition															
	C1*	C2±	C1*	C2±	C1*	C2±	C1*	C2±	C1*	C2±	C1*	C2±	C1*	C2±	C1*	C2±
90PC/10PFA	26.5	4.0	38.0	17.0	46.0	26.0	48.0	31.0	54.5	47.0	59.6	50.5	68.0	59.5	72.5	68.0
80PC/20PFA	28.0	5.0	33.5	17.0	37.5	25.0	42.0	33.0	51.5	44.0	53.5	46.5	67.5	57.0	75.0	61.0
70PC/30PFA	21.5	10.0	30.0	15.5	36.0	19.5	41.0	24.5	48.5	42.0	52.0	44.5	68.0	61.0	73.0	64.0
70PC/30GGBS	24.5	9.0	32.0	17.0	37.0	22.5	49.5	31.0	-	-	56.5	46.0	68.5	57.0	74.0	69.0
60PC/40GGBS	18.5	3.5	30.0	12.0	38.5	18.0	45.5	24.5	-	-	58.5	40.5	71.5	62.0	81.5	68.0
50PC/50GGBS	9.0	1.5	20.5	6.0	28.5	9.5	-	-	46.0	23.5	53.5	28.5	68.0	55.0	73.0	62.5
100PC-Control	43.5	13.0	49.5	23.0	54.0	30.0	58.0	38.0	-	46.0	67.0	56.5	69.0	64.5	70.5	70.0

* C1 = Sealed room temperature curing (@ 20° C & 55% RH)

± C2 = Sealed winter curing (@ 7°C)

Table 4-10 Compressive cube strength for C3 curing regime (Trial2)

	Compressive cube strength MPa	
	Test Age days	
Concrete Mix	28 days	56 days
90PC/10PFA	71.0	79.5
80PC/20PFA	68.5	76.0
70PC/30PFA	70.0	74.5
70PC/30GGBS	72.0	75.0
60PC/40GGBS	72.0	82.0
50PC/50GGBS	68.0	74.5
100PC-Control	77.0	79.0

4.6.1. Compressive Strength development of GGBS concrete (Trial2)

Compressive strength development for GGBS concrete mixes under the C1 curing regime (summer temperature 20 °C) is compared with the PC concrete mix and presented in Figure 4.1. It can be seen from Figure 4.1 that all of the concrete mixes have nearly the same 28th day strength but there is a greater increase in the compressive strength of 60PC/40GGBS than the other mixes at Day 56 and all concrete mixes containing GGBS had higher strength than the PC only concrete at this age. It is concluded that the concrete containing 30 %, 40 % and 50 % GGBS gains more strength than the PC concrete after the age of 28 days which is according to the literature of Khatib and Hibbert (2005) reviewed in Chapter 3. At the age of Day 56 the strengths of 70PC/30GGBS, 60PC/40GGBS and 50PC/50GGBS are 5 %, 15.5 % and 3.5 % higher than the 100PC-Control concrete mix respectively.

Early age strength of GGBS concrete mixes is lower than the PC concrete mix. There is not much difference in the strengths of concrete at Day 7. At Day 1 concrete mixes containing GGBS have lower strengths compared to the PC concrete mix. Except 50PC/50GGBS, all of the other concrete mixes cured in the C1 environment had strengths in the range of 18 to 43 MPa at the age of one day which is enough to be used in fast track construction. 50PC/50GGBS concrete had strength of 28 MPa at the age of two days when cured under the summer temperature (C1) of 20 °C.

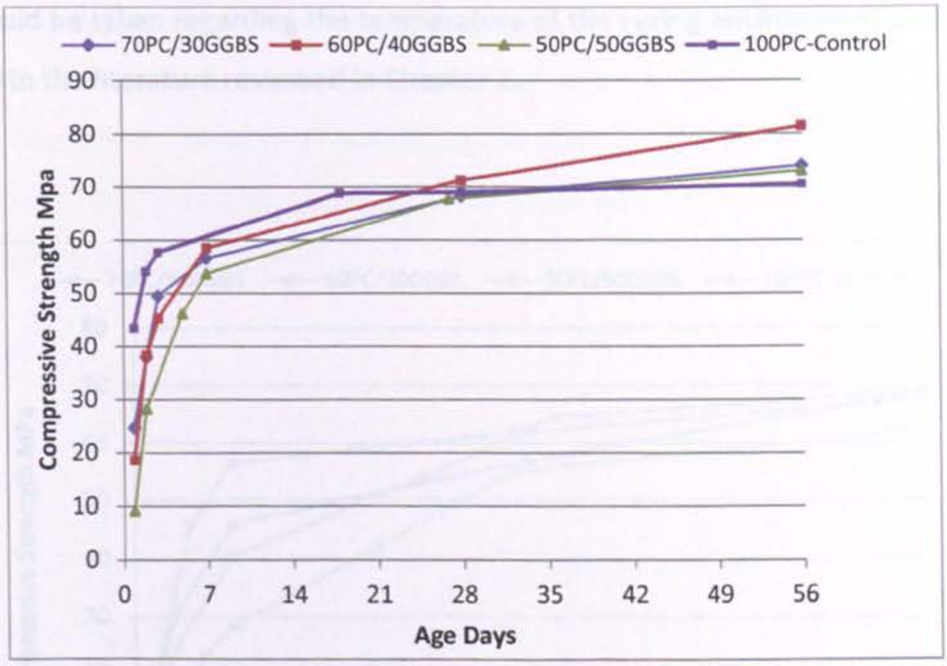

Figure 4-1 Compressive strength development of GGBS concrete cured at C1 environment

The strength development of GGBS concrete under curing regime C2 (winter temperature (7 °C) is presented in Figure 4.2. The compressive cube strength at the age of 28 days is similar for 60PC/40GGBS and 100PC-Control concrete mixes and is slightly lower for 70PC/30GGBS and 50PC/50GGBS concrete mixes because of the lower curing temperature. At the age of 56 days all concrete mixes have similar strength except the 50PC/50GGBS concrete mix has slightly lower strength than the other concrete mixes, but higher than the 100PC-Control mix. It can be seen in Figure 4.2 and Table 4.9 that the 28th day strength of the concrete mixes reduced for curing regime C2 and is in the range of 57 MPa to 64.5 MPa as compared to 68.5 MPa to 71.5 MPa for curing regime C1. The compressive cube strength for GGBS concrete mixes cured in C2 environment at the age of one day are less than 10 MPa compared to the control mix C strength of 13 MPa. At the curing environment C2, 70PC/30GGBS and 60PC/40GGBS concrete mixes gained enough strength of 23 MPa and 18 MPa respectively at the age of 2 days but 50PC/50GGBS concrete gained strength of 17 MPa at the age of 3 days. It was observed that curing regime C2 has reduced the first day strength for GGBS concrete slightly more than the PC concrete. It is concluded that in winter for GGBS concrete up to 50 % special care should be taken regarding the temperature of the curing environment at early ages as noted in the literature reviewed in Chapter 2.

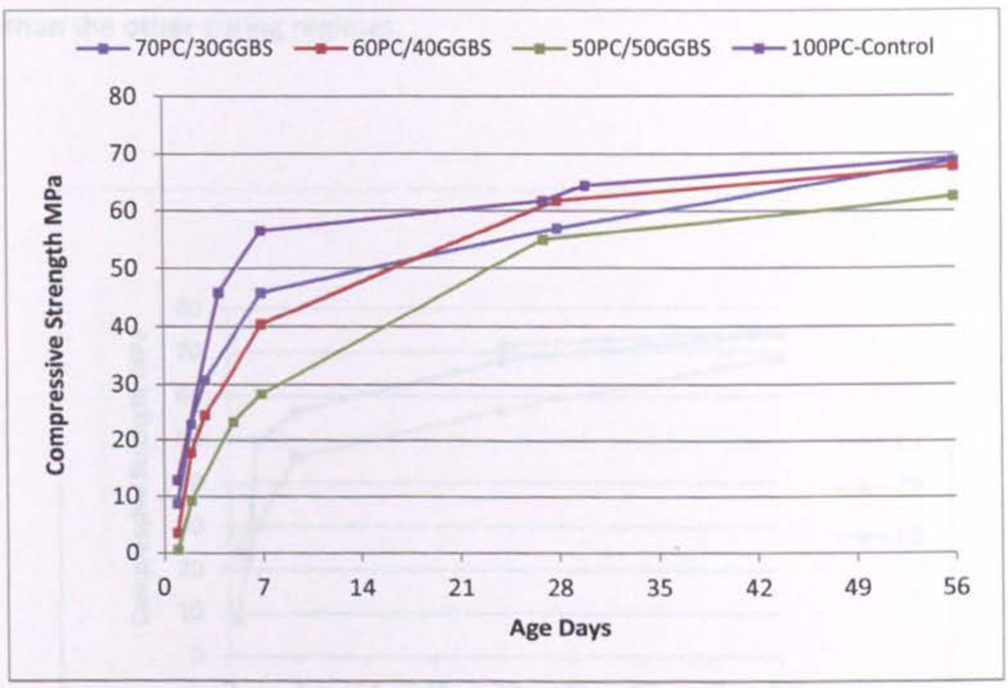

Figure 4-2 Compressive strength development of GGBS concrete cured at C2 environment

The strength development for 70PC/30GGBS, 60PC/40GGBS, 50PC/50GGBS and 100PC-Control concrete mixes under different curing regimes are compared in Figure 4.3 to Figure 4.6 respectively. In all concrete mixes the strength development under curing regime C2 is lower than the strength development under curing regime C1 and C3. As in curing regime C1, concrete was cured in sealed plastic bags to minimize the loss of moisture, all the concrete mixes have nearly the same twenty eighth day strength as that of the concrete mixes cured under regime C3, except the 100PC-Control concrete mix which has gained more strength at the age of 28 and 56 days under regime C3 than the other curing regimes. From these results, it can be concluded that at the curing temperature of 20 °C for GGBS concrete mixes there is not much difference in the ultimate strength if it is cured in sealed bags to minimize the loss of moisture or cured under water. For PC concrete mixes the ultimate strength is higher if it is cured under water than the other curing regimes.

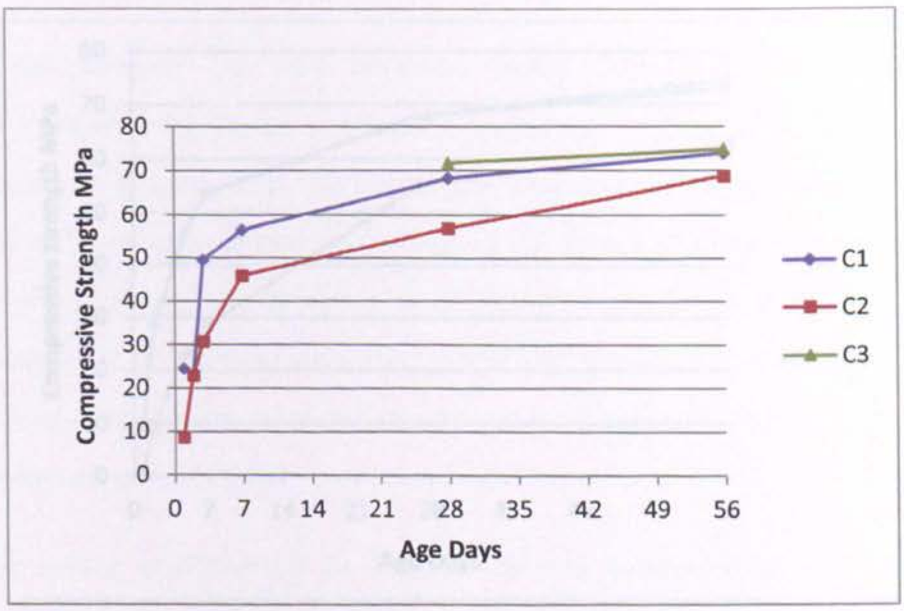

Figure 4-3 Compressive strength development of 70PC/30GGBS concrete mix

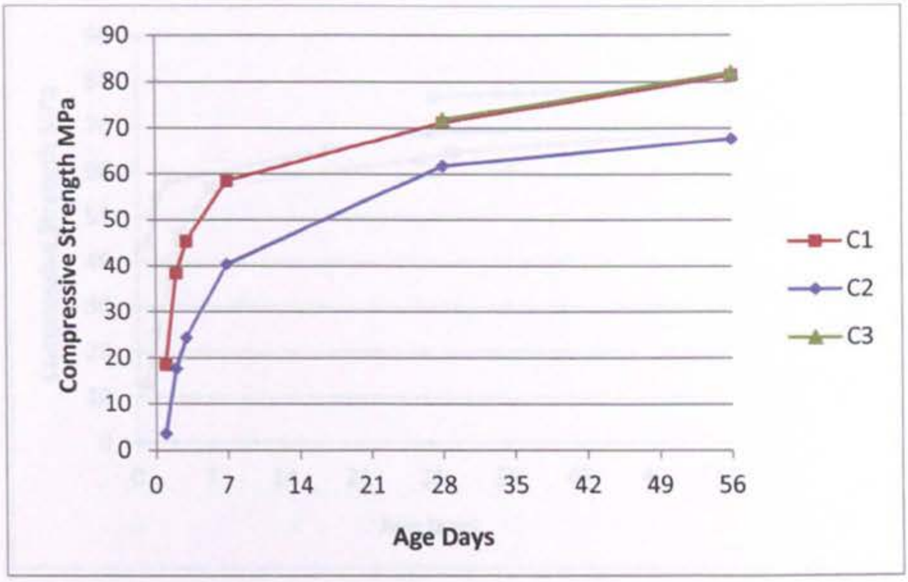

Figure 4-4 Compressive strength development of 60PC/40GGBS concrete mix

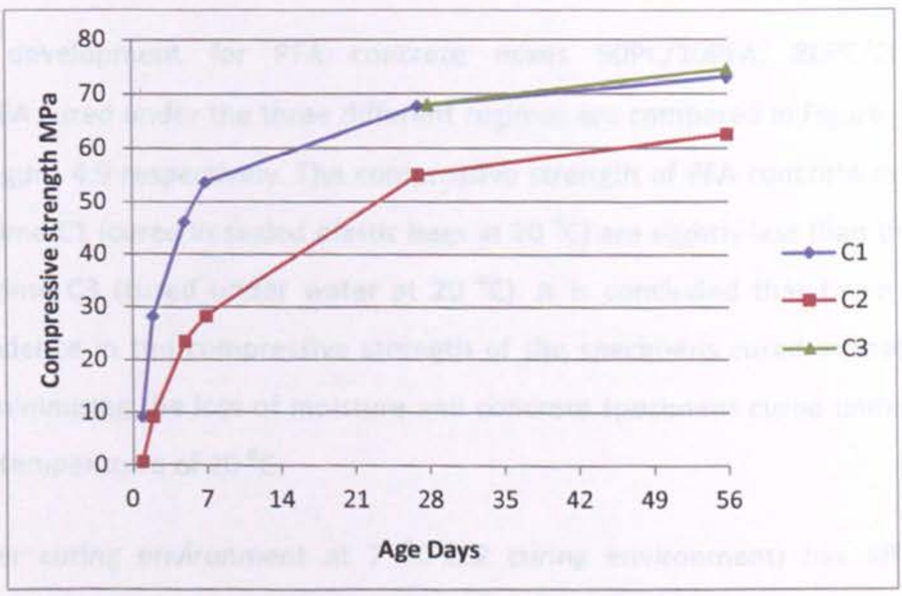

Figure 4-5 Compressive strength development of 50PC/50GGBS concrete mix

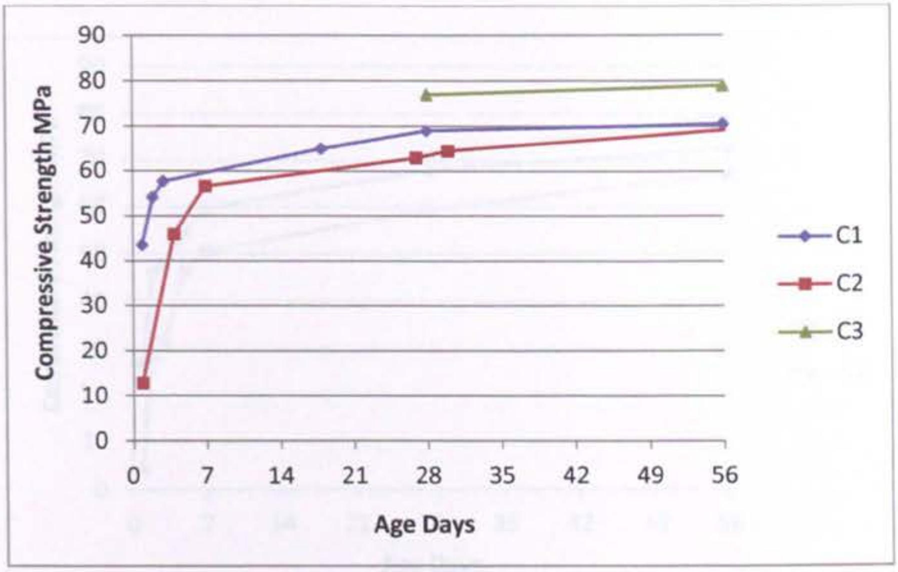

Figure 4-6 Compressive strength development of 100PC-Control concrete mix

4.6.2. Compressive Strength Development for PFA concrete (Trial2)

Strength development for PFA concrete mixes 90PC/10PFA, 80PC/20PFA and 70PC/30PFA cured under the three different regimes are compared in Figure 4.7, Figure 4.8 and Figure 4.9 respectively. The compressive strength of PFA concrete mixes cured under regime C1 (cured in sealed plastic bags at 20 °C) are slightly less than those cured under regime C3 (cured under water at 20 °C). It is concluded that there is a close correspondence in the compressive strength of the specimens cured in sealed plastic bags for minimising the loss of moisture and concrete specimens cured under water at the same temperature of 20 °C.

The winter curing environment at 7 °C (C2 curing environment) has affected the strengths of all the concrete mixes including 100PC-Control concrete mix but has reduced the strength of PFA concrete more than the 100PC-Control concrete mix. In winter it is recommended that the curing temperature of the PFA concrete be kept at least 20 °C at early ages to gain enough strength to be used in the fast track construction and to enhance the long-term strength of PFA concrete, an extended curing time is required.

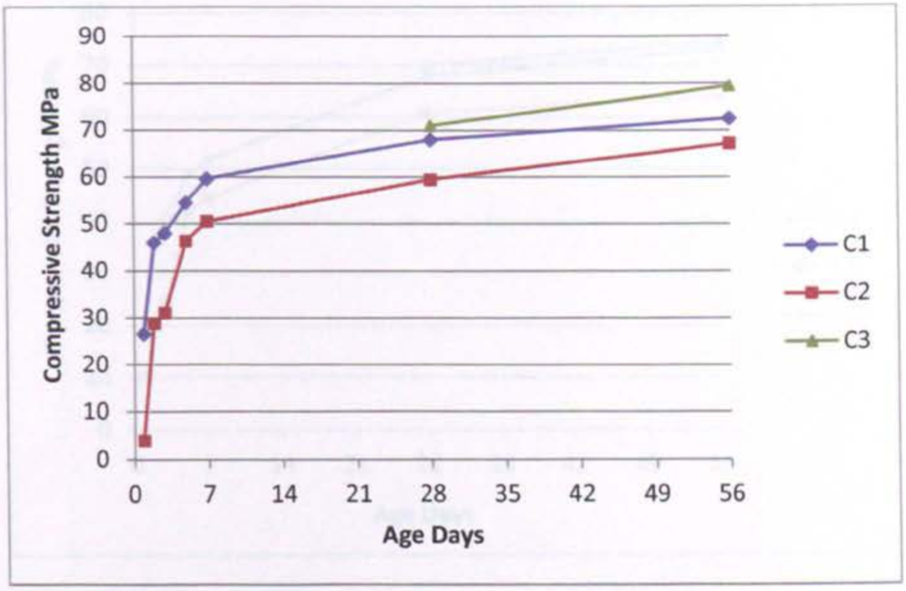

Figure 4-7 Compressive strength development of 90PC/10PFA concrete mix

Figure 4-8 Compressive strength development of 80PC/20PFA concrete mix

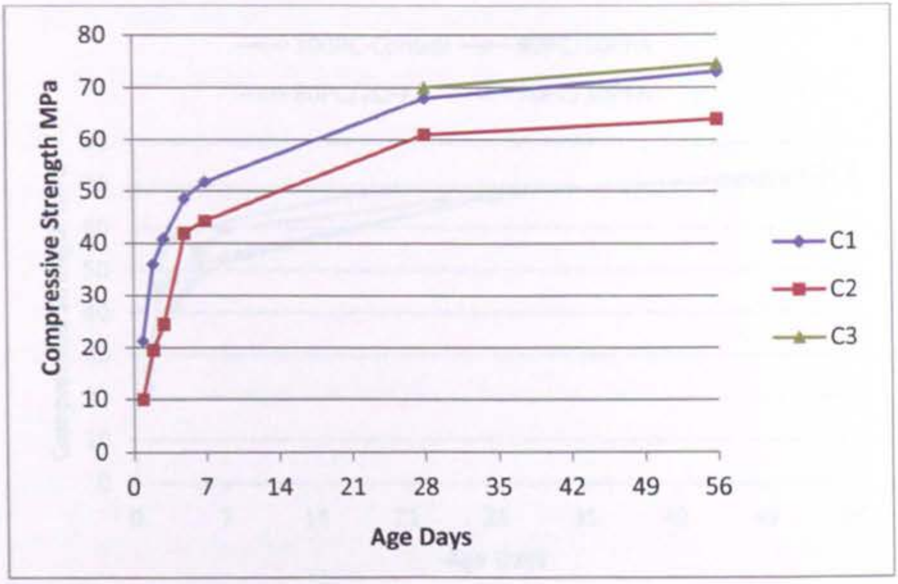

Figure 4-9 Compressive strength development of 70PC/30PFA concrete mix

In Figure 4.10 the strength development of PFA concrete mixes has been compared with the PC concrete mix cured under regime C1 (20°C). PFA concrete has shown similar behaviour as to that of GGBS concrete with the early age strength development slow due to the slow pozzolanic reactions between PFA and the lime ($Ca(OH)_2$) generated by the PC concrete. Under curing regime C1 the strength of 90PC/10PFA, 80PC/20PFA and 70PC/30PFA concrete mixes are 26.5 MPa, 28.0 MPa and 21.5 MPa respectively at the age of one day compare to 43.5 MPa for the, 100PC-control concrete mix.

There is a close correspondence in the twenty eight days compressive strength of PFA concrete mixes cured under the C1 regime and that of the PC concrete which is according to the findings of Dhir et al (1984), reviewed in Chapter 2. As expected PFA concrete mixes have gained more strength than the PC concrete at the age of 56 days which is according to the research work reviewed in Chapter 2. The 80PC/20PFA concrete has the highest compressive strength at the age of 56 days and is 6.5 % higher than the 100PC-Control concrete mix.

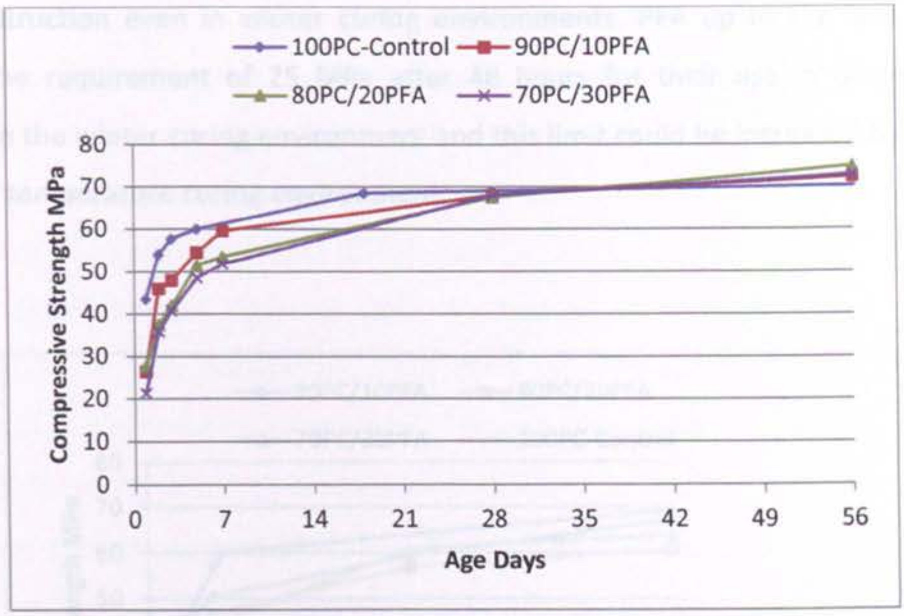

Figure 4-10 Compressive strength development of PFA concrete for C1

Strength development of PFA concrete mixes is compared with the 100PC-Control concrete mix cured under regime C2 (7 °C) in Figure 4.11. It can be seen in Figure 4.11 that curing regime C2 has affected the early age strength of both PFA and PC concrete mixes. Compressive strength of concrete at the age of 28 days is reduced by 7 % (for100PC-Control) to 19 % (for 50PC/50GGBS) for different concrete mixes when cured under the winter environment compared to the summer curing environment. There is a slight reduction in the twenty eighth and 56th day strength for PFA concrete mixes compared to the 100PC-Control concrete mix cured under the C2 regime. For curing regime C2 the early age strength of PFA concrete mixes is lower than the 100PC-Control concrete mix. For concrete cured in C2 curing environment strengths of 90PC/10PFA, 80PC/20PFA and 70PC/30PFA concrete mixes is 26.0 MPa, 25.0 MPa and 19.5 MPa respectively at the age of two days and are 7.6 %, 12.9 % and 43.6 % respectively lower than the 100PC-Control concrete mix. The compressive strengths of PFA concrete mixes at the age of two days cured under the winter curing environment are more than the minimum strength (10 MPa) requirement for soffit formwork removal. It can be concluded that PFA up to the level of 30 % can be used in concrete required for fast track construction even in winter curing environments. PFA up to the level of 20 % satisfies the requirement of 25 MPa after 48 hours for their use in post-tensioned concrete in the winter curing environment and this limit could be increased by providing a summer temperature curing environment.

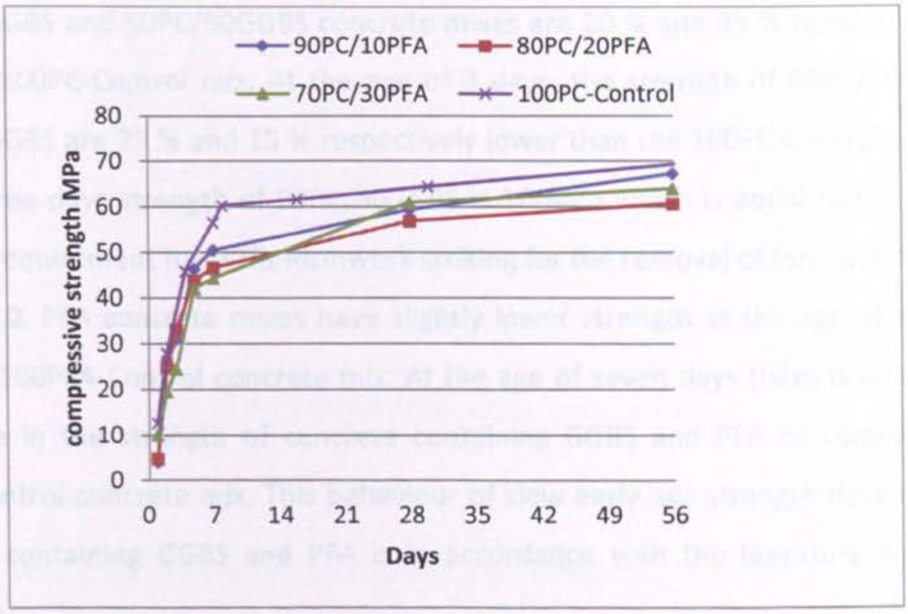

Figure 4-11 Compressive strength development of PFA concrete mixes for C2

4.7. Compressive strength development & Form work removal time

The compressive cube strength development of concrete mixes, designed for twenty eight days characteristic strength of 30 MPa for the testing flat slab specimens, given in Table 5.1 in Chapter 5, is presented in Table 4.11.

The time requirement for striking formwork, as per BS 8110 part-1 (1997) is 4 days for soffit formwork to slabs and 12 hours for vertical formwork (temperature to be 16 °C and above). “Shorter periods can be used if the concrete strength is at least 10 N/mm2 (in-situ) or twice the stress to which the slab is likely to be subjected, whichever is the greater provided that the striking at that time would not result in unacceptable deflections." The compressive strength development of different concrete mixes for slab specimens at different ages is shown in Figure 4.12. Figure 4.12 shows that concrete containing GGBS and PFA have lower early strength than the 100PC-Control concrete mix and this reduction in strength increases with the level of GGBS and PFA in concrete. At the age of one day the strength of 70PC/30PFA and 80PC/20PFA is about 15 % lower than the 100PC/Control concrete mix. At the age of one day, the strength of 60PC/40GGBS and 50PC/50GGBS concrete mixes are 20 % and 35 % respectively lower than the 100PC-Control mix. At the age of 3 days, the strength of 60PC/40GGBS and 70PC/30GGBS are 25 % and 15 % respectively lower than the 100PC-Control mix. At the age of three days strength of 50PC/50GGBS is 10 MPa which is equal to the minimum strength requirement for soffit formwork striking for the removal of formwork according to BS 8110. PFA concrete mixes have slightly lower strength at the age of three days than the 100PFA-Control concrete mix. At the age of seven days there is no significant difference in the strength of concrete containing GGBS and PFA as compare to the 100PC-Control concrete mix. This behaviour of slow early age strength development of concrete containing GGBS and PFA is in accordance with the literature reviewed in Chapter 2.

At the age of 28 days all of the concrete mixes have nearly the same strength with only slight differences. The 60PC/40GGBS and 70PC/30PFA have 10 % and 15 % higher strengths at the age of 28 days than the 100PC-Control mix. At the age of 56 days concrete containing GGBS and PFA gained more strength than the 100PC-Control concrete which is according to the literature of Johari et al (2011) reviewed in Chapter 2. 50PC/50GGBS and 70PC/30PFA have 15 % and 20 % higher strength respectively than the 100PC-Control mix.

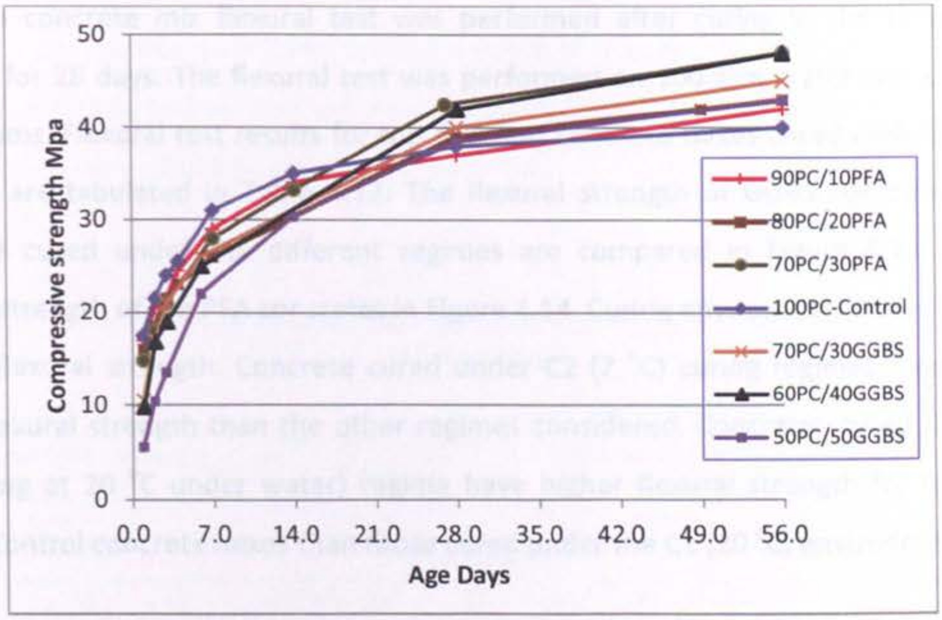

Figure 4-12 Compressive strength development

Table 4-11 Compressive cube strength development of flat slab concrete

MIX	Compressive Cube Strength MPa							
	Age at test, days							
	1	**2**	**3**	**4**	**7**	**14**	**28**	**56**
100PC-Control	17.5	21.5	24.0	25.5	31.0	35.0	38.0	40.0
90PC/10PFA	17.0	20.5	21.4	24.5	29.0	34.0	37.0	42.0
80PC/20PFA	16.0	18.5	21.5	23.0	26.0	-	39.0	43.0
70PC/30PFA	14.8	18.0	21.0	-	28.0	33.3	42.0	48.0
70PC/30GGBS	10.5	17.0	21.0	-	26.0	-	40.0	45.0
60PC/40GGBS	9.8	16.9	19.0	-	25.0	-	42.0	48.0
50PC/50GGBS	5.5	10.4	13.5	-	22.0	30.3	38.5	43.0

4.8. Flexural strength

For each concrete mix flexural test was performed after curing in the three curing regimes for 28 days. The flexural test was performed on 100 mm x 100 mm x 500 mm mini beams. Flexural test results for the different concrete mixes cured under different regimes are tabulated in Table 4.12. The flexural strength of GGBS concrete and PC concrete cured under the different regimes are compared in Figure 4.13, and the flexural strength of the PFA concretes in Figure 4.14. Curing environments have an effect on the flexural strength. Concrete cured under C2 (7 °C) curing regimes have slightly lower flexural strength than the other regimes considered. Concretes cured under the C3 (curing at 20 °C under water) regime have higher flexural strength for GGBS and 100PC-Control concrete mixes than those cured under the C1 (20 °C) environment.

The 60PC/40GGBS concrete mix gained slightly more flexural strength than the other concrete mixes cured under different curing regimes. The 70PC/30GGBS and 50PC/50GGBS concrete mixes have slightly higher flexural strength than the 100PC-Control concrete mix, which was expected according to the literature reviewed.

Curing environments have an effect on the flexural strength of PFA concrete mixes and the flexural strength of PFA concrete mixes are reduced after being cured under the C2 regime compare to the concrete cured under other regimes. The 80PC/20PFA concrete mix has the highest flexural strength compared to the other concrete mixes cured in the C1 environment and is 11.5 % more than the flexural strength of 100PC-Control concrete mix. The 90PC/10PFA and 70PC/30PFA concrete mixes have slightly higher flexural strength than the 100PC-Control concrete mix cured under the C1 regime. PFA concrete mixes have slightly lower flexural strength than the 100PC-Control concrete cured under environment C3.

The winter temperature curing environment, C3 reduces the flexural strength of PFA concrete. In order to enhance the flexural strength of concrete, curing time needs to be extended for a long time and the temperature of the curing environment needs to be kept at least 20 °C which is according to the literature reviewed in Chapter 2. It is concluded that the concrete mixes designed for equal twenty eight days strength, the use of GGBS up to 50 % and PFA up to 30 % slightly increase the twenty days flexural strength in comparison to PC only concrete, which is according to the research of Khatib and Hibbert (2005) and Solanki and Pitroda (2013), reviewed in Chapter 2 and is due to the better microstructure and packing of concrete.

Table 4-12 Twenty eight days cube/cylinder compressive strength, flexural strength& modulus of elasticity

Compressive Cube Strength			Compressive Cylinder Strength			Flexural Strength			Modulus of Elasticity		
MPa			MPa			MPa			GPa		
Curing Conditions											
C1	C2	C3	C1	C2	C3	C1	C2	C3	C1	C2	C3
90PC/10PFA											
68.0	59.5	71.0	55.0	50.0	58.5	6.0	5.0	5.5	42.0	40.0	42.8
80PC/20PFA											
67.5	57.0	68.5	53.0	48.5	49.0	7.0	5.0	6.5	41.5	39.1	42.2
70PC/30PFA											
68.0	61.0	70.0	50.5	42.5	43.0	6.0	6.0	6.5	39.1	37.5	39.6
70PC/30GGBS											
68.5	57.0	72.0	56.5	49.0	58.5	6.5	6.0	7.0	40.0	38.5	40.5
60PC/40GGBS											
71.5	62.0	72.0	57.0	48.0	58.0	6.5	6.0	7.0	40.5	39.0	41.0
50PC/50GGBS											
68.0	55.0	68.0	53.0	47.5	54.0	6.5	6.0	7.0	40.5	38.5	40.0
100PC-Control											
69.0	64.0	77.0	56.0	55.0	57.5	6.0	6.0	7.0	39.8	38.5	39.8

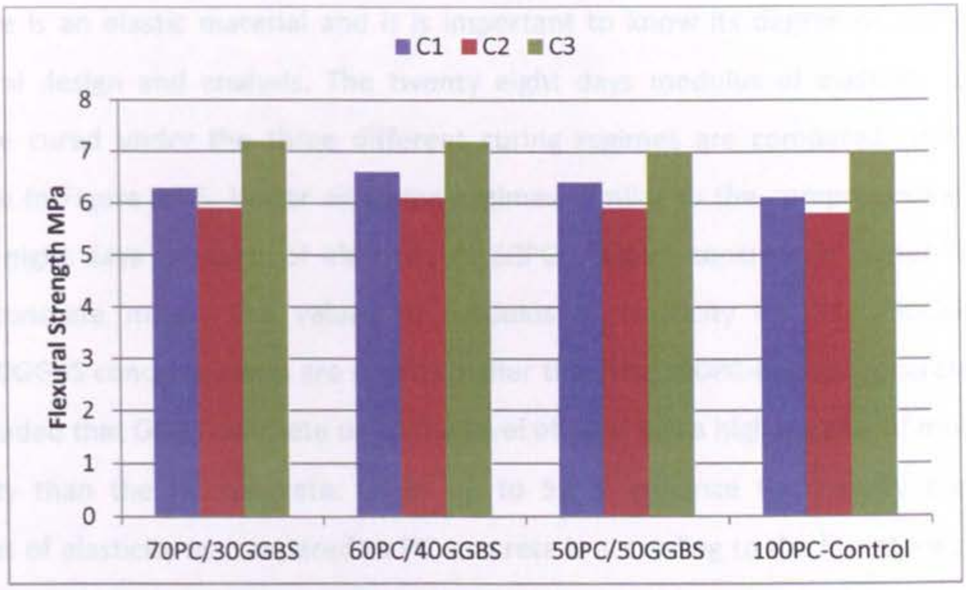

Figure 4-13 Flexural strength of GGBS concrete at the age of 28 days

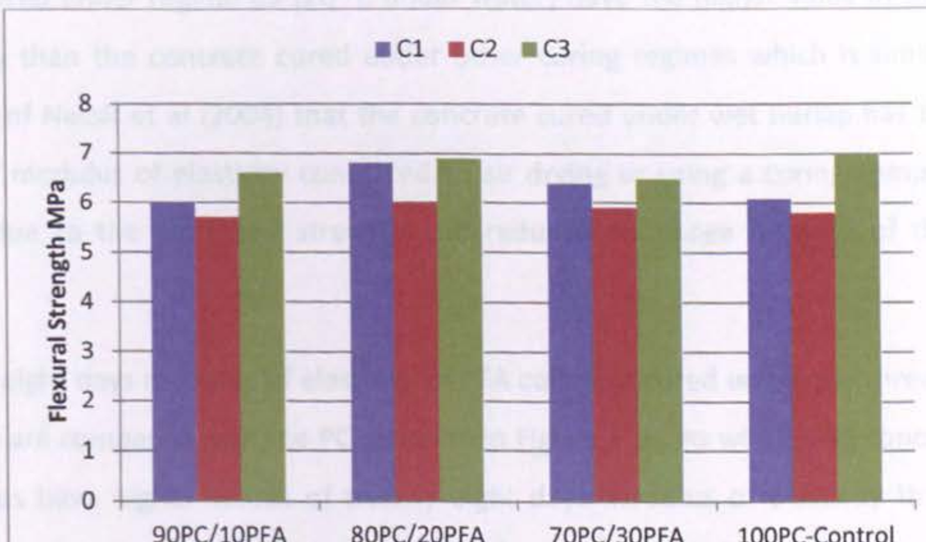

Figure 4-14 Flexural strength of PFA concrete at the age of 28 days

4.9. Static Modulus of Elasticity

Concrete is an elastic material and it is important to know its degree of elasticity for structural design and analysis. The twenty eight days modulus of elasticity of GGBS concrete cured under the three different curing regimes are compared with the PC concrete in Figure 4.15. Under all curing regimes, similar to the compressive strength, twenty eight days modulus of elasticity of 60PC/40GGBS concrete is higher than the other concrete mixes. The values of modulus of elasticity for 50PC/50GGBS and 70PC/30GGBS concrete mixes are slightly higher than the 100PC-Control concrete mix. It is concluded that GGBS concrete up to the level of 50 % has a higher value of modulus of elasticity than the PC concrete. GGBS up to 50 % enhance the twenty eight days modulus of elasticity as compared to PC concrete is according to the literature of Johari et al (2011) reviewed in Chapter 2 and is due to the better microstructure development and cementitious reactions of GGBS in concrete.

The curing regime C2 (7 °C) has an adverse effect on the twenty eight days modulus of elasticity of GGBS and PC concrete, similar to the compressive strength values. Concrete mixes cured under regime C3 (20 °C under water) have the higher value of modulus of elasticity than the concrete cured under other curing regimes which is similar to the findings of Nassif et al (2004) that the concrete cured under wet burlap has the higher value of modulus of elasticity compared to air drying or using a curing compound and this is due to the increased strength and reduced shrinkage because of the use of burlaps.

Twenty eight days modulus of elasticity of PFA concrete cured under the three different regimes are compared with the PC concrete in Figure 4.16. As with GGBS concretes, PFA concretes have higher values of twenty eight days modulus of elasticity than the PC concretes. Compared with the 100PC-Control concrete mix, the value of twenty eight days modulus of elasticity increased by 8 % for 90PC/10PFA concrete mix and 6 % for 80PC/20PFA concrete mix cured in the C1 (20°C) environment. The 70PC/30PFA concrete mix has nearly a similar value of modulus of elasticity as that of the 100PC-Control concrete mix. The increase in the modulus of elasticity of PFA concrete is in accordance with the literature of Nassif et al (2005) reviewed in Chapter 2. According to their observations up to 30 % replacement of PC with PFA in concrete will have the same value of twenty days modulus of elasticity and the replacement below 30 % will enhance the modulus of elasticity. This is due to the better microstructure and packing of concrete with more fine particles of PFA.

The curing regime C2 (7 °C) has affected the values of modulus of elasticity for all concrete mixes. The modulus of elasticity of PFA concrete mixes and the PC concrete mixes cured under C2 curing environment are reduced.

All the concrete mixes cured under curing regime C3 (20 °C under water) have higher values of modulus of elasticity than the other curing regimes. The results of the C1 curing regime are slightly lower than the C3 curing regime. It is concluded that proper curing of PFA concrete under water at 20 °C (C2 curing) or by the prevention of loss of moisture (C1 curing) at 20 °C enhances the modulus of elasticity.

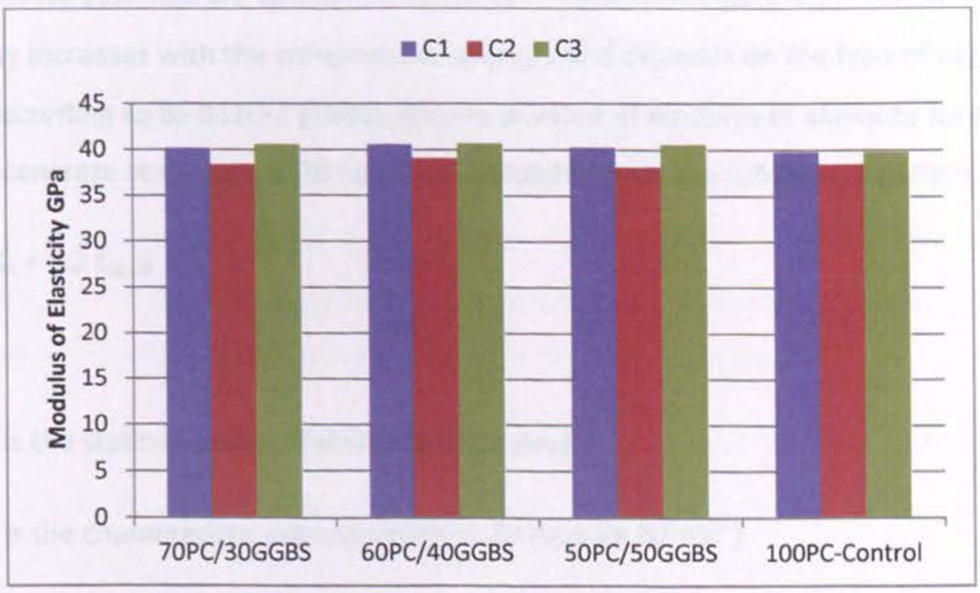

Figure 4-15 Modulus of elasticity of GGBS concrete at the age of 28 days

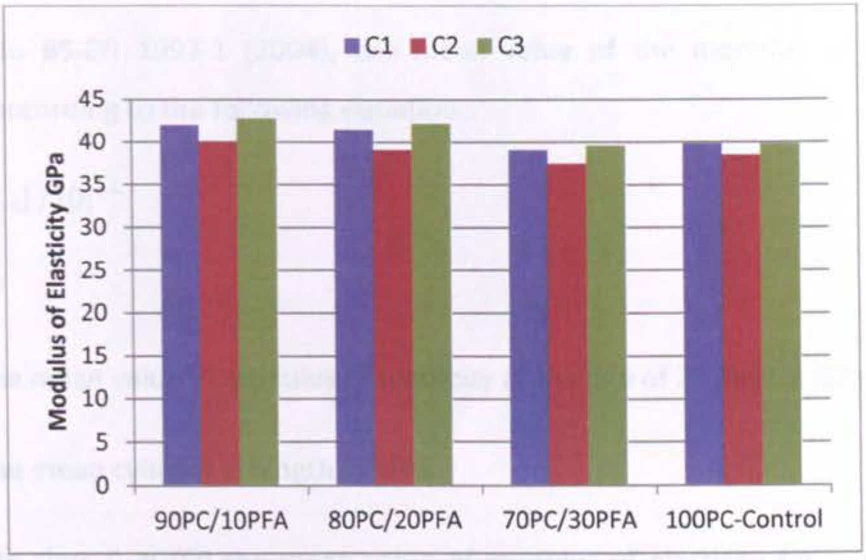

Figure 4-16 Modulus of elasticity of PFA concrete at the age of 28 days

The modulus of elasticity values for different concrete mixes, at the age of 28 days are compared with the compressive cube strength in Figure 4.17. Compressive cube strengths for each mix are written on top of each column in Figure 4.17. The modulus of elasticity increases with the compressive strength and depends on the type of aggregate used. According to BS 8110-2 (1985), the mean value of modulus of elasticity for normal weight concrete at the age of 28 days, are calculated from the following equation.

$E_{c,28} = K_o + 0.2\, f_{cu,28}$

Where

$E_{c,28}$ is the static modulus of elasticity at 28 days.

$f_{cu,28}$ is the characteristic cube strength at 28 days (in N/mm^2)

K_o is a constant closely related to the modulus of elasticity of the aggregate (taken as 20 kN/mm^2 for normal-weight concrete).

For strength class C 40/50 the mean value of modulus of elasticity for normal weight concrete will be as follows.

$E_{c,28} = 20 + 0.2 \times 50 = 30$ GPa

According to BS-EN 1992-1 (2004), the mean value of the modulus of elasticity is estimated according to the following equation.

$E_{cm} = 22\ [(f_{cm})\ /10]^{0.3}$

Where

E_{cm} is the mean value of modulus of elasticity at the age of 28 days in GPa.

f_{cm} is the mean cylinder strength in MPa.

For strength class C 40/50 the mean value of modulus of elasticity for normal weight concrete is calculated as follows, according to BS-EN 1992-1 (2004).

$E_{cm} = 22\ [(40)\ /10]^{0.3} = 33.4$ GPa

The estimate of the mean value of modulus of elasticity for normal weight concrete, according to BS-EN 1992-1 (2004), is slightly more than the estimate of BS 8110-2 (1985).

For 100PC-Control mix, the twenty eighth Day, compressive cube strength is 68.0 MPa and the compressive cylinder strength is 56.0 MPa according to Table 4.12. The elastic modulus of elasticity will be 33.6 GPa according to BS-EN 1992-1 (2004) and 36.9 GPa according to BS-EN 1992-1 (2004). The modulus of elasticity determined in the laboratory is 39.8 GPa, which is slightly more than these estimates.

In Table 4.13 the experimental values of modulus of elasticity, estimates of BS 8110-2 (1985) and BS EN 1992-1 (2004) are presented. It can be seen that the experimental values of modulus of elasticity are slightly higher than estimates of the codes of practices. The ratio of cylinder /cube strength is also presented in the Table 4.13 and is in the range of 0.75 to 0.84.

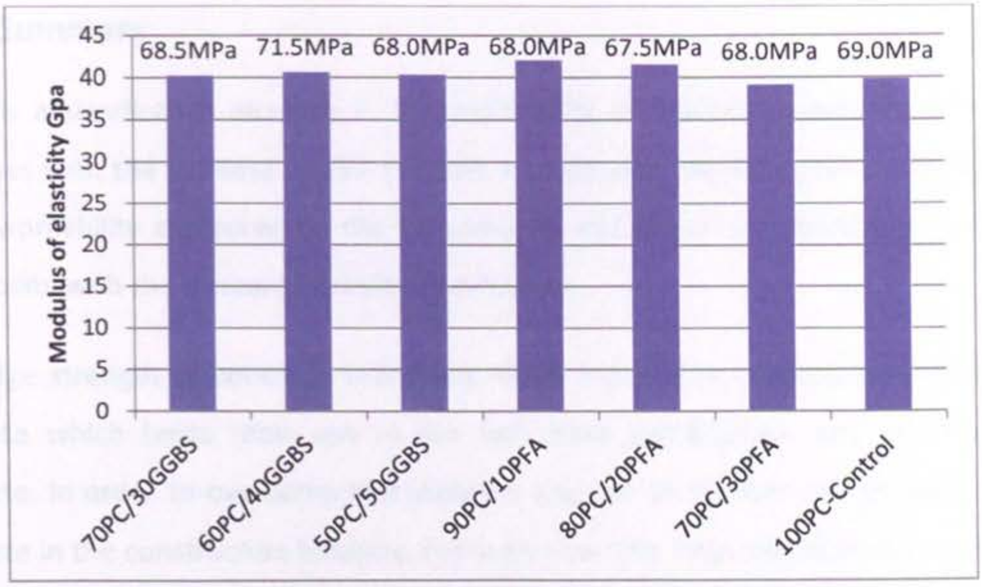

Figure 4-17Modulus of elasticity and compressive cube strength

Table 4-13 Modulus of elasticity estimates

Concrete mixes	Cube strength MPa	Cylinder strength MPa	Cylinder/Cube strength	Experimental Modulus of elasticity GPa	BS8110 estimates GPa	BSEN1992-1 estimates GPa
70PC/30GGBS	68.5	56.5	0.83	40.0	33.5	37.0
60PC/40GGBS	71.5	57.0	0.84	40.5	34.5	37.0
50PC/50GGBS	68.0	53.0	0.78	40.5	33.5	36.5
90PC/10PFA	68.0	55.0	0.8	42.0	33.5	36.5
80PC/20PFA	67.5	53.0	0.75	41.5	33.5	36.5
70PC/30PFA	68.0	50.5	0.75	39.0	33.5	36.0
100PC-Control	69.0	56.0	0.8	40.0	34.0	37.0

4.10. Summary

There is a significance increase in the workability of concrete containing PFA and it increases with the increase of PFA content in concrete. Concrete containing GGBS has more workability compared to the PC concrete and there is a significant increase in workability with the increase in replacement level.

Early age strength of concrete containing GGBS and PFA is reduced compared to PC concrete which limits their use in the fast track construction and post-tensioned concrete. In order to overcome this problem and due to the demand of high strength concrete in the construction industry, the water/cement ratio was kept low, so that the concrete containing GGBS or PFA gains enough high early strength to be used in fast track construction. Superplasticiser was used to increase the workability of concrete. Concrete was cured under three different curing environments.

For summer curing environment (20 $^{\circ}$C) and storage of concrete in sealed bags to minimise the loss of moisture, 70PC/30GGBS and 60PC/40GGBS concrete mixes gained enough strength of 24.5 MPa and 18.5 MPa respectively at the age of one day and 50PC/50GGBS concrete mix gained enough strength of 28 MPa at the age of two days. These strengths are greater than the minimum value of 10 MPa required for soffit formwork striking. It is concluded that high early age strength of GGBS concrete can be achieved by reducing the water/cement ratio and by providing a proper curing environment.

Concrete containing GGBS up to 50 % has the same 28 day compressive strength as the PC concrete when cured under normal temperatures and gains more strength than the PC concrete at the age of 56 days. Concrete containing 40 % GGBS has the highest compressive strength compared to the other concrete mixes at the age of 56 days and is 15.5 % more than the strength of 100PC-Control concrete mix.

From the compressive cube strength results of various concrete mixes used, it is concluded that curing environments affect the rate of strength gain for all concrete mixes. There is a significant reduction in the strength gain of concrete cured under winter conditions and all the concrete mixes including 100PC-Control concrete mix could not gain enough strength at the age of one day to be used in post-tensioned concrete when cured under winter curing conditions. The 70PC/30GGBS and 60PC/40GGBS concretes gained strengths of 23.0 MPa and 18.0 MPa respectively at the age of two days, which is sufficient to be used in fast track construction. The 50PC/50GGBS concrete mix gained enough strength of 15 MPa at the age of three days, which is more than sufficient for the removal of formwork. It is concluded that in winter for GGBS concrete up to 50 % special care should be taken regarding the temperature of the curing environment at early ages to gain enough strength.

From the results it can be concluded that at the curing temperature of 20 °C there is not much difference in the strength of GGBS concrete if it is cured in sealed bags to minimize the loss of moisture or it is cured under water. For the PC concrete mix the ultimate strength is higher if it is cured under water than the other curing regimes.

In winter curing conditions, the strength of PFA concrete mixes at the age of one day is lower than the 100PC-Control mix. In winter it is recommended that the curing temperature of the PFA concrete be kept at least 20 °C during the early ages to gain enough strength to be used in fast track construction. Early age strength development is slow due to the slow pozzolanic reactions between PFA and the lime ($Ca\ (OH)_2$) generated by the PC. For winter curing environment C2 (7 °C) the early age strength of PFA concrete mixes is lower than the 100PC-Control concrete mix. For concrete cured in the C2 environment, the strength of 90PC/10PFA, 80PC/20PFA and 70PC/30PFA concrete mixes are 26.0 MPa, 25.0 MPa and 19.5 MPa respectively at the age of two days and are 7.6 %, 12.9 % and 43.6 % respectively lower than the 100PC-Control concrete mix.

The compressive strength of concrete at the age of 28 days is reduced by 7 % (for100PC-Control) to 19 % (for 50PC/50GGBS) for different concrete mixes when cured under winter environments compared to the summer curing environment. There is a slight reduction in the 28th and 56th day's strength of PFA concrete mixes compared to PC concrete mix cured under winter curing conditions.

The compressive strength of PFA concrete mixes at the age of twenty eight days and cured under summer curing environment are the same as that of the PC concrete and gains more strength than the PC concrete at the age of 56 days. Compressive strength of 80PC/20PFA concrete cured under summer curing environment is 6.5 % higher than the 100PC-Control concrete mix at the age of 56 days and the strength of 90PC/10PFA concrete mix and 70PC/30PFA concrete mix are slightly higher than the 100PC-Control concrete mix at this age.

The normal strength concrete containing GGBS up to 50 % has the same 28 day compressive strength as the PC concrete when cured under normal temperature and gain more strength than the PC concrete after 28 days. Although the compressive cube strength of concrete containing 50 % GGBS at the age of three days is less than the PC concrete, it is greater than the minimum value of 10 MPa required for soffit formwork striking.

The flexural strength of concrete is increased with the addition of GGBS and PFA in concrete, compared to the flexural strength of PC concrete. The flexural strength of concrete cured under water has the higher value than the concrete cured under other curing environments.

Similar to flexural and compressive strength, elastic modulus of concrete is increased with the addition of GGBS and PFA in concrete and is affected with the curing environment.Compared with the 100PC-Control concrete mix, the value of twenty eight days modulus of elasticity is increased by 8 % for 90PC/10PFA and 6 % for 80PC/20PFA concrete mixes. The 70PC/30PFA concrete mix has a similar value of modulus of elasticity to that of the 100PC-Control mix.

5. PUNCHING SHEAR RESISTANCE OF FLAT SLABS

5.1. Introduction

According to Desai (2000), punching shear strength is the critical parameter of flat slabs and often governs the design requirement for flat slabs. Punching shear failure is the breaking of the portion surrounding a column from the rest of the slab. The critical design requirement for flat slabs is often based on provision against punching shear failure. The punching shear capacity of flat slabs depends on the strength of concrete, tension steel, column size and depth of slab. Punching shear capacity of flat slabs could be increased by providing shear reinforcement. In this chapter, punching shear resistance of flat slabs without any shear reinforcement is examined.

Punching shear tests were carried out on flat slab specimens at Kingston University London. Portland cement in the concrete of flat slab specimens was partially replaced by GGBS and PFA to reduce the embodied CO_2 of flat slabs and the results were compared with the PC concrete slab specimen.

Punching shear test results of the flat slabs specimens are presented in this chapter. Average mid-span deflections of slab specimens under the influence of different loads are presented. The cracking pattern of the slabs under the influence of load was also captured and presented in this chapter.

The main reinforcement strain and the concrete strain on the compression face of the slab specimen were also measured and presented in this chapter.

As there was no facility available at Kingston University for testing slabs so the setting up of testing rig for checking the punching shear resistance of flat slabs was a big part of the project.

5.2. Slab Specimen

Nine reinforced concrete slab specimens were tested for punching shear strength. Dimensions were the same for all slab specimens i.e. 1150 mm x 1150 mm in plan and with a depth of 120 mm. The specimens had a square central stub column, 200 mm x 200 mm. This slab size was chosen by keeping in mind various factors which included casting, curing, handling, disposal and testing of the slab specimen in the laboratory. In the flat slab tests reported by Chana & Desai, (1992), slab specimens for punching shear strength were simply supported at the nominal line of contraflexure, which is assumed to be at a distance of 0.2 L from the centre of the column. (L = span of the slab).The same concept has been used by the Author in the flat slab tests reported in this chapter and the specimens are meant to represent a flat slab continuous over columns at 2.5 m centres. Each slab specimen had holes drilled for rods and the rods were connected on to the steel sections placed over the slab to hold it down. This was meant to offer reactions on the sides of a square 990 mm x 990 mm, corresponding to the nominal lines of contraflexure at 495 mm from the centre of the column (0.2 L of the span) as shown in Figure 5.1.

All slab specimens were cast in timber moulds and were covered under plastic sheets after casting. The specimens were cured at laboratory temperature of 20 ^{o}C and water was manually sprinkled on them for 7 days. The plan and elevation of a typical flat slab specimen for punching shear strength tests are shown in Figure 5.1.

5.3. Reinforcement

In eight slab specimens, twelve high yield 10 mm ribbed bars were used as tension reinforcement in both directions at 100 mm centres, which gave the reinforcement ratio ρ_{st} as 0.82. With 15 mm as the clear cover, the mean effective depth "d" for the two-way reinforcement is given as 95 mm. Seven 6 mm bars were provided in each direction at 180 mm centres, as nominal reinforcement near the other face of the slab. The stub column was provided with four 12 mm high yield bars and three 8 mm links at equal centres.

In the ninth slab a central layer of 8 mm high yield rib bar at 150 mm centres was used to check its effect on punching shear, which makes the reinforcement ratio of central bars ρ_b equal to 0.35 and is about half the flexural reinforcement ratio. Reinforcement details for a typical slab specimen are given in Figure 5.2.

According to Table 4.2 of BS EN 1992-1-1 (2004) the minimum cover to reinforcement is equal to the diameter of the reinforcement bar which is 10 mm in the current case of study. For structural class S4 (design working life of 50 years) and exposure class XC1 (Dry or permanently wet) the minimum cover is 15 mm according to Table 4.4 N of BS-EN 1992-1-1(2004). This value can be further reduced to 10 mm by reducing the structural class by 1 according to Table 4.3 N of BS EN 1992-1-1(2004). The maximum size of aggregate used in slabs was 14mm.

5.4. Mix Proportions

Since the test rig was developed from scratch in the laboratory, a trial test was carried out on a specimen made with PC concrete to ensure that it worked correctly as expected. With minor modifications to the testing rig, further tests were performed. The diameter of the support holes in the slab was increased so that they can tolerate the movement of the supporting rods, passing through the slab without touching the sides of the slab. The steel frame which was used for putting the inserts in the floor holes was screwed on the supporting rods from top so that it could hold the rods and prevent any movement of the rods. After these minor modifications to the testing rig, it served its purpose successfully.

Concrete mixes designed for flat slab specimens were different from those used in Chapter 5. Normal strength concrete was designed for flat slab specimens, which is a common practice. Concrete was designed for 28 days compressive strength of 30 MPa with a margin of 5 MPa for all slab specimens, except the trial specimen which was designed for 40 MPa with a margin of 5 MPa. The design compressive strength for all the slab specimens was kept the same to check the effect of partial replacement of cement with GGBS and PFA in concrete on punching shear strength performed after 28 days. The mix proportions of the slab mixes are given in Table 5.1.The concrete mixes used for different RCC slab specimens are summarized below.

100PC- Trial represents the 100% PC concrete trial mix.

100PC-Control represents the 100% PC concrete control mix.

90PC/10PFA represents the concrete mix with 90% PC &10% PFA by weight.

80PC/20PFA represents the concrete mix with 80% PC & 20 % PFA by weight.

70PC/30PFA represents the concrete mix with 70% PC & 30 % PFA by weight.

70PC/30GGBS represents the concrete mix with 70% PC & 30% GGBS by weight.

60PC/40GGBS represents the concrete mix with 60% PC & 40% GGBS by weight.

50PC/50GGBS represents the concrete mix with 50% PC & 50% GGBS by weight.

100PC-CB represents the 100%PC concrete mix with central layer of reinforcement bars.

Table 5-1 Concrete Mix Proportions

Mix	Constituent Materials kg/m^3						
	Free Water	PC	PFA	GGBS	Aggregate		W/C ratio
					Coarse	Fine	
100PC-Trial	195	375	-	-	1220	575	0.52
100PC-Control	195	325	-	-	1245	600	0.6
90PC/10PFA	185	298	33		1204	580	0.56
80PC/20PFA	180	279	70	-	1260	520	0.52
70PC/30PFA	170	258	111	-	1276	560	0.47
70PC/30GGBS	195	227.	-	97.5	1245	600	0.6
60PC/40GGBS	195	195	-	130	1245	600	0.6
50PC/50GGBS	195	162	-	162	1245	600	0.6

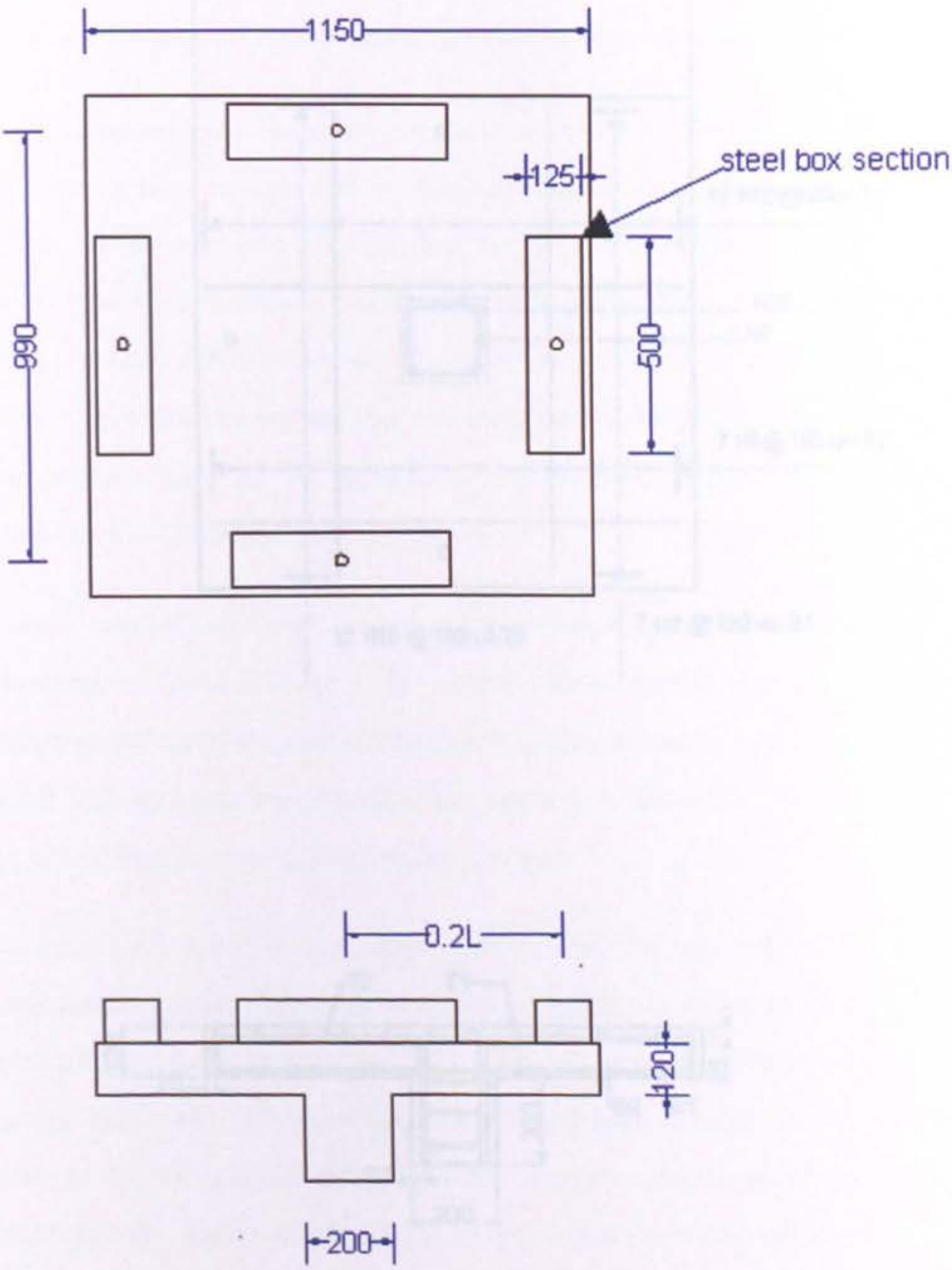

Figure 5-1 Specimen for Punching shear strength test

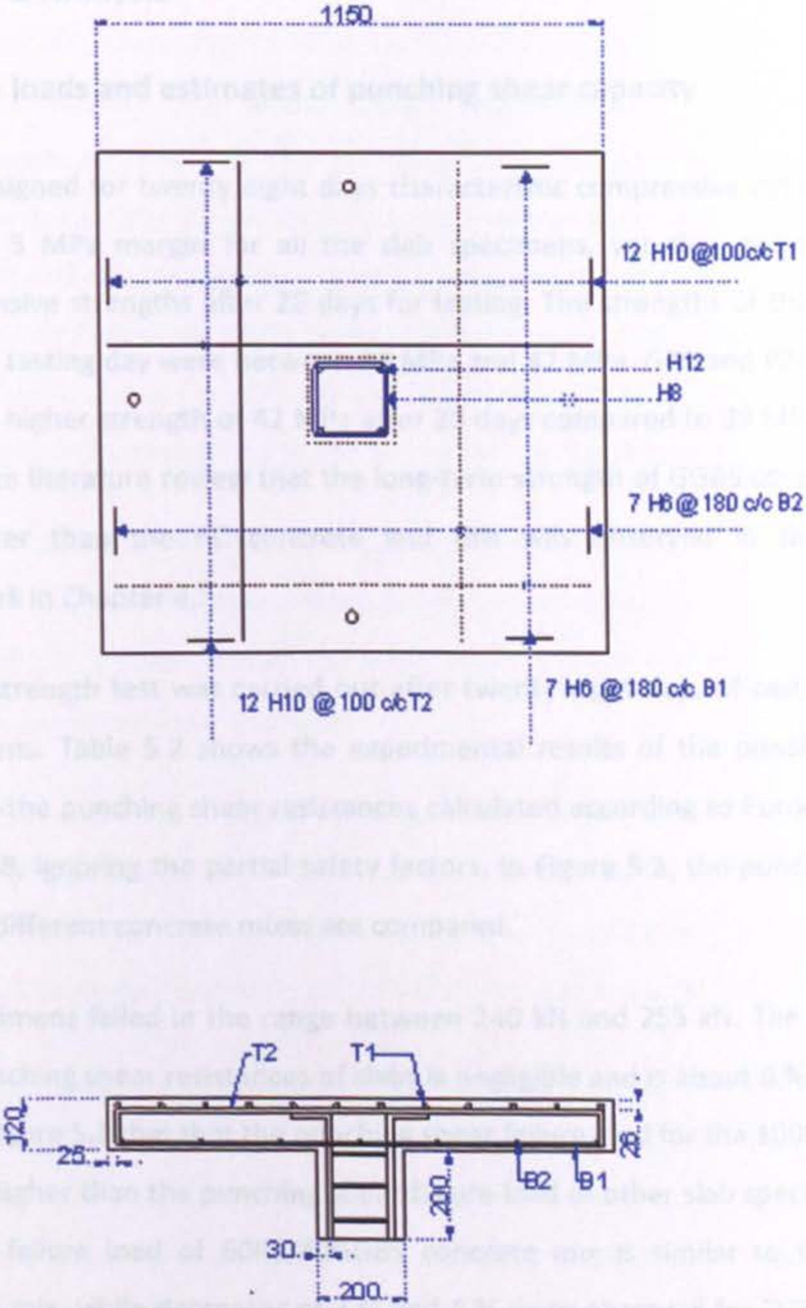

Figure 5-2 Reinforcement details

5.5. Results & Analysis

5.5.1. Failure loads and estimates of punching shear capacity

Concrete was designed for twenty eight days characteristic compressive cube strength of 30 MPa with 5 MPa margin for all the slab specimens, yet they gained slightly different compressive strengths after 28 days for testing. The strengths of the concrete specimens at the testing day were between 39 MPa and 42 MPa. G40 and P30 concrete mixes gained the higher strength of 42 MPa after 28 days compared to 39 MPa for G50. It is clear from the literature review that the long-term strength of GGBS concrete after 28 days is higher than the PC concrete and this was observed in the current experimental work in Chapter 4.

Punching shear strength test was carried out after twenty eight days of casting, for all the slab specimens. Table 5.2 shows the experimental results of the punching shear failure loads and the punching shear resistances calculated according to Eurocode 2, BS 8100 and ACI 318, ignoring the partial safety factors. In Figure 5.3, the punching shear failure loads for different concrete mixes are compared.

All the slab specimens failed in the range between 240 kN and 255 kN. The difference between the punching shear resistances of slabs is negligible and is about 6 %. It can be observed from Figure 5.3 that that the punching shear failure load for the 100PC-Control concrete mix is higher than the punching shear failure load of other slab specimens. The punching shear failure load of 60PC/40GGBS concrete mix is similar to the 100PC, control concrete mix, while decreases of 2 % and 4 % were observed for 70PC/30GGBS and 50PC/50GGBS concrete mixes respectively.

The punching shear failure load of 90PC/10PFA, 80PC/20PFA and 70PC/30PFA concrete slab specimens is 2 %, 6 % and 4 % lower than the 100PC-Control concrete mix. Overall and as noticed above, there was a negligible reduction in the punching shear strength of flat slab specimens made with GGBS up to 50 % and PFA up to 30 %.

It can be concluded from these results that partial replacement of PC with GGBS up to 50 % and PFA up to 30 % has negligible effect on the punching shear strength and can be used in flat slabs without any special design requirements unless the required compressive strength is achieved. The punching shear strength of the reinforced concrete slab is independent of the cementitious material used in the concrete and depends on the concrete compressive strength, reinforcement ratio and the shape factor as described by Park et al (2011), Desai (2000) and in different codes of practices. As discussed in Chapter 2, the use of GGBS and PFA in concrete as a partial replacement of PC saves embodied CO_2 emissions and the embodied energy of concrete.

It can be seen in Table 5.2 that the ratios (V_F/V) between the experimental punching shear failure load (V_F) and the failure load estimated according to BS 8110 (V) are between 1.04 and 1.11 ignoring the partial safety factor and the 40 MPa limit in the BS 8110 equation for punching shear strength.

The ratios (V_F/V_{Rdc}) between the experimental failure loads and the failure loads estimates (V_{Rdc}) of Eurocode 2 (BS EN 1992-1), ignoring partial safety factors are between 1.18 to 1.26 which are very similar to the results of Clarke (1987). The ratio indicates that the Eurocode 2 estimates are lower than the actual experimental results.

The ratios V_F/V_{uo} between the experimental failure loads and the failure loads calculated according to ACI 318 are between 1.12 and 1.20. From these observations it can be concluded that BS 8110 estimates for punching shear strength, ignoring the partial safety factor, are close to the real experimental punching shear strength values and the estimates of the Eurocode2 and ACI318 for punching shear strength are over-conservative, which is in accordance with the literature reviewed in Chapter 2.

Table 5-2 Experimental & Code predictions of Punching shear strength

MIX	Strength	Punching shear strength						
		Experimental	EC2	BS8110	ACI318			
		V_F	V_{Rdc}	V	V_{uo}	V_F/V_{uo}	V_F/V_{Rdc}	V_F/V
	MPa	kN	kN	kN	kN			
100PC-Control	40	255	203	229	212	1.20	1.26	1.11
90PC/10PFA	40	250	203	229	212	1.18	1.23	1.09
80PC/20PFA	41	240	204	231	214	1.12	1.18	1.04
70PC/30PFA	42	245	207	233	217	1.13	1.18	1.05
70PC/30GGBS	40	250	203	229	212	1.18	1.23	1.09
60PC/40GGBS	42	255	207	233	217	1.18	1.23	1.09
50PC/50GGBS	39	245	202	227	209	1.17	1.21	1.08
100PC-CB	40	250	203	229	212	1.18	1.23	1.09

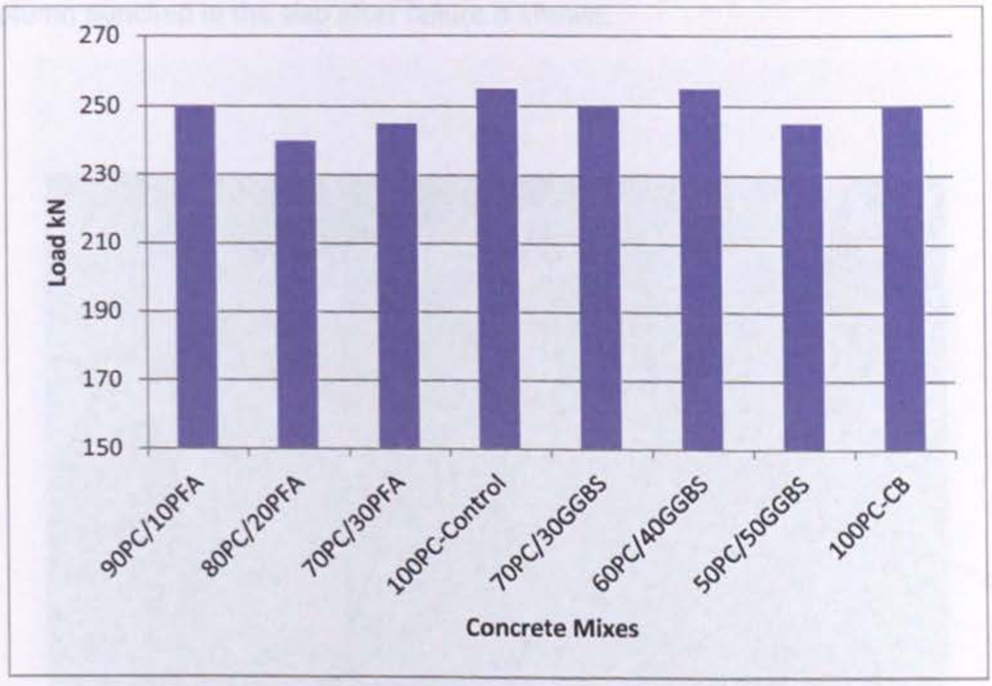

Figure 5-3 Experimental Punching Shear failure Loads

5.5.2. Crack Pattern

The pattern of cracking on all the specimens was similar. Radial cracks formed in the middle of the slab, extending gradually to the edges. Some circumferential cracks developed before punching shear failure. The pattern of crack development was the same as explained by Chana (1991). The average crack width at the point of failure was similar for all slab specimens and was in the range of 0.25 mm to 0.35 mm which is similar to the maximum crack width of 0.31 mm, recorded by Chana (1991) in his research on punching shear resistance of RCC flat slabs, reviewed in Chapter 2.

In Figure 5.4 to Figure 5.9 the final pattern of the cracks after failure is shown for different slab specimens. In Figure 5.10, the critical perimeter at which the slab failed in punching shear is shown. It can be seen from Figure 5.10 that the critical perimeter is at a distance of about 250 mm from the centre of the loaded column and it was in a similar range for the other slab specimens, which are approximately equal to 2.0d (190 mm) from the face of the column as calculated in Eurocode2. In Figure 5.11 and Figure 5.12, the column punched in the slab after failure is shown.

Figure 5-4 80PC/20PFA slab specimen after failure

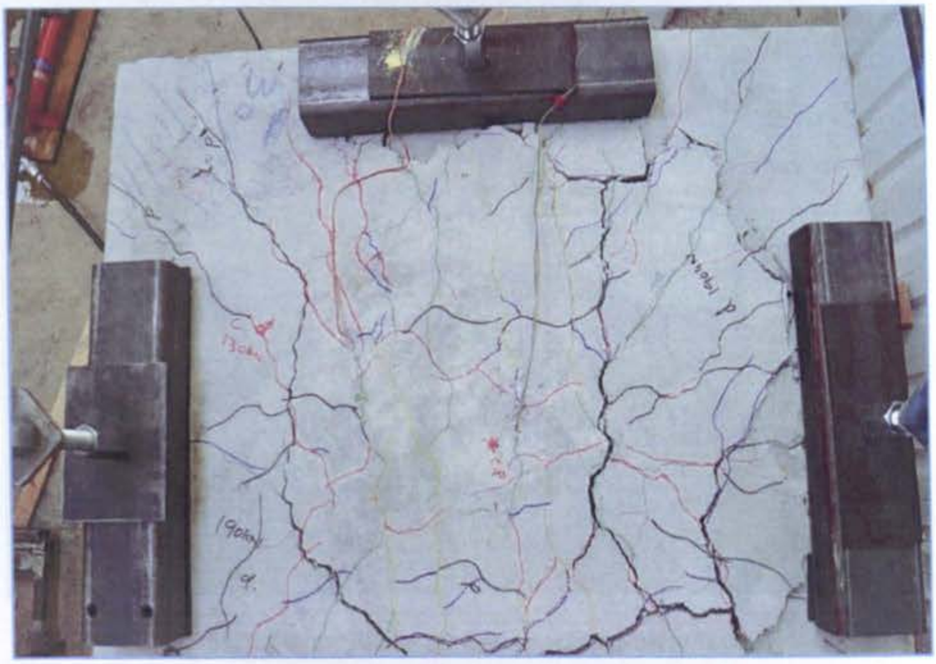

Figure 5-5 70PC/30PFA slab specimen after failure

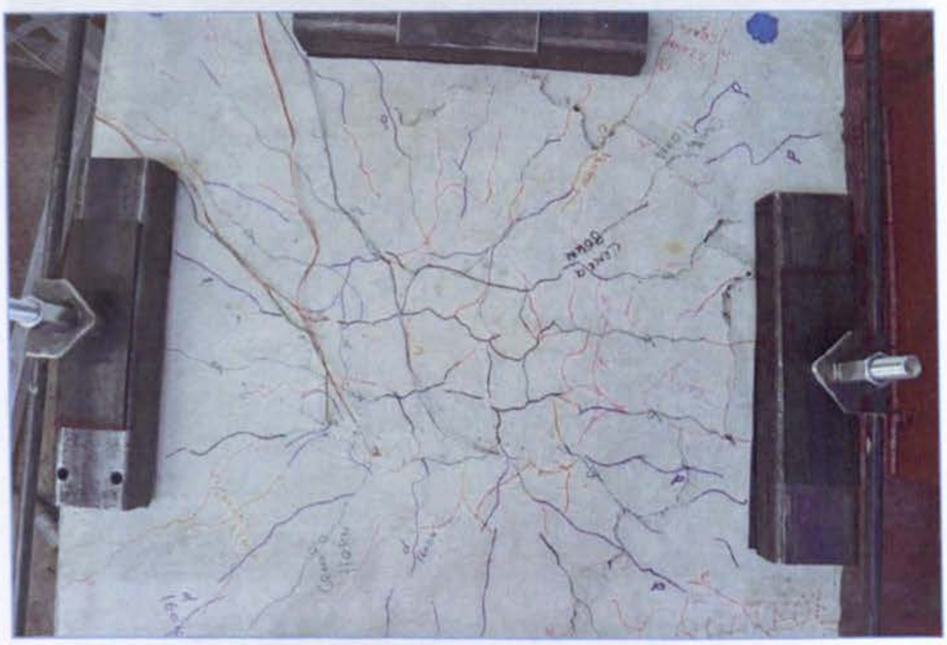

Figure 5-6 100PC-Control slab specimen after failure

Figure 5-7 60PC/40GGBS slab specimen after failure

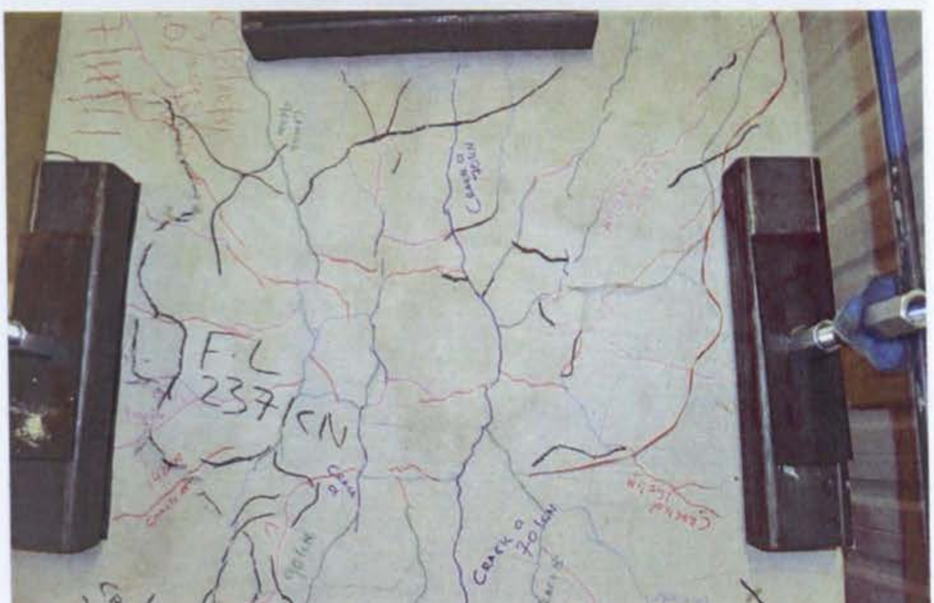

Figure 5-8 50PC/50GGBS slab specimen after failure

Figure 5-9 100PC-CB slab specimen after failure

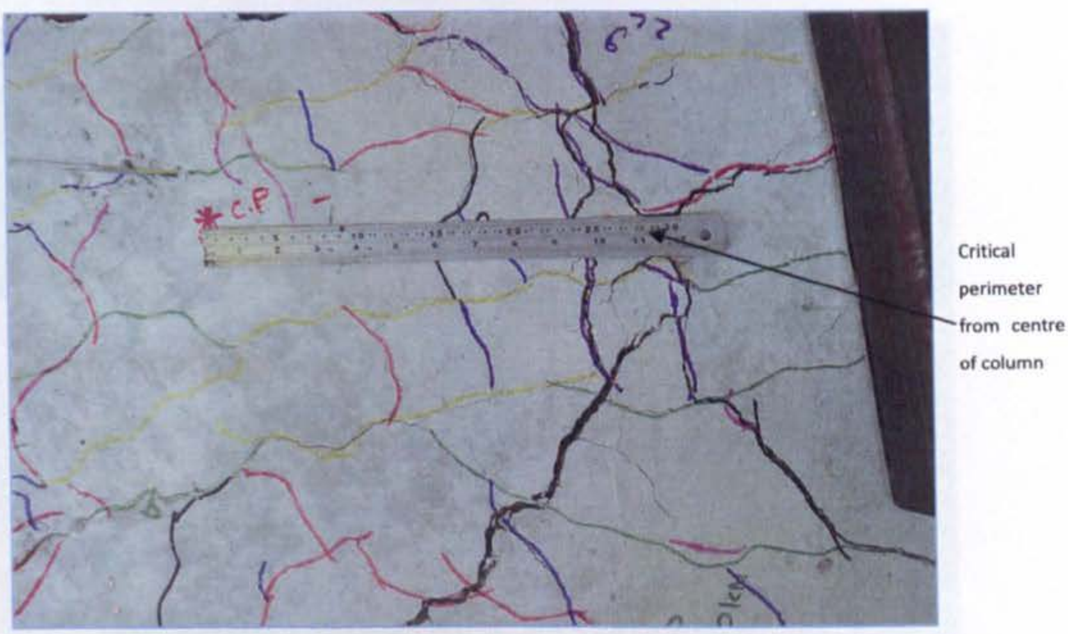

Figure 5-10 Critical perimeter of punching shear

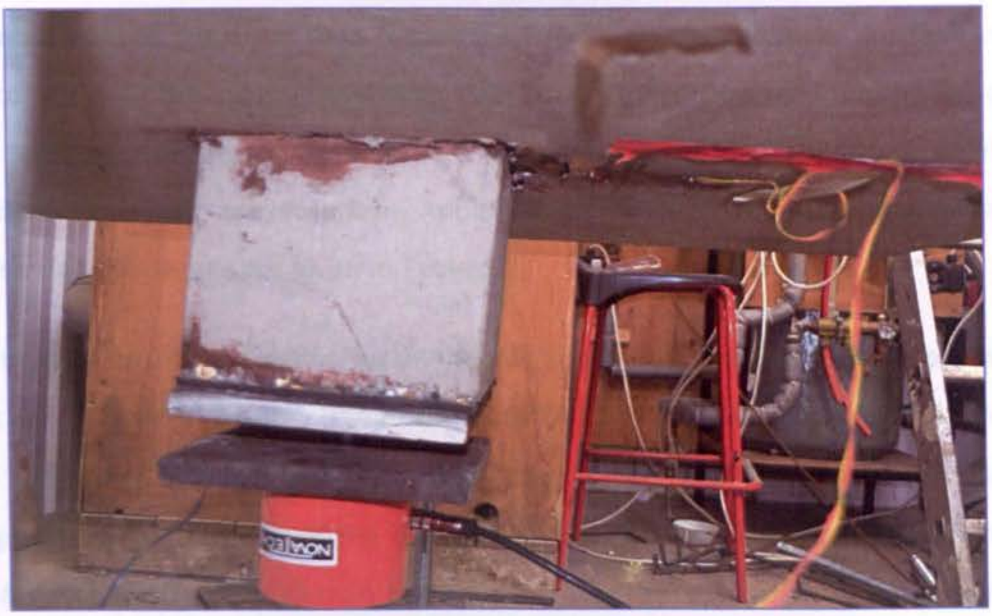

Figure 5-11 50PC/50GGBS Punched column in the slab after failure

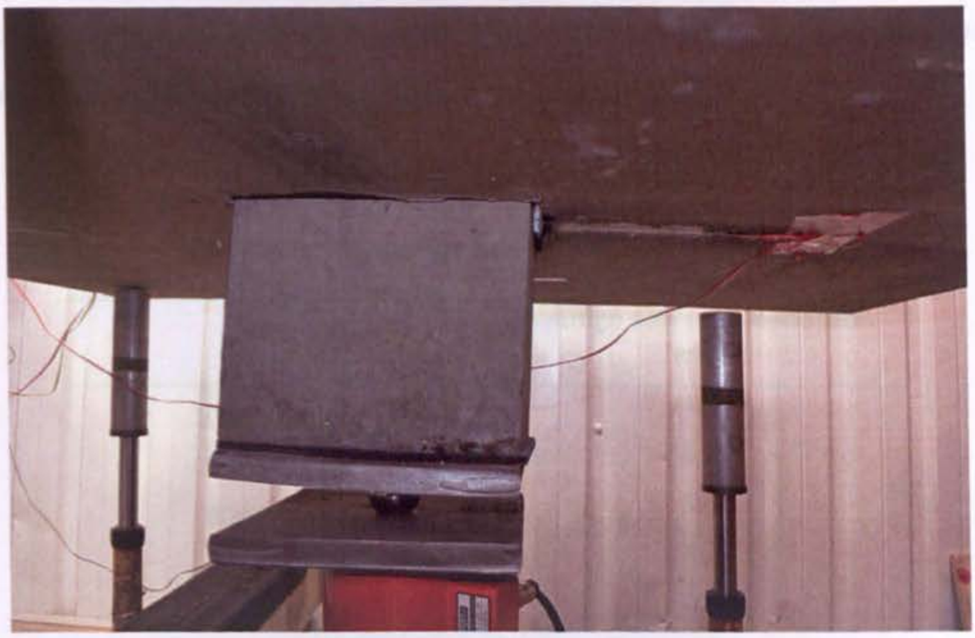

Figure 5-12 80PC/20PFA Punched column in the slab after failure

5.5.3. Mid Span deflection

The data collected from the data logger for LVDT's was analysed in the Excel spread sheet to calculate the average mid-span deflection. Deflections recorded by the central LVDT's were adjusted to take into account the deflection of the supports. The details of the adjustments are presented in Appendix C. Average mid-span deflections for different slab specimens are given in Table 5.3.

The average mid-span deflection for all the slab specimens was between 3.6 mm and 4.7 mm. It can be seen in Table 5.3 that 90PC/10PFA and 80PC/20PFA slab specimens have nearly the same average mid span deflection as the 100PC control specimen. The average mid span deflection of the 70PC/30PFA slab specimen is about 17 % higher and 60PC/40GGBS is 10 % higher than the 100PC-Control specimen. The average mid-span deflection was marginally higher, by 0.4 mm, in the slab specimens containing GGBS up to 50 %. This is not considered significant, suggesting that no special design requirements are necessary. The average mid- span deflection of the 100PC/30PFA concrete slab specimen is about 0.7 mm higher than the 100PC-Control slab specimen but is not considered significant with regard to design requirements.

Table 5-3 Average mid span deflections

Mixes	Average mid span deflection mm
100PC-Control	4.0
90PC/10PFA	4.2
80PC/20PFA	3.8
70PC/30PFA	4.7
70PC/30GGBS	4.2
60PC/40GGBS	4.4
50PC/50GGBS	4.3
100PC-CB	3.6

Load-Displacement curves for different concrete slab specimens are shown in Figure 5.13 and it can be seen that the average mid-span deflections at the failure load are almost similar for all concrete mixes.

The average mid span deflections of GGBS mixes are compared with the control mixes in Figure 5.14 and it can be seen that the GGBS slab specimens have similar average mid-span deflection- load curves to the 100PC-Control slab specimens.

In Figure 5.15 the average mid-span deflection at different levels of load applied for PFA mixes are compared with the 100PC-Control mix. Average mid span deflection-load curves for 90PC/10PFA and 80PC/20PFA slab specimens are almost the same as the 100PC-Control slab specimen. The average mid-span deflection of 70PC/30PFA slab specimen at failure load is slightly higher than the 100PC-Control slab specimen.

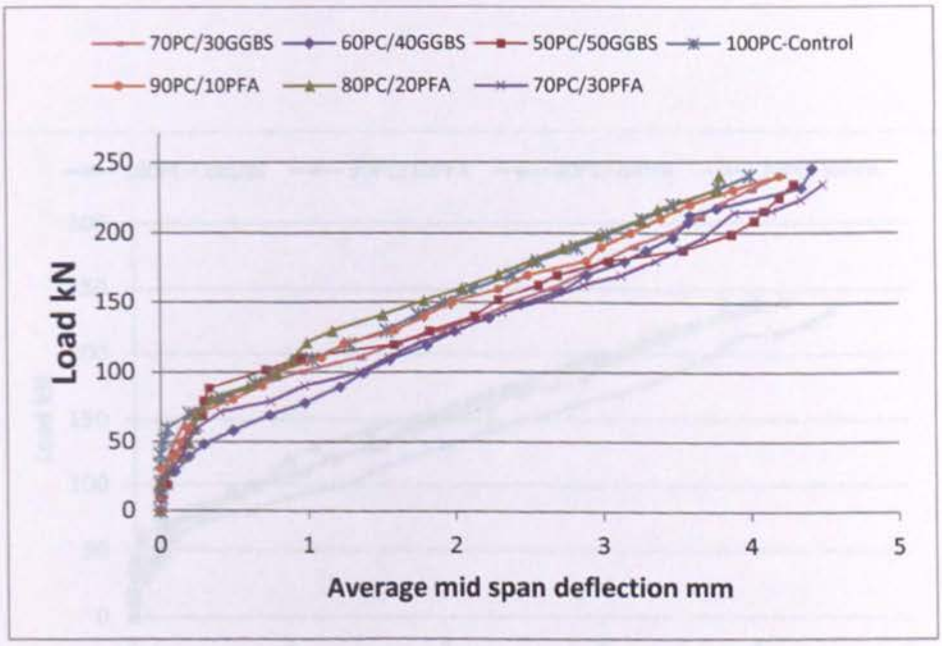

Figure 5-13 Load-displacement curve

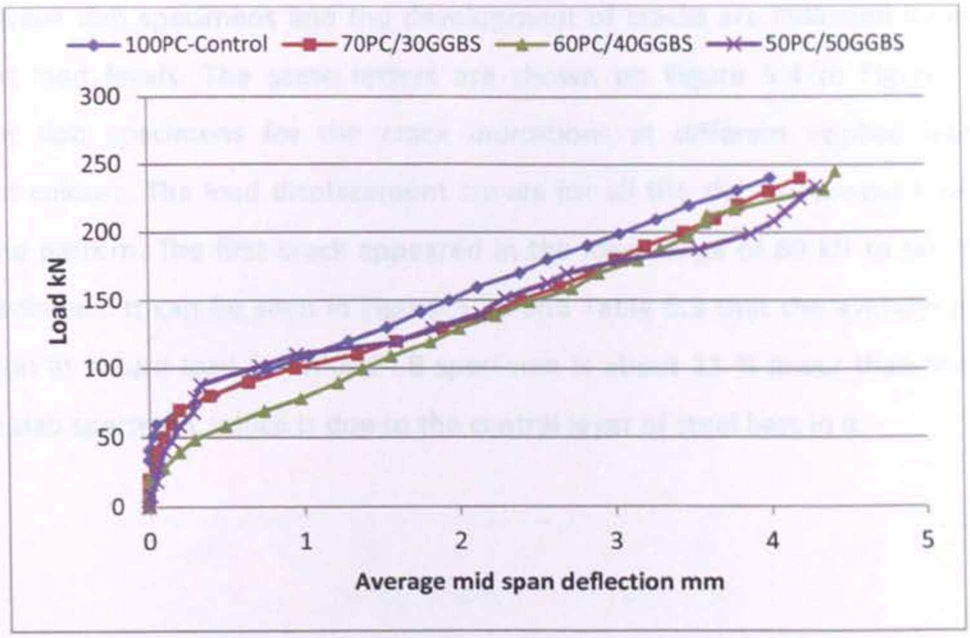

Figure 5-14 Average mid span deflection of GGBS mixes

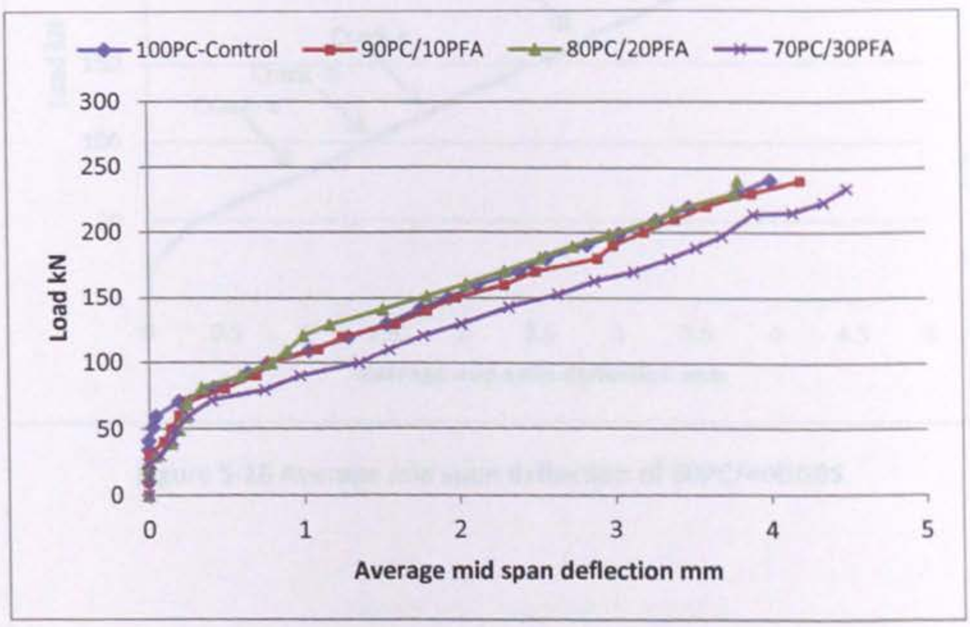

Figure 5-15 Average mid span deflection of PFA mixes

In Figure 5.16 - 5.21 the average mid-span deflection and applied load curves are shown for different slab specimens and the development of cracks are indicated by letters at different load levels. The same letters are shown on Figure 5.4 to Figure 5.10 for different slab specimens for the crack indications at different applied loads with different colours. The load displacement curves for all the slab specimens have nearly the same pattern. The first crack appeared in the load range of 80 kN to 90 kN for all slab specimens. It can be seen in Figure 5.21 and Table 5.3 that the average mid-span deflection at failure load for 100PC-CB specimen is about 11 % lower than the 100PC-Control slab specimen, which is due to the central layer of steel bars in it.

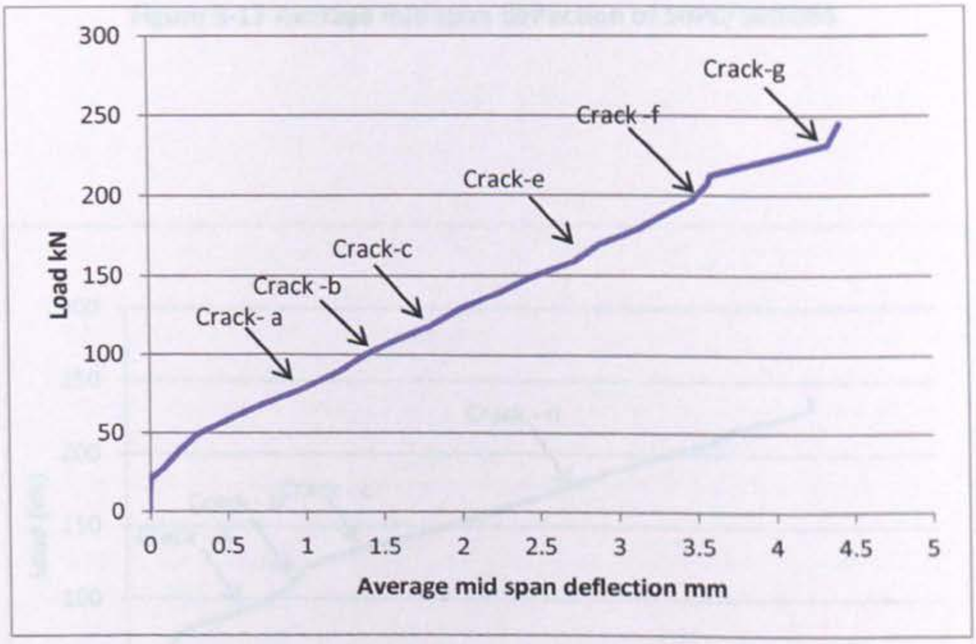

Figure 5-16 Average mid span deflection of 60PC/40GGBS

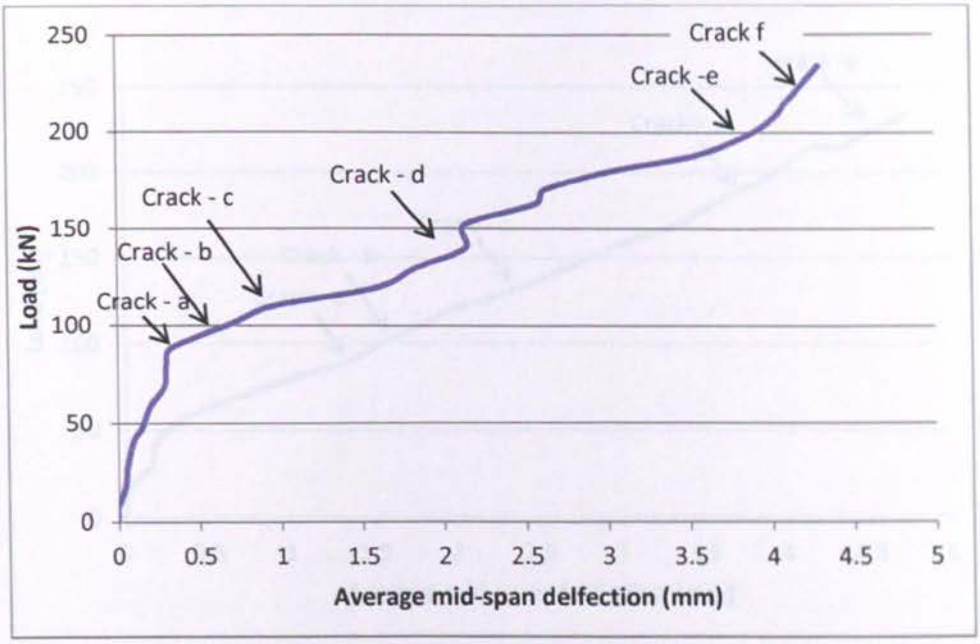

Figure 5-17 Average mid span deflection of 50PC/50GGBS

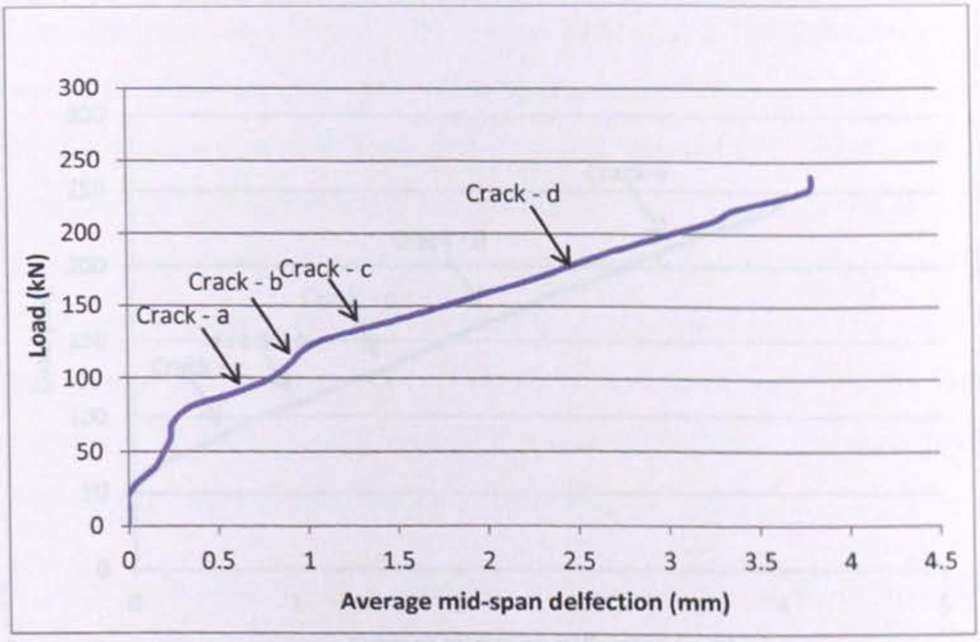

Figure 5-18 Average mid span deflection of 80PC/20PFA

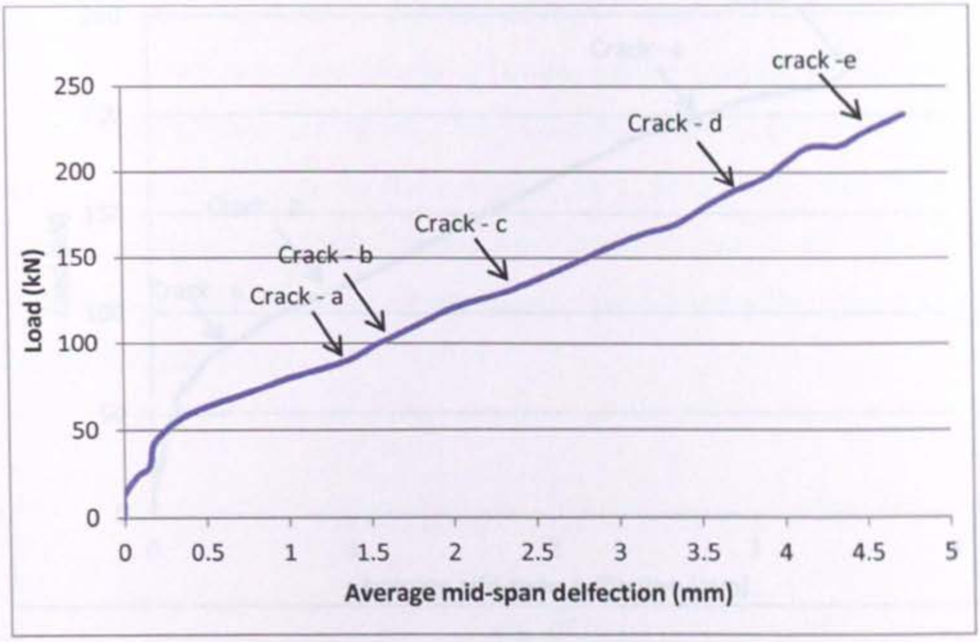

Figure 5-19 Average mid span deflection of 70PC/30PFA

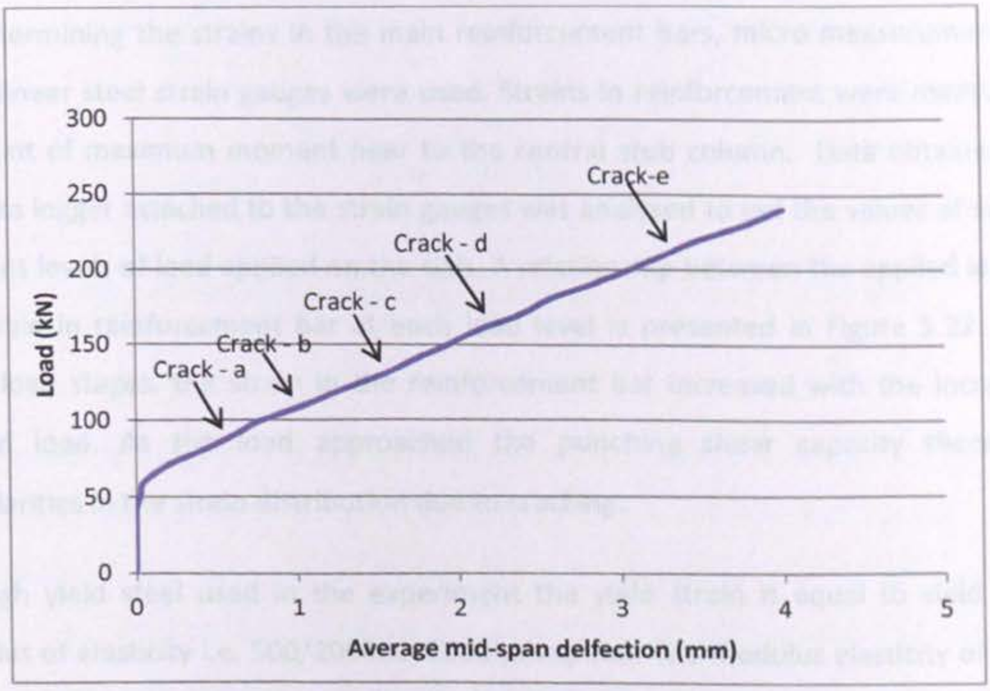

Figure 5-20 Average mid span deflection of 100PC-Control

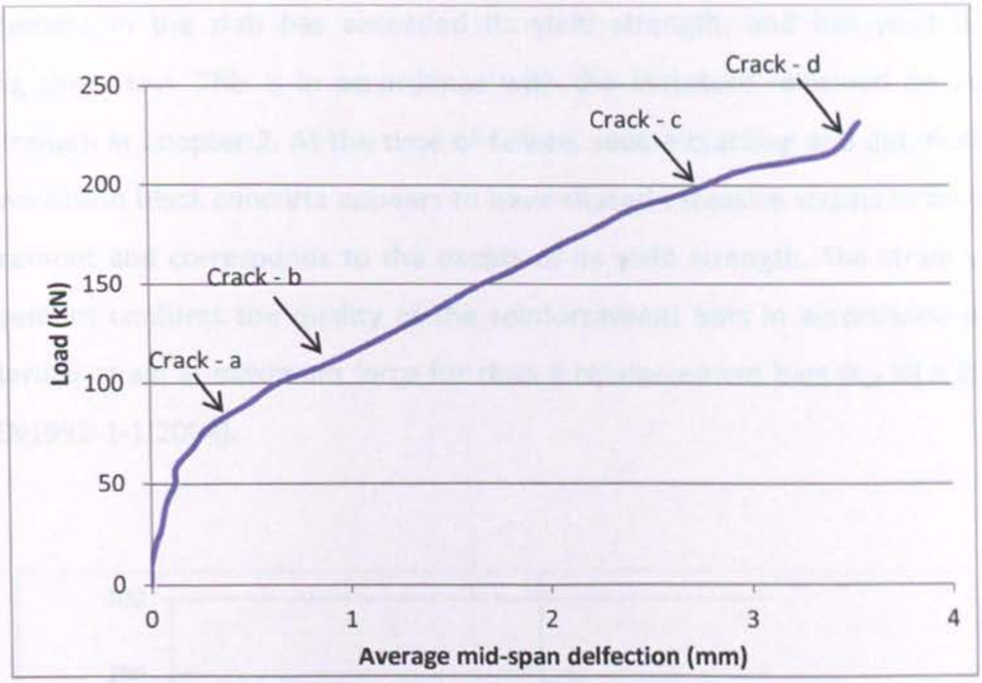

Figure 5-21 Average mid span deflection of 100PC-CB

5.5.4. Reinforcement Strain

For determining the strains in the main reinforcement bars, micro measurements CEA series linear steel strain gauges were used. Strains in reinforcement were measured at the point of maximum moment near to the central stub column. Data obtained from the data logger attached to the strain gauges was analysed to get the values of strain at different levels of load applied on the slab. A relationship between the applied load and the strain in reinforcement bar at each load level is presented in Figure 5.22. At the initial load stages, the strain in the reinforcement bar increased with the increase of applied load. As the load approached the punching shear capacity there were irregularities in the strain distribution due to cracking.

For high yield steel used in the experiment the yield strain is equal to yield stress/ modulus of elasticity i.e. 500/200000=2500 μmm/mm. The modulus elasticity of steel is taken as 200000 MPa and the yield stress for grade 500 steel is 500 MPa. From the strain results of steel in the slab specimens it can be seen that the strain in the steel was in the range of 2500 to 3500 μmm/mm at the point of failure. It means that the reinforcement in the slab has exceeded its yield strength, and has yielded in the punching shear test. This is in accordance with the literature reviewed on punching shear strength in Chapter 2. At the time of failure, severe cracking and deterioration in the compression block concrete appears to have caused excessive strains in the tension reinforcement and corresponds to the excess of its yield strength. The strain value of reinforcement confirms the quality of the reinforcement bars in accordance with the characteristic strain at maximum force for class A reinforcement bars (ε_{uk} %) ≥ 2.5 given by BS-EN1992-1-1(2004).

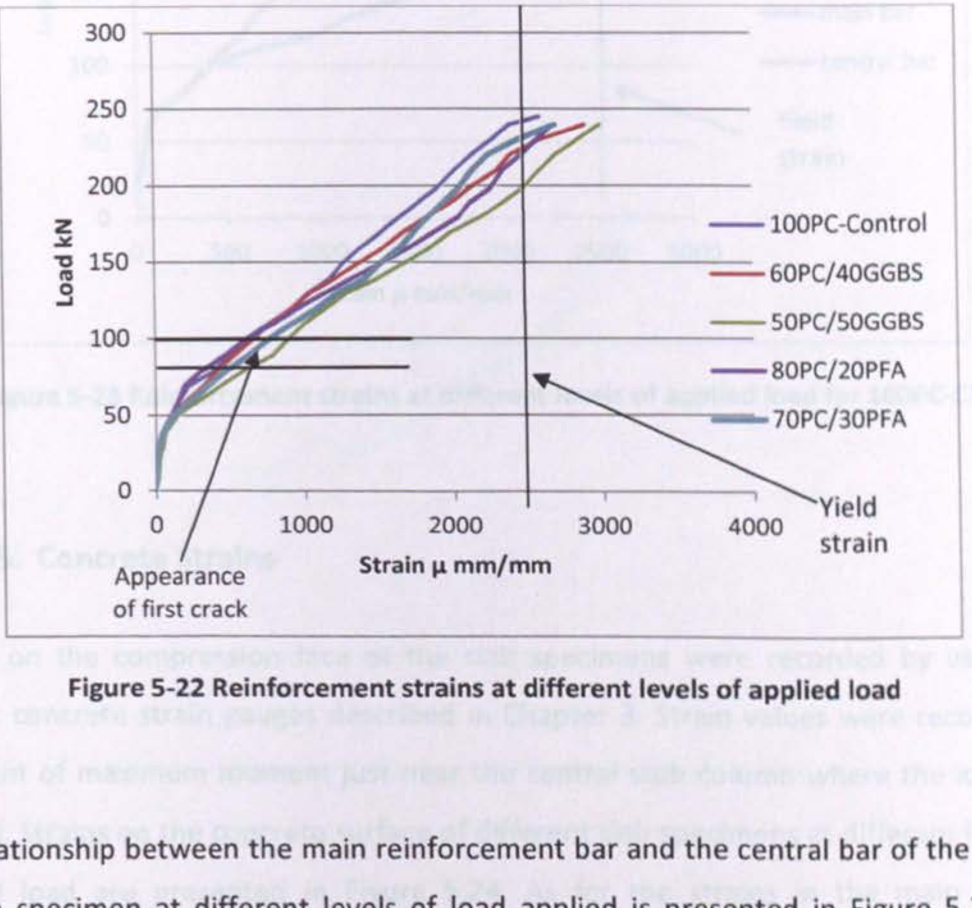

Figure 5-22 Reinforcement strains at different levels of applied load

The relationship between the main reinforcement bar and the central bar of the 100PC-CB slab specimen at different levels of load applied is presented in Figure 5.23. The strain values in the main bar are similar to the strain values of other slab specimens. Strain in the central bar is less and has not yielded. The idea of providing the central bars was that it will help in making a bond between the steel and the concrete and will add to the punching shear strength of the slab as proved by Desai (1997) for thicker slab sections. This idea did not work in the current specimen because of the very thin slab specimen with so many layers of reinforcement which did not allow the concrete to make a strong bond with steel. Further research is recommended for the use of central bars to enhance the punching shear strength in thin slab sections.

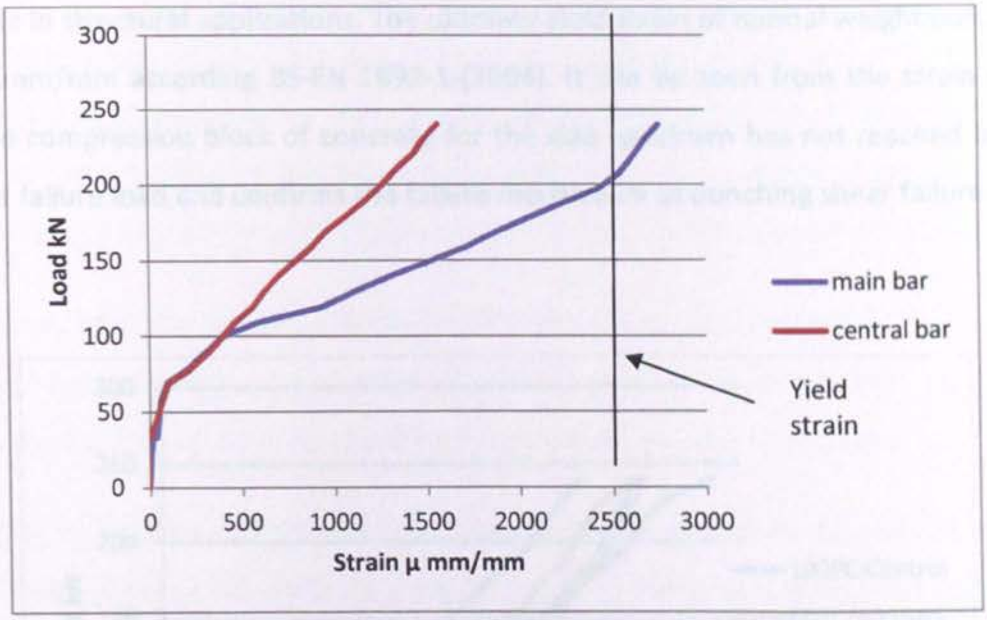

Figure 5-23 Reinforcement strains at different levels of applied load for 100PC-CB

5.5.5. Concrete Strains

Strains on the compression face of the slab specimens were recorded by using the electric concrete strain gauges described in Chapter 3. Strain values were recorded at the point of maximum moment just near the central stub column where the load was applied. Strains on the concrete surface of different slab specimens at different levels of applied load are presented in Figure 5.24. As for the strains in the main tension reinforcement, strain on the compression face of concrete slab increased with the increase in load applied and as this reached the punching shear failure load, there were irregularities in the strain values of concrete. The strains on the compression face of the slab specimens almost all had same pattern. It can be seen from Figure 5.24 that strain in the 100PC-Control specimen is less than the other slab specimens at different stages of load applied and this difference increased after the appearance of the first crack on the tension face of the slab until the failure load. It can be seen that the strain on the compression face of concrete increased with GGBS and PFA level at a given applied load. Overall there is a negligible difference in the strain values of concrete for all slab specimens at the failure load and suggest no specific consideration needs to be given for their use in structural applications. The ultimate yield strain of normal weight concrete is 3500 μmm/mm according BS-EN 1992-1-(2004). It can be seen from the strain results that the compression block of concrete for the slab specimen has not reached its yield point at failure load and confirms the failure mechanism as punching shear failure.

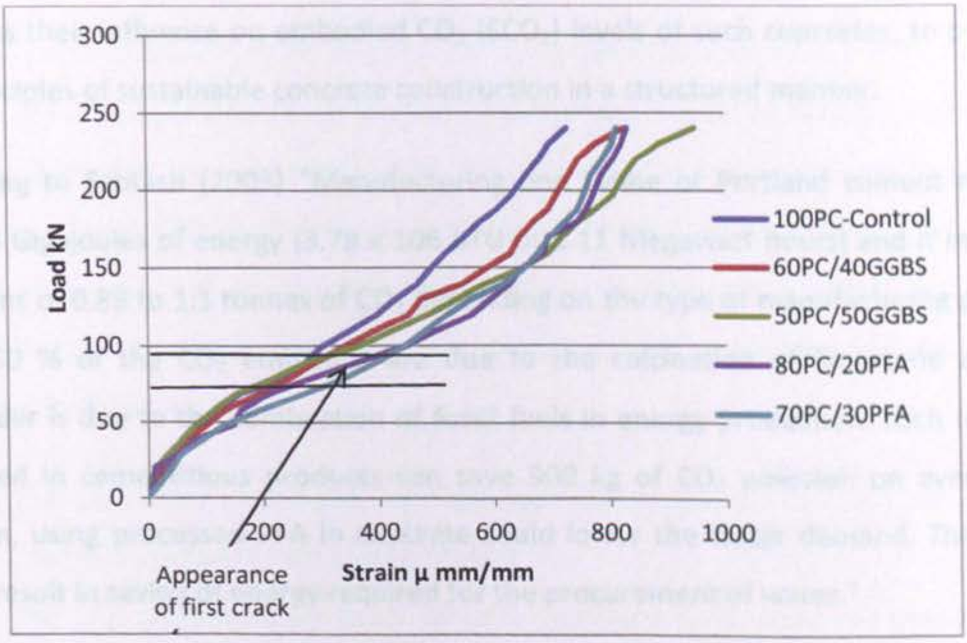

Figure 5-24 Concrete Strains at different levels of applied load

5.6. Quantification of the Environmental benefit of GGBS and PFA in concrete

The concrete technology research has demonstrated that GGBS and PFA can be used as PC replacement in composite mixes, without any adverse effect on the strength of structural concrete. On the other hand, carefully designed composite mixes can improve the microstructure of concrete and, hence, its durability of concrete (e.g. structures subjected to chemical attack), and they can also enhance the structural properties of concrete. Tests reported in this thesis have shown that the concretes made with partial replacement of PC with PFA and GGBS can perform well in flat slab construction.

It is essential that the benefits afforded by PFA and GGBS concretes should be quantified to assess their influence on embodied CO_2 (ECO_2) levels of such concretes, to promote the principles of sustainable concrete construction in a structured manner.

According to Scotash (2005) "Manufacturing one tonne of Portland cement requires about 4 Gigajoules of energy (3.78 x 106 BTU or 1.11 Megawatt hours) and it results in emissions of 0.89 to 1.1 tonnes of CO_2 depending on the type of manufacturing process. Some 50 % of the CO_2 emissions are due to the calcination of limestone and the remainder is due to the combustion of fossil fuels in energy production. Each tonne of PFA used in cementitious products can save 900 kg of CO_2 emission on average. In addition, using processed PFA in concrete could lower the water demand. This factor would result in saving of energy required for the procurement of water."

According to the data published by The Concrete Centre (2007) the embodied CO_2 of Portland cement, produced in the UK is 930 kg/tonne production of PC. This figure is cradle to factory gate and does not include the transport from the place of manufacture to the concrete plant.

According to the data published in Hanson UK Performance and sustainability report (2011) the amount of CO_2 produced for the production of one tonne of cement in 2010 is 0.8 tonnes.

For a general idea about the environmental benefit of GGBS and PFA in concrete flat slab, a five storey flat slab building with a plan dimension 50 m x 10 m and slab thickness 200 mm is considered as an example to work out the savings in CO_2 emissions and energy consumption for various concrete mixes and are presented in Table 5.4. Table 5.1 shows a concrete mix with 30 % replacement of PC with PFA. This mix had 258 kg of PC and 111 kg of PFA in a cubic metre of concrete. This mix has satisfied the 28 days cube strength requirement. However, its use can mean a 67 kg reduction in PC in comparison to 100PC-Control mix which is 62 kg of CO_2 emissions per cubic metre of concrete, i.e. 0.93 x 67 kg. If one were to have a 200 mm thick slab in a five storey building, 10 metres wide and 50 metres long, a 30 % replacement of PC with PFA would mean a 31 tonne reduction in CO_2 emission.

As discussed in Chapter 2, GGBS and PFA are both industrial by-products and a negligible amount of CO_2 results from their production compared to that of Portland cement. For example from one tonne of GGBS, 0.05 tonnes of CO_2 is produced compared to 0.93 tonnes for a typical Portland cement. Similarly the production of GGBS and PFA require very small amounts of energy compared to that required for Portland cement. The savings in CO_2 emissions and the energy consumptions, presented in Table 5.4 are calculated as follows.

- The quantities of cement required per cubic meter of concrete for different concrete mixes are taken from Table 5.1.
- The CO_2 emissions per cubic metre of concrete production are calculated for different concrete mixes and are based on the CO_2 emissions of PC production which is 0.93 tonnes of CO_2 release for each tonne. The CO_2 emissions for other materials of concrete are ignored in this comparison.
- The volume of concrete in the five storey flat slab only is calculated for this example as 500 m^3
- The CO_2 emissions in tonne are calculated for the concrete used in the five storey flat slab example for different concrete mixes using the same relation.

- Savings in the amount of CO_2 released in the concrete of the flat slab example for different concrete mixes are calculated in comparison to 100PC-Control concrete mix.
- Energy consumption per cubic meter of concrete production for different concrete mixes is calculated for different concrete mixes and is based on the energy released in the production of one tonne of PC which is 4 Giga Joules (GJ) of energy consumption per tonne.
- Energy consumption for the concrete used in the five storey flat slab example is calculated in GJ for different concrete mixes using the same relation.
- Savings in energy consumption of concrete used in the five storey flat slab example for different concrete mixes are calculated in comparison to 100PC-Control concrete mix.

The values of CO_2 emissions and energy consumption are based on the amount of concrete used only in the flat slabs excluding the foundations, columns and beams. The quantification of sustainability enhancement should be refined and formalised through further research, so that it can apply to all alternative constituents of concrete.

Table 5-4 Savings in CO_2 emission & energy consumption for flat slab concrete mixes

Concrete Mixes	PC used	CO_2 emission of concrete	CO_2 emission of concrete in flat slab example	Savings of CO_2 in flat slab example	Energy consumption	Energy consumption of concrete in flat slab example	Savings of energy in flat slab example
	kg/m^3	kg/m^3	Tonne	Tonne	GJ/m^3	GJ	GJ
100PC-Control	325	302.3	151.1	0	1.3	650	0
90PC/10PFA	298	277.1	138.6	12.6	1.19	596	54
80PC/20PFA	279	259.5	129.7	21.4	1.12	558	92
70PC/30PFA	258	239.9	120.0	31.2	1.03	516	134
70PC/30GGBS	227	211.1	105.6	45.6	0.91	454	196
60PC/40GGBS	195	181.4	90.7	60.5	0.78	390	260
50PC/50GGBS	162	150.7	75.3	75.8	0.65	324	326

It can be seen in Table 5.4 that for a typical five storey office building of plan 50 m x 10 m with a 200 mm thick slab a reduction of about 75.8 tonnes of CO_2 emissions and 326 Giga Joules energy savings can be achieved by using GGBS up to 50 % in the flat slab concrete only, excluding the concrete used in the columns, beams and foundation etc. A reduction of about 31 tonnes of CO_2 emissions and saving of 134 Giga joules of energy can be achieved by using PFA up to 30 % in the same building compared to the PC concrete. It is evaluated that a reduction of 152 kg of CO_2 and 0.65 GJ of energy consumption per cubic metre of concrete can be achieved by using 50PC/50GGBS concrete compared to the 100PC-Control concrete mix. A reduction of 62.5 kg of CO_2 and 0.27 GJ of energy per cubic metre of concrete can be achieved by using 70PC/30PFA concrete mix instead of 100PC-Control concrete mix.

The environmental benefits of Trial 2 concrete mixes, from Chapter 5 are presented in Table 5.5. The twenty eight days compressive cube strength achieved by Trial 2 mixes under summer curing environment (20 °C) is between 67.5 MPa and 71.3 MPa. The amount of PC used in the Trial 2 mixes has been taken from Table 4.2. The CO_2 emissions per cubic metre of concrete production are calculated for different concrete mixes and are based on the CO_2 emissions of PC production which are 0.93 tonnes of CO_2 release for each tonne of PC production. The CO_2 emissions for other materials of concrete are ignored in this comparison. The energy consumptions per cubic metre of concrete production for different concrete mixes are calculated for different concrete mixes and are based on the energy released in the production of one tonne of PC which is 4 Giga Joules (GJ) of energy consumption. It can be seen that the maximum saving in CO_2 emissions is 213 kg/m^3 and 0.91 GJ/m^3 in energy consumptions for the 50PC/50GGBS, compared to the 100PC-Control mix. There is a saving of 124 kg/m^3 of CO_2 emissions and reduction of 0.53 GJ/m^3 of energy consumptions by using 70PC/30PFA concrete mix.

Table 5-5 Savings in CO_2 & energy consumptions for Trial 2 concrete mixes

Concrete Mixes	w/c Ratio	PC used	CO_2 emissions	Saving in CO2 compare to 100PC	Energy consumption	Saving in energy Consumption compare to 100PC
		kg/m^3	kg/m^3	kg/m^3	GJ/m^3	GJ/m^3
100PC-Control	0.35	457	425		1.83	
90PC/10PFA	0.375	360	335	90	1.44	0.39
80PC/20PFA	0.325	370	344	81	1.48	0.35
70PC/30PFA	0.325	324	301	124	1.30	0.53
70PC/30GGBS	0.35	320	298	127	1.28	0.55
60PC/40GGBS	0.35	274	255	170	1.10	0.73
50PC/50GGBS	0.35	229	213	213	0.92	0.91

5.7. Summary

Flat slab specimens in which cement is replaced partially by GGBS up to 50 % and PFA up to 30 % has no adverse effect on punching shear capacity and the mid-span deflection. Based on the tests carried out and the materials used, concrete containing GGBS and PFA can be used in flat slab structures without any special requirements for design and the same design rules for PC concrete can be used without any modification.

The relationship between average mid-span deflections at different levels of load applied has a similar pattern for PFA, GGBS and PC concrete mixes. The average mid span deflection at the failure load for the different slab specimens was in the range of 3.6 mm for 100PC-Control specimen and 4.7 mm for 70PC/30PFA concrete slab specimen. The average mid span deflection for the slab specimen containing GGBS and PFA is slightly more than the 100PC-Control slab specimen but is negligible to be considered for design considerations and also confirms the failure of the slab specimens as punching shear.

The pattern of cracking on all specimens was similar. Radial cracks formed in the middle of the slab, extending gradually to the edges. Some circumferential cracks developed before punching shear failure. The average crack width at the point of failure was similar for all slab specimens and was in the range of 0.25 mm to 0.35 mm.The first crack appeared in the load range of 80 kN to 90 kN for all slab specimens. In all the slab specimens, the critical perimeter of the crack was at a distance of about 2.0 d from the face of the loaded column, which confirms the authenticity of the critical perimeter distance of 2.0 d from the face of the column, given by Eurocode 2.

There is no significant difference in the concrete strain on the compression face of the slab specimen and steel strain in the main reinforcement for different RCC slab specimens containing GGBS and PFA, which is important for their use in structural applications.

In the CB slab specimen, the central layer of reinforcement was provided to enhance the punching shear strength of the slab. Punching shear strength of the slab with this was

not improved although it helped in reducing the average mid-span deflection, and further research is recommended for the provision of central bars to enhance the punching shear strength of thinner slabs.

The concrete designed for the characteristic compressive cube strength of 30 MPa, a reduction of 152 kg of CO_2 and 0.65 GJ of energy consumption per cubic metre of concrete can be achieved by using 50PC/50GGBS concrete compared to a 100PC-Control concrete mix. A reduction of 62.5 kg of CO_2 and 0.27 GJ of energy per cubic metre of concrete can be achieved by using a 70PC/30PFA concrete mix instead of 100PC-Control concrete mix. For high strength concrete a saving of 213 kg/m^3 of CO_2 emissions and a reduction of 0.91 GJ/m^3 in energy consumption can be achieved by using a 50PC/50GGBS compared to the 100PC-Control mix. A saving of 124 kg/m^3 of CO_2 emissions and reduction of 0.53 GJ/m^3 of energy consumption can be achieved by using a 70PC/30PFA concrete mix.

6. CONCLUSIONS & RECOMMENDATIONS FOR FUTURE WORK

6.1. Conclusion

6.1.1. Workability

- From the slump test results in Chapter 4, it is concluded that concrete containing GGBS and PFA are more workable and easy to finish compared to PC concrete. The slump values of concrete increase with GGBS and PFA contents in concrete, because of the higher fines content.

6.1.2. Early age compressive strength of concrete

- From the strength development results in Chapter 4, it is concluded that at low water/cement ratio (0.35), concrete containing GGBS or PFA gains enough high early age strength to be used in post-tensioned concrete and fast track construction.
- For summer curing environments (20 °C) and storage of concrete in sealed bags to minimise the loss of moisture, the strength achieved by 40 % GGBS concrete, was 18.5 MPa at the age of one day and the strength achieved by 50 % GGBS concrete was 28.0 MPa at the age of two days. These strengths are greater than the minimum value of 10 MPa required for soffit formwork striking, which means that these mixes can be used in the fast-track construction.
- From the results in Chapter 4, it is concluded that there are significant reductions in the rate of strength gain of concrete cured under winter curing conditions (7 °C), compared to those of summer curing under water (20 °C) and storage of concrete in sealed plastic bags, to minimise the loss of moisture. All concrete mixes including PC concrete could not gain enough strength at the age of one

day, when cured under winter conditions but they gained enough strength (more than 10 MPa) at the age of two days to be used in fast track construction, except 50 % GGBS concrete mix which gained enough strength at the age of three days. It is concluded that in winter for concrete containing GGBS up to 50 %, special care should be taken regarding temperature increase of the curing environment at the early age to gain enough strength. The heating of concrete buildings, to increase the temperature for curing is a common practice in cold areas.

- It is concluded from the compressive strength results in Chapter 4 that in winter curing conditions the strength of PFA concrete mixes at the age of one day is lower than the control PC concrete mix. In winter it is recommended that the curing temperature of PFA concrete be kept at a level of at least 20°C during early ages to gain enough strength to be used in fast-track construction.
- It is concluded that at the specified water/cement ratio of 0.35, PFA concrete up to the level of 30 %, if cured under summer conditions (20 °C), has enough strength of 21.3 MPa (more than the 10 MPa limit) for the removal of form work, at the age of one day to be used in the fast-track construction, where early removal of form-work or the structure subjected to high early loads is the requirement.

6.1.3. Compressive strength of concrete at the age of 28 and 56 days

- From the compressive strength development of GGBS concrete results presented in Chapter 4, it is concluded that concrete containing GGBS up to 50 % has the same 28 day compressive strength as PC concrete, when cured under summer temperatures (20 °C) and gains more strength than the PC concrete at the age of 56 days which agrees with previous research done by others and reviewed in Chapter 2. Concrete containing 40 % GGBS has the highest compressive strength compared to the other concrete mixes at the age of 56 days and is 15.5 % more than the strength of PC concrete.

- From the results of compressive strength development of PFA concrete in Chapter 4, it is concluded that twenty eight days strength of PFA concrete mixes cured under the summer temperature (20 °C) environment are the same as that of the PC concrete and gains more strength than this at 56 days, which confirms previous research and reviewed in Chapter 2. Compressive strength of 20 % PFA concrete cured under the summer curing environment is 6.5 % higher than PC concrete at the age of 56 days. The strength of 10 % PFA and 30 % PFA are slightly higher than the PC concrete at this age.
- Under the winter curing environment compressive strengths of PFA and GGBS concrete mixes at the age of 28 and 56 days are less than the compressive strength of the PC concrete mix at the same age, which shows that the lower winter curing temperatures reduces the strength of GGBS and PFA concrete more than the PC concrete. It is concluded that to enhance the long-term strength of PFA concrete, an extended curing temperature of about 20 °C is required
- The curing environment affects the rate of strength gain for all concrete mixes. From the results it can be concluded that at the curing temperature of 20 °C there is not much difference in the strength of GGBS concrete if it is cured in sealed bags to minimize the loss of moisture or it is cured under water. For the PC concrete mix the ultimate compressive strength is higher, if it is cured under water than the other curing regimes.

6.1.4. Flexural Strength

- From the flexural strength results in Chapter 4, it is concluded that the flexural strength of concrete is increased with the addition of GGBS and PFA in concrete, compared to the flexural strength of PC concrete. Concrete containing GGBS up to 50 % and PFA up to 30 % have higher values of flexural strength than the PC concrete when cured under the summer curing environment (20 °C), which confirms the previous research reviewed in Chapter 2. Concrete containing 20 % PFA has higher flexural strength than the other PFA concretes. The twenty eight days flexural strength of 30 %, 40 % and 50 % GGBS concrete mixes are 3.3 %, 8.2 % and 4.9 % higher respectively than the PC concrete mix cured under the summer temperature.

- Curing environments have an effect on the flexural strength of GGBS and PFA concrete mixes and this is reduced after being cured under winter environments (7 °C) compared to summer temperatures of 20 °C in sealed plastic bags or under water.
- GGBS concrete and the PC concrete mixes cured under water at 20 °C have higher flexural strength than the concrete cured in sealed plastic bags at 20 °C.

6.1.5. Modulus of Elasticity

- From the modulus of elasticity results in Chapter 4, it is concluded that concrete containing GGBS and PFA have higher values of modulus of elasticity than the PC concrete at the summer curing temperatures (20 °C), which is according to the literature reviewed in Chapter 2. The value of twenty eight days modulus of elasticity is increased by 8 % for 10 % PFA and 6 % for 20 % PFA concrete mix. The 30 % PFA has nearly similar value of modulus of elasticity as that of the control PC concrete mix. Values of the twenty eight days modulus of elasticity of concrete containing 30 %, 40 % and 50 % GGBS are respectively 1 %, 2 % and 1.3 % higher than the PC concrete mix, cured under the summer curing environment.
- The winter curing environment has an adverse effect on the twenty-eight days modulus of elasticity values of GGBS, PFA and PC concrete, similar to the compressive strength values. It is concluded that proper curing of PFA and GGBS concrete under water at 20 °C or by the prevention of loss of moisture and storing at 20 °C enhances the modulus of elasticity.
- Concrete mixes cured under water at 20 °C have the higher value of modulus of elasticity than the concrete cured in sealed plastic bags at 20 °C.

6.1.6. Flat slabs

- From the Punching shear strength results presented in Table 5.2, it is observed that the punching shear failure load of RCC concrete flat slab containing 40 % GGBS is similar to the punching shear strength of PC only RCC flat slab and the punching shear strength decreases by 2 % and 4 % for RCC concrete flat slabs containing 30 % GGBS and 50 % GGBS respectively.
- The punching shear failure load of RCC flat slabs containing 10 %, 20 % and 30 % PFA are 2 %, 6 % and 4 % lower respectively than the RCC flat slab containing PC only. Overall there was a negligible reduction in the punching shear strength of flat slab specimens made with GGBS up to 50 % and PFA up to 30 %.
- From the experimental results of punching shear strength in chapter 5, it can be concluded that ACI 318 and BS 8110 estimates for punching shear strength, ignoring the partial safety factors are very similar to the real experimental punching shear strength values and the estimates of the Euro code 2 for punching shear strength are over-conservative.
- Based on the materials used and the experimental results of Punching shear strength of flat slabs, it is concluded that no special design requirements are necessary for the punching shear strength of RCC flat slabs containing GGBS up to 50 % and PFA up to 30 %, and the same design equations for punching shear strength, given in different codes of practice can be used without any further modifications.
- The pattern of cracking on all the specimens was similar. Radial cracks formed in the middle of the slab, extending gradually to the edges. Some circumferential cracks developed before punching shear failure. The pattern of cracks development was the same as explained by Chana (1991). The average crack width at the point of failure was similar for all slab specimens and was in the range of 0.25 mm to 0.35 mm, which is similar to the results of the punching shear strength test of RCC flat slabs performed by other researchers and reviewed in Chapter 2. The first crack appeared in the load range of 80 kN to 90 kN for all slab specimens.

- It was noted that the critical perimeter was at a distance of about 250 mm from the centre of the loaded column which was approximately equal to 2.0 d from the face of the column as calculated in Eurocode 2. Experimental results confirmed the authenticity of the critical perimeter at a distance of 2.0 d from the face of the column.
- At the initial load stages the strain in the reinforcement bars increased, with applied load. As the load approached the punching shear capacity there were irregularities in the strain distribution due to cracking. Similar behaviour was noticed by previous researchers in punching shear strength tests of RCC flat slabs.
- There is no significant difference in the strain values of concrete and steel for different concrete slab specimens. Strains in the steel were in the range of 2500 to 3500 μ mm/mm at the point of failure. From the strain results of steel it is clear that the reinforcing steel yielded just before the punching shear failure.
- Strains on the compression face of the slab specimens have almost the same pattern for all slab specimens. There is a negligible difference in the strain values of concrete at the failure load.
- Average mid-span deflection for all the slab specimens was between 3.6 mm and 4.7 mm. Average mid span deflection of 30 % PFA concrete was about 17 % higher and 40 % GGBS concrete was 10 % higher than the PC concrete control specimen. The average mid-span deflection was marginally higher by 0.3mm, in the slab specimens containing GGBS up to 50 %. This is not considered significant and no special design requirements are deemed as necessary. Average mid span deflection for 30 % PFA mix for flat slab specimen is 0.7 mm higher and 0.2 mm lower for 20 % PFA concrete than the PC concrete mix slab specimen. The average mid slab deflections for all slabs were recorded and were very small, which confirmed the failure mechanism as punching shears failure.

- In one of the slab specimen a central layer of reinforcement was provided to enhance the punching shear strength. The punching shear strength of the slab with the central bar was not improved although it helped in reducing the average mid span deflection. The idea of providing the central bars was to help in making a bond between the steel and the concrete which will add in the punching shear strength and further research is recommended on the provision of central bar reinforcement to enhance the punching shear strength of thinner slabs.
- It was noted that the failure of slabs with central steel was less brittle than for slabs without any central bars. The average mid span deflection at failure load for the slab specimen with the central layer of steel bars is about 11 % lower than the slab specimen without central bars. Strain values in the main bar are similar to the strain values of other slab specimens. The strain in the central bar is less and has not yielded.
- For a typical five storey office building of plan 50 m x 10 m with a 200 mm thick slab, a reduction of about 73 tonnes of CO_2 emissions and 324 Giga joules of energy savings can be achieved by using GGBS up to 50 % in the flat slab concrete only excluding the concrete used in columns, beams and foundation etc. A reduction of about 30 tonnes of CO_2 emissions and saving of 134 Giga joules of energy can be achieved by using the PFA up to 30 % in the same building compared to PC concrete.

6.2. Future Recommendations

- To check the effect of silica fume in RCC flat slabs for punching shear strength.
- To check the RCC flat slabs of different strengths and containing GGBS and PFA in different percentages for punching shear strength.
- To check the affect of GGBS, PFA and silica fume in reinforced concrete flat slabs on deflection, further testing is recommended.
- Due to the demand of high strength concrete (HSC) in high rise buildings, testing of high strength concrete containing GGBS, PFA and silica fume for punching shear strength of flat slabs.
- To enhance the punching shear strength of thin flat slabs by providing a central layer of reinforcement, further testing is recommended. Different sizes of central layer of reinforcement should be used and their effects on punching shear strength evaluated.
- Evaluation of creep for PFA, GGBS and silica fume concrete and analysing its effect on deflection is recommended.

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APPENDIX A CONCRETE ADMIXTURES

Concrete Admixtures

GRACE

ADVA® 650 & 655

High Performance Readymix Superplasticisers With Extended Slump Life

Description

ADVA® 650 and ADVA® 655 are high performance superplasticising admixtures specifically designed for the production of readymix concrete.
The products are designed to be used in a wide range of readymix concrete applications where, in addition to a superplasticising effect, long slump retention is required.

The extended slump retention of ADVA® 650 and ADVA® 655 make the products ideal for long or difficult pours where workability of up to two hours is needed. ADVA® 655 offers the greatest slump retention and the choice of product should normally be based on concrete trials with the materials and mix design to be used on the job.

The products are based on next generation polycarboxylate polymers and offer concrete producers the advantages of the latest advances in concrete admixture technology.

ADVA® 650 and ADVA® 655 conforms to EN 934-2 and are manufactured under closely controlled conditions to give a high quality consistent product.

Advantages

- ADVA® 650 and ADVA® 655 are especially suitable for producing high workability concrete, where extended slump retention is required.
- Matching the correct product to the mix design and application can produce extended slump life over normal superplasticisers, even where difficult cements are used, allowing long and difficult pours to be achieved.
- High workability flowing concrete can be obtained by incorporating ADVA® 650 or ADVA® 655 into a concrete mix designed for a 50mm slump.
- ADVA® 650 and ADVA® 655 have minimal impact on setting time when used at normal dosages.
- Providing suitable mix designs are employed, superplasticised concretes, based on ADVA® 650 or ADVA® 655 remain cohesive. Normal pump mixes are recommended for this application.
- ADVA® 650 and ADVA® 655 can be used to effect high range water reductions, leading to considerable increases in compressive strength. Impermeability and durability are correspondingly improved.

Typical Properties

Appearance: Clear amber/straw liquid.
Air Entrainment: 1% approx.
Chloride Content: Nil.

Method Of Use

ADVA® 650 and ADVA® 655 are supplied ready for use. When producing normal workability concrete it should be added in its supplied form with part of the batching water, after the addition of the cementitious component.

After addition of admixture, a further mixing cycle of at least 2 minutes is recommended to enable ADVA® 650 and ADVA® 655 to efficiently disperse the mix components.

Addition Rates

Range: 500ml-1300ml per 100kg cement.
As with most products of this type, the magnitude of the effect obtained with ADVA® 650 and ADVA® 655 is governed by the quantity of product used and the specific nature of the concrete and its constituent materials.
It is necessary, therefore, to assess performance under site conditions using site materials to determine optimum dosage and effect on both plastic and hardened concrete properties, such as cohesiveness, workability retention, set characteristics, early rate of strength gain, ultimate compressive strength and shrinkage when these are of consequence. **As a guide to these trials, an addition level of 0.4-0.7% ADVA® 650 and ADVA® 655 volume/ weight of cement is recommended.** For advice and assistance with your trials we would recommend that you consult Grace.

APPENDIX B

- Water absorption of fine aggregates (Trial 1)

M2 = 2564.7g

Temperature of water 19oC

M3 = 1615.5g

Temperature = 19oC

M1 = 1598.4g

M4 = 1586.2g

Apparent particle density = ρa = ρw M4 /M4 – (M2 –M3)

= 0.9984 * 1586.2 / 1586.2 –(2564.7 – 1615.5) =2.486 Mg/m3

Particle density on an oven –dried basis = ρrd = M4 / M1 – (M2 – M3)

= 1586.2 / 1586.2 – (2564.7 -1615.5) = 2.44

Particle density on a saturated and surface – dried basis ρssd = M1 / M1 – (M2 – M3)

= 1598.4 / 1598.4 –(2564.7 – 1615.5) = 2.46

Water Absorption (as a percentage of the dry mass) after immersion for 24 hours

WA24 = 100*(M1 – M4)/M4 = 100(1598.4 – 1586.2) / 1586.2 = 0.77%

- Water absorption of coarse aggregate (Trial 2)

M2 = 2409.4

M3=1248.2

M1=1370.8

M4=1352.2

WA=100*(1370.8-1352.2)/1352.2=1.375%

- Water absorption of fine aggregate (Trial2)

M2=1996.8

M3=1570.9

M1=1164.2

M4=1156.7

WA=100*(1164.2-1156.7)/1156.7=0.65%

- Water content of coarse aggregate

3280.3-3246.9=33.4grams

Dry weight=3246.9 grams

Water content=33.4/3246.9*100=1.03%

- Water content of fine aggregate

3047.2-3002.9=44.3grams

DRY WEIGHT=3002.9

WATER CONTENT =44.3/3002.9*100=1.48%

APPENDIX C SLAB DEFLECTIONS

In the Figures C-1 to C-12 the deflection readings of slab specimens with support movements and corrected mid span deflections are presented.

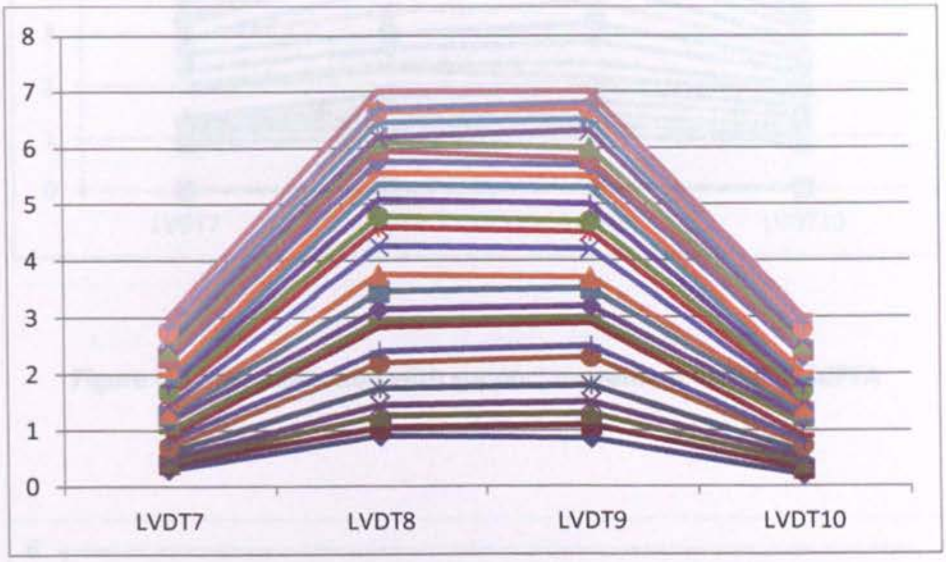

Figure C-1 Slab deflection with support movement for 100PC-Control

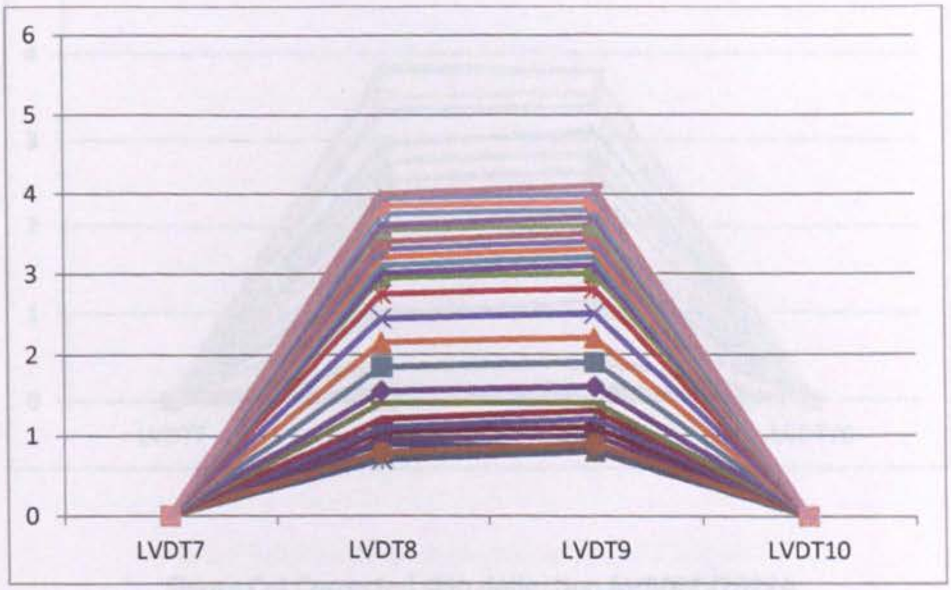

Figure C-2 Corrected slab deflection for 100PC-Control

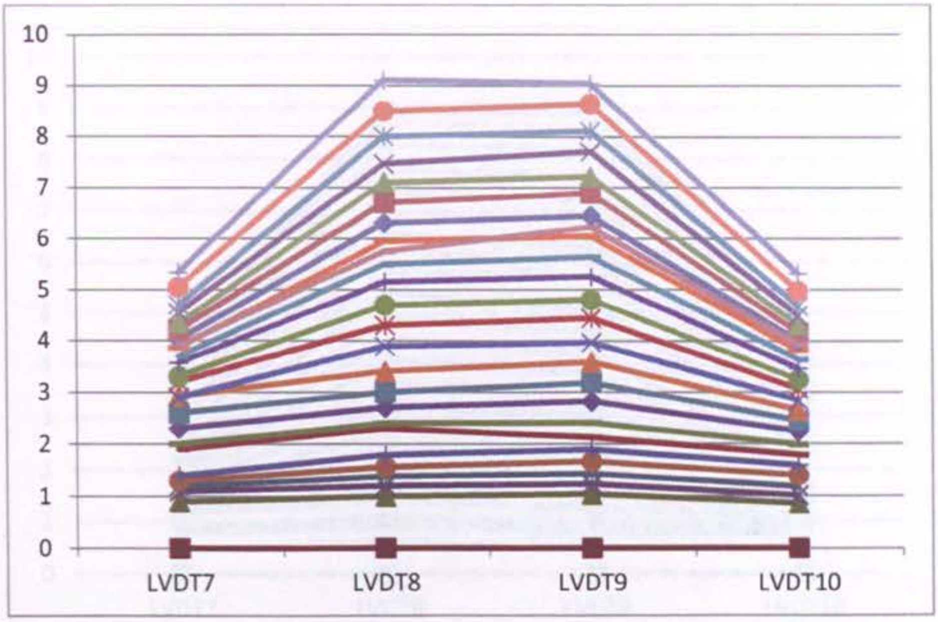

Figure C-3 Slab deflection with support movement for80PC/20PFA

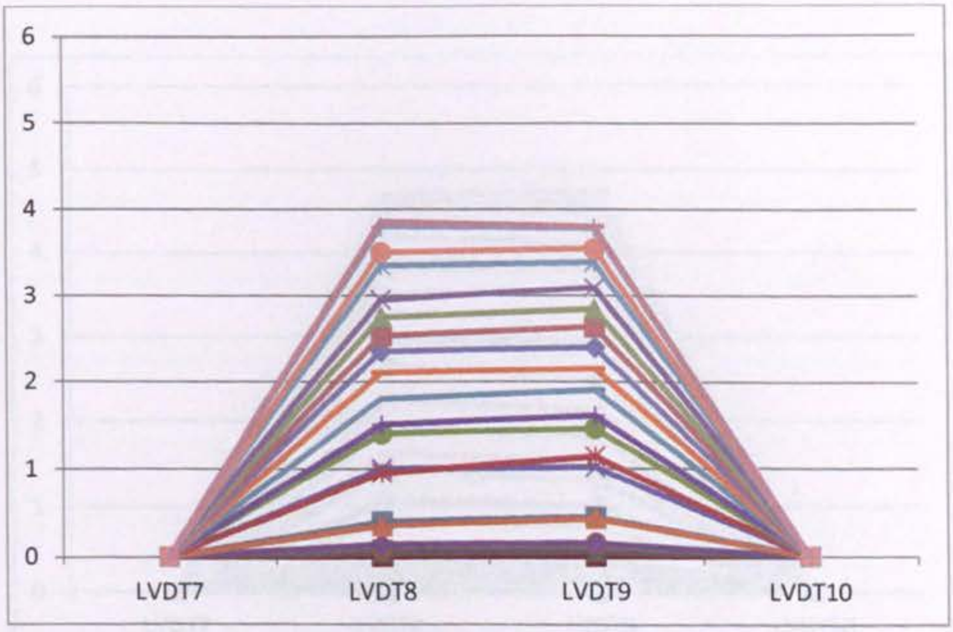

Figure C-4 Corrected slab deflection for80PC/20PFA

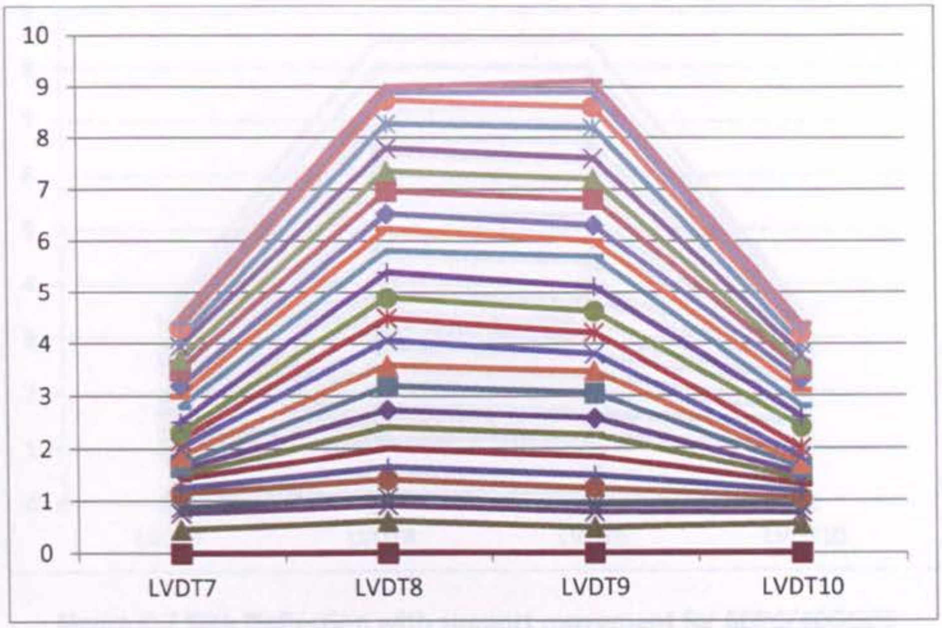

Figure C-5 Slab deflection with support movement for 70PC/30PFA

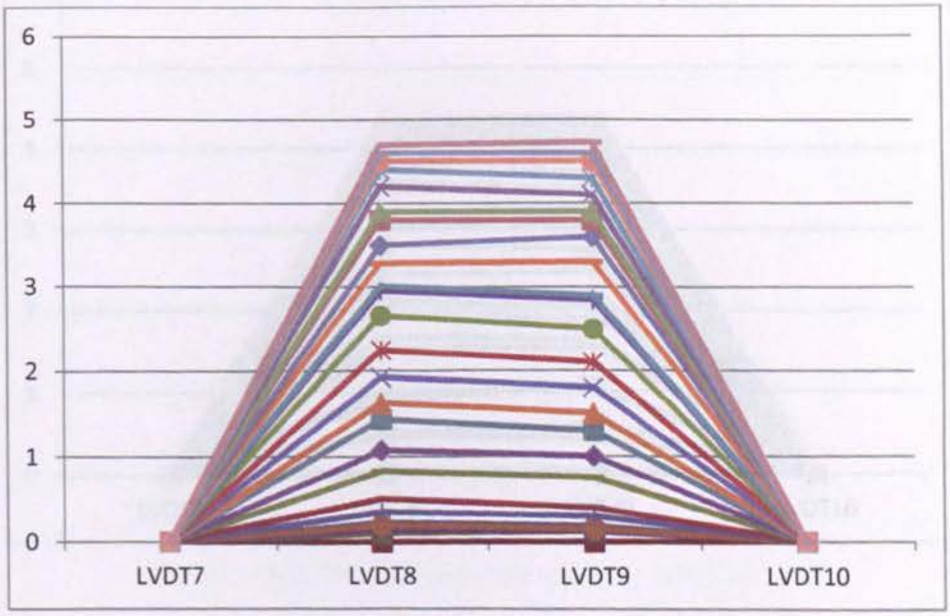

Figure C-6 Corrected Slab deflection for 70PC/30PFA

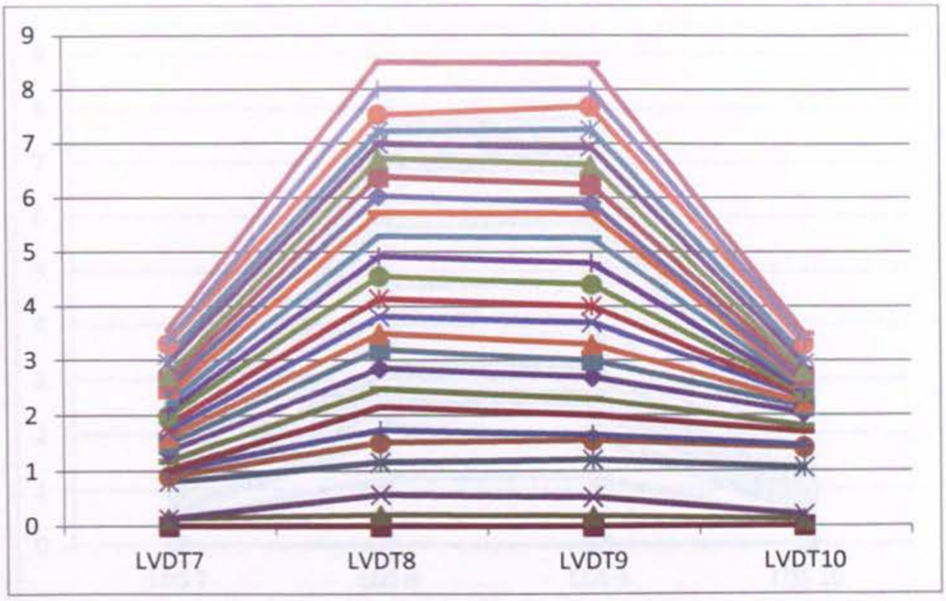

Figure C-7 Slab Deflection with support movement for 60PC/40GGBS

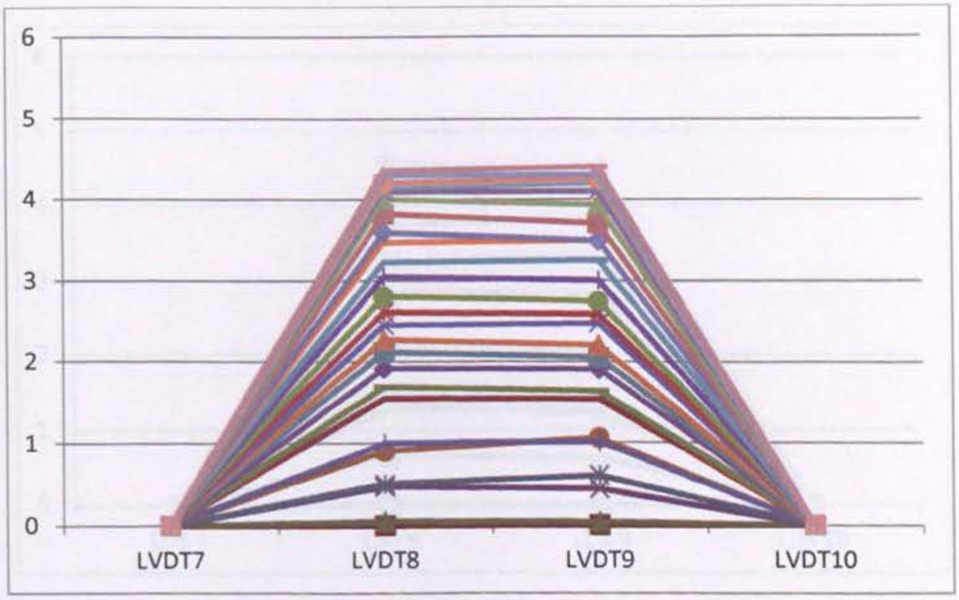

Figure C-8 Corrected Slab deflection for 60PC/40GGBS

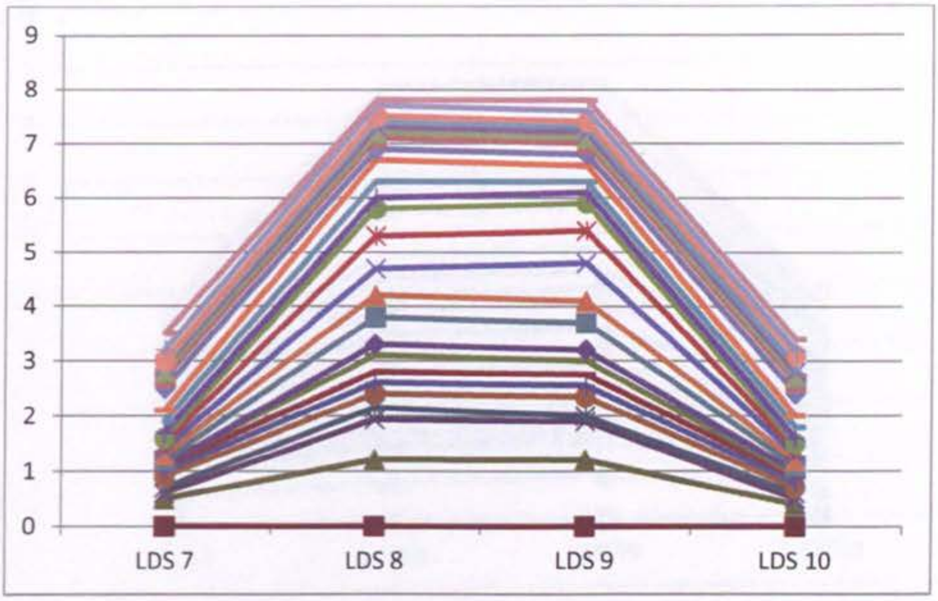

Figure C-9 Slab Deflection with support movement for 50PC/50GGBS

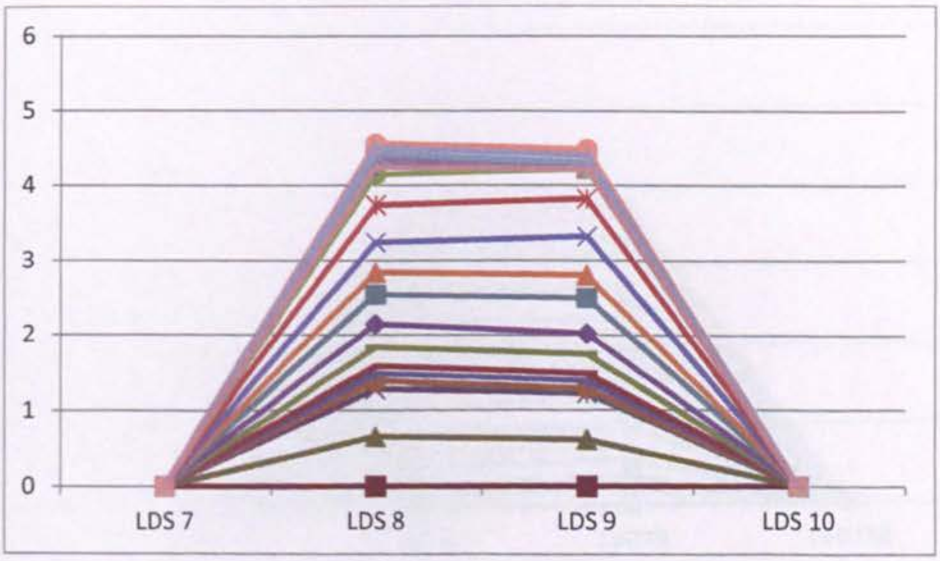

Figure C-10 Corrected Slab deflection for 50PC/50GGBS

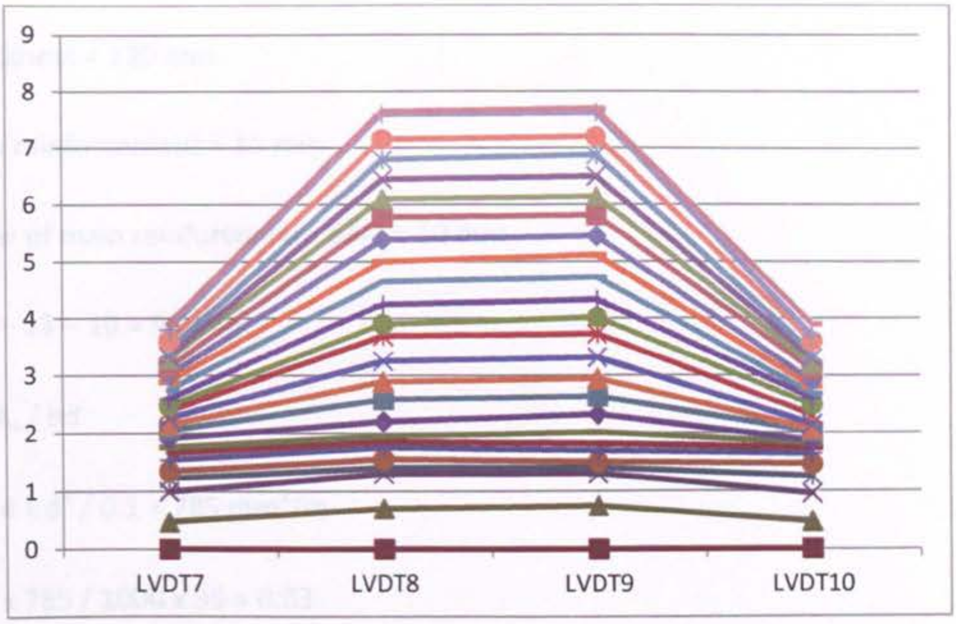

Figure C-11 Slab Deflection with support movement for 100PC-CB

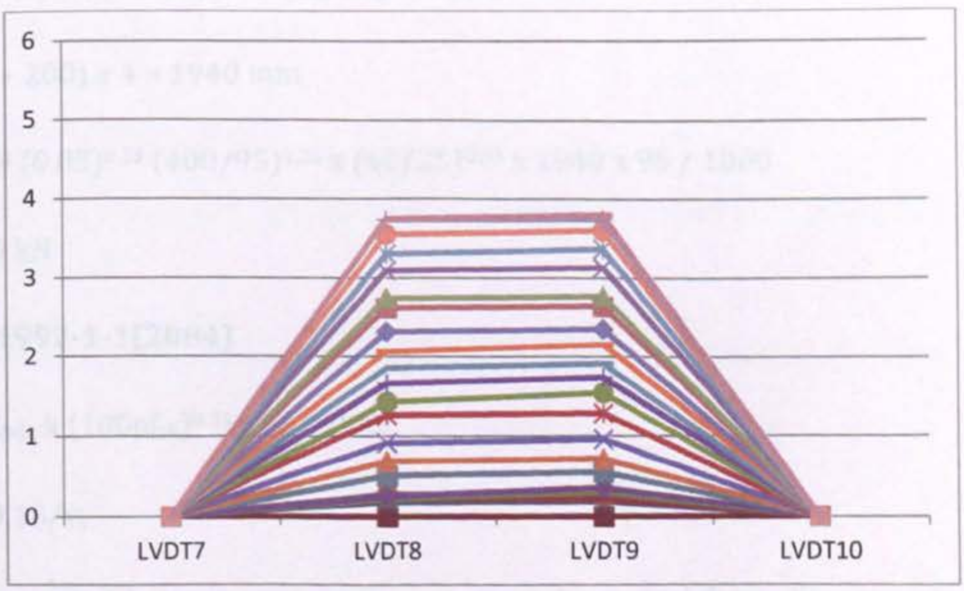

Figure C-12 Corrected slab deflection for100PC-CB

Punching shear calculations

Slab thickness = 120 mm

Cover to reinforcement = 15 mm

Diameter of main reinforcement bars = 10 mm

d = 120 – 15 – 10 = 95 mm

$\rho = 100A_s / bd$

$A_s = \pi / 4 \times d^2 / 0.1 = 785\ mm^2/m$

$\rho = 100 \times 785 / 1000 \times 95 = 0.83$

For $f_{cu} = 40$ MPa

BS 8110 (1997)

$V = 0.79\ (\rho)^{0.33}\ (400/d)^{0.25} \times (f_{cu}/25)^{0.33} \times ud/1000$

$u = (3d + 200) \times 4 = 1940$ mm

$V = 0.79\ (0.83)^{0.33}\ (400/95)^{0.25} \times (40/25)^{0.33} \times 1940 \times 95 / 1000$

V = 229 kN

BS EN 1992-1-1(2004)

$V_{Rdc} = C_{Rdc}\ k\ (100\rho f_{ck})^{0.33} \times ud/1000$

$C_{Rdc} = 0.18/\gamma_c$

γ_c is the partial factor for concrete, we ignore the partial factor for concrete for this exercise.

f_{ck} is the cylinder crushing strength and is taken as 0.8 times the cube crushing strength.

so f_{ck} is 32 MPa

$k = 1 + (200/d)^{0.5} \leq 2$

k= 2.45 so value of k used is 2

$u = u_o + 4\pi d = 800 + 1193.8 = 1994$ mm

$V_{Rdc} = 0.18 \times 2\ (0.83 \times 32)^{0.33} \times 1994 \times 95/1000$

$V_{Rdc} = 203$ kN

ACI 318 (2008)

$V_{uo} = ud\ (V_n)\ /1000$

V_n is the smallest of the following values

$V_n = (1+2/\beta_c)\ \sqrt{f_c}\ /6 = (1+2/2)\sqrt{32}/6 = 2.94$

$V_n = \{(\alpha_s\ d/u) +2\}\sqrt{f_c}\ /12 = \{(40 \times 95/1180)+ 2\}\sqrt{32}/12 = 2.45$

$V_n = \sqrt{f_c}\ /\ 3 = \sqrt{32}\ /\ 3 = 1.89$

α_s is 40 for internal column

And β_c is the ratio of the column longer span/shorter span and should be equal to or greater than 2.

f_c is the cylinder crushing strength and is taken as 0.8 times the cube crushing strength

so f_c is 32 MPa

So $V_n = 1.89$

u is taken at a distance of 0.5d from the face of the column and is equal to (d+200) x 4. u = 1180 mm

$V_{uo} = 1180 \times 95 \times 1.89\ /\ 1000 = 212$ kN