Bounding recycled aggregate pavement mixtures using hydraulic binders and cold bitumen emulsion.

ZHAO, Bin.

Bounding Recycled Aggregate Pavement Mixtures Using Hydraulic Binders and Cold Bitumen Emulsion

Bin Zhao

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ABSTRACT

With the current emphasis on sustainable development, recycling in the construction industry including highway planning, design, construction and maintenance has become a default option. Traditionally, recycled aggregate has been employed as filling or capping materials. However, the need to replace virgin materials in higher grade applications and reduce landfill has stimulated the need to enhance their performance. The requirements of using low energy and low environmental impact binders such as bituminous emulsion and industrial by-products as hydraulic binders whilst maintaining a long shelf life presented a further challenge.

The primary aim of this research was to investigate methods by which a mixture of recycled aggregate composed of road planings, concrete demolitions and bricks with proprietary bitumen emulsion as binder could be enhanced to comply with the prevailing specifications and performance requirements for pavement materials, by using novel combinations of bituminous emulsions and latent hydraulic binders.

The preliminary investigation focussed upon the establishment of an appropriate method of compaction of bitumen emulsion recycled aggregate mixtures to ensure results were consistent and representative of field performance. The main investigation evaluated the environmental conditions including freeze-thaw, low and high humidities and varying temperatures upon the behaviour and performance of loose pre-compacted and compacted recycled product using a range of novel latent hydraulic binders and bituminous emulsion combinations. Test methods included Indirect Tensile Stiffness Modulus test, Repeated Load Axial test, Indirect Tensile Fatigue test, Compressive Strength test and a novel modification of the Indirect Tensile Strength test was proposed and developed for enhance assessment of performance.

The key findings were that whilst bitumen emulsion mixtures could perform adequately, the addition of a latent hydraulic binder enhanced the mixture's performance in terms of mechanical properties and withstanding extreme conditions exemplified by freeze-thaw and high humidity, whilst maintaining shelf life. However, it was deduced that the performance could be heavily influenced by the condition, consistency and composition of the recycled aggregate. It is recommended that further work should focus on rigorously investigating the influence of recycled components on mixture properties to optimise their performance for given applications, and extended to include tar bound material.

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LIST OF ABBREVIATIONS

AASHTO	The American Association of State Highway and Transportation Officials
BACMI	British Aggregate Construction Materials Industry
BBA/HAPAS	British Board of Agrément, the Highway Authorities Product Approval Scheme
BFS	Blast furnace slag
BS	British Standard
BS DD	British Standard Draft for Development
BOS	Basic oxygen steel slag
CBM	Cement bound granular materials
CBR	California Bearing Ratio
CI	Confidence interval
CIRIA	Construction Industry Research and Information Association
CRCP	Continuously reinforced concrete pavement
C&D W	Constriction and demolition waste
DBM	Dense Bitumen Macadam
DETR	Department of Environment, Transport and the Region
DMRB	Design Manual for Roads and Bridges
DSR	Dynamic Shear Rheometer
EAM	Emulsified asphalt mixture
F-T	Freezing and thawing test
GBS	Granulated Blastfurnace Slag
GGBS	Ground Granulated Blastfurnace Slag
HAUC	Highway Authorities and Utilities Committee
HMB	High modulus base
HRA	Hot rolled asphalt
IBA	Incinerator Button Ash
ITFT	Indirect Tensile Fatigue test
ITS	Indirect Tensile Strength test
ITSM	Indirect Tensile Stiffness Modulus test
JRC	Jointed reinforced concrete pavement
LOI	Loss on ignition
MCHW1	Manual of contract documents for highway work

MCHW2	Notes for Guidance on the manual of contract documents for highway works
MSWI	Municipal Solid Waste Incinerator
MSA	Million standard axle
NAT	Nottingham asphalt tester
OMC	Optimum moisture content
PFA	Pulverised Fuel Ash
PCSMs	Permanent cold lay surface materials
Pen	Penetration
PQC	Pavement Quality Concrete
PRD	Percentage refusal density
RAP	Recycled asphalt planings
RH	Relative Humidity
RLAT	Repeated Load Axial test
SBM	Slag Bound Materials
SHRP	Strategic Highway Research Program
SHW	Specification for Highway Works
SMA	Stone Mastic Asphalt
SPC	Static pressure compression
SWPE	Scott Wilson Pavement Engineering
TFV	Ten Percent Fines Value
TRL	Transport research laboratory
URC	Jointed un-reinforced concrete pavement
WRAP	Waste & Resources Action Programme

LIST OF SYMBOLS

μ	micron (metre x 10^{-6})
m	Metre
km	Kilometre
kg	Kilogram
E	Strain
G*	Complex shear modulus
Δ	Phase angle
T	Temperature
t	Tonne
υ	Poisson's ratio
Mol/litre	Moles of solute per litre

CHAPTER 1 INTRODUCTION

1.1 BACKGROUND

In line with the UK government policy on recycling and encouraged by the successful experience of other countries, Barnsley Metropolitan Borough Council set up an aggregate recycling plant. The project was undertaken in co-operation with two industrial companies, Total Bitumen and MultiServ International. The recycled aggregate processed in the recycling plant comprised of demolished concrete, masonry and road planings. The incoming recycled aggregate was first crushed into two sizes, 0/10mm and 10/20mm respectively, and then mixed in certain proportions to create a mixture with a gradation following a 20mm Dense Bitumen Macadam (DBM). The aggregates were further mixed with a patented slow set bitumen emulsion and the mixed material was then stockpiled for piecemeal application. Since the mixture is mixed at ambient temperature, no heating up is undertaken of either the aggregate or the bitumen emulsion. The final product is therefore referred to as cold mixed materials. The Metropolitan Council and the companies involved were looking at various applications for such cold mixed materials, such as surfacing, base/binder course, trench reinstatement for less trafficked roads, footpaths and cycle-tracks. Figure 1-1 presents trials of cold mixes for surfacing and patching respectively, conducted by the cooperating companies.

Cold mixes as surface materials

Cold mixes as reinstatement materials

Figure 1-1: Cold mixes application

The School of Environment and Development at Sheffield Hallam University was approached to investigate the suitability of such cold mixed materials as reinstatement materials. The following requirements were emphasised by the industrial companies as those required for successful cold mixed reinstatement materials:

- Shelf life: The cold mixed materials were intended to be mixed and then stockpiled for long periods between mixing and application. A long shelf life was necessary for the successful operation of the recycling plant. This is a clear advantage of cold mixed materials over traditional hot mixed materials.
- Physical performance: British Board of Agrément (BBA) together with the Highway Authority Product Approval Scheme (HAPAS) has set out a guideline for the approval and certification of Permanent Cold-lay Surfacing Materials (PCSMs). The companies hoped that their product could be certified as Permanent Cold-lay Surfacing Materials by BBA/HAPAS.

With the supply of bitumen emulsion and aggregate from the companies, research was conducted, initially looking at the performance of bitumen emulsion recycled aggregate. Subsequently, the research was extended to investigate the performance of bitumen emulsion recycled aggregate mixture with the addition of Ground Granulated Blastfurnace Slag (GGBS) at a later stage. The applications of such cold mixture were extended beyond reinstatement materials.

Aggregate recycling has been a focal research topic in recent years. A great deal of research has been conducted, utilizing recycled aggregate for various purposes. Relevant parts of the British Standards, European Standards and Highway Design Specifications have been re-written to facilitate the application of recycled aggregate. Both bitumen emulsion and GGBS have been proposed as binders for aggregate recycling purpose. Research and application of bitumen emulsion as binder in the recycling process has been undertaken and widely reported, but no research on the aggregate recycling with bitumen emulsion and GGBS as binders has been published in the public domain.

1.2 AIMS AND OBJECTIVES

The primary aim of this research was to investigate methods by which a mixture of recycled aggregate composed of road planings, concrete demolitions and bricks with proprietary bitumen emulsion as binder could be enhanced to comply with the prevailing specifications and performance requirements for pavement materials by using novel combinations of bituminous emulsions and hydraulic binders. The aim was achieved by:

- Comparing the performances of typical cold mixtures using locally available aggregate and proprietary bitumen emulsion supplied by the co-operating companies in this project, with the required performance of reinstatement and general pavement materials.

- Devising a method of improving the performance of the mixture of bitumen emulsion and recycled aggregate without compromising the requirements of shelf life.
- Exploring the behaviour and properties of cold mixes with and without cementitious binder considering the prevalent UK climatic conditions.
- Reviewing the conventional test methods and developing modifications where appropriate to assess emerging unique characteristics of the modified cold mixed materials.
- Developing a pavement design based on the performance of the enhanced cold mixed materials.

1.3 METHODOLOGY

The rationale for the programme of research was based upon an initial review of existing knowledge, previous research and established industrial practice. This was followed by an identification of key criteria for assessing the performance and use of conventional and hybrid bituminous mixed materials, particularly with reference to environmental factors and climatic sensitivity within the conditions prevailing in the UK. The link between the principal objectives and the research methodology is presented in Figure 1-2.

The experimental investigation was split into appropriate stages:

- The initial focus was to investigate a range of compaction methods e.g. Marshall Compaction, Gyratory Compaction etc, to establish an appropriate means of compaction to be used as the baseline comparator throughout the main investigation that would ensure consistent results representing of field compaction, preferably with a good historical data base.
- The main investigation focussed upon investigating the behaviour of a range of binder combinations, mainly latent hydraulic binder and bitumen emulsion, under the effect of a range of environmental conditions frequently encountered in the UK. These included exposing both loose pre-compacted and compacted mixtures to freeze-thaw cycles, low and high humidities and varying temperatures and measuring their effects upon the performance.

The most widely employed performance indicator for bituminous materials is stiffness modulus and for hydraulic bound materials is compressive strength. The stiffness modulus test is a non-destructive test and the same specimen is able to be monitored over time revealing the performance evolution process. This is a great advantage in monitoring the performance evolvement of latent hydraulic bound materials, which develop their stiffness and strength over a long period. As a result, the Indirect Tensile Stiffness Modulus (ITSM) test was employed throughout this research process as the primary test method. Tests related to rut resistance and fatigue resistance were also conducted.

1.4 ORGANISATION OF THE THESIS

The research findings and recommendations are included in the twelve chapters and schematically presented in Figure 1-3.

Part I: Introduction

- Chapter 1: The background, aims and objectives of this project are introduced.

Part II: Literature review

- Chapter 2: The market, benefits and limitations related to aggregate recycling are introduced.
- Chapter 3: Firstly, the performance indicators and related test methods of the bitumen are introduced, and then the types and application of various bituminous products, such as bitumen emulsion, bitumen aggregate mixture are summarised.
- Chapter 4: The current recycling techniques are introduced and compared.
- Chapter 5: A summary of the test methods employed in this research are listed in this chapter.

Part III: Experimental work

- Chapter 6: The aim of this chapter is to select the compaction method for use in this research project from a series of compaction methods developed for hot mixes design. The selected compaction method should be able to fabricate samples with consistent performance comparable to that compacted by roller on site.
- Chapter 7: The performances of cold mix specimens, composed of bitumen emulsion and recycled aggregate curing at various moisture and temperature conditions, are presented in this chapter.

- Chapter 8: The aim of this chapter is to investigate the effectiveness of the addition of GGBS into the bitumen emulsion recycled aggregate mixtures. The investigation process included a preliminary research and a full-scale investigation.
- Chapter 9: Various factors influencing the performance of the cold mixes composed of recycled aggregate with bitumen emulsion and GGBS as binder are investigated in this chapter, including the type and composition of recycled aggregate, the curing temperature and the accelerators.
- Chapter 10: A comparison of cold mixes, hot mixes and cement-bound recycled aggregate mixes is reported in this chapter. It is helpful to understand a new material further by comparing it with well-understood established materials.

Part IV: Pavement design

- Chapter 11: A pavement design based on the performance of the cold mixed recycled aggregate with bitumen emulsion and GGBS as binder was undertaken and presented in this chapter.

Part V: Conclusions and recommendations

- Chapter 12: The conclusions, limitations and recommendations of this research are presented in this chapter.

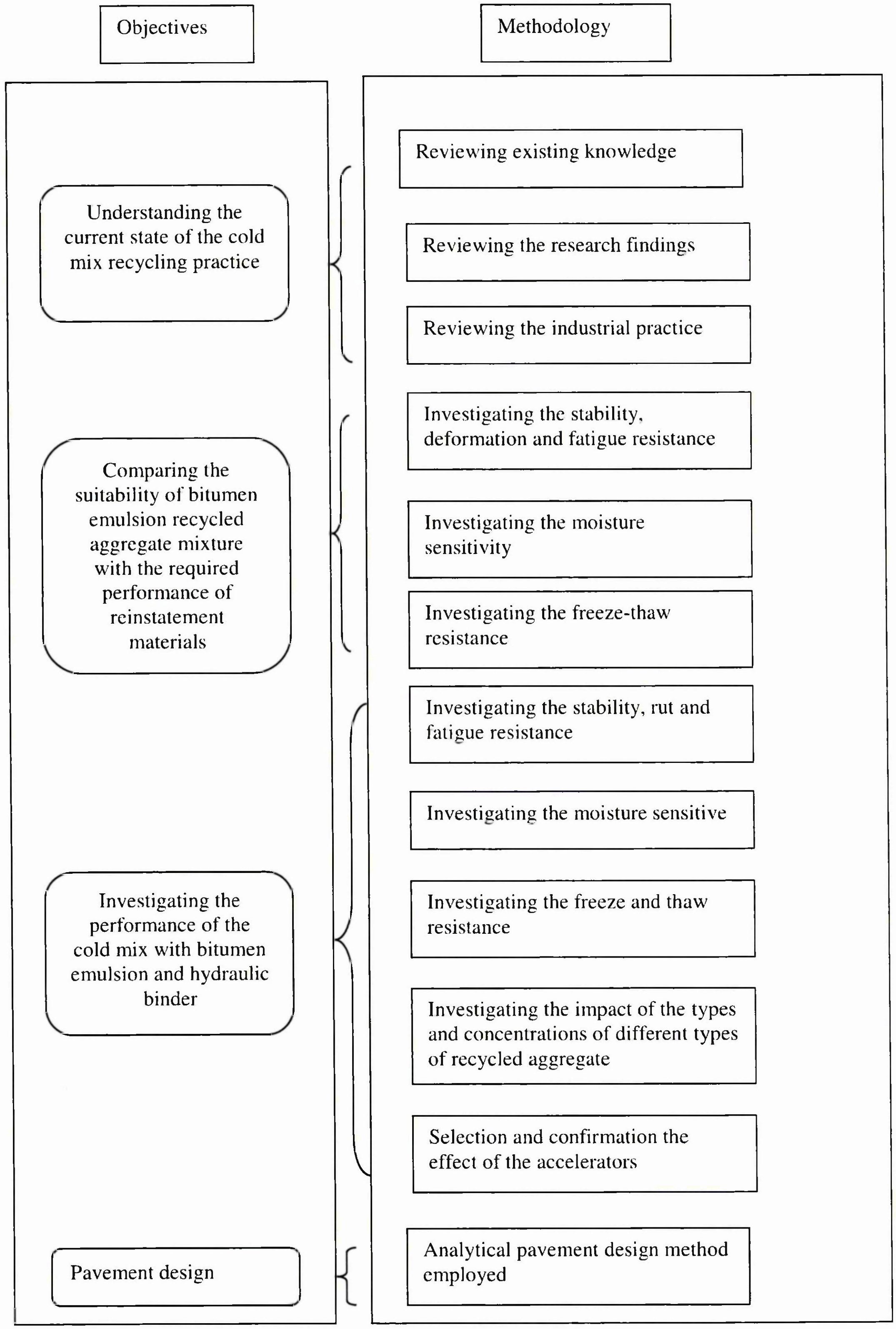


Figure 1-2: Research methodology

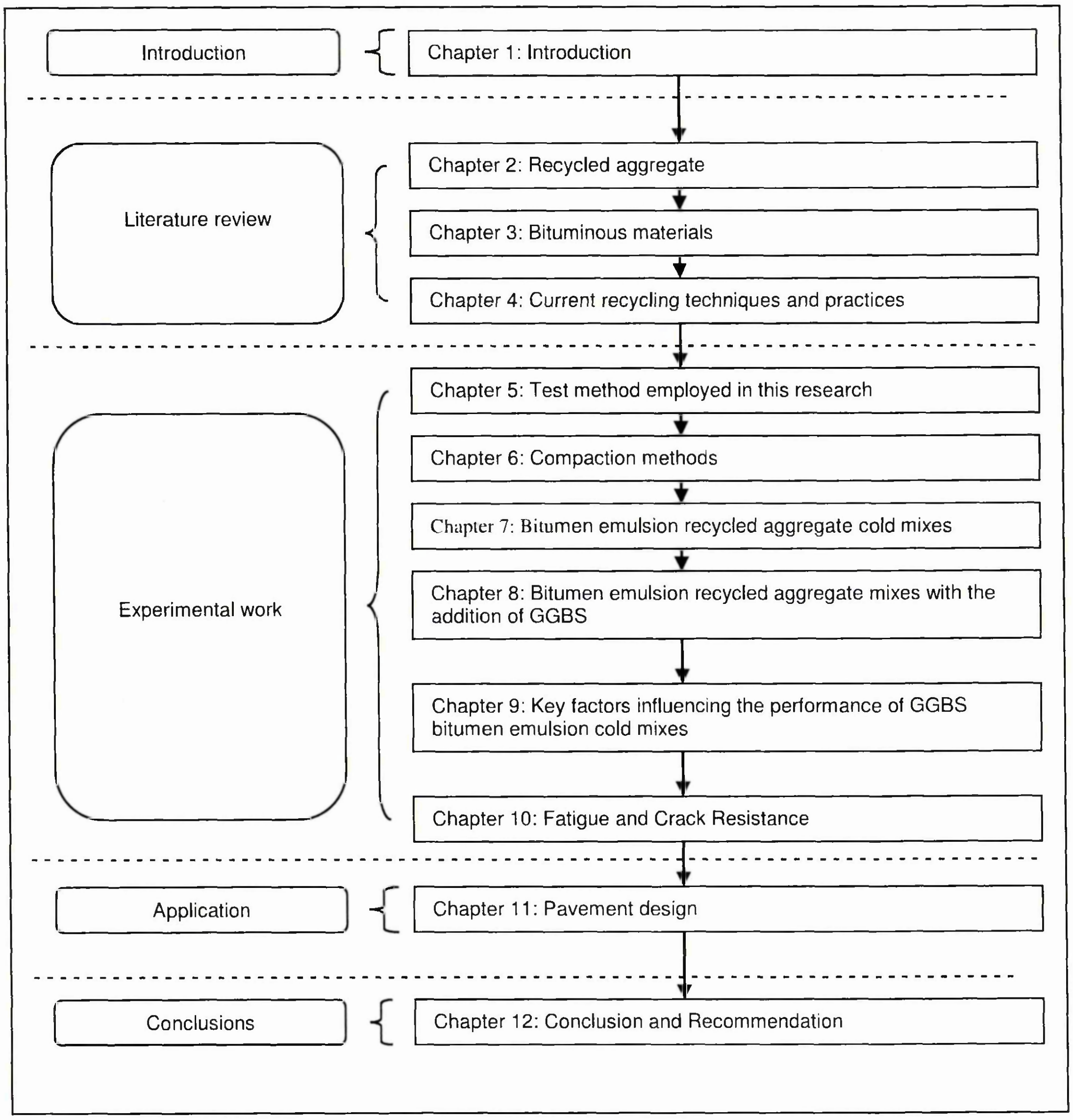


Figure 1-3: Schematic organisation of the thesis

CHAPTER 2 RECYCLED AGGREGATE

2.1 BACKGROUND

Concern over the protection of the environment and over energy savings has led to a growth of interest in the re-use and recycling of construction materials. The output of construction and demolition waste, excavation waste and road planings in 2001 was nearly 100 Million tonnes. This is by far the largest source of any recyclable materials in England and Wales (Reid and Jostein, 2002). As road building accounts for one-third of the total aggregate demand (Figure 2-1) (BACMI, 1992) and the requirements for aggregate in the lower layer of pavement are relatively easier for recycled aggregate to fulfil, pavements have been a very good application for recycled/secondary aggregate. Such recycling practice saves virgin aggregate, cuts landfill demand and possibly reduces transport if the recycling is conducted in-situ. Because of all these benefits, aggregate recycling practice and research is encouraged all over the world. Recycled/secondary materials that can now be readily used in road construction include recycled construction and demolition materials, asphalt planings, pulverized fuel ash (PFA), china clay sand, slate aggregate, steel and blast furnace slag, colliery spoil, incinerator bottom ash, crushed glass, recycled tyres and recycled plastic.

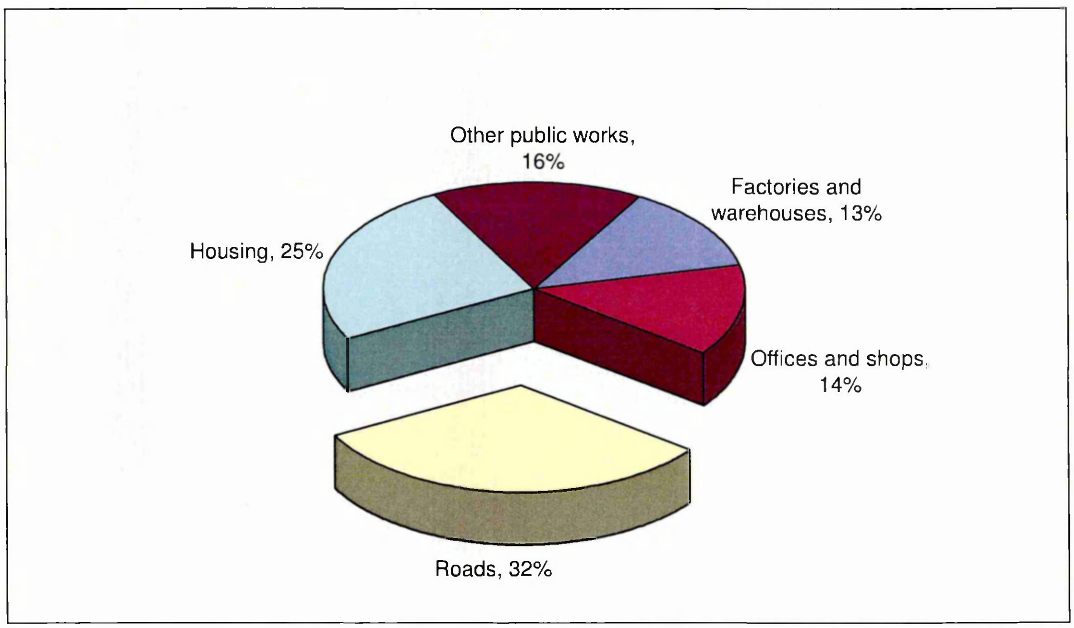


Figure 2-1: Aggregate consumption distribution in 1990 (BACMI, 1992)

Recycled aggregate refers to the aggregate that has been used previously in construction. Recycled aggregate may comprise:

- Construction and Demolition Waste (C&DW): the demolition of building and other man-made structures results in a range of waste materials, including concrete, brick, masonry, metal and timber etc. The excavation of trenches by the utility companies and for the installation of cable television is currently providing a significant source of such material.
- Asphalt road planings: asphalt road planings, typically comprising bituminous binder and aggregates, are removed from the surface of roads prior to maintenance work by machinery designed specifically for this purpose.

The term 'secondary aggregates' refers to mineral wastes and industrial by-products. The potential sources of secondary aggregates include:

- Colliery spoil: this represents the waste materials generated when extracting deep-mined coal.
- China Clay Waste: China clay is used in the paper and ceramic industries. For every tonne of China clay, an average of nine tonnes of waste is produced. Coarse sand is the main component of the waste and the sand possesses desirable engineering properties. It can be applied in both bound and unbound layers.
- Slate Waste: the slate is used as roofing material. Slate waste comes from the cutting and splitting of large blocks of slate. It has been estimated that the average ratio of waste to final product is 20:1.
- Pulverized Fuel Ash (PFA): PFA is the resultant ash carried out of a furnace by gas following the combustion of pulverised coal in coal-fired power stations. It is a fine powder that resembles Portland cement in fineness and colour. Because of its pozzolanic properties, it is widely used in the road base and foundation stabilisation process, usually activated by lime or cement. Fly ash bound materials as road base have many advantages over the traditional cement bound materials, for example the mixture hydrates at a slow rate, less heat is generated in the hydration process, and is therefore less likely to generate wide cracks, which is a common problem for cement bound materials.
- Incinerator bottom ash (IBA): the ash from Municipal Solid Waste Incineration (MSWI) is providing an increasingly significant source of secondary aggregates in the UK. High quality Incinerator Bottom Ash (IBA) can be used successfully

as embankment fill, road base material, in the production of asphalt material and in concrete building blocks.

- Blast Furnace Slag: blast furnace slag is a by-product obtained in the manufacture of pig-iron in a blast furnace, and is formed by the combination of the earthy constituents of the iron ore with the limestone flux. By employing different rates of cooling, three main types of slag are produced: air -cooled slag, which is crushed and used as an aggregate; granulated slag which is ground up for use in cement; and pelletised slag which is used as a light weight concrete aggregate. Ground granulated blast furnace (GGBS) is used in this project, exploiting its latent hydraulic properties.
- Steel slag: steel slag is a by-product that resembles igneous rock. The use of steel slag as an aggregate is constrained as it contains free lime and magnesia and requires weathering before use. Currently most of the steel slag used in road construction is in the asphalt layers.
- Foundry sand: foundry sand comes from utilising sand to prepare mould for casting. *'In the UK, the major reuse of foundry sand has been as structural fill to raise grade for construction site and highway embankment (Coventry et al, CIRIA 513, 1999).'*
- Crushed glass: crushed glass as aggregate has some apparent advantages including high hardness and almost zero water absorption. Crushed glass is able to replace virgin aggregate, producing bituminous asphalt base course material and replacing sharp sand as loose fill materials in road construction process (WRAP, 2003).
- Used tyres: whole tyres have been prevented from going to landfill since 2003 and shredded tyre waste will be prevented from going to landfill from 2006, therefore outlets for used tyres are urgently needed. Used in hot mixes, the shredded tyres are likely to absorb the bituminous binder at high temperature and therefore cold mix is a more appropriate recycling technique for tyres (Hulme and Day, 2004).

Although the UK is fortunate in having an abundant supply of primary aggregates, the producers and users of aggregates are now under heavy pressure to reduce consumption of primary resources and to switch to recycled and secondary aggregate.

2.2 PAVEMENT CLASSIFICATION

Pavements can be classified as flexible, rigid and composite pavement. Flexible pavements are those in which the surfacing and base materials are bound with bituminous binder. Rigid pavements comprise pavement quality concrete (PQC) used for the combined surfacing and base. Depending on if the concrete pavement is reinforced and how it is reinforced, the rigid concrete slab is classified as:

- Jointed un-reinforced concrete pavement (URC): un-reinforced concrete pavement has frequent transverse joints (approximately 5m apart) to prevent thermal cracking.
- Jointed reinforced concrete pavement (JRC): this type of pavement has less transverse joints than the plain concrete above.
- Continuously reinforced concrete pavement (CRCP): CRCP have much heavier reinforcement and joints are used only when necessary for construction purposes. A ground beam anchorage is required at terminations of every CRCP.

There are two variations of the flexible and rigid pavement.

- Flexible composite: where the surface course and the upper base material are bound with bitumen on a lower base of cement bound materials (CBM).
- Rigid composite: also referred to as Continuously Reinforced Concrete Base (CRCB), where an asphalt overlay or surfacing of at least 100mm on top of continuously reinforced concrete base.

The construction of rigid pavement is mainly a specialist job and often requires complex and expensive laying machinery. Rigid pavements are generally temperature and freeze thaw resistant and often have a long service life with minimum maintenance. They are extremely suitable for situations where high stresses are expected, such as airports and dockyards. Although widely used as road construction materials in countries like US, Canada and China, current practice in carriageway construction still favours flexible pavements.

2.3 FUNCTION AND MATERIALS OF THE PAVEMENT LAYERS

The layer classification for flexible as well as rigid pavement is illustrated in Figure 2-2. The flexible and flexible composite pavements are composed of three principal layers, namely foundation, base and surfacing. The rigid pavement is simply divided into surface slab and foundation. The concrete slab in the rigid pavement is equivalent to the combination of surfacing and base course in flexible pavement.

The layered pavement design is intended to economize the aggregate usage. In a layered road pavement structure, the aggregate quality, in terms of the durability and bearing capacity, increases from the bottom upwards, i.e. the specification requirement for any given layer is always higher than that of the layers immediately beneath it. This means that the same material could be used to construct a particular layer and all the underlying layers. The function of the road layers is summarised as follows:

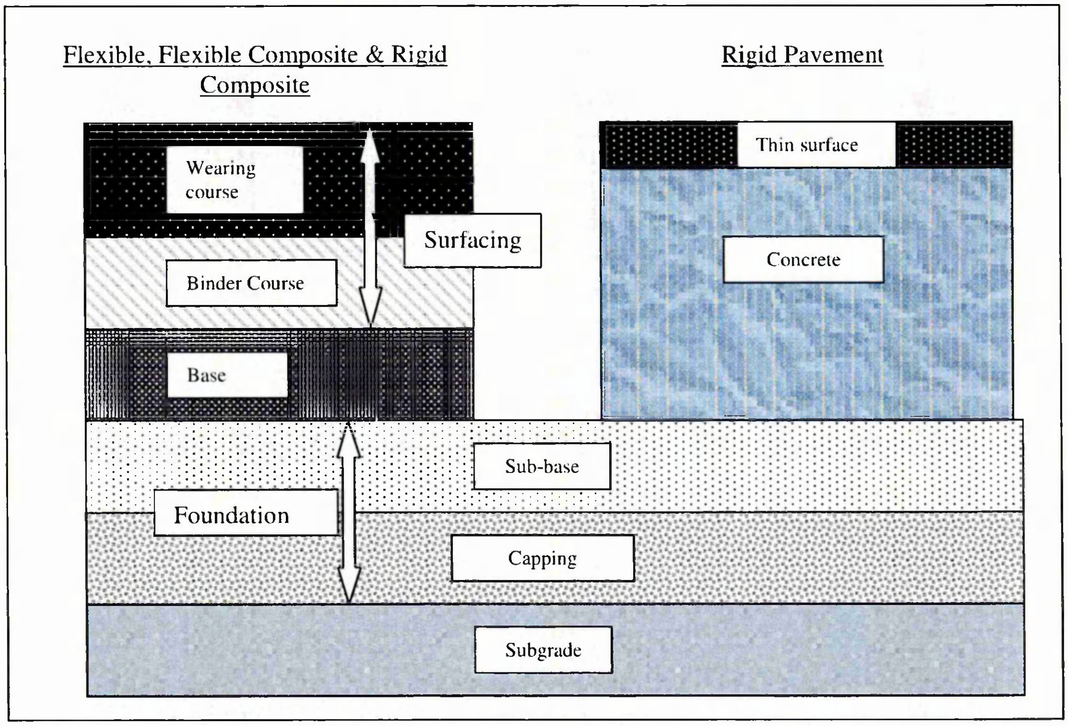


Figure 2-2: Pavement layers (DMRB7, HD 23/99)

- Foundation

Foundation includes capping and subbase layers. The foundation performs as a platform upon which more expensive and structurally significant materials can be laid and compacted. Most of the recycled aggregate currently are recycled into the foundation layers. Four types of subbase materials are specified in the MCHW:

Type 1 unbound mixture: Type 1 unbound mixture shall be made from crushed rock, crushed slag, crushed concrete, recycled aggregate or well burnt non-plastic shale and may contain up to 10% by mass of natural sand that passes the 4mm test sieve.

Type 2 unbound mixture: Type 2 unbound mixture shall be made from natural sands, gravels, crushed rock, crushed slag, crushed concrete, recycled aggregate or well burnt non-plastic shale.

Type 3 (open graded) unbound mixtures: Type 3 (open graded) unbound mixture shall be made from crushed rock and crushed blast furnace slag.

Category B (close graded) unbound mixtures: Category B (close graded) unbound mixture shall be made from crushed rock, crushed blast furnace slag or recycled concrete aggregate. Recycled concrete aggregate used in Category B (close graded) unbound mixtures shall not contain more than 5% asphalt.

- Base

This is the main structural layer of the pavement and is required to distribute the applied traffic loading so that the underlying layers are not overstressed. The cold mixed materials are intended for this layer. This layer of the pavement must resist permanent deformation and fatigue cracking from repeated loading.

The base course can be bituminous or cementitious bound. Continuously graded aggregate and hard bitumen binder are commonly used in this layer to offer high load distribution capabilities. Cement bound materials (CBM) generally have the advantage of high strength and less moisture sensitivity but are notoriously crack prone, which could develop to the surface of the pavement. Such cracking commonly referred to as reflective cracking. Many methods have been developed to curb this problem and in the UK, all the cement bound material that have an average 7 day compressive strength over 10.0N/mm^2 must have transverse cracks induced during construction. These transversely induced cracks are normally spaced at 3m centres. '*The use of cement treated base in flexible composite pavement has been given a new lease of life through the introduction of pre-cracking techniques. These appear successful and allow more closely spaced, finer cracks to be established which are less damaging for the bituminous layers (Brown, 1998).*'

Cold mixes with bitumen emulsion as well as hydraulic binder as base materials performs somewhere between bituminous and hydraulic bound materials. Such materials tend to possess higher stiffness than hot mixed materials and less crack prone than hydraulic bound materials.

- Surfacing

The surfacing is normally composed of two layers, binder and wearing course (Figure 2-2). The binder course is to provide a good shape and regular surface on which to lay the relatively thin wearing course as well as distribute traffic loads over the base course. The wearing course is the most expensive layer of the pavements and places strict requirements on the aggregate involved. Its functions include load distribution, skid resistance, drainage and some aesthetic effect. Apart from road planings, which are assumed to have fulfilled the aggregate requirements for surface course in its previous application, other recycled aggregate, such as mortar and concrete demolitions are seldom used in this layer.

2.4 CHALLENGES IN ROAD CONSTRUCTION

Several outstanding questions have to be addressed for the application of recycled aggregate in road construction:

- Supply and demand.
- Environmental concern.
- Physical performance.
- Economic consideration.

2.4.1 Supply and demand

Firstly, there needs to be a sufficient supply of C&DW and a market for it in road construction. This is the pre-condition to justify this research. According to the handouts from WRAP (2004), '*approximately 250 million tonnes of aggregates are used each year in England and Scotland as raw materials for construction. Of this supply, around 55 million tonnes are sourced from recycled or secondary resources. However, estimates suggest that a further 20-25 million tonnes a year of suitable recycled and secondary materials could be used for construction aggregates within the built environment.*' Within the recycled part, if classifying the application into low utility, intermediate utility and high utility according to Table 2-1, most of the recycled aggregate went into low utility applications (Table 2-2).

Table 2-1: Recycled aggregate utilization (Winter, 2001)

Material	Low utility	Intermediate utility	High utility
Bituminous planings and breakout	Haul road; general fill	Capping layer; sub-base	Hot and cold recycled bituminous materials
Construction demolition and excavation waste (excluding soil)	General fill	Building and road foundation materials, excluding concrete, foundation aggregates and additives	Building and structural units (for example, blocks); concrete and cement constituents; structural road construction layers

Table 2-2: Recycled aggregate in Scotland in 1999 (Winter, 2001)

Category	Arising (10^3 t)	Landfilled	Total recycled	Low utility	Recycled intermediate utility	High utility
		10^3t (Percentage of Arisings)		10^3t (Percentage of Total Recycled)		
Construction and demolition waste	8,728	4,504 (52)	4,224 (48)	3,957 (94)	112 (2)	155 (4)
bituminous planings and breakout	613	138 (22)	476 (78)	355 (75)	85 (18)	35 (7)
Industrial wastes and by-products	906	*554 (61)	*787 (87)	467 (59)	100 (13)	220 (28)
All Categories	10,247	*5,196 (51)	*5487 (54)	4779 (87)	297 (5)	410 (7)

* The sum of landfilled and total recycled exceeds the arisings due to the recycling of old, or legacy, and imported industrial wastes for which there are no arisings.

Barritt (2003) made estimates of the end use of recycled and secondary aggregates from a survey of 260 suppliers of recycled aggregates and showed that:

- 70% was used as 'fills' and this includes general fill, capping and structural backfills. This does not include unprocessed C&DW.
- 18% was used as sub-base.
- 5% was used in asphalt.
- 7% was used in concrete or as graded coarse aggregate.

From the above figures, 70% of the recycled aggregate used as fill and 18% of which used as subbase, which is equivalent to 88% used in the middle or low utility applications according to Winter's classification (2001) presented in Table 2-1. As a rule, aggregate for the bound layer is more expensive than for unbound layers. Application of recycled aggregate in bound layers will enhance the market value of recycled aggregates and make the separation, processing and transportation economically worthwhile.

In summary, there are large amounts of recycled/secondary aggregates yet to be reused. Among the recycled aggregates, most were recycled into low utility applications. There are researchers looking at the possibility of applying recycled aggregate including bricks, masonry into concrete or into hot bituminous mixes (Khalaf and Devenny, 2004). This research is focusing on recycling construction demolition and road planings, which constitute the largest part of recycled/secondary aggregate and its application in the pavement with bitumen emulsion and hydraulic binder. Such research will benefit not only the companies involved but also contribute to aggregate recycling in general.

2.4.2 Environmental concerns

The environmental impact associated with recycled and secondary aggregates operations could be associated with the following aspects (DETR, 2000):

- The impacts of land take and ancillary development.
- Dust.
- Noise.
- Water pollution and ground contamination.
- Vibration.
- Gaseous emissions and odour.
- Transport impacts in addition to the above (such as severance, congestion and delay).

Careful management and operation can avoid most of the problems mentioned above. The most controversial issue might be the potential water and soil pollution. '*The recycled aggregates may be contaminated as a result of previous use. Any oil, solvents or other contaminating material in the waste may be transported with the suspended matter. Where contaminants are soluble in water, such as road salt, these may also pollute the water. Concrete dust may be a potential problem since the lime contained in the concrete may dissolve to give an alkaline solution, although the quantity of lime present is rarely likely to be sufficient to significantly raise the pH of the water. Chlorinated solvents, if present, may dissolve in the water, drain as a separate layer or contaminate land. Lubricating oil and fuel oil may also form a separate layer and certain toxic components of the oil may dissolve in the water or contaminate land (DETR, 2000).*'

The Environment Agency is concerned about the long-term leaching problem from the recycled and secondary materials. Wider use of secondary materials in road construction will depend on users being able to demonstrate that these applications will have no deleterious effects on surface or ground water. Most contaminants are unlikely to be a concern if the materials are used in cement or bitumen bound form, as the exposure to percolating water is greatly reduced. If they are intended for use in unbound layers, however, it may be necessary to carry out a risk assessment to demonstrate that they will not pose a threat to the controlled waters. The bitumen emulsion mixed materials are partially coated by bitumen, which greatly reduced the potential leaking problem, if the recycled aggregate is safe for unbound layers, then it is safe for use in bitumen emulsion bound layers.

Since the potential pollution comes from the diffusion of hazardous substance into the water system through the contact between water and recycled aggregate, a leaching test is commonly employed to reveal the potential pollution of alternative materials. Research in this field has been conducted and reported, for example, CIRIA report 167 (Baldwin et al, 1997) and ALT-MAT[1] (Reid et al, 2001). Different recycled aggregate and industrial by-products were examined in CIRIA Report 167 to investigate their potential contamination on ground and surface water when used in the unbound layer of the pavement. The recycled/secondary materials were divided into three groups according to their potential to cause pollution as shown in Table 2-3. The report concluded that none of the by-products considered would give rise to serious concern about ground and surface water pollution if used in normal applications. ALT-MAT is a collaborative research project partly funded by the European Commission Directorate General for Transport, many case studies of the application of different recycled/secondary aggregates in Europe were included in the report. Their cases showed that the recycled aggregates were used successfully in unbound layers without any pollution problems, even Municipal Solid Waste (which is classified as Group 3 in Table 2-3) was used safely without causing any ground water pollution in France and Denmark.

Table 2-3: Recommended category for use based on leaching test (CIRIA Report 167)

Recommended category for use based on leach test		By product investigated in this project
Group 1	Unrestricted use (similar to limestone)	China clay sand
		Blast furnace slag
		BOS slag (Basic oxygen slag)
		Colliery spoil
		Rubber crumb
		Slate quarry residues
		Spend oil shale
Group 2	Unrestricted use, some restriction may apply for unbound material	Black top planings
		Cement kiln dust
		Demolition debris – brick rubble
		demolition debris - crushed concrete
		EAF (Electric Arc Furnace slag)/refractory blend sewage sludge incinerator ash
Group 3	Some restrictions will apply to unbound materials	Municipal solid waste incinerator ash (MSWI)

The recycled materials involved in this project, which are composed of asphalt planings, bricks and concrete demolitions, can be classified into Group 2 based on Table 2-3, which is safe to be used in unbound layer. In bitumen emulsion cold mixes,

[1] ALT-MAT: Alternative Materials in Road Construction.

the aggregates were partly coated with bitumen, therefore the potential pollution problems are further reduced.

2.4.3 Physical performance

The fact that recycled aggregate is mainly used in low and medium utility applications could be attributed to the following two reasons:

- The recycled aggregates often contaminated by clay, soil, wood, paper etc. Such materials are impossible to be used for high utility applications without proper segregation.
- The recycled aggregates have a lower physical performance than virgin aggregates.

Segregation of recycled aggregate can be carried out at source during the demolition or construction period or can be achieved by processing the mixed materials to remove the foreign materials. Segregation at source is most efficient in terms of energy utilization, economics and time (Reid, 2003). The current demolition industry, with highly skilled operators and very expensive specialised equipment, is well equipped to sort different recycled aggregate at source (Hurley et al, 2001). For recycling plants, there is a protocol to guarantee the output 'fit for purpose' (construction research communications, 2000), by visual inspection of the incoming materials and testing the bulk density, grading, fine content and particle shape at various times. Such measures are also helpful to secure the recycled aggregates from potential water and soil pollution.

Even with strict separation and processing, the recycled aggregates are still inferior to virgin aggregate in terms of consistency and physical performance. For example, the aggregate intended for pavement surface must have high abrasion resistance and proper surface texture. The performance of the recycled aggregate is highly variable and generally unable to satisfy these requirements, except recycled asphalt planings (RAP), which are assumed to have originally fulfilled such requirements. The recycled aggregate in this project includes RAP as well as mortar and crushed concrete, mainly intended for binder and base courses. Various properties are required for the aggregate in base/binder course, such as gradation, hardness, soundness, etc. It is relatively easy to achieve the required gradation but recycled aggregates frequently fail to meet the hardness requirement. Ten Percent Fine Value (TFV) and Aggregate Impact Value (AIV) (BS 812 Part 111, BS 812 Part 112, 1990) are often employed to assess the aggregate hardness. Table 2-4 lists the TFV requirements of different pavement layers extracted MCHW1 and Table 2-5 presents the typical properties and the suggested applications for recycled aggregate. Because of the masonry content, based on the figures suggested in Table 2-5, the recycled aggregate involved in this project can be assumed to have a TFV value of 70kN. This is much lower than the required hardness for bitumen bound base or surface materials, where over 100kN is required as shown in Table 2-4. Therefore, the recycled aggregate with masonry is not suitable for the base and binder course of the motorway or trunk road, where the use of material is governed by the Highway Agency.

Table 2-4: TFV Requirements for aggregate in bituminous and cementitious bound material (MCHW1)

Type of bound	Application	10% fines value (min, kN)
Cementitious bound	Pavement wearing surface	100
Cementitious bound	Other	50
Bituminous bound	Thin surface course systems	180 (Clause 942, MCHW1)
Bituminous bound	Bituminous base and surfacing materials	140 (Clause 901, MCHW1)
Unbound	Sub-base, Type 1 or Type 2	50 (Clause 803, 804, MCHW1)

Table 2-5: Classes of recycled aggregate and suggested applications (BRE 433, 1998)

Recycled aggregate classification	Original (normal circumstances)	Brick content by weight	TFV (usually found)	Unbound/cement bound material applications
Class I	Brickwork	0 – 100%	70kN	Capping layer 6F1 or 6F2, CBM1 or CBM2
Class II	Concrete	1 – 10%	100kN	Sub-base Type 1, Type 2, CBM3, CBM4, CBM5
Class III	Concrete and brick	0 – 50%		Capping layer 6F1 or 6F2, or CBM1, CBM2

Note: CBM1, CBM2, CBM3, CBM4 and CBM5 refer to cement bound materials with compressive strength of 2.5, 4.5, 6.5, 15.0 and 20.0MPa at 7 days.

In Britain, there is some 370,000 km of roads, of which motorway and trunk roads make up less than 5%. This leaves some 355,000 km of other roads, which are the responsibility of Local Authorities (Dunhill, 2004). The recycled aggregate involved in this project is more likely to find a market in the road system controlled by local authorities and in footpaths and cycle tracks.

Highway agency has been actively encouraging research and practice in recycling. Table 2-6 is extracted from HD35/04 of DMRB, which offers a provisional guidance for the application of recycled aggregate in highway construction. Since most local authorities consult DMRB in drafting their own design specification, similar guidance can be found in the design specifications of many city councils.

Table 2-6: Specification for Highway Works: Application of secondary and recycled aggregates (HD35/04, DMRB7)

Application	Pipe Bedding	Embankment and Fill	Capping	Unbound Mixtures for Sub-base	Hydraulic Bound Mixtures for Sub-base and Base	Bitumen bound layers	PQ concrete
MCHW Series Number	500	600	600	800	800	900	1000
Blast furnace Slag	✓	✓	✓	✓	✓	✓	✓
Burnt colliery spoil	✓	✓	✓	✓	✓	×	×
China Clay Sand/Stent	✓	✓	✓	✓	✓	✓	✓
Coal fly ash/Pulverised fuel ash (CFA/PFA)	✓	✓	✓	×	✓	✓	✓
Foundry Sand	✓	✓	✓	✓	✓	✓	✓
Furnace Bottom ash (FBA)	✓	✓	✓	×	✓	×	×
Incinerator Bottom Ash Aggregate (IBAA)	✓	✓	✓	✓	✓	✓	✓
Phosphoric Slag	✓	✓	✓	✓	✓	✓	✓
Recycled Aggregate	✓	✓	✓	✓	✓	✓	✓
Recycled Asphalt	✓	✓	✓	✓	✓	✓	✓
Recycled Concrete	✓	✓	✓	✓	✓	✓	✓
Recycled Glass	✓	✓	✓	✓	✓	✓	×
Slate Aggregate	✓	✓	✓	✓	✓	✓	✓
Spent Oil Shale	×	✓	✓	✓	✓	×	×
Steel Slag	✓	✓	✓	✓	✓	✓	×
Un-bunt Colliery spoil	×	✓	×	×	✓	×	×

Key: ✓: Specific (permitted as a constituent if the material complies with the specification MCHW1 or General Provision (permitted as a constitute if the material complies with the specification MCHW1 but not named within the specification MCHW 1). ×: Not permitted.

2.4.4 Economic consideration

Recycled aggregate will only be used when it is cheaper or at least equal to primary aggregates. The UK is fortunate in having rich aggregate resources. Because of the extra cost of transport, sorting and processing, the recycled aggregate often turns out to be more expensive than virgin aggregate. Quite often, the recycled aggregates are cheaper if they are used where they are produced, because of the saving in transport costs.

In order to make the price of aggregate better reflect the true social and environmental costs of quarrying and to reduce demand for virgin aggregates and encourage the use of recycled aggregate, the government has acted to adjust the economic balance in favour of recycling by means of the landfill tax and the aggregates levy. The landfill tax was introduced in October 1996 and was set at two rates. A lower rate of £2 per tonne for inactive (or inert) wastes and £15 per tonne from 1 April 2004 for active waste. The aggregate levy was introduced at £1.60 per tonne on primary aggregates from April 2000.

2.5 SUMMARY

Road construction consumes one thirds of the total aggregate consumption in the UK and the requirements are relatively easy to fulfil comparing with other applications. There are potential problems associated with recycled aggregate such as noise, water pollution etc, but research has proved that with proper management, all these problems can be contained. Pavement can be divided into layers including surfacing (wearing course and binder course), base, subbase and capping layers. Currently most of the aggregate in road construction are recycled into the capping or subbase layer and there are still large amount of un-recycled aggregate. There are environmental and economic benefits to recycle the aggregate into high utility applications, such as road base, which is the aim of this research.

CHAPTER 3 BITUMINOUS MIXTURES

3.1 INTRODUCTION

As introduced in previous chapters, pavements can be broadly classified as 'rigid' and 'flexible' pavement. Rigid pavement is composed of aggregate and hydraulic binders and mainly used in areas with high static stress, such as airport aprons and dockyards and may be reinforced with steel mesh in many cases. Flexible pavement is composed of bitumen and aggregate. Most roads are actually built with bituminous materials and the existing concrete pavement will be surfaced with bituminous materials in the UK.

Tar was first used in the pavement to cover up the dust stirred up by passing traffic. Over the years, tar has been replaced by bitumen because tar is produced from the destructive distillation of coal and, as other energy sources have already replaced coal as the main energy sources, there is no longer enough tar available. However, tar bound materials can often be found in the pavement rehabilitation process.

Bitumen can be natural asphalt and rock asphalt, and the most well-known natural asphalt is lake asphalt which is located in Trinidad. However, the most commonly used binder now is the bitumen from oil refineries, and therefore the world supply of crude oil is crucial to the supply of bitumen. Bitumen is the heavy 'end' of the residual left after others, such as gasoline, kerosene have been distilled. Bitumen is a thermoplastic material and is characterised by its cohesiveness, adhesiveness and visco-elasticity. At elevated temperature, it behaves as a viscous liquid and can readily coat the dried and heated aggregates and fines, whilst conversely at lower temperatures such mixtures stiffen. At ambient temperature, when properly designed, such materials are stiff enough to bear heavy traffic load and last for many years. In general, the response and performance of bitumen is mainly dependent upon several external factors:

- Temperature

This is the most critical parameter affecting performance. Bitumen is a visco-elastic material and behaves as viscous liquid at high temperature and as elastic solid material at low temperature.

- Rate of loading

At short time of loading, the bitumen responds like an elastic solid, however, at long time of loading, it behaves as a viscous liquid.

- Nature of bitumen

Many experimental methods have been developed over the years to characterise the properties of the bitumen with the aim of predicting the properties of the bitumen aggregate mixtures during the mixing, transport, laying and compaction processes. Of these, the two simple empirical, comparative tests methods - penetration and the softening point test as shown in Figure 3-1, are fundamentally important and widely employed in the bitumen industry.

The penetration test is presented in BS EN 1426:2000 (BSI, 2000) and the ring and ball test is detailed in BS EN 1427:2000 (BSI, 2000). In the penetration test, a needle weighted 100g is allowed to penetrate into a bitumen sample for 5 seconds, the sample having been conditioned to a temperature based on the hardness of the bitumen. The depth of penetration in units of 0.1mm is reported as the penetration depth. The penetration is widely used in describing the hardness of the bituminous mixture. For example, DBM190, DBM125 and DBM50 are the most widely used base course materials, where DBM is the acronym of Dense Bitumen Macadam and 190, 125 and 50 represents the bitumen binder penetration, the lower the penetration value, the harder the materials. For example, DBM190 is suitable for footpath while DBM50 is the most commonly used material for the base course of motorway.

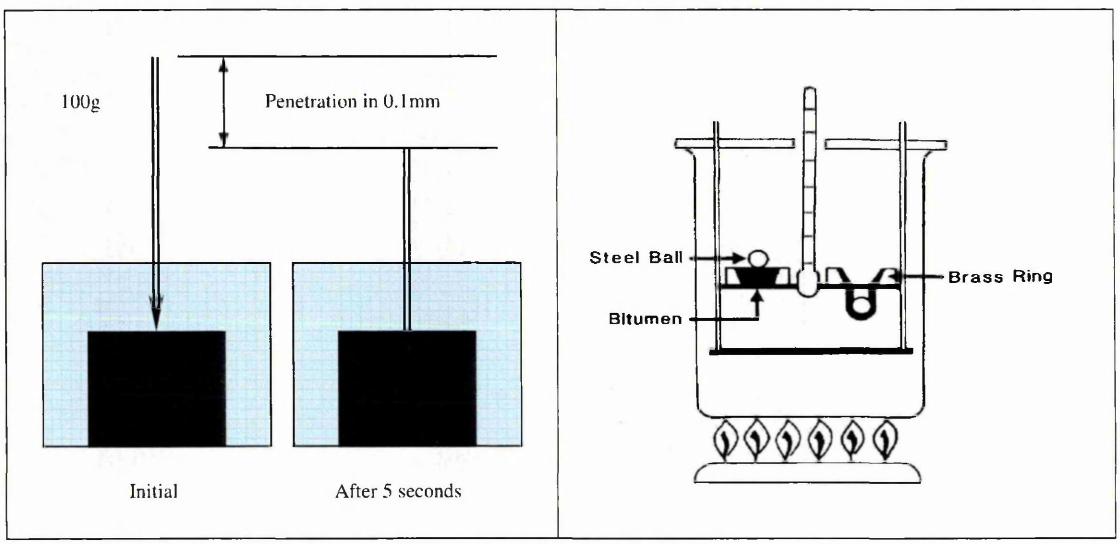


Figure 3-1: Penetration (left), Ring and Ball test (right) illustration

The Ring and Ball test describes the temperature dependence of the bitumen. In the test, two horizontal discs of bituminous binder, cast in shouldered brass rings, are heated at a controlled rate in a liquid bath while each supports a steel ball as presented in Figure 3-1. The bitumen is heated to enable a given rate of increasing temperature. At a certain temperature, the bitumen in the brass spring ring becomes too soft to sustain the weight of the ball. The steel balls will fall through the brass ring. The average temperature for the two balls falling to a distance of around 25mm is reported as softening point.

Although conventional bitumen performs very well in most cases, the increase in permitted heavy goods vehicle weights and increased axle loads has resulted in the more frequent use of modified bitumen, especially in the surface course. The modified bitumen is made by adding various additives, such as sulphur, rubber or thermoplastic polymer to traditional penetration grade bitumens. The advantages of modifying bituminous binders and mixtures with a polymer are:

- Obtaining softer mixtures at low service temperature and increasing the stiffness at high temperature.
- Increasing the structural strength.
- Improving workability and compaction.
- Improving marginal asphalt binders.
- Allowing thicker binder films on aggregate which gives the mixture better workability and fatigue resistance.
- Improving bonding and reducing stripping of bitumen and aggregate.
- Reducing bleeding at high temperature.
- Improving resistance to ageing or oxidation.
- Reducing structural thickness of road pavement layers.

The disadvantage of the modified bitumens is their higher price compared with traditional bitumen. However, because of the large scale application, the price has greatly reduced.

With the addition of polymer, the rheological properties over various temperatures cannot be predicted simply by the penetration and softening point test. For example, bitumens with similar penetration and softening point may have widely different physical performance. A Bohlin Dynamic Shear Rheometer (DSR) is often used to characterize the performance of modified bitumen. The DSR is a dynamic oscillatory test that can be used to measure the linear visco-elastic properties of bituminous binders. It applies a sinusoidal strain to a sample of bitumen sandwiched between two parallel circular disks. The resulting stress is monitored through the torque at the top disc as a function of temperature and frequency. Measurements of shear stiffness and shear viscosity can be obtained at different temperatures, frequencies and strain levels.

Two of the most commonly used parameters obtained from the DSR test are the magnitude of the complex shear modulus ($|G^*|$) and the phase angle (δ). The magnitude of the complex shear modulus is the ratio of peak stress to peak strain as shown in Figure 3-2. It represents the stiffness of the bitumen under the conditions of testing and is independent of the phase angle. The phase angle is defined as the phase difference between the peak stress and peak strain. A material with a high phase angle (approaching 90^o) represents a highly viscous response, whereas a material with a low phase angle (approaching 0^o) represents an elastic response (Airey, 1997).

The major feature of the DSR is that it has the ability to test bitumen over a range of loading times and temperatures. It is possible to measure the stiffness and visco-elastic response of the bitumen at different temperatures and loading times to describe the rheological behaviour of the bitumen throughout its viscoelastic region.

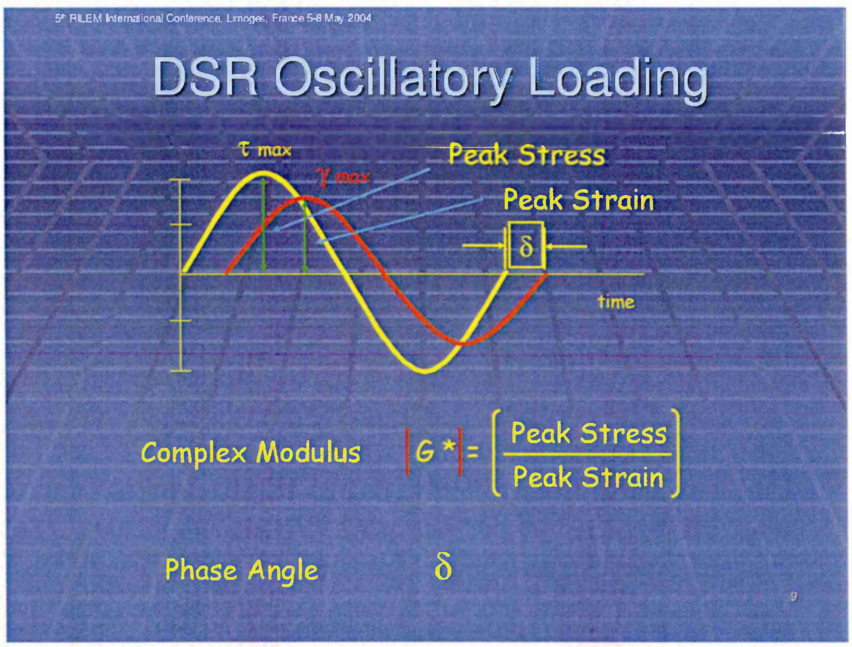


Figure 3-2: DSR test mechanism

In the process of producing hot mixed bituminous materials, both bitumen and aggregate have to be heated to an elevated temperature before being mixed together and such mixed materials have to be kept above certain temperature before laying and compaction.

The majority of today's roads are paved with hot mixed asphalt concrete. Bitumen is a relatively small fraction (typically 5-7% by mass) of the total mixture and acts as a visco-elastic binder between the mineral aggregate particles (stones, sand and filler). Hot mixture production takes place typically between 150°C and 180°C. Spreading and compaction on the road takes place typically between 130°C and 160°C. Aggregate, sand and bitumen are heated to high temperatures to enable the bitumen to coat the mineral aggregate particles properly and to make the total mixture fluid enough to allow good workability during mixing, laying and compacting.

3.2 COLD BITUMINOUS BINDER

3.2.1 Overview

Although hot mixed bituminous materials have been performing very well over the years, efforts have been made to save energy and directly mix the bitumen with aggregates at the ambient temperature. The most widely employed cold bituminous binders include:

- Cutback bitumen

Cutback bitumen is produced by blending penetration bitumen with suitable oils, such as gasoline or kerosene to make it more fluid (less viscous) and more convenient for using at a lower temperature than is necessary for penetration bitumen. The main use of cut-back bitumen in roads is for surface dressing and in making coated macadams for patching and for lightly trafficked areas. Currently, bitumen emulsion has replaced cutback bitumen in these applications due to environmental concern. '*In many areas, cut-back bitumen has been banned because of Volatile Organic Compound (VOC's). The restriction on VOC's is intended to reduce 'Ground Level Ozone' formation. VOC's are a group of photochemically - reactive pollutants that combine with nitrogen oxides in the presence of sunlight to form ozone (Kollaros and Athanasopoulou, 1999)*.'

- Bitumen emulsion

Bitumen emulsion is produced by shearing the bitumen into tiny globules, which are further dispersed in water and stabilised by an emulsifier to make it convenient to use at lower temperatures than for penetration bitumen.

- Foamed bitumen

Foamed bitumen is a bituminous binder produced by injecting cold water under controlled conditions, sometimes with certain additives, into hot penetration grade bitumen before application through specially designed nozzles on a spray bar. The foamed bitumen expands to 10-15 times the original volume of the penetration grade bitumen, then coats the aggregates. Foamed bitumen is a well-established technology for lightly trafficked roads and a comparison with bitumen emulsion cold mixes is presented in the next chapter.

Cold mixed materials possess many valuable properties such as long shelf life, and operate at ambient temperatures. Since the bituminous binder and the aggregate are not heated up in the mixing process, the viscosity of the binder is high and unable to coat the aggregate as good as the hot bitumen is capable of. The cold mixtures have a low workability resulting in high void content after compaction. The materials also take much longer to reach the performance required in the pavement design process.

Since bitumen emulsion is one of the binders investigated in this research, it is introduced in detail.

3.2.2 Bitumen emulsion

3.2.2.1 Introduction

An emulsion is a dispersion of fine droplets of one liquid in another liquid. In contrast to solutions, the two liquids are co-existent rather than mutually mixed. In the case of a bitumen emulsion these are bitumen, which is a liquid with a very high viscosity, and water. Normally, in good bitumen emulsions, the droplets are in the order of 1 to 30μm in diameter. The bitumen content is normally in the region of 60 to 70% but can be as low as 40% or as high as 80%. The globules of bitumen are termed the disperse phase, as they are discrete droplets, and the water is the continuous phase in which the droplets are suspended (Needham, 1996). Bitumen emulsions are used at lower (ambient) temperatures and mixed with mineral aggregates termed 'cold mixes'.

Fundamentally, there are two types of road emulsions - cationic and anionic emulsions. The terms cationic and anionic stem from the electrical charges surrounding the bitumen globules. If an electrical current is passed through an emulsion containing negatively charged particles of bitumen, they will migrate to the anode and the emulsion is hence referred to as anionic. Conversely, positively charged globules will move to the cathode and the emulsion is known as cationic and the aqueous phase is normally acid (Whiteoak D, 1991). The bitumen globules in anionic emulsion have a negative charge (-) on the surface and such globules tend to adhere to and coat that aggregate with a positive surface charge (+); the bitumen globules in cationic emulsion have a positive charge (+) over the surface and such globules tend to coat better on aggregate with negative surface. As most road building aggregate used in the UK possesses a negative surface, cationic emulsion is more effective

The majority of bitumen emulsions are manufactured using colloid mills. Bitumen is heated to a liquid state in order to pass through the mill head. The temperature required is dependent on the penetration grade of bitumen in question; as a rule of thumb, 100°C above the softening point. Water resolved with emulsifier, consisting of a long hydrocarbon chain terminating in either a cationic or an anionic functional group, and bitumen is pumped towards the mill head at a calculated rate. Under the shear force created by the high-speed rotating rotor, the bitumen is broken into small globules. Once the droplets are formed they must be stabilized against coalescence both within the colloid mill and afterwards. Emulsifiers adsorbed at the surface of the droplets provide an electric or steric repulsion energy barrier between the droplets and hence stabilize the bitumen emulsion. The diagram in Figure 3-3 is a representation of the emulsification process.

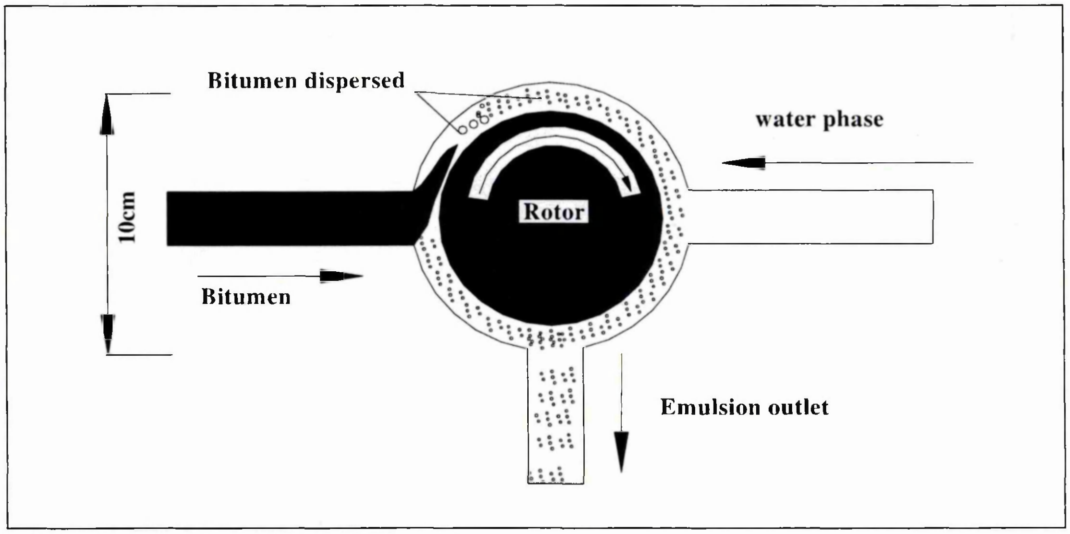


Figure 3-3 Diagram of emulsification process (Needham D, 1996)

Bitumen emulsion is marked by a three parts code the first part of which, either A or K, indicates anionic or cationic emulsion. The second part, 1 to 4, indicates the breaking rate or stability, the higher the number the greater the stability. The breaking rate here refers to the speed at which the bitumen emulsion droplets reverse back to continuous bitumen. The third part of the code, 40 to 70, indicates the bitumen content of the emulsion. For example:

K1 - 70 is a cationic emulsion, rapid setting with a bitumen content of 70%.

A2 - 50 is an anionic emulsion, semi-stable, with a bitumen content of 50%.

Bitumen emulsion must revert to a continuous bitumen film in order to fulfil its role as a binder in road material. The setting of an emulsion is a complex process which is not fully understood, but the setting might broadly follow the steps shown in Figure 3-4. Many factors influence the setting process, such as aggregate type, moisture content, weather conditions, the presence of other additives such as lime, cement etc, and compaction. After making contact with the aggregate, some emulsifiers in the emulsion are absorbed onto the aggregate surface and the loss of emulsifier causes the emulsion to become unstable, leading to the movement of emulsion droplets to the aggregate surface. Some aggregates, such as the demolished concrete used in this project containing un-hydrated lime, will neutralize the acid in a cationic emulsion causing the pH to rise and destabilize the bitumen emulsion and inducing the bitumen globules to set and cure on the aggregate. A crucial factor in the coalescing process is the loss of water and the breaking down of the surface tension in the individual bitumen particle walls so that a single coherent bitumen film is formed.

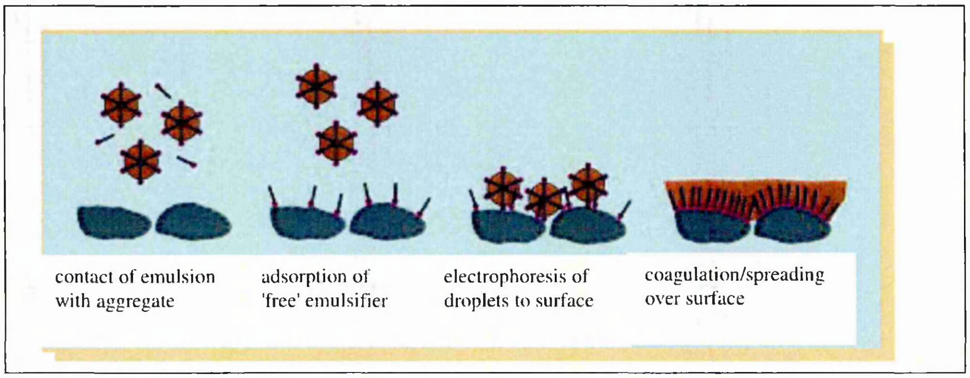


Figure 3-4: Possible stages in the setting of a cationic bitumen emulsion (Akzo Nobel Asphalt applications, 2004)

3.2.2.2 Applications

Perhaps more bitumen emulsion is used in roofing than road paving, but only pavement related applications are listed below. Each application places particular demands on the emulsion and there is a considerable amount of variation between countries on the choice of emulsion for each application. Anionic emulsion is rarely used outside North America.

- Surface dressing (chip seal) can be used successfully on all types of roads, from the country lane that carries only an occasional vehicle to trunk roads and motorways carrying thousands of vehicles a day. The purposes of surface dressing include providing both texture and skid-resistance to the surface, sealing the road surface against ingress of water and arresting disintegration, hence extending the life of the pavement and assisting sustainable development (Nicholls, 1997). In the surface dressing process, binder is sprayed onto the road and chippings spread over and rolled in before the binder is cured. A high percentage of bitumen emulsion is used for surface dressing (Table 3-1). Surface dressing can be conducted with bitumen emulsion as well as cut-back bitumen, with bitumen emulsion developing final strength more quickly than cut-back bitumen. Surface dressing is a cost-effective process used for improving the texture and sealing small cracks in deteriorating pavement surfaces. Surface dressing has a short laying season because the chippings have to adhere to the binder at a temperature close to ambient and the binder has to maintain sufficient cohesive strength to resist traffic force when the road is opened. Rain can wash away the unbroken bitumen emulsion and high humidity can retard the breaking of bitumen emulsion. All these potential problems have to be avoided.

Table 3-1: Percentage of bitumen emulsion used for surface dressing out of the whole bitumen emulsion consumption

Countries	Percentage (%)
United Kingdom	75
France and Germany	60
Spain	55
Netherlands	50

- Slurry surface and micro-surface are two very similar techniques used as surface treatments to improve surface texture and correct minor irregularities. Slurry surfacings range in thickness from about 2mm to 8mm and micro surfacing from about 10mm to 20mm. Slurry surfaces are suitable for footways or areas only occasionally trafficked, while micro-surfacing is suitable for all categories of roads. These processes are not considered as adding significant strength to the pavement, but act more like surface dressing and, by sealing an oxidized and fretting surface, will extend the life of the existing road surface as well as improving the skid resistance and appearance of the road. Unlike surface dressing, micro-surface is mixed before paving and consists of bitumen emulsion, coarse aggregate, fine aggregate, ordinary Portland cement and perhaps, an amount of retard agent (Summer, 2003).

- Cold mixes made of bitumen emulsion and aggregate are widely used on the less trafficked road. *'The major problem with the use of bitumen emulsions in road mixes is that relatively high void contents are needed to allow the water to escape during compaction and service. Furthermore, strength is developed relatively slowly. For both of these reasons emulsion mixes are suitable only for sites carrying modest traffic loads. As a result, the material has been used very little in the UK (Whiteoak, 1990).'*

- Tack coat is an established technique for providing a thin additive film of bitumen binder between an existing road surface and an overlay or between courses in road construction. The tack coat can minimize the dust on existing surfaces and provide an additive surface for overlay. The bitumen used for this work is either class A1-40 or K1-40. Bond coat is similar to tack coat but usually uses polymer modified bitumen or higher grade bitumen.

- Prime coat is used on unbound and lime or cement-stabilized base materials to provide good adhesion for a hot mix layer. The prime coat should penetrate the base to a depth of 1-4cm. It may be necessary to mix the emulsion with the surface of the base and re-compact in order to achieve this penetration, especially with stabilized base (Akzo Nobel, 2003).
- Sealing and curing of pavement layers: bitumen emulsion may be used to provide a water resistant membrane of bitumen to seal road bases, sub-bases and sub-grade and so to protect ingress of water or water loss from surface evaporation.

Other applications, such as fog seal and soil stabilization, do not account for a high percentage and hence are not detailed here.

3.3 HOT BITUMINOUS MIXTURES

Hot mixed materials are the default choice for the road pavement. There are about 2.2 million miles of paved roads in the United States (US) of which 94% are surfaced with bituminous materials whereas in the United Kingdom this figure is about 90% (Huang, 1993). Hot mixed materials production involves warming up bitumen and aggregate to an elevated temperature and mixing the two together. During mixing, the hot bitumen should be readily able to coat the dried and heated mineral aggregate in a relatively short period of time. There are essentially three types of premixed bituminous materials (Figure 3-5).

- Hot rolled asphalt (HRA) (BS 594: 2003)

HRA is composed of a rich mortar and a gap grading where the coarse aggregate does not interlock. The mortar of fine aggregate, filler and high viscosity binder is a major contributor to the strength and performance of the laid materials.

- Macadam (BS 4987: 2003)

Macadam, also known as asphalt concrete, is composed of continuously graded aggregates. The interlocking of the aggregate particles is a major contributor to the strength of the compacted material.

- Porous asphalt (PA) and Stone mastic asphalt (SMA)

PA and SMA are composed of high stone content, full coarse aggregate interlock and a gap grading. The materials are single size in their grading, with few fines filling their grading, so the material has a high void content. This material can provide a free draining and quiet surface but the service life is shorter than HRA surface.

HRA has historically been the most widely used wearing course material for trunk roads. The texture is formed by rolling coated chippings into the asphalt mat. HRA is also used in the base and binder course where the requirements merit such high quality materials. The bitumen used in HRA ranges from 30/45Pen to 100/150Pen. The aggregate in HRA is a gap-graded material with the gap between the fine and coarse aggregate. Stresses imposed on the pavement by the traffic are distributed mainly through the mortar. Hot rolled asphalt has been working very well in the UK and it offers excellent durability because of water impermeability. Hot rolled asphalt used as the wearing course is often finished with coated chippings, which not only offers high skidding resistance but may also produce a pavement with different colours, which has a functional or decorative effect for city streets.

Whereas HRA is a gap-graded material, in coated macadam the aggregates are continuously graded and the strength and resistance to deformation is achieved mainly through aggregate interlock. The bitumen fills the voids between the aggregates, lubricates the aggregates so a better compaction effect can be achieved and binds the aggregates together. Coated macadam is mainly used as base course material. The base course is the main structural layer in the pavement and often uses low penetration bitumen, sometimes as low as 20/30 Pen, which is termed High Modulus Base (Nunn and Smith, 1997).

With current increased concern over safety and the environment, new products, most prominently stone mastic asphalt and porous asphalt have been introduced as surface materials.

Stone mastic asphalt (SMA) was developed in Germany as a deformation-resistant surface material. SMA has a stone skeleton of interlocking crushed rock coarse aggregates, comprised largely of single-sized stone of a size appropriate to the laying thickness and required surface texture (Loveday and Bellin, 1998). The single sized nature of the aggregate skeleton leaves a relatively high void content between the aggregate particles which are partly filled with a binder rich mastic mortar (Nicholls, 1998). The possible 'run-off' of the binder is overcome by the addition of cellulose or rock-wool fibres. It has the advantage of high stability and, due to the stone content, does not require coated chippings to provide skid resistance (Wignall et al, 1999).

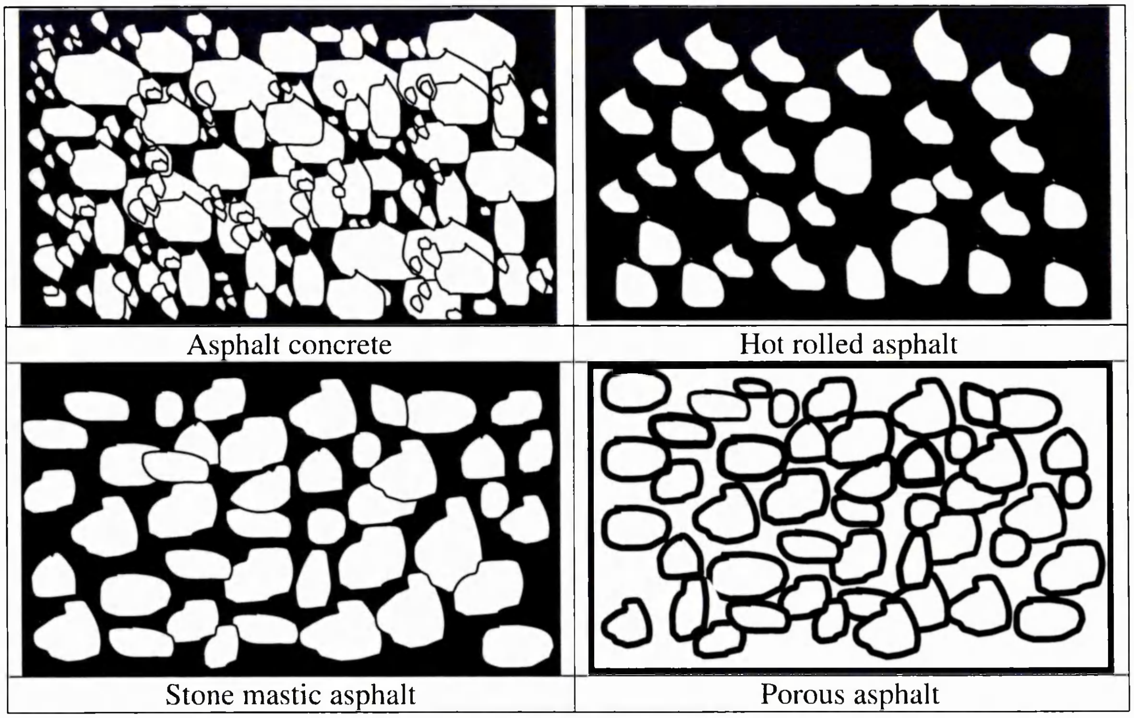


Figure 3-5: Typical cross-sectional views of the commonly used bituminous materials

Another development has been porous asphalt (PA). The primary advantage of PA is its effect in reducing the spray from road surface and the noise level generated. PA has a high void content, around 25% in general, and consequently will absorb rainwater like a sponge. The surface water is carried away on the underlying dense basecourse (Nicholls, 1998). '*The noise from all vehicles running on porous macadam surfaces was significantly lower than that generated on conventional bitumen surfacings for similar levels of skid resistance: 3 to 4 dB (A) lower in dry conditions and 7 to 8 dB (A) lower in wet conditions (Whiteoak, 1991).*' In order to reduce the surface water run-off, which may cause flooding, the French have developed 'reservoir pavement' or 'drainage pavement' (Figure 3-6). The basic idea of such design is *that 'the rainfall on the surface infiltrates across the entire surface into a porous unbound sub-base material (reservoir layer). Here, water can accumulate before it is dissipated more slowly into the sub-soil or it is removed through drains into the main surface water drainage system. This process reduces the runoff (or outflow) that thereby mitigates the adverse effects of storm water surges and the risk of flooding as the result of the drainage system becoming overloaded (Nunn M E, 2000)'.* To realize the objective of reservoir pavement, the surface has to be porous asphalt (PA).

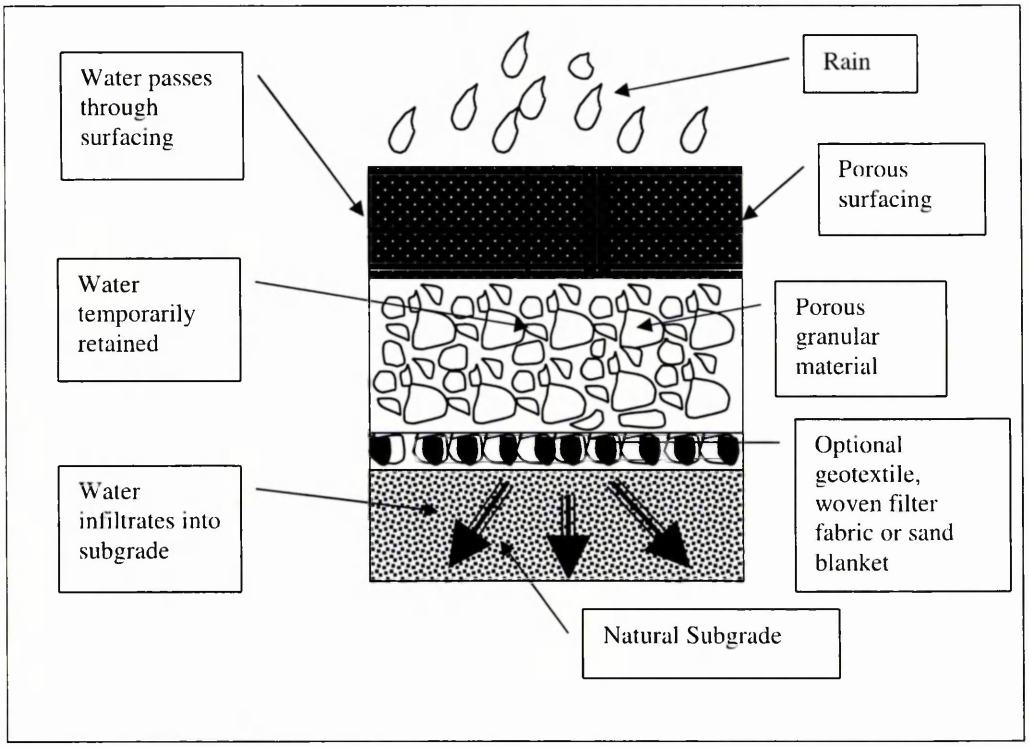


Figure 3-6: Concept of a reservoir pavement (Nunn M E, 2004)

Thin surfacing, including SMA and PA, contribute relatively little to the overall strength of the road, and it is the structural layers which are responsible for the distribution of the traffic load. The current default choice of surface materials is thin surface instead of HRA (HD 36/99). The definition of thin surface is a proprietary bituminous product with suitable properties to provide a surface course, which is laid at a nominal depth of less than 40mm. Its advantage is that it can restore skid-resistance and provide some regulating ability whilst minimizing the need to raise ironwork and the loss of headroom at over-bridges. Many of the proprietary thin surface materials are actually SMA.

3.4 SUMMARY

Bitumen has a long history as a pavement binder material. Penetration and softening point tests are employed to characterise the bitumen. With the increasing use of modified bitumen, the DSR test was introduced to characterise the properties of modified binder over various temperatures and loading conditions. The hot mixed materials are basically categorised as hot rolled asphalt, coated macadam or asphalt concrete, porous asphalt and stone mastic asphalt.

The applications of bitumen emulsion in road construction include surface dressing, tack coat, bond coat, joint or crack sealing etc. It has also been used as paving or reinstatement materials in the structural layer in some countries.

CHAPTER 4 RECYCLING TECHNIQUES

4.1 INTRODUCTION

The recycling process can be classified in different ways. According to the place where recycling happens, the recycling process can be classified as in-situ (where recycling takes place on-site) versus ex-situ (where reclaimed material is processed off-site). According to the binder used, the recycling process can be classified as bituminous or hydraulic bound recycling. In the case of bituminous recycling, the binder can be applied hot or cold. In the cold recycling process, the binder can be bitumen, hydraulic binders or a mixture of the two, the cold bitumen can be applied as foamed bitumen or bitumen emulsion, hydraulic binders can be cement or PFA/lime. In many cases, the hybrid of bitumen with hydraulic binder has proved to be very successful. In-situ recycling only recycles the original pavement material, whilst off-site recycling can deal with aggregates from various sources. The possible recycling methodologies are summarised graphically in Figure 4-1. The recycling process in this project can be classified as off-site recycling with a hybrid of bitumen and hydraulic binder.

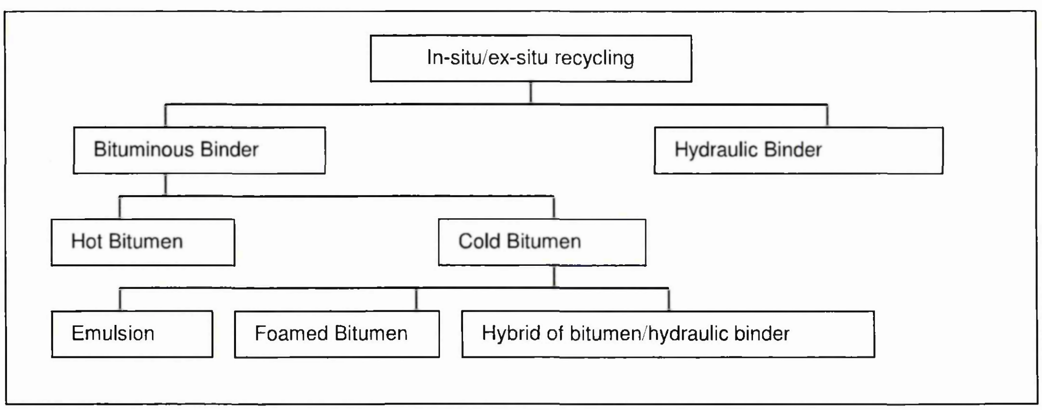


Figure 4-1 Classification of recycling methodology

Among the above-mentioned recycling techniques, the cold mixes with foamed bitumen and hydraulic binders such as cement and PFA/lime are the most widely applied in the UK. It is estimated that 1.5 million tonnes of cold mixes (foamed bitumen binder) are used per annum, over 95% being off-site plant mixed materials (Walsh, 2004).

4.2 HOT RECYCLING

Hot recycling can be conducted in situ or off site. Because of the strict requirements on the aggregates, in practice the hot mix recycling is only applicable to Recycled Asphalt Planings (RAP), in which case, the aggregate are assumed to have fulfilled the requirements in the previous application. The available RAP recycling information in Europe for 2001 is presented in Table 4-1, which indicates that a high percentage of road asphalt is not recycled effectively.

Table 4-1: The use of reclaimed asphalt in asphalt in 2001 (EAPA, 2004)

Country	Available materials (x 10^3 tonnes)	% of the new product which contains reclaimed materials	Total HMA (in 10^6 tonnes)	Country	Available materials (x 10^3 tonnes)	% of the new product which contains reclaimed materials	Total HMA (in 10^6 tonnes)
Austria	1500	3	9.5	Italy	15	5	39.8
Belgium	20	25	4.5	Latvia			0.6
Croatia	710	10	1.8	Netherlands	3000	60	7.7
Czech Republic	220	16	4.3	Norway	520	4	4.1
Denmark	200	31	2.8	Poland	750	0.3	11.2
Finland	<5000	5-10	3.6	Slovakia	80	40	2.5
France	15,000	<10	40.5	Romania	4.6		1.1
Germany	1200	60-65	63	Slovenia	50	15	1.4
Hungary	1200		2.9	Sweden	1000	19	6.7
Iceland	5	0	0.3	Switzerland	1750		5.1
Ireland			3.1	United Kingdom	5000		26.5

Note: HMA denotes hot mixed asphalt.

4.2.1 Hot in-situ recycling

Hot in-situ recycling generally involves heating existing pavement surfaces, scarifying, adding rejuvenator, fine aggregate or hot mix as required, mixing, re-profiling and compacting this hot mixture in a continuous operation. This process is suitable for pavements with only superficial rather than structural distresses, including ravelling, coarse aggregates loss, and slight to moderate cracking. Roads suitable for hot in-situ recycling should have a minimum of 75 mm of hot-mix asphalt in place and typically 25mm to 50mm will be scarified and milled during the recycling process. Thinner asphalt surfaces can easily be torn apart by the horizontal shear stresses of the banks of scarification teeth, and broken loose from the base. Hot in situ recycling includes repaving and remix process.

'*When the road pavement surface needs replacement, because of deterioration without excessive hardening of the binder, new surfacing material can be overlaid as a veneer on the heated, scarified and re-profiled existing road surface. This process is* *known as Repave (HD 31/94).'* The overlay is added while the old surface is still hot, thus ensuring that a good bond is established between the old and the new material. '*Remix process is an adaptation of the repave system where the machine is fitted with a small mixing unit. Material from the scarified surface is augured into the pugmill mixer where it is blended with hot freshly-mix new material. The recycled mix is placed evenly on the heated surface to form the replacement wearing course which must conform to the appropriate standard (HD35/04, DMRB7).*' A road train for hot in-situ recycling is presented in Figure 4-2. All operations in the Repave/Remix process including warm up, pulverisation, adding new aggregate or binder, mixing and compaction can be completed with such a train.

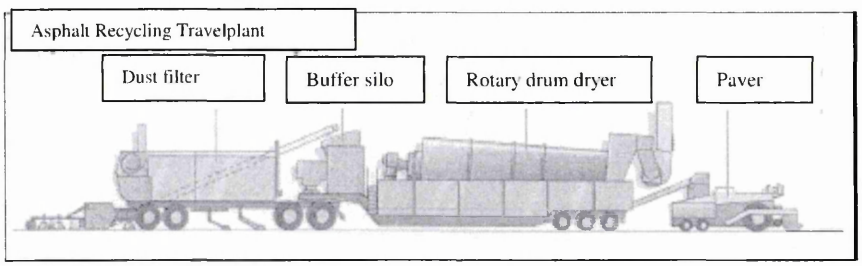


Figure 4-2: Road train for hot in-situ recycling

A comparison between a new wearing course and a Repave/Remix wearing course was conducted by the TRL and it was found that the performances are quite similar, '*from the rut, texture and skidding resistance results, it was concluded that there were no systematic differences between the performance of conventional wearing course, Repave and Remix. The performance of the treatment was however site dependent, indicating that the pavement structure on which wearing courses are placed has a strong influence on performance (Edwards and Mayhew, 1989).'* Hot in-situ recycling is also widely used in the USA, according to Kazmierowski et al (1999), the Ministry of Transport and the Regional Municipality of Ottawa - Carleton have rehabilitated in excess of 80 projects using in situ hot-mix techniques since 1987. There are two potentially limiting environmental factors which require consideration and improvement in the hot in-situ recycling process. The first is that there can be considerable gaseous emissions (blue smoke) at times and the second is that the rejuvenators typically used must meet increasing strict Health and Safety requirements.

4.2.2 Hot off-site recycling

For off-site recycling, '*Reclaimed bituminous materials may be used in the production of bituminous surface course, binder course including binder and regulating* *course, and base. The maximum amount of reclaimed bituminous material permitted shall be 10% in surface course and 50% in all other layers (Clause 902, MCHW1).*' A trial was conducted by TRL comparing hot mixes with virgin and RAP aggregate, which found that major properties such as elastic modulus, fatigue resistance and permanent deformation resistance were quite similar. '*It was identified that the main limiting factor to achieve higher levels of RAP reuse in Hot Mixed Asphalt wearing course was the possibility of dangerously high temperatures in the mixing plant's bag filter unit and the possible increase of emissions from the mixing plant (Woodside et al, 2000).*'

There are cost savings available by using recycled asphalt planings as shown in Table 4-2, the higher the recycled material content, the lower the production cost (Cornelius and Edwards, 1991).

Table 4-2: Component costs of producing HRA road base (Percentage of total) (Cornelius and Edwards, 1991)

Cost items		Cost in percentage (%)		
		No RAP	40% RAP	60% RAP
Transport cost	Delivery of planings	0	9	13
	Delivery of asphalt	29	29	29
Raw Material Cost	Stone	19	14	11
	Sand	7	3	1
	Filler	3	0	0
	Bitumen	39	24	14
Mixing plant fuel		3	3	3
Total (%)		100	82	71

RAP: recycled asphalt planings. NO RAP, 40% RAP and 60% RAP: 0, 40 and 60% out of the total aggregate employed are composed of road asphalt planings.

4.3 COLD RECYCLING

Cold recycling can be conducted using hydraulic or bituminous binder or a mixture of the two as introduced in Figure 4-1. Hydraulic/pozzolanic binders are widely employed in road construction and maintenance, mainly in the lower base and subgrade layers. The commonly used hydraulic binders include cement, hydrated/unhydrated lime, PFA and blast furnace slag. Cold bitumen technology has been in use for road construction in this country. Traditionally, cold bituminous mixtures were made of cut-back bitumen until after the issue of '*Specification for the Reinstatement of Openings in Highways*' by the Highways Authority and Utilities Council (HAUC) in 1992, which basically demanded that the cold mixed materials attain similar properties to hot mixed materials. Cut back bitumen cold mixes are unable to meet the stiffness and rut resistance requirements and the flux oil contained in cut-back bitumen is detrimental to the environment. As a result, this application has largely been replaced by bitumen emulsion. Worldwide, cold bitumen is generally applied as bitumen emulsion or foamed bitumen, the two different procedures producing products with similar properties. In the UK, "cold mixes" generally refers to the foamed bitumen mixes. The reason is that bitumen emulsion cold mixes have a high moisture content, since nearly half of the bitumen emulsion is water. The strength development of cold mixes relies on moisture evaporation, which is very problematic considering the UK weather conditions.

Cold bituminous mixes are regarded as having the following advantages over traditional hot bituminous mixed materials (DETR, 1998):

- Potential energy saving and reduction of carbon dioxide emissions.
- Potential improved binder durability due to the absence of binder hardening during cold mixing.
- Improved storage characteristics, giving greater operational flexibility and reduced materials wastage.
- Health and safety benefits due to the elimination of the handling of bitumen at a temperature greater than 100°C, together with the avoidance of the associated emissions to the atmosphere.
- Reduced seasonality of operations due to a wider ambient temperature window for laying.
- Enabling all kinds of recycled/secondary aggregate, even contaminated aggregate, to be used where the heating process would cause unacceptable degradation or fume emission.

The development of cold bitumen mix in the UK has been driven largely by the utility sector, the prolonged shelf-life and relaxed temperature restriction being especially suitable for trench reinstatement where the requirement for each trench is limited and a large amount of trench work takes place at the same time. The cold bituminous mixes are also widely used in the stabilization of lightly trafficked road, footpath and cycle tracks.

According to the primary binder type and their rate of curing, cold mixes can be classified into four types as follows (Merrill et al, 2004):

- Quick hydraulic (QH): with hydraulic only binder(s) including cement.
- Slow hydraulic (SH): with hydraulic only binder(s) excluding cement.
- Quick visco-elastic (QVE): with bituminous and hydraulic binder(s) including cement.

- Slow visco-elastic (SVE): with bituminous only or bituminous and hydraulic binder(s) excluding cement.

The materials classification above is graphically illustrated in Figure 4-3.

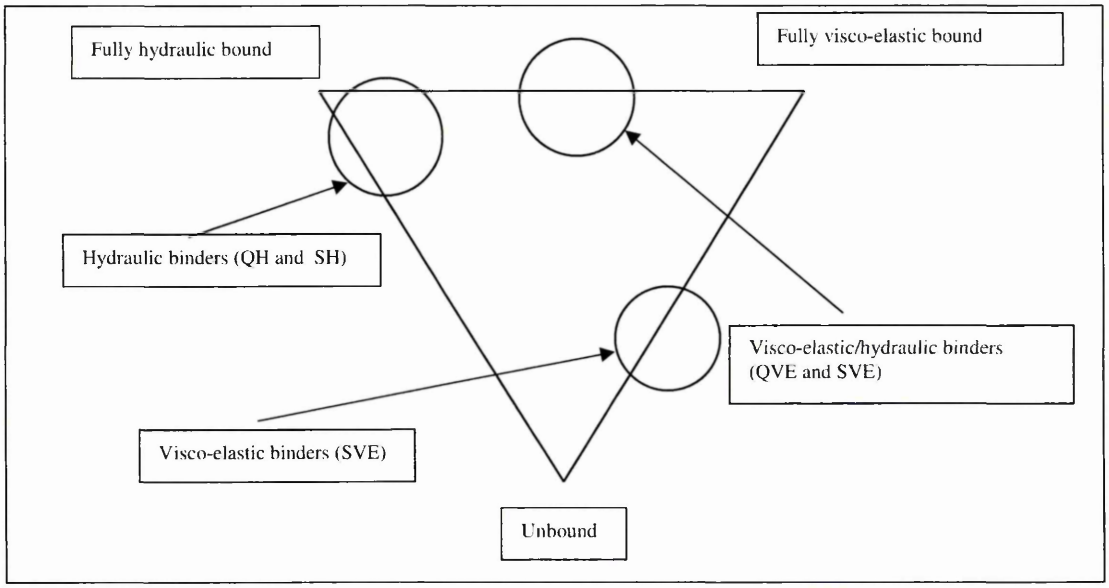


Figure 4-3: Identification of material families

Mixes with cement as binder have to be put into use within a few hours after mixing. The word 'quick' is added when cement is used as binder for this reason. PFA or GGBS can only hydrate with the activation of cement or lime. It could take weeks or months for the mixes to reach design stiffness. The word 'slow' is added to refer to the mixes when PFA or GGBS is added as binder. The bitumen is a visco-elastic binder. The bond develops after cooling if the bitumen is mixed with the aggregate at an elevated temperature; if mixed at ambient temperature, it takes much longer to develop the visco-elastic bond. Cold mixed bituminous materials are therefore named as slow visco-elastic material.

4.3.1 Cement

Cement is the most widely used binder in the recycling process. The recycled materials with the addition of cement have to be compacted soon after mixing with water or the mixture will lose its workability. The design of cement recycled materials can be based on the existing specifications (Clause 800 and 1000, MCHW 1 and 2). Cement as a binder is adaptable to most conditions and can cope with all kinds of recycled aggregates. It is readily available at reasonable cost and there are accepted methods of working to satisfy Health and Safety requirements. The major problem with cement bound materials is the potential reflective cracking. Because of this potential problem, all cement bound materials that have an average 7 days' compressive strength of 10.0N/mm^2 must have transverse cracks induced during construction. With pre-cracking, more closely spaced finer cracks are established which are less likely to merge into big cracks and develop to the surfacing bituminous layer.

4.3.2 PFA activated with lime or cement

Pulverised fuel ash (PFA), or fly ash as it is known in many countries, is a by-product of the burning of pulverised coal in power stations. In a typical modern coal-fired power station, approximately 80% of the ash will be produced as a fine powder known as PFA. Being light and fine, the PFA is carried in the gas stream out of the boiler, ultimately being collected by mechanical arrestor and/or electrostatic precipitators. PFA is similar in colour to Portland cement with pozzolanic and self-hardening properties and has a specific gravity of approximately two thirds of Portland cement. Because of its pozzolanic properties, it can partly replace Portland Cement (BS 3892, 1997). PFA activated with lime or cement as binder has been used in the construction and reinstatement of carriageways. One successful example was in a construction project on the A52 between Kingshill and Froghall in 1997, where the mix design consisted of 3% CaO + 12% PFA + 85% recycled planings (Aggregate Advisory Service (AAS), 1999). The material was used in the base and binder course and was covered with new bitumen surfacing. The material is placed and compacted like soil, but pozzolanic reactions occur over time between the PFA and the lime, leading to the formation of cementitious compound with high stiffness and compressive strength. Because of the slow strength gain, the pavement can be reworked soon after compaction if necessary. The added lime not only activates the PFA, but also acts on the clay particles in the pavement. For example, where high clay content is present in the foundation stabilization process, lime is often added to agglomerate the clay into particles before cement treatment. PFA activated with lime or cement in the cold in-situ recycling process is also reported in the US (Thomas et al, 2000; Crovetti, 2000).

4.3.3 Blast furnace slag (BFS)

BFS is formed as a by-product of the manufacture of iron in the blast furnace. In the furnace, the slag is the molten stratum above the molten iron. BFS issues from the blast furnace as a molten stream at a temperature of 1400°C to 1500°C (Hewlett, 1988). Different products are obtained, according to the kind of process used in cooling the molten slag.

If cooled slowly under ambient conditions, then a hard lump slag is produced which can subsequently be crushed and screened for use as road stone and concrete aggregate. If the molten slag is cooled and solidified by adding a controlling quantity of water, air or steam, then a lightweight expanded material is produced, which can be used as lightweight aggregate after crushing and grading.

If the molten slag is cooled and solidified by rapid water quenching to a glassy state, little or no crystallization occurs. This process results in the formation of sand size fragments which, when activated with lime, demonstrate cementitious properties and act as a binder for any suitable aggregate. The resulting bound material is known as Slag Bound Material (SBM) and has been specified as a kind of subbase material (Clause 805, MCHW1). In SBM, slag acts as a binder activated by lime based activator and accounts for about 10% to 25% by mass. One successful trial has been reported with the following composition (DETR, 1998):

- 78.5% of either 28mm limestone with 6% added water or 28mm air-cooled BFS aggregate with 7% added water.
- 20% granulated BFS.
- 1.5% activator of lime-based proprietary catalyst.

SBM are not only used as subbase materials, they can also be used as base course materials and in trench reinstatement. Because the aggregates are used without heating up and drying off as required by hot mix, energy is saved. *'Slag bound materials are suitable for road construction in remote areas, where a short delivery time might be impossible and for base materials for small-scale reinstatement/maintenance work, where maintaining a high temperature over a full day would otherwise require insulated or heated trucks (DETR, 1998)'.*

When crushed or milled to very fine cement-size particles, ground granulated blast furnace slag (GGBS) has cementitious properties, which makes a suitable partial replacement for, or additive to, Portland cement. Latent hydraulic binders such as GGBS are widely used in blended cements combined with active hydraulic binders such as Portland cement. It is known that concrete incorporating GGBS has improved workability, lower initial heat of hydration and has superior resistance to alkali-silica reaction. Such properties make it very suitable for sea-water work and other very aggressive environments (Miura and Iwaki, 2000). Unlike PFA, which has to be activated by lime or cement, GGBS contains calcium oxide (CaO) and it can hydrate slowly without relying solely on an activator. In road construction, GGBS has been proved to have an effect of preventing sulphate attack of lime-stabilized kaolinite (Tasong et al, 1999). GGBS is rarely used by itself as a cementitious binder as the hydration is too slow for most practical uses. However, this slower hydration rate was a key reason why GGBS was employed in this project as hydraulic binder.

4.3.4 Foamed bitumen

Foamed bitumen is a bituminous binder produced by injecting cold water under controlled conditions, sometimes with certain additives, into hot penetration grade bitumen before application through specially designed nozzles on a spray bar (Milton and Earland, 1999). The potential of foamed bitumen for use as a soil binder was first realised in 1956 by Dr Ladis H Casnyi at the Engineering Experiment Station in Iowa State University. The original process consisted of injecting steam into hot bitumen, and later the process was modified to adding cold water rather than steam into the hot bitumen, as shown in Figure 4-4. The bitumen foaming process thus became much more practical and economical for general use (Muthen, 1999).

The foamed mix stabilisation as base course is already included in Clause 948 of MCHW1 and MCHW2. Foamed bitumen mixes can be processed in-situ or ex-situ. Milton and Earland (1999) suggested that foamed mixes could be applied on roads with traffic levels routinely up to 10 million standard 8 tonne axles (msa) and, where good quality aggregates exist in the road, up to 20msa or possibly more. The material is generally regarded as suitable for use as foundation and base course.

The foamed bitumen is characterised in terms of expansion ratio and half-life. The expansion ratio of the foam is defined as the ratio between the maximum volume achieved in the foam state and the final volume of the binder once the foam has dissipated. The half-life is the time, in seconds, between the moment the foam achieves maximum volume and the time it dissipates to half of the maximum volume. For specific bitumen, the expansion ratio and half-life also depends on the cold water added in the foaming process. A longer half-life and a bigger expansion ratio are favourable for foamed mix to coat the aggregate, but the two parameters go in different directions with the addition of water, as shown in Figure 4-5. The amount of water added is a balance between expansion ratio and half-life. According to Milton and Earland (1999), 1 - 3% of water is generally added in the foaming process.

The aggregate temperature in the mixing process also profoundly influences the coating effect of the bitumen foam. At ambient temperature, only fines can be coated. The strength of foamed mixes comes from the interlock of coarse aggregate and bitumen and fines mortar. Warming up the aggregate before mixing means that even the coarse aggregate can be coated, so a better performance, such as higher stability, can be achieved (Jenkins, 1999). Bowering and Martin (1976) suggested a critical temperature range of between 13°C and 23°C for the minimum aggregate temperature before foam treatment, below which mixes obtained are of poor quality.

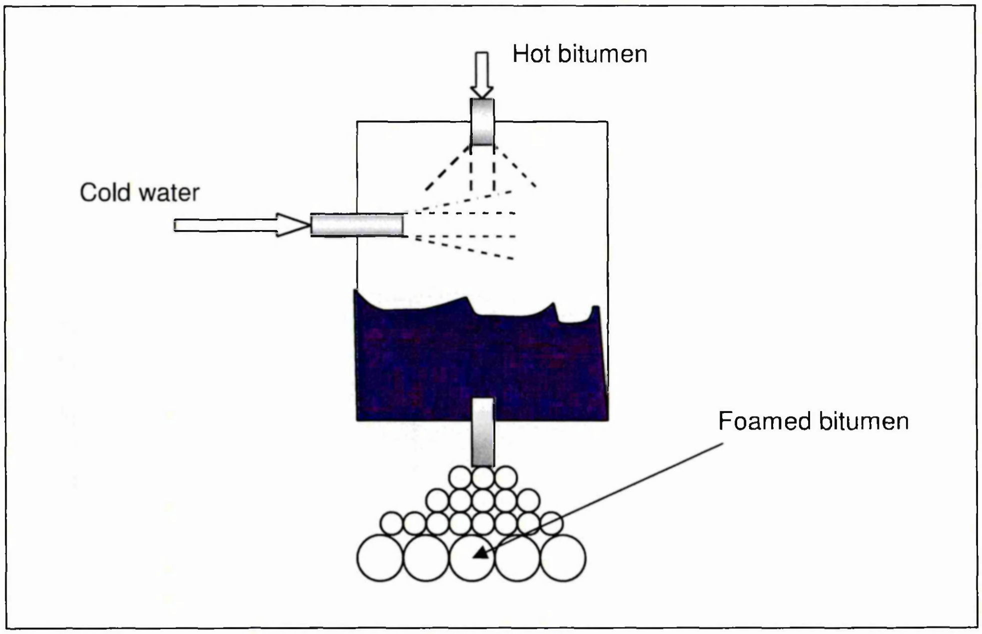


Figure 4-4: Foamed bitumen producing process

'In foamed-asphalt mixes, the optimum bitumen content often cannot be clearly determined as it can in the case of hot mixed asphalt. The range of binder contents (BC) that can be used is limited by the loss in stability of the mix at the upper end of the range and by water susceptibility at the lower end (Muthen, 1999).' The binder is closely related to the fines content. The fines content of the aggregate is an important consideration and should preferably be above 5% (Ruckle, 1982). As the bitumen foam can actually only partly coat the coarse aggregate, it is the bitumen and fine mortar that bind the coarse aggregates together. Ruckel (1982) suggested a bitumen content as shown in Table 4-3.

Jostein (2003) suggested a formula between foamed bitumen content and percentage of fine passing 0.075mm sieve as Equation 4-1:

$$P_a = 0.14 \times p_{0.075} + 2.6$$ Equation 4-1

Where:

p_a = Minimum binder content in weight percent (residual) ($P_a >$ 3.0%).

$p_{0.075}$ = Percentage of aggregate materials less than 0.075mm.

The binder content (P_a) derived from Equation 4-1 (Jostein, 2003) is similar to the value suggested by Ruckel (1982). In the case of in-situ recycling, where the aggregates are mainly composed of road planings, a bitumen content at 3.5±0.75% is suggested (Milton and Earland, 1999).

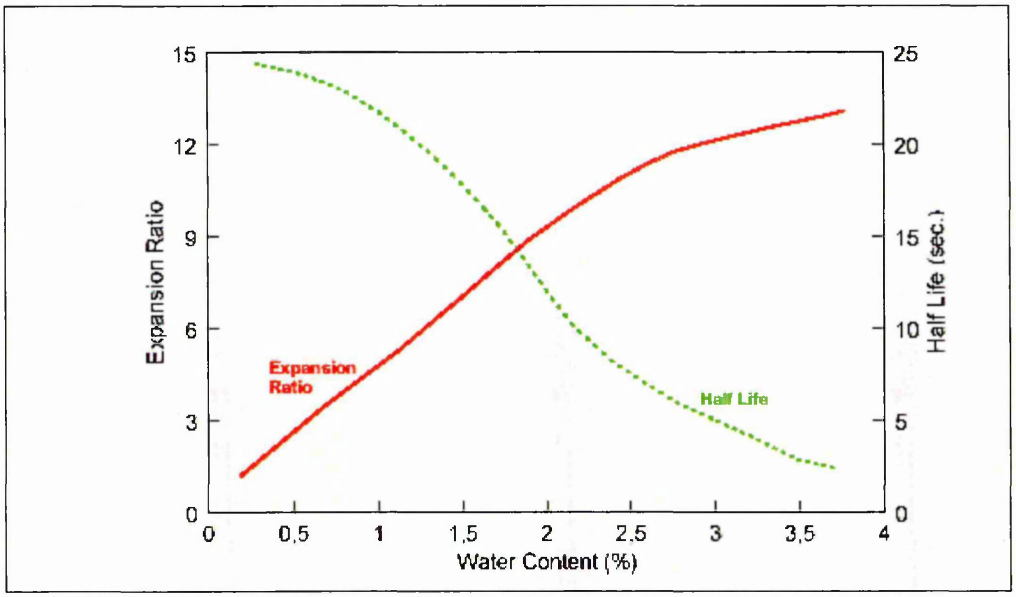


Figure 4-5 Duration of the foam (half life) and the expansion ratio (after Muthen, 1999)

Table 4-3 Bitumen content of foamed bitumen mixture

Passing 4.75mm sieve (%)	Passing 0.075mm sieve (%)	Foamed bitumen (%)
<50 (gravels)	3 – 5	3
	5 - 7.5	3.5
	7.5 – 10	4
	> 10	4.5
>50 (sands)	3 – 5	3.5
	5 - 7.5	4
	7.5 – 10	4.5
	> 10	5

'*It is possible to use penetration grade bitumen from grade 40 to over 200 as the base bitumen for foaming. Higher penetration grades tend to foam better, but the lower grades produce materials of higher stiffness. On balance, the foamed bitumen recommended for cold in-situ recycling is based on penetration grade 100 (Milton and Earland, 1999).*'

Foamed bitumen mixes can be produced in-situ or ex-situ. Ex-situ mixed materials can be stockpiled more or less indefinitely if protected from the sun and wind. With the in-situ mix process, the materials treated can be trafficked immediately after compaction and the hindrances caused by the job site execution can thus be kept to an absolute minimum.

4.3.5 Bitumen emulsion

Compared with foamed bitumen, where special foaming facilities are needed, bitumen emulsion cold-mix can be mixed at ambient temperature by traditional hot-mix facilities and paved by the traditional paving facilities, hence the initial capital investment is reduced. Such cold mixed materials have been proved very successful in many countries, with the most prominent being the French 'Grave Emulsion' and US 'Type 1' material.

Grave emulsion was first specified in France in 1974. This cold mixed material generally has a residual bitumen content of about 3.5%. It can be stockpiled, spread with a blade grader and compacted at ambient temperatures. It is mainly used for re-profiling lightly trafficked roads and has been claimed effective for preventing the transmission of thermal and shrinkage cracks when laid over materials bound with hydraulic binders. Grave emulsion is generally designed with less than 15% void content. The strength of Grave emulsion comes mainly from the interlock of the aggregates. Grave emulsion is not used by all regional authorities in France. It is mainly used in the warmer and drier southern regions due to the water sensitivity of emulsion based mixtures. The Type-1 cold mix in the US, which is dense graded and has been claimed to have a performance comparable to asphalt concrete, is mainly used in the mid-Atlantic and southern states (Leech, 1994).

There is no universally accepted emulsified or cut-back asphalt-aggregate mix-design method. Depending on the application, aggregate type and gradation, different bitumen contents are reported to be in use. According to DETR Report 85 (1998), *'the optimum residual binder content for the particular aggregate grading was estimated from the binder content of the nearest equivalent hot-mix asphalt mixture. Adjustments were then made, depending on the observed behaviour of the laboratory mixtures.'* MS-14 (Asphalt Institute, 1990) suggested that the bitumen content should be decided according to Equation 4-2:

$$P = (0.05A + 0.1B + 0.5C) \times (0.7) \qquad \text{Equation 4-2}$$

Where:

$P =$ Percentage by weight of asphalt emulsion, based on weight of graded mineral aggregate (%).

$A =$ Percentage of mineral aggregate retained on 2.36mm sieve (%).

$B =$ Percentage of mineral aggregate passing 2.36mm sieve and retained on 75μm sieve (%).

$C =$ Percentage of mineral aggregate passing 75μm sieve (%).

Although Grave emulsion and Type-1 EAM (emulsified asphalt macadam) have successfully been used in France and US, their application in UK has not been very successful. Taylor (1997) attributed the slow development of cold mix in UK to the following reasons:

- A lack of awareness of the significance of changes in grading and surface chemistry in the early days by both bitumen producers and road stone manufacturers.
- The widespread use of dense continuously graded rather than open-textured gap-graded mixes hindering effective water evaporation.

4.3.6 Cold mixes comparison

Table 4-4 is a general summary of the advantages and limitations of different recycling technologies. Foamed bitumen and bitumen emulsion cold mixes are reported to have similar physical properties (Lewis and Collings, 1999; Jostein, 2003) as shown in Table 4-5. According to Leech (1994), cold mix is energy saving compared with hot mixes, but the advantage will be lost if an active hydraulic binder such as Portland cement is added (Table 4-6). With the addition of latent hydraulic binder instead of active hydraulic binder, the workability is intact, hence the advantage of energy saving can be maintained.

Table 4-4 Cold mix comparison (Lewis and Collings (1999) with modifications)

Materials	Advantage	Disadvantage
Cement bound materials	Easy to apply as a powder or slurry, less expensive than bitumen emulsion and improves material's moisture resistance.	Shrinkage cracking can be a problem and no stockpile life.
Bitumen emulsion bound materials	Easy to mix, transport and compact, long stockpile life, flexible and fatigue resistant layer can be produced from the mixture, can be reworked and relocated at early age.	Usually more expensive than cement or foamed bitumen, emulsion treatment can be a problem when in situ moisture contents are high, weak initial strength.
Bitumen emulsion/cement bound materials	The cement/emulsion combination produces higher strength, cures quicker and is more moisture resistant than emulsion alone. If properly designed, it is not prone to shrinkage cracking.	More expensive than either cement or emulsion alone, also more expensive than foamed bitumen, short shelf life and less energy saving than bitumen emulsion cold mixes.
Foamed bitumen bound materials	Foamed bitumen is easy to apply, not prone to shrinkage cracking, less expensive than bitumen emulsion or combination of bitumen emulsion and cement, less water addition, rapid strength gain. The road can be trafficked immediately after compaction is complete,	Bitumen has to be warmed up to high temperatures, aggregate should have between 5% and 15% passing the 75µm sieve size. If this is not the case, the grading should be rectified by importing and spreading a layer of suitably graded aggregate over the layer to be recycled.

Materials	Advantage	Disadvantage
	can be stockpiled, relocated and reworked if necessary in their early life.	Coarse aggregate just partially coated. Purpose built foamed bitumen stabilization equipment is required.
Foamed bitumen/cement bound materials	With 1-2% cement added, higher strength especially early strength achieved, more moisture resistant and less expensive than bitumen emulsion plus cement.	It has the same bitumen temperature and aggregate grading requirements as foamed bitumen, more expensive than foamed bitumen alone, cannot be stockpiled, more energy consumption than foamed bitumen mixes
Latent hydraulic binder (PFA or GGBS activated by lime or cement)	Easy to apply, long shelf life, less expensive than cement, high long term strength, reduced potential thermal cracking compared with cement as binder	Weak early strength

Table 4-5: Comparison of the mixes with different binders (Lewis and Collings, 1999)

Test parameter	Cement (3%)	Bitumen emulsion (3.5% net bitumen)	Emulsion/cement (3.5% net bitumen plus 2% cement)	Foamed bitumen (3.5%)	Foamed bitumen/cement (3.5% bitumen plus 1% cement)
Unconfined compressive strength (MPa)	3	n/a	n/a	n/a	n/a
Indirect tensile strength (dry) (kPa)	250	200	500	200	500
Indirect tensile strength (soaked) (kPa)	n/a	80	250	80	300
Marshall stability (dry) (kN)	n/a	10	20	10	20
Resilient modulus (dry) (MPa)	5 000	2 000	3 500	2 000	3 500

Table 4-6: Comparing energy requirement for cold mixed and hot mixed materials

Energy components	Semi-dense graded mixes (3.6% residual bitumen) Energy (MJ/t)		Dense graded mixes (6% residual bitumen) Energy (MJ/t)	
	Cold-mix	Hot-mix	Cold-mix	Hot-mix
Material	110	91	373	105
Mixing and laying	30	270	30	320
Transport	53	50	60	53
Totals	193	411	463	478

In this project, latent hydraulic binder was used instead of active hydraulic binder. Such cold mixes have the advantage of energy saving compared with hot mixes as indicated in Table 4-6.

4.4 SUMMARY

With the emphasis on sustainable development, research has been focusing on effectively using recycled/secondary aggregate. Aggregate recycling for pavements can be conducted by hot or cold recycling, both can be conducted in-situ or off-site. Because of the strict requirements on aggregate, hot recycling is practically limited to road planing only, while cold mix can cope with all kinds of aggregates, from virgin aggregate to marginal aggregates, even glasses and shredded tyres. Cold mixed materials can be bound by bitumen emulsion and foamed bitumen or hydraulic binders. Currently foamed bitumen with hydraulic binder such as cement or PFA/lime is widely accepted in the industry. It is suggested that a hybrid of bitumen and hydraulic binder could mitigate the crack potential of cement bound material whilst attaining higher strength/stiffness than materials bound with only bitumen emulsion.

CHAPTER 5 TEST METHODS OVERVIEW

5.1 INTRODUCTION

The aim of this chapter is to present a general introduction of the test methods employed in this research. Some of the test methods are further discussed in later chapters where the test is referred to in more detail.

Conventional bound layers of pavement are composed either of hot bituminous or hydraulically bound materials. For bituminous bound materials, the design process basically varies the aggregate gradation and binder content in order to achieve the target volumetric and physical performance. The volumetric properties are embodied by air void content while the physical performances are often related to the pavement's ability to distribute load and failure mode. The latter represented by permanent deformation and fatigue cracking. For hydraulic bound materials, the design process involves controlling the compressive strength by changing the cement content. Cold mixed bituminous materials, especially with the addition of latent hydraulic binders such as PFA/lime, which is very common in UK, are different from conventional bituminous materials. Initially, they behave more like granular materials but over time, both bituminous and hydraulic bonds build up and the materials act in between bitumen and hydraulic bound materials.

The following tests are widely employed in the pavement materials design process. They have also been conducted or referred to in this research:

- Indirect Tensile Stiffness Modulus test (ITSM) (BS DD 213: 1993).
- Repeated Load Axial test (RLAT) (BS DD 226: 1996).
- Indirect Tensile Fatigue test (ITFT) (BS DD ABF: 1995).
- Indirect Tensile Strength test (ITS) (BS EN 12697-23: 2003).
- Compressive Strength test (BS EN 12390-3: 2001).
- Maximum Theoretical Density test (BS DD 228: 1996).

ITSM, RLAT and ITFT are commonly referred to as NAT tests because all these tests are conducted with the Nottingham Asphalt tester. NAT tests and Maximum Theoretical Density test are designed for bituminous materials. ITS test has two versions, one relates to bitumen bound materials (BS EN 12697-23: 2003) and the other is for hydraulic bound materials (BS EN 13286-42: 2003). The version for bitumen bound materials is employed in this project. Compressive strength test is designed for hydraulic bound materials by detecting the peak load at failure. Bituminous materials continuously deform under compressive load and there is no clear peak value in the compressive process, therefore it is not possible to conduct a compressive strength test. Bitumen emulsion cold mixes with the addition of latent hydraulic binder behave similarly to hydraulic bound materials in the long term, hence the compressive strength test can be conducted.

5.2 PAVEMENT MATERIAL PERFORMANCES AND TEST METHODS

There are a set of well-established test methods for hot mixed materials. The cold mixes design has been aiming at achieving similar performances to hot mixes. Cold mix design currently adopts ideas and experimental methodologies from hot mix design, but cold mixes have unique performance, which cannot be represented by hot mixed materials. Thom (2004) suggested that *'the most appropriate way forward in understanding and therefore using these materials, is to consider them as exceptionally high quality layers rather than as equivalent to hot-mix materials.'*

The hot mixed materials are composed of three parts, which are air, bitumen and aggregates. The cold mixes have extra water content (Figure 5-1). In the bituminous material design process, the following properties are considered (Indiana Department of Transport, 2004).

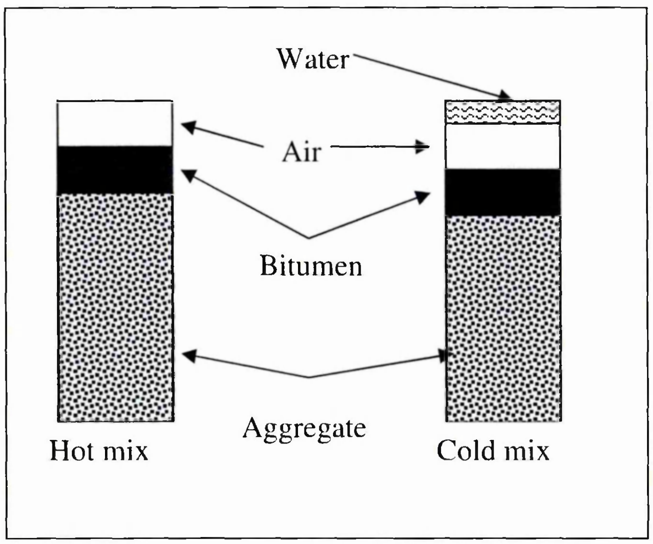


Figure 5-1: Mixture components in hot and cold mix

- Stability
- Durability

- Permeability
- Workability
- Flexibility
- Fatigue resistance

These properties are actually interrelated, for example, materials with good workability generally have low void content after compaction and materials with low void content tend to have high stability.

5.2.1 Stability

The stability of hot mixed bituminous materials relates to its ability to resist shoving and rutting under load after compaction. A stable pavement maintains its shape and smoothness under repeated loading, whereas an unstable pavement develops permanent rut deformation. Stability is related to the binder penetration, void content and aggregate gradation. Tests relate directly to stability include the Marshall test (BS 598-107:1999), Repeated Load Tri-axial test, Wheel Track test (BS DD 184:1990), Repeated Load Axle test (RLAT), Indirect Tensile Stiffness Modulus test (ITSM) and Indirect Tensile Strength test (ITS). Stability is the most important property in the materials design process, as it reflects the ability of a layer to spread load and thereby reduce stress concentration at the base of the bituminous layers and into the underlying layers.

5.2.1.1 Marshall Test

The optimum binder content of the HRA is designed by the Marshall test, which is directly linked to stability. In the Marshall design process, a number of specimens with different binder contents are compacted and then compressed at a rate of 50mm/min. The load on the specimen and the displacement are monitored and the maximum load is recorded as stability and the corresponding displacement at this point is recorded as flow. Thus a relationship between bitumen content and stability, mix density, and compacted aggregate density for each mix across a range of incremental binder contents is established. The mean of the maximum stability, mix density and compacted aggregate density is used as the basis for deciding the design binder content.

Marshall design has a very long history and a rich resource has been built up over years. It is helpful to understand the performance of a new material by comparing it with well understood materials through the Marshall test. However, as an empirical test method, there is no stiffness modulus derived from this test and the result cannot be related to fundamental properties.

5.2.1.2 Indirect tensile stiffness modulus test (ITSM)

With the development of analytical design methods and the adoption of performance specifications, fundamental properties such as the stress–strain relationships are more and more important. The stress-strain relationship is represented by the stiffness modulus, which is calculated by dividing stress by strain. The bituminous material is a visco-elastic material, which means that the stress–strain relation is not linear as in the case of metals in the elastic phase as presented in Figure 5-2. Factors including stress level, temperature, loading rate and duration of load all affect the stiffness modulus. Figure 5-3 illustrates that a pavement with a higher stiffness modulus is able to distribute the traffic load to a wider area, hence the stress level per unit area is reduced and the potential damage to the sub-grade is alleviated.

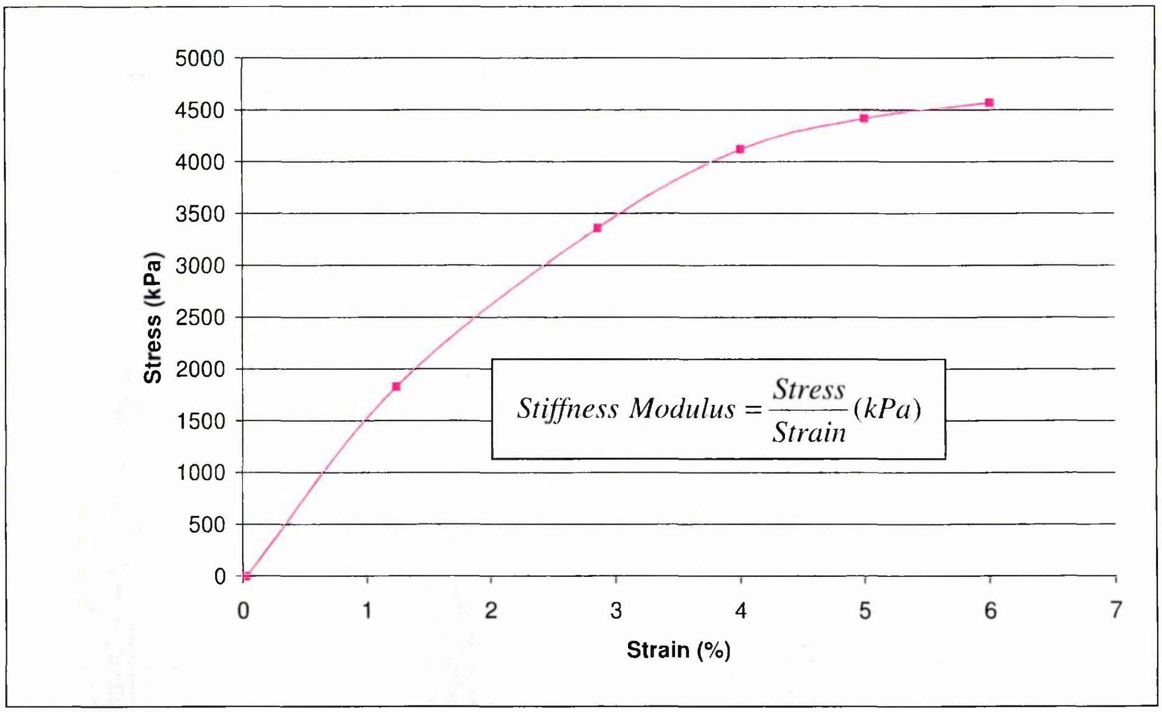


Figure 5-2: Typical non-linear relationship between stress and strain

Several stiffness modulus test methods have been presented in BS EN 12697–26 (2004), including the bending beam test, indirect tensile test, uniaxial tension-compression test etc. All these tests are conducted at certain temperatures, with fixed stress or strain level. The most widely used stiffness modulus test in the UK is Indirect Tensile Stiffness Modulus test (ITSM). In the test, a vertical load is applied through two steel platens and tensile stress is generated at right angles to the load. The vertical load and horizontal deformation are monitored in the test (Figure 5-4). By calculating the tensile stress at the centre of the specimens and detecting the horizontal deformations generated at the two transducers, the stiffness modulus is calculated (Equation 5-1).

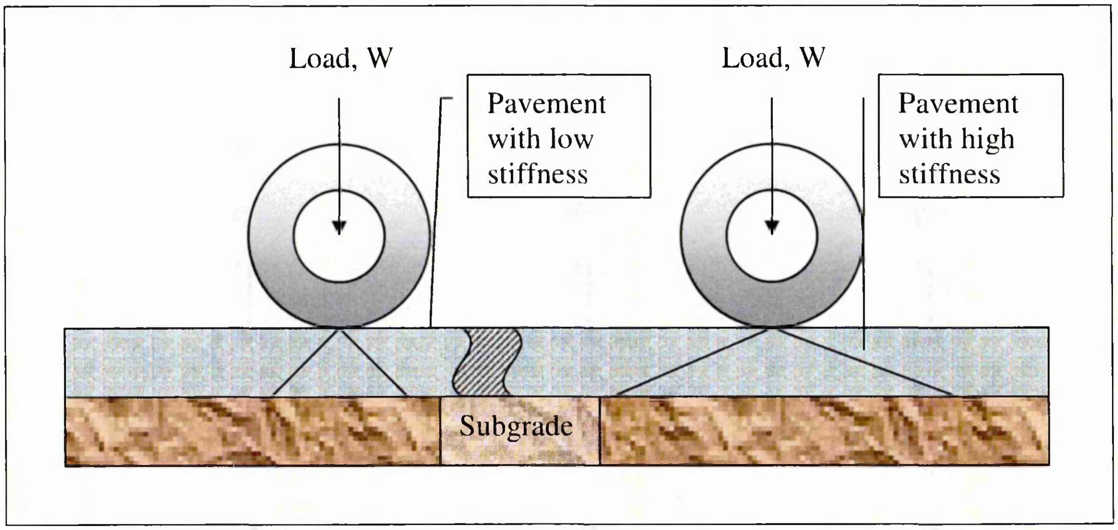


Figure 5-3: Spread of wheel load pressure through pavement structure

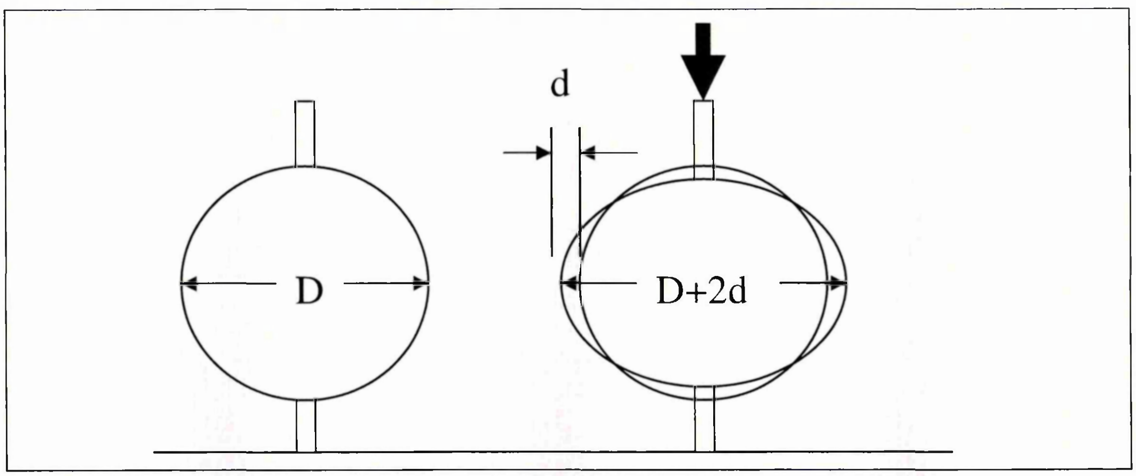


Figure 5-4: Sketch of the ITSM test

$$ITSM = \frac{L}{(d \times t)}(\upsilon + 0.27)$$ **Equation 5-1**

Where:

L Peak value of the applied vertical load (N)

d Peak horizontal deformation resulting from the applied load (mm)

t Mean thickness of the test specimen (mm).

υ Poisson's ratio for the bituminous mixture at the temperature of test.

In the ITSM test, cylindrical specimens with 100mm or 150mm in diameter may be employed, depending upon the aggregate size. In the case of normal aggregate size i.e. less than 20mm, specimens with a diameter of 100mm are regarded as appropriate, otherwise 150mm diameter specimens are used. As the stiffness modulus is different at varying stress levels and deformation conditions, the applied load and the horizontal deformation are strictly controlled in the test process. *'UK experiences indicated suitable values of peak horizontal deformation are 7±2μm for a 150 mm nominal diameter specimen and 5±2μm for a 100 mm nominal diameter specimen (BS DD 213: 1993).'* The bitumen is a viscous material and its stiffness varies with temperatures, therefore the ITSM values for the same materials will be different at different temperatures. *'Bituminous materials are highly sensitive to temperature, and the ITSM test is conducted at 20°C although other test temperatures may be used (BS DD 213, 1993)'.* In Equation 5-1, the Poisson's ratio is the ratio between vertical and horizontal strain. For hot mixed bituminous materials, the normal Poisson's ratio at 0°C is 0.25, at 20°C is 0.35 and at 30°C is 0.45.

The ITSM test set up is further explained in Figure 5-5, which was produced by SWPE in 2004.

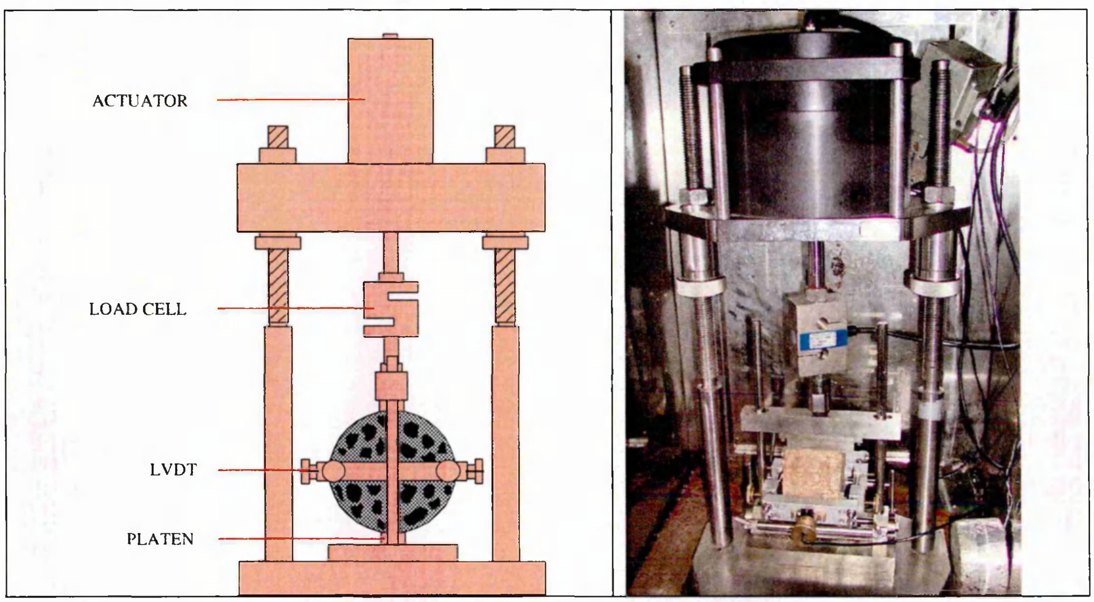


Figure 5-5: Illustration of ITSM test (SWPE, 2004)

In comparison with other methods to determine the stiffness of bituminous materials, (e.g. direct tension and/or compression or bending beam), the ITSM testing has several advantages and disadvantages as detailed in Table 5-1.

Table 5-1: Advantages and limitations of ITSM test

Advantages	Limitations
1. The test is simple, quick to conduct. 2. Non-destructive if testing conditions ensure elastic response. 3. Fewer problems with manufacturing specimens, e.g. specimens are moulded or cores. 4. Thin specimens can be tested as the equipment can accommodate specimens with thickness of between 25–75mm. 5. A biaxial stress state exists during the test which better represents field conditions than the stress condition found in flexural testing.	1. The method relies on theoretical analysis using elastic theory. 2. Using an assumed Poisson's ratio which makes the test less reliable than direct tension/compression or flexural tests. 3. Susceptible to the development of permanent deformation if tested at high temperature. Temperature higher than 40°C should be avoided.

In this project, the nominal size of the aggregate used is 20mm, hence 100mm diameter specimens were appropriate. As indicated in Equation 5-1, deformation and Poisson's ratio are needed in calculating the stiffness modulus. Although 5μm are recommended in the NAT test user's manual for 100mm diameter hot asphalt specimens, 7μm horizontal deformations were used in this project considering cold mixed materials are soft and high deformation can be generated under limited load. Poisson's ratio was assumed as 0.35 in this project, although this is normal figure for hot mixed materials. For cold mixed materials, especially with the addition of hydraulic binder, which has a performance somewhere between hydraulic and bituminous bound materials, the Poisson's ratio could be different. '*Kolias and Williams have studied Poisson's ratio for various cemented materials ranging from stabilized cohesive soil to lean concrete. Static values close to 0.15 were found at an age of 28 days* (Croney, 1997).' For cold mixed materials with latent hydraulic binder, the Poisson's ratio experiences changing from a very high value as unbound granular materials to a very low value as hydraulic bound material, this change happens over a long time and it is very difficult to track this transformation. Nevertheless, the ITSM test was very successful in detecting the stiffness development of cold bituminous materials in this project as well as in other research work (Needham, 1996; Ibrahim, 1998) with a Poisson's ratio assumed at 0.35.

The ITSM test was the major test method undertaken to detect the cold mixes performance throughout this research project. The results are presented in the following chapters.

5.2.1.3 Compressive Strength test

By definition, the compressive strength of a material is that value of uniaxial stress reached when the material fails completely, it is mainly used to characterise the performance of hydraulic bound materials and the set up is pictured in Figure 5-6. The results can be used for both control and compliance purposes. Laboratory prepared or field cores, which could be cylinders or cubes (laboratory only), are loaded to failure in a compression testing machine. The maximum load sustained by the specimen is recorded and the compressive strength of the concrete is calculated by dividing maximum load with the area of cross-section. Both pavement quality concrete and cement bound materials, which are widely used in road construction, are specified by their compressive strength.

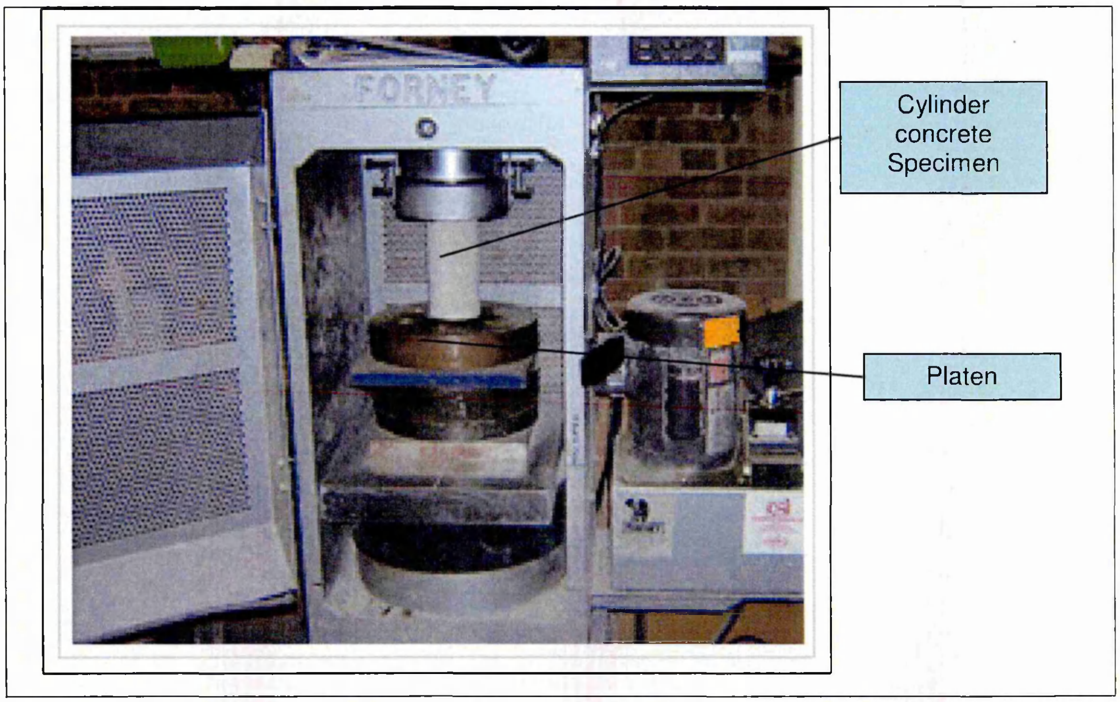


Figure 5-6: Illustration of concrete compressive strength test

In the pavement analytical design process, the tensile strength instead of compressive strength is often employed to predict the pavement residual life. Tensile strength test is not normally conducted and a relationship developed by Croney (1997) could be used to predict flexural strength from compressive strength:

$$f_f = c \times f_c \qquad \text{Equation 5-2}$$

Where:

c coefficient, 0.11 for gravel and 0.16 for crushed rock

f_f tensile strength (N/m^2)

f_c compressive strength (N/m^2)

This relationship has been employed in the current specification for highway design, which is HD26/06, DMRB (Highway Agency, 2006).

Hot bituminous materials are not suitable for the compressive strength test because such materials continuously deform under the compressive load and no maximum compressive force can be detected. Therefore, the compressive strength test was only conducted on hydraulic bound materials in this research project and the results have been presented in Chapter 9.

5.2.1.4 Indirect Tensile Strength test (ITS)

The ITS test has been widely used in estimating moisture resistance (AASHTO[2] designation: T 283-99) where 100mm or 150mm cylindrical specimens are used. In this test, specimens are brought to the specified test temperature, placed in the compression testing machine between the loading strips and loaded diametrically to failure. The maximum load is recorded and the tensile stress at the centre of the specimen is calculated. When measuring the moisture resistance, specimens are split into two groups, one group cured in ambient conditions and the other group cured in water. By comparing the tensile strength of specimens cured in ambient conditions with specimens immersed in water, the moisture resistance performance of the cold mix can be established.

In the project, this test is employed to assess the contribution from bitumen emulsion towards the cold mixes and to compare the performance of cold mixes with hot mixes. The detailed test procedures and the key findings are presented in Chapter 10. A photo of ITS test set up from SWPE is presented in Figure 5-7.

[2] AASHTO is the acronym of the American Association of State Highway and Transportation Officials. AASHTO is a leading source of technical information on design, construction and maintenance of highways and other transportation facilities, including aviation, highways, public transit, rail and water in the US.

Figure 5-7: ITS test set up, reproduced from SWPE (2004)

5.2.1.5 Wheel Track test and Repeated Load Axial test (RLAT)

When a flexible pavement is subjected to a wheel load, it undergoes both recoverable and irrecoverable deformations because the bitumen is a visco-elastic material. The irrecoverable deformation leads to pavement rutting, which is the result of the accumulation of small irrecoverable strains under repeated loads. Permanent deformation is one of the major failure modes of bituminous material. The external factors related to rutting include hot weather, high axle load, slow traffic, uphill gradient section. The internal factors might be high void content, soft binder and smooth aggregate. The following measures in the pavement design process can be taken in order to improve the resistance to rut deformation:

- Using hard binder or modified binder.
- Reducing the binder content.
- Increasing the coarse aggregate content.
- Designing flexible composite pavement instead of flexible pavement in sections liable to rut.

Test methods assessing the permanent deformation performance include Marshall test, Wheel Track test, Repeated Load Triaxial test, Dynamic Creep test, Static Creep test and Hveem Stabilometer. The most widely used test methods are Wheel Track test (BS 598-110: 1998) and Repeated Load Axial test (RLAT).

The specimen for Wheel Track test can be in different sizes and the laboratory Wheel Track test is generally conducted on 200 mm diameter cylindrical cores or on lab compacted slabs between 35 mm and 50 mm thick, at 45°C or 60°C. In the Wheel Track test, a wheel with a rubber tyre keeps on running on a test specimen, exerting a pressure similar to those experienced on the road until the wheel-track deformation reaches a depth of 15 mm or for 45 min, whichever is the sooner (Figure 5-8). The wheel track rate is expressed as mm/h and the mean of six cores is used as the valid result. The Wheel Track test is widely used to detect the deformation resistance. The test was not conducted in this project but the results from previous researchers have been referred to in this thesis.

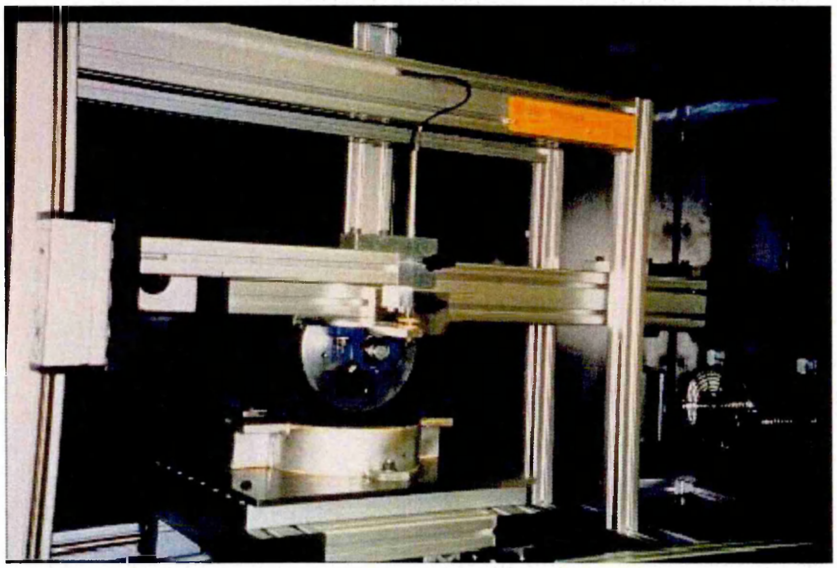


Figure 5-8: Sketch of Wheel Track test, adopted from SWPE (2004)

Alternatively, the NAT test can also be employed in detecting the deformation resistance. In the NAT RLAT test, a cylindrical specimen of 100 mm or 150 mm in diameter is put under two platens, as presented in Figure 5-9 and subjected to a static stress of 10 kPa, nominally square wave, of one second duration followed by a rest period of one second duration was applied repeatedly for 3600 cycles. The test is normally conducted at 30°C. The developed deformation is monitored continuously. The strain is calculated by Equation 5-3 as follows:

$$\varepsilon_{d(n,T)} = \frac{\Delta h_o}{h}$$ Equation 5-3

Where

$\varepsilon_{d(n,T)}$ is the axial strain of the specimen after n applications of load at temperature T (°C).

Δh is the axial deformation (change in distance between the specimen loading surfaces) (mm).

h is the original distance between the specimen loading surfaces (mm)

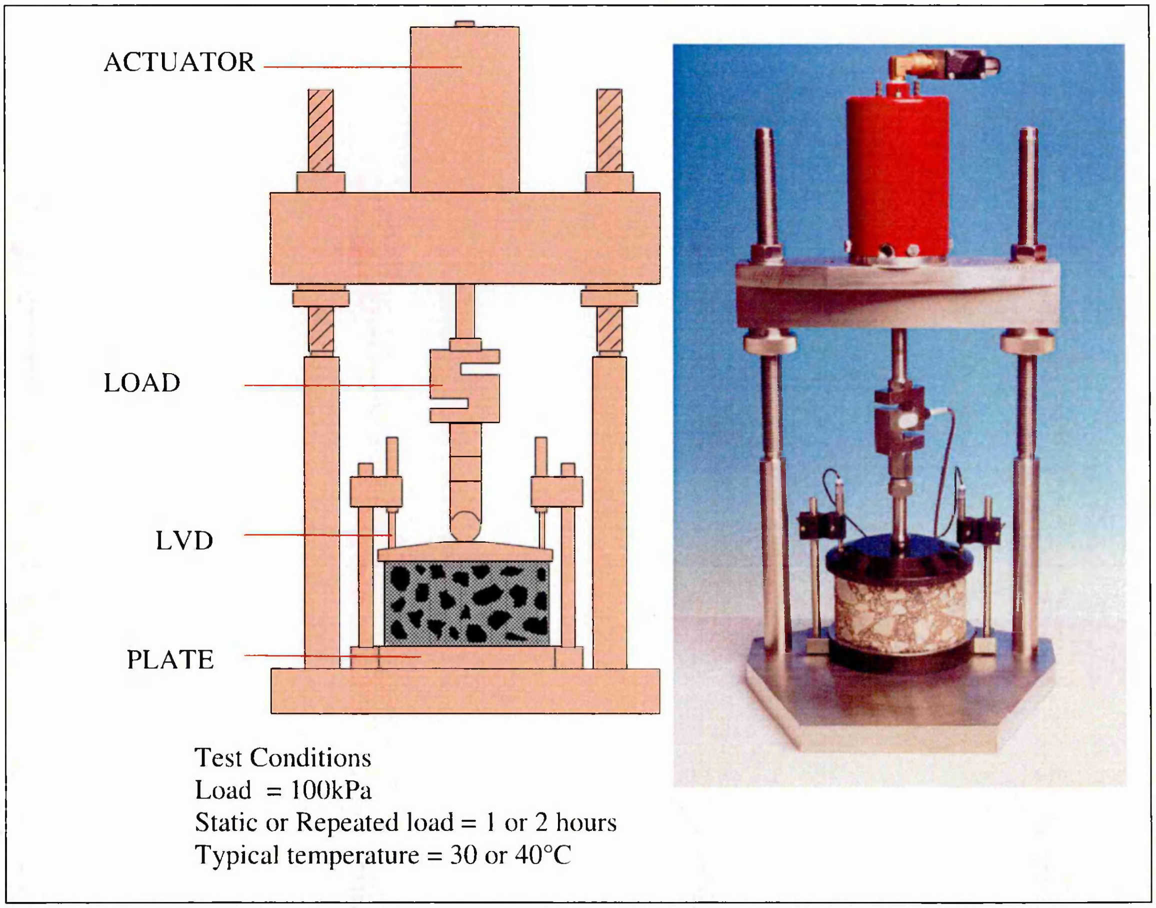


Figure 5-9: Sketch of RLAT test adopted from SWPE (2004)

Over the years, two versions of the RLAT test have been developed, which are the confined and un-confined RLAT test. In the confined RLAT test, there is a plastic band wrapped around the specimen to simulate the confinement of the materials in the pavement, which is claimed to be more realistic than an unconfined RLAT test.

In this project, the unconfined RLAT test was conducted to assess the rut resistance. The RLAT test is included in the BBA/HAPAS (2001) guideline to evaluate the performance of the cold mixed materials, which suggests that the strain level should be less than 16,000μm for 1800 load applications or less than 20,000μm for 3600 load applications for cold mixes to be accepted as certified PCSMs[3] (Table 5-2).

[3] PCSMs: Permanent Cold-lay Surface Materials

Table 5-2: BBA/HAPAS PCSM elastic stiffness and resistance to permanent deformation requirements

Permanent cold-lay surfacing material	Minimum property requirement at 20°C		
	50 Pen hot-laid elastic stiffness (MPa)	100 Pen hot-laid elastic stiffness (MPa)	200 pen hot-laid elastic stiffness (MPa)
20mm nominal size base course	4000	2400	900
10 mm nominal size base course	3800	1900	800
6 mm nominal size wearing course	2800	1400	600
All materials	Resistance to permanent deformation at 30°C Repeated load uniaxial creep resistance to permanent deformation, shall not exceed: 16,000 microstrain for 1800 load applications or 20,000 microstrain for 3600 load applications.		

5.2.2 Fatigue property

Fatigue cracking is a major mode of failure of traditional hot mixed materials. Fatigue failure can be described as the phenomenon of fracture under repeated or fluctuating stress having a maximum value generally less than the tensile strength of the material (Whiteoak, 1991). Pavement structures are subjected to tensile stress coming from both traffic load and thermal expansion/contraction. Brown (1995) further defined fatigue in bituminous pavements as the phenomenon of cracking. It consists of two main phases, crack initiation and crack propagation and is caused by tensile strains generated in the pavement by not only trafficking but also by temperature variations and construction practice. Fatigue failure can happen to both surface and base courses. Previous studies have shown that strain rather than stress is the principal fatigue criterion for crack initiation rather than stress. The magnitude of the tensile strain is dependent upon the exerted load, the stiffness of the layer carrying the load and the stiffness of the underlying layers (i.e. the amount of support). At same loading level and the pavement thickness, the pavement constructed with materials of higher stiffness generated lower strain at the bottom of the pavement layer therefore longer pavement life can be expected.

Various fatigue test methods have been developed over the years, including trapezoidal beam fatigue test and the three or four points bending beam test (BS: EN 12697-24: 1999). Although these reported test methods give equivalent and reproducible results (Brown et al, 1995), the specimens are not easy to fabricate. Compared with other test methods, the indirect tensile fatigue test (ITFT) is relatively easy to conduct. The ITFT test facility and test condition are graphically illustrated in Figure 5-10. The specimens employed are cylindrical which can be easily fabricated in laboratory or extracted from field cores.

In the NAT (Nottingham asphalt tester) fatigue test, cylindrical specimens of 100 mm diameter are repeatedly loaded to failure at various strain levels as presented in Figure 5-10. A relation is drawn between strain level and the number of load applications to failure, in order to compare the fatigue properties of different materials. This test is further explained in Chapter 10.

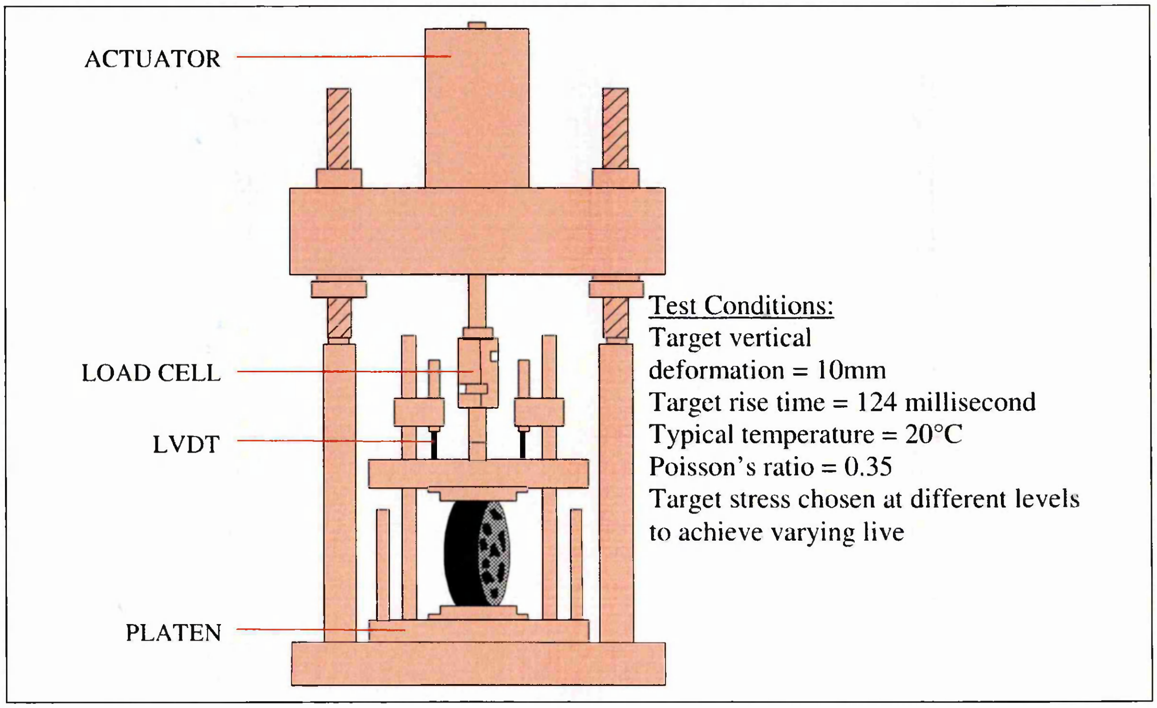


Figure 5-10: ITFT test illustration (SWPE, 2004)

5.2.3 Durability

The durability of bituminous pavement is its ability to resist factors such as changes in the binder oxidation, and disintegration of the aggregate. These factors can be the result of weather, traffic or a combination of the two. High binder content, sound aggregate and low void content makes positive contributions towards the durability. High binder content helps to reduce the void content and sound aggregate resists disintegration under bad weather and heavy traffic. The aggregate soundness test is detailed in BS 812-121:1989. In the test, a sample of aggregate in the size range 10.0 mm to 14.0mm is subjected to five cycles of immersion in a saturated solution of magnesium sulphate, followed by oven-drying at 105°C to 110°C. This subjects the sample of aggregate to the disruptive effects of the repeated crystallization and re-hydration of magnesium sulphate within the pores of the aggregate. The percentage of the mass of materials retained on the 10mm sieve compared with the initial mass of specimens is termed as aggregate soundness value. When used in bituminous materials, the soundness value should be greater than 75 (Clause 900, MCHW1).

In the cold mixes, the aggregates are only partially coated and thus the aggregates are even more exposed to the influence of the environment than hot mixes. Although no aggregate durability test was conducted in this project, it can be assumed that the properties of the recycled aggregates are highly variable and there is no guarantee of unique high soundness value. This is another reason limiting the application of recycled materials to the less trafficked road.

5.2.4 Permeability

Permeability is closely related to void content. Hot mixed base and binder course are usually designed to be impermeable while the surface layer is designed to be porous or with a certain level of texture to reduce the noise and water splash, except in the case of newly developed reservoir pavements. In the case of the reservoir pavement, even the base is designed to be porous to facilitate the water penetrating to the lower layer.

'*The void content in a hot mix will influence the pavement quality. Low void content causes instability and excessive deformation and rutting. On the other hand, high void contents may cause post compaction, ingress of water and reduced durability. For asphalt concrete with dense grading curve, it is commonly accepted that void contents below 2% should be avoided. A void content between 2% and 5% is regarded as favourable, while a void content in the range of 7% to 10% is undesirable (Roar and Jostein, 2000).*'

The void content of bituminous materials can be calculated according to Equation 5-4.

$$Air\,Void\,Content\ (\%) = \frac{(Maximum\ theoretical\,density\ -\ Bulk\ density)}{Maximum\ theoretical\ density} \times 100$$

Equation 5-4

The maximum theoretical density test procedure is outlined in BS DD 228 (1996). The test is also referred to as the Rice Method. In the test, a portion of coated aggregate is first oven-dried to a fixed weight, the test sample is further submerged in water and subject to partial vacuum for 15 min, then the volume of the test sample after submerging is calculated. Since after partial vacuum conditioning all the pores on the aggregate surface are filled with water, the volume calculated in such a way is assumed to be the volume of aggregate and bitumen. As a result, the density is referred to as maximum density. The test procedure is also outlined in BS 598-104 (1989).

Air void content assessments were undertaken in this project and the results are presented in Chapters 7, 8 and 9.

5.2.5 Workability

The term 'workability' is commonly used to describe the ease, or otherwise, of mixing, laying and most importantly compacting a bituminous material. Less workable mixes are not as mobile during application and compaction (Whiteoak, 1990). The workability of hot mixed materials is governed by several factors. A high fines content mix will be less workable than a low fines content mix and one having angular crushed rock aggregates will be less workable than one having rounded aggregates. A major factor in the workability of a mixed material, however, is the viscosity of the binder which is governed by the temperature of the material (BACMI, 1992).

There is no universally accepted test method on workability. An asphalt producer, Nynas Group, developed a simple facility similar to the shear box test as shown in Figure 5-11 and Figure 5-12 to test the workability of bituminous material. For hot mixes, workability is directly related to the void content after compaction and the required workability is achieved by controlling the temperature in the mixing and compaction process. Jostein (1999) suggested that cold mixes have a lower workability than traditional hot mixes based on the test results presented in Figure 5-13, which was conducted using the Nynas Workability tester illustrated in Figure 5-11 and Figure 5-12.

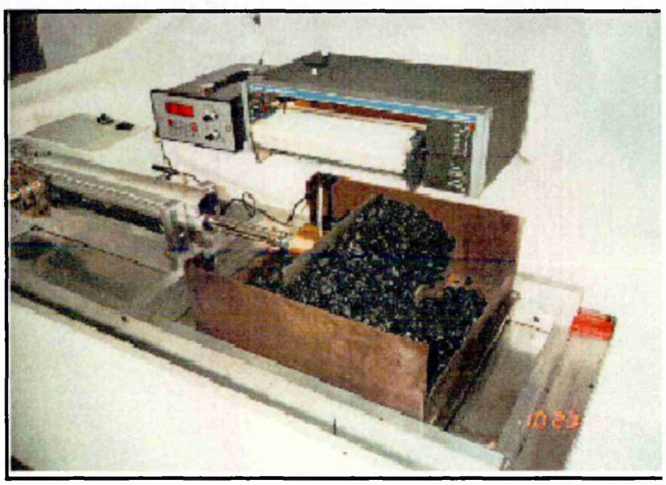


Figure 5-11: Nynas workability tester

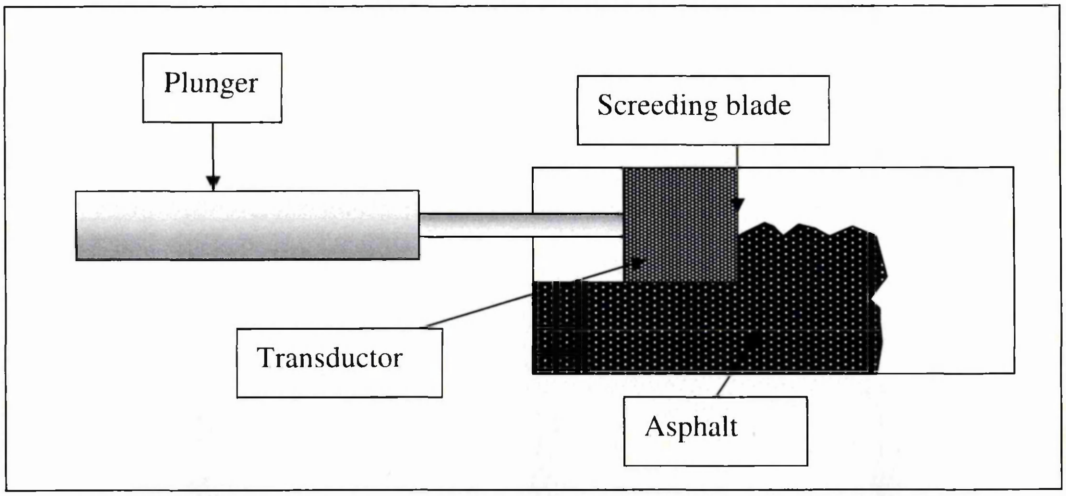
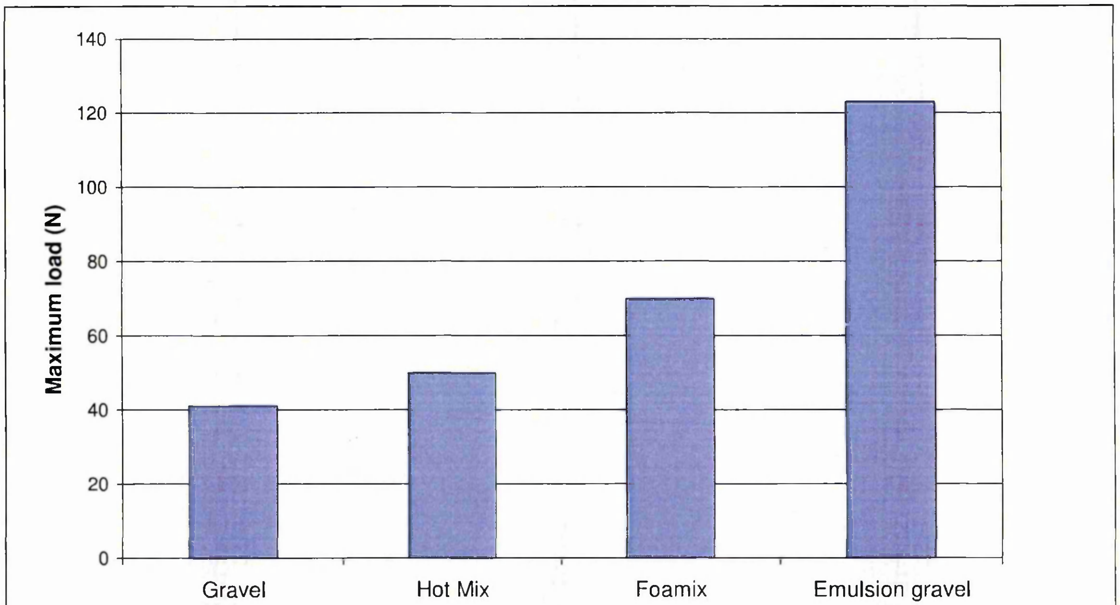


Figure 5-12: Schematic illustration of Nynas workability tester



Note: Foamix is a proprietary foamed asphalt product.

Figure 5-13: Comparing workability of different materials

5.3 SUMMARY

Test methods related to bituminous and hydraulically bound materials are introduced in this chapter. The performance of bituminous materials may be assessed by its stability, stiffness, strength, fatigue cracking resistance, permeability and workability.

Stability is related to the capability of the material in a pavement layer to distribute load to the lower layers and is normally measured by the Marshall test. Stiffness, which is assessed using the ITSM test, is a measurement of the stress-strain relationship of the materials under prescribed controlled conditions; because bituminous materials are visco-elastic and the stress -strain relationship is not linear.

Strength, normally compressive, is used to characterise the resistance to load of hydraulically bound materials but this test is not suitable for hot asphalt materials because such materials cannot sustain compressive loads for a maximum force to be obtained. However, Indirect Tensile strength has a different mode of failure to the compressive strength test and may be used to characterise hydraulic and bituminous materials.

Irrecoverable deformation and the resultant rutting is a major cause of failure in asphalt pavement. The Wheel-track test and RLAT test have been developed to measure the deformation resistance. Both tests are conducted at an elevated temperature and load applied at a frequency simulating the traffic load. In Wheel-track test, the load is applied through a rubber tyre whilst in RLAT test, the load is applied through a pair of platen.

Fatigue cracking is another major mode of failure of asphalt pavement. The fatigue cracking, which is a result of fracture under repeated or fluctuating applications of stress, could originate from the pavement surface or the underside of the asphalt layers. The ITFT test measures the fatigue resistance of materials by measuring the number of cycles that results in failure at a given strain level.

Permeability is closely related to void content. Impermeable materials are normally required on the road surface and bridge deck to inhibit water penetrating to the lower layers.

Workability describes the ease with which material may be laid and compacted, and in turn influences the void content. Previous work has demonstrated that cold mixtures have a lower workability than their hot mix counterparts.

CHAPTER 6 COMPACTION METHODS

6.1 INTRODUCTION

Compaction method has a fundamental impact upon the physical properties of the specimens. Over the years, different compaction methods have been developed, in most cases designed for hot mixed materials. The objective of this part of the research was to review the relative characteristics of different compaction methods in the context of cold mixes and choose one for further research work.

Ideally, the laboratory compaction should link up with the field compaction, but in the event this was not possible due to the enforced constraints of this project, therefore other researchers' work was referenced.

The most prominent compaction methods developed over the years include:

- Marshall compaction.
- Percentage refusal density compaction (PRD).
- Slab compaction.
- Gyration compaction.
- Static pressure compression compaction.

These compaction methods are detailed as follows:

6.1.1 Marshall compaction

The concept of the Marshall method of designing paving mixture was formulated by Bruce Marshall, a former Bituminous Engineer with the Mississippi State Highway Department. The U.S. Army Corps of Engineer, through extensive research and correlated studies, improved and added certain features to Marshall's test procedure, and ultimately developed mix design procedure (The Asphalt Institute, 1995). In Marshall mix design process, the specimens are prepared by placing bituminous materials in a mould and then compacting with a hammer. The mould has an inner diameter of about 100mm and the hammer weighs about 4.5kg and is designed to drop onto a specimen from a height of 457mm. Generally, 35, 50 and 75 blows on both sides of the specimens are employed in the Marshall specimen preparation process, corresponding to relatively light, normal and heavy compaction in the field.

'The main advantages of Marshall compaction is its worldwide use and the

extensive amount of practical long-term experience gathered. This explains why it is often used as a reference in comparing results. However, many researchers have expressed doubts regarding the suitability of the method to simulate field compaction adequately. The classical Marshall compaction hammer does only produce impact but does not impose any kneading motion, as it is done in the field by a Rolling-Wheel compactor. Other disadvantages are the geometrical restriction of the specimen in terms of maximum aggregate size and the restricted possibility to determine a densification curve. In addition, it is not possible to produce beams for mechanical testing, such as fatigue (Jonsson et al, 2002).'

6.1.2 Percentage refusal density (PRD) compaction

The PRD specimens were compacted manually with a hammer and a split mould, the mould being usually 100mm or 150mm diameter. The PRD test was designed to evaluate the maximum refusal density of a hot asphalt mixture and is often employed to evaluate the field compaction, for example field compaction is often controlled at 95% of PRD density. The detailed operation procedure is described in BS 598-104 (1989). Since the compaction is conducted manually, it is especially suitable for field conditions where facilities are limited, but the compaction process is tedious and noisy. It would have been impracticable to compact the number of specimens needed for this project with this compaction method.

6.1.3 Slab compaction

Slab compaction simulates field compaction better than other compaction methods because the load is applied not via a vibrating force or static load but through a roller, which is more like the action of a full scale roller in the field. The modern slab compactor as presented in Figure 6-1 has many advanced functions. Compaction effort can be controlled by roller passes, load exerted, the thickness of the slab or the final density of the slab. Unfortunately, such a modern slab compactor was not available for this research and a slab compactor with only limited function was therefore employed. The employed slab compactor had a slab size of 305 mm x 305 mm with a thickness of 50mm and rolled using a steel-clad roller-compactor with a load of 2.68 kg per linear mm across the width of the specimen.

The problem with slab compaction is that most test methods require cylinder specimens but slabs of cold mixed materials are too weak and friable to make a core.

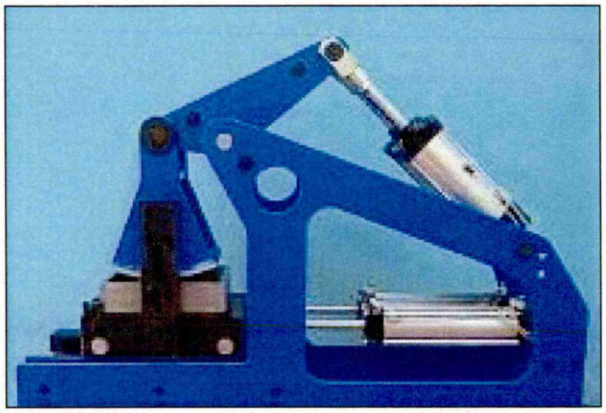


Figure 6-1: Slab compactor

6.1.4 Gyration compaction

'*The development of the gyratory concept is attributed to Philippi, Raines and Love of the Texas Highway Department. The first Texas gyratory "press" was a manual unit used on an experimental basis from 1939 to 1946 (Harman T et al, 2001)*'. After years of development and with a specification of SHRP[4], this compaction device is well established all over the world. Gyration compaction is considered to be the most effective method for taking moulded specimens, which are representative of the material laid and compacted in the road. The detailed compaction process is described in BS EN 12697-31 (2000). There are two types of mould for a gyration compactor, for hot mix and cold mix respectively. The mould for a cold mixture has a slot on its wall to facilitate water running off during the compaction process which in turn will reduce the positive pore water pressure built up in compaction, hence a fuller compaction is achievable. As shown in Figure 6-2, the specimen is compacted in a cylindrical mould which is clamped so that it is unable to rotate. The axis of the mould is inclined at an adjustable angle to the vertical (∅ in Figure 6-2). With such an angle existing, a vertical compressive stress is applied to the specimen and horizontal shear stresses generated in the mixture, which have a kneading effect. It is easier for aggregates to orientate themselves during compaction under such kneading compression than under transient vertical load or under static load.

The compaction can be controlled by the number of gyrations or the density of the specimens. Both 100mm and 150mm specimens can be fabricated by Gyration compaction. Compared with Marshall compaction, where only 100mm specimens are made, gyration compaction can accommodate larger aggregate size.

[4] SHRP: Strategic Highway Research Program

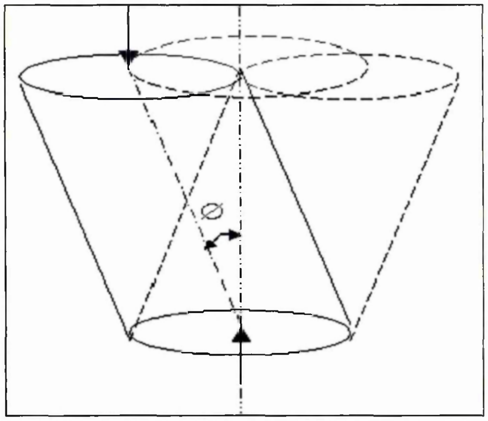


Figure 6-2: Sketch of gyration compression process

6.1.5 Static pressure compression compaction (SPC)

Static pressure compression compaction is widely used in fabricating cold mix specimens and is claimed to be more suitable for cold mixes. The cold mixes have relatively high moisture content and, in the static compressive compaction, the static load lasts for minutes, therefore the extra water contained has an opportunity to run off and hence a better compaction can be achieved. Whilst in Marshall or PRD compaction process, a transient load is exerted on the specimen and there is no time for the water to run off in such a short period, hence pore pressure builds up preventing the material from being compacted effectively.

Many researchers have employed SPC in their research work. Jostein (2001) stated '*in Norway, static [pressure compression] compaction or a gyratory compactor is used for compaction of specimens*' and he further claimed that *'the static compaction yields similar densities as compared to rolling in the field*'. He conducted the SPC as follows:

1. The load is increased gradually from 0 to 8 tonnes within two minutes.
2. The maximum load of 8 tonnes is maintained for two minutes.
3. The specimen is unloaded and de-moulded.

Needham (1996) employed SPC in his research on bitumen emulsion cold mixed materials by simply exerting a 20 tonne load onto cold mixes held in a Marshall mould. The most well known SPC is the 'Duriez method', in which samples with a diameter of 80mm or 120mm are compacted with static load (NF 98-251-4, 1992).

There are criticisms of SPC, *'The Duriez test, conducted using the current procedure for hot mixes, is neither able to predict in situ behaviour or differentiate between materials, in particular, application of heavy loads (120kN) for a long duration*

(5 minutes) makes it impossible to detect the differences between the consistency of different emulsions and residual binders. For different formulations the densities of the specimens moulded are very similar and frequently higher than those obtained on site (Serfass, 2002).'

6.2 EXPERIMENTAL PROGRAMME

6.2.1 Introduction

This part of the experiment was conducted from April to July 2002. Marshall compaction, Slab compaction and PRD compaction were conducted in the University laboratory and Gyration compaction was conducted in a laboratory of the cooperating companies. The dates and places of different compaction methods performed are detailed in Table 6-1.

Table 6-1: Dates and places of specimens compaction

Method	Date of compaction	Place of compaction
PRD compaction	2nd May 2002 and trimmed level at 4th September 2002	University laboratory
Marshall compaction	30th April 2002	University laboratory
Slab compaction	12th May 2002	University laboratory
Gyration compaction	12th May 2002	Collaborating company

The experiment was conducted in the following sequence:

- Specimen fabrication.
- Laboratory curing.
- Monitoring ITSM over time.
- Void and maximum theoretical density test.

Ideally, the specimens compacted in the laboratory should be comparable with the cores from the field, unfortunately, the field cores were unable to be obtained, so other research work where field cores were investigated had to be referred for comparison.

The materials employed in the compaction method evaluation were collected from the recycling plant of the co-operating company composed of recycled aggregate with 4.5% residual bitumen. Unfortunately, this batch of materials had a low workability, which was initially attributed to overdosed bitumen emulsion (it was claimed that over 5% bitumen had been added while the design bitumen content is 4.5%). However, an alternative explanation could be that the aggregate involved was the first batch of the recycled aggregate crushed in 2001, and that the aggregate was crushed and mixed without proper weathering. Freshly crushed aggregate, especially with high concrete demolition content, has high un-hydrated residual lime content which tends to absorb the emulsifier and destabilize the bitumen emulsion. Ideally, the void content should be controlled at less than 12%, which is required by BBA/HAPAS (2001) for cold mixed materials for use in footpath, footway and cycleway, but this batch of specimens developed higher void content, attributed to the low workability. Notwithstanding the problem, this project went ahead using this material, because of the time limitation and the fact that the aim at this stage of the project was to compare and assess the effects of compaction methods.

6.2.2 Marshall compaction

Marshall compaction was carried out following the procedures outlined in BS 598-107 (1990). For hot mixes, 30, 50 and 75 blows on both sides of the specimens are often applied, representing light, medium and heavy field compaction. As cold mixes have a lower workability than hot mixes and more compaction energy is required, hence only 50 and 75 blows were used in this project. It was found that around 1000 gram of cold mixed materials produced a specimen of approximately 60mm in thickness. The specimens were left in the mould for 24 hours before extrusion. Aggregate breakdown, mostly bricks as a results of the impact hammer, was observed. Hardly any water seeped out of the mould in the compaction process but the papers separating the cold mixes from the platen soaked up and had to be replaced while turning the specimen around to compact the other side. This might be because the materials had been stockpiled for some time before collection and the surplus water had already drained away.

6.2.3 PRD compaction

California bearing ratio (CBR) moulds, which were 150mm in diameter, were employed to conduct CBR test. Around 2.5kg of cold mixed material was put into a CBR mould, covered with a filter paper and then a collar was screwed on the mould. A Kango hammer was used to compact the specimen. The vibrating hammer was first conditioned by running on a blank specimen for 2min before proceeding with the specimen compaction. In the specimen compaction process, the foot of the hammer moved over the specimen in a sequence of N, S, W, E, NW, SE, SW, NE and at each position, the hammer stayed for 2 to 10 seconds until the total compaction period amounted to 2 minute ± 5 second. The compaction process proved noisy and tedious.

Eight PRD specimens were compacted. After compaction, the specimens were left to cure in the laboratory. The two surfaces of the PRD specimens were not parallel and they were later trimmed parallel in preparation for the ITSM test.

6.2.4 Slab compaction

In slab compaction, the load was exerted through a roller, which is similar to the process of field roller compaction. The slab compactor employed in this project was very primitive and could only exert a constant compaction force on the slab. The slab size adopted was 300mm x 300mm x 50mm. The compaction energy was controlled by the roller passes. In the first few rolls, the cold mixture in the slab moved with the movement of the roller, an uneven surface was created as a result and the compaction force was not evenly distributed. The slabs reached stable thickness after 20 rolls, nevertheless, 32 passes suggested by Jacobs (1997) were adopted with the aim of making sure that full compaction was achieved. After compaction, the slabs were cured in the laboratory and de-moulded a few months later when the specimens were fully cured.

Slab compaction was also tried with the modern slab compactor owned by one of the collaborating companies, which is similar to the slab compactor shown in Figure 6-1. The first few rolls were programmed to exert light force to stabilize the materials within the slab, and full compaction force built up gradually, producing an evenly compacted specimen. Unfortunately, this compacted slab broke in the de-moulding process, because it was too soft to de-mould soon after compaction. Considering the extra resources needed to pursue further slab compaction using the slab compactor of the co-operating company, no further effort was made.

The slabs compacted were very friable and eight cores were finally finished after numerous trials and after consulting the successful experience of other companies.

6.2.5 Gyration compaction

The gyration compaction was conducted with the help of the co-operating companies. There are two types of mould, for hot and cold mixes respectively. The mould for cold mixes has a slot on its wall to facilitate water seeping out. *'In Superpave Gyration compaction, a constant vertical pressure of 600kPa is applied to compress the mix, while the upper part of the mould simultaneously rotates around the vertical axis in a nominally constant angle of 1.25°C (Jonsson, 2002).'* In this project, consulting the experience of the co-operating companies, 600kPa vertical pressure was applied on the cold mixes in the mould, rotating at 1.5°C angle from the vertical axis at 30 rpm. As shown in Figure 6-3, the thickness of the specimen became very stable after 30 gyrations. In the event, 60 gyrations were employed to ensure that the specimens were properly compacted.

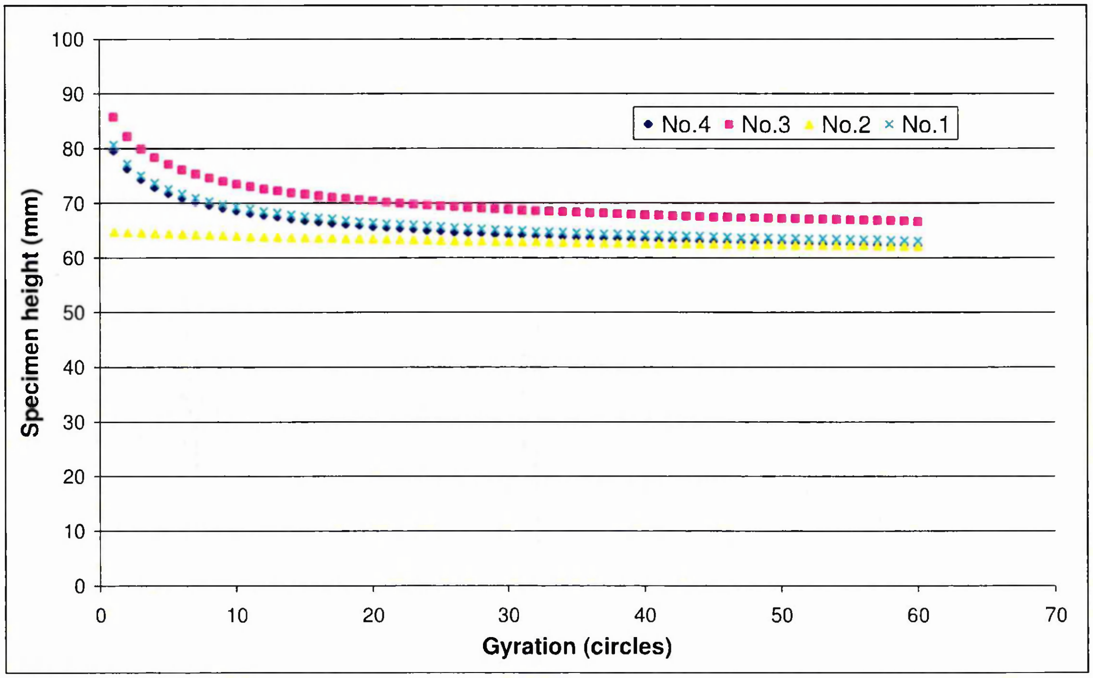


Figure 6-3: Gyratory compaction - specimen height over gyration number

The gyratory compactor can be set to stop at certain number of gyrations or at certain void content. In this project, the specimens were compacted according to the plan presented in Table 6-2:

- 3 specimens with Ø150mm and 6 specimens with Ø100mm were compacted with 60 gyrations
- 4 specimens with Ø100mm were compacted with target void content of 15% and 10% respectively, which is achieved after 9 and 32 gyrations.

Table 6-2: Gyration compaction arrangement

Number of specimen	Specimen label	Diameter (Ø) (mm)	Target gyrations	Target void content (%)
6	G1 to G6	100	60	
2	G7, G8	100		10
2	G9, G10	100		15
3	G11,G12,G13	150	60	

In the gyration compaction process, the height of the specimen is monitored so it is possible to control the target bulk density, but the cold mixes were found to bounce back after unload. As a result, the final density is less than the density indicated by the Gyratory compactor. In the compaction process, only negligible water seeped out of the slots on the wall of the mould. The specimens were extruded immediately after compaction and cured in the laboratory.

6.2.6 SPC compaction

Although the static pressure compression compaction is reputed to be very effective for cold mixes, it was not conducted in this project because of the author's experience with Ten Percent Fine Value test (BS 812-111:1990) conducted on recycled aggregates. In order to find out the aggregate compressive strength, TFV test was conducted on the mixture of road planings, bricks and concrete demolitions. In the test, under prolonged compression force, all the aggregates agglomerated together forming a hard block and it took some effort to extrude the material from the mould. The bitumen on the RAP, which warmed up in the compression process, bound all the aggregate together and caused the problem. This proved that recycled aggregate containing RAP is not suitable for lengthened compression. This agglomeration does not represent field compaction, where the transient rolling instead of prolonged static compression load is exerted.

6.3 DISCUSSION

6.3.1 Relationship between specimen density and void content

Maximum theoretical density test was conducted according to the Rice Method (BS DD 228:1996), which is presented in detail in Chapter 5. Two portions were tested on 8th August 2002 and the average maximum theoretical density was 2.378 tonne/m^3. The bulk density was measured following BS 598-104:1989. The void content was calculated using Equation 6-1 and summarized in Table 6-3, Figure 6-4 and Figure 6-5.

$$Void\ Content = (1 - Bulk\ Density / Maximum\ Density) \qquad \text{Equation 6-1}$$

The summarised result revealed that the gyration compactor produced specimens with the highest density and the lowest void content, the specimens also possessed consistent properties represented by its low standard error and coefficient of variance. It was also the most efficient compaction method. Only 9 or 10 gyrations produced specimens with higher density and lower void content than the Marshall specimens employing 50 or 75 blows. The specimens produced by Marshall compactor using 50 or 75 blows are very similar to the specimens from the PRD compaction in terms of density and void content; slab compaction is the least efficient compaction method in this test.

Table 6-3: Summarisation of the compaction methods

Compaction method		Number of specimen	Density (kg/m^3)			Void content (%)		
			Mean	Standard deviation	Standard error	Mean	Standard Deviation	Standard error
Gyratory	Ø100mm, 60 gyrations	6	2037	19	8	14.3	0.8	0.3
	Ø100mm, targeting at 10% void content	2	2005	35	25	15.7	1.7	1.2
	Ø100mm, targeting at 15% void content	2	1950	14	10	17.9	0.7	0.5
	Ø150mm, 60 gyrations	3	2027	12	7	14.8	0.4	0.3
Marshall	Marshall 50 blows	13	1922	22	6	19.2	0.9	0.3
	Marshall 75 blows	7	1933	21	8	18.7	0.9	0.3
PRD		7	1920	27	10	19.2	1.2	0.4
Slab		8	1874	40	14	21.2	1.8	0.6

Note: Ø diameter.

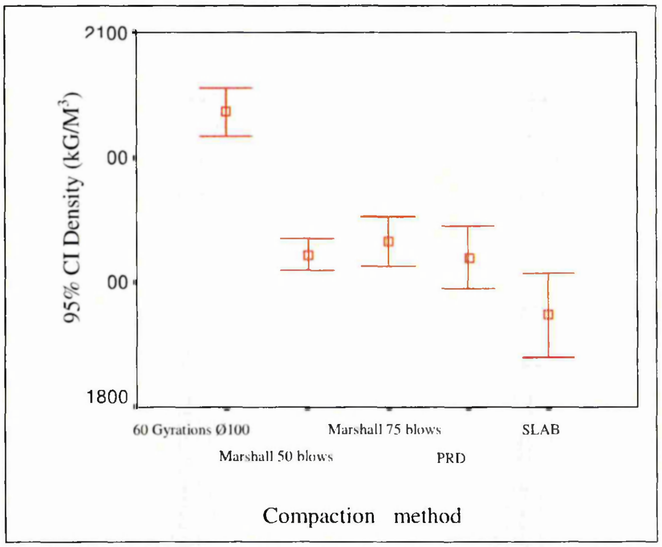


Figure 6-4: Specimen density vs. compaction methods[5]

6.3.2 Stiffness modulus comparison

In order to compare the effectiveness of different compaction methods, the ITSM test was conducted to monitor the stiffness development over time. The ITSM test is a non-destructive test, so the same group of specimens can be monitored over time. As indicated in the literature review, the properties of cold mixes change over time with the evaporation of water. By monitoring the stiffness development over time, the comparison between various compaction methods takes the whole curing process into

[5] 95% CI: 95% confidence interval

consideration. The test results monitored over time are presented in Figure 6-6. The ITSM results conducted on 18/12/2004 were selected and are summarised and presented in Table 6-4 and Figure 6-7. This is because by then, most of the specimens had been curing in the laboratory for half a year and the stiffness had reached a stable condition.

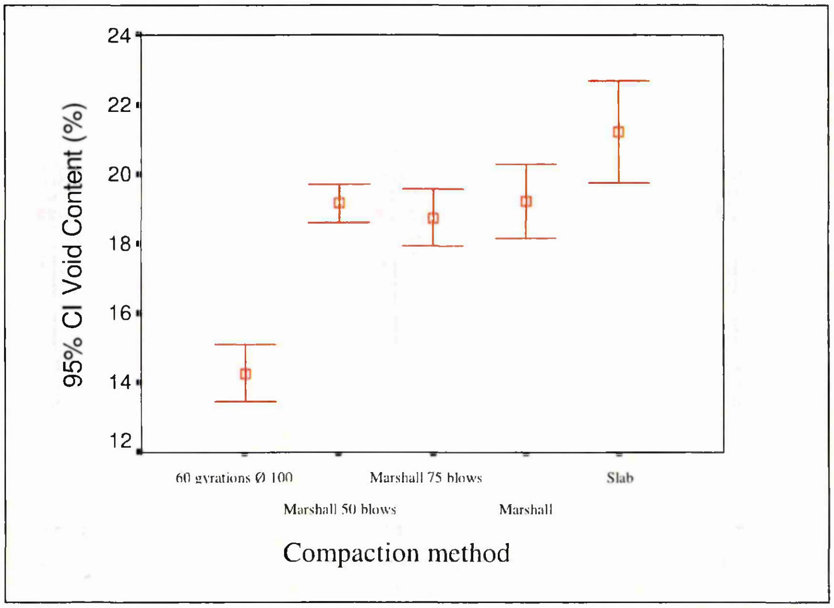


Figure 6-5: Void content vs. compaction methods

Table 6-4: ITSM results on 18/12/2002 for specimens from different compaction methods

Compaction method	Number of Specimens	Mean ITSM (MPa)	Standard deviation (MPa)	Standard Error (MPa)	Coefficient of variation (%)
Gyratory Ø 100mm	6	1589	123	50	7.8
Gyratory Ø 150mm	3	1527	83	48	5.4
Marshall Ø 100 @ 75 blows	8	1022	92	33	9.0
Marshall Ø 100 @ 50 blows	13	796	107	27	13.5
PRD Ø 150mm	8	1079	149	53	13.8
Slab Ø 100mm	8	558	58	21	10.4

In Figure 6-6, the specimens were gaining stiffness rapidly over the first two months after compaction and then levelled off, irrespective of the compaction method employed. This reflected the fact that the stiffness gain is closely related to the water evaporation and the break of bitumen emulsion. Figure 6-6 also revealed that, although the stiffness rose over time, the relative ranking position of different compaction methods were very consistent. The stiffness values of the specimens from various compaction methods were ranked in the following order from the start: Gyration compaction ϕ100mm 60 gyrations, Gyration compaction ϕ150mm 60 gyrations, PRD compaction ϕ150mm, Marshall compaction ϕ100mm at 75 blows, Marshall compaction ϕ100mm at 50 blows and Slab compaction.

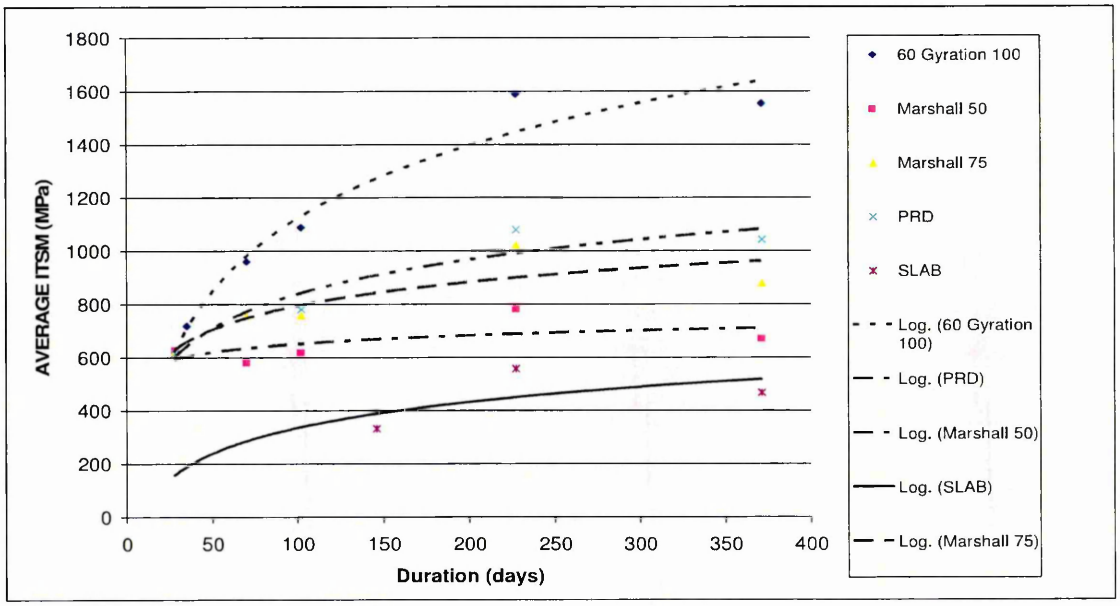


Figure 6-6: ITSM development over time

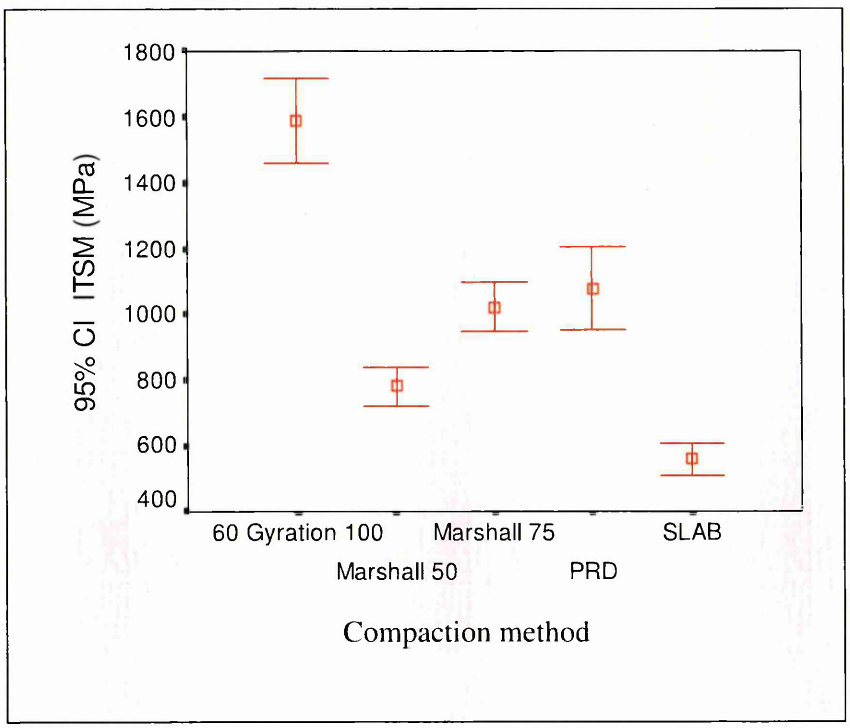


Figure 6-7: ITSM test result for specimen fabricated by different compaction methods on 18/12/2003

As shown in Table 6-4, the specimens produced by Gyratory compaction have the highest stiffness and the lowest coefficient of variation. This suggested that the

Gyratory compaction is an efficient compaction method and produces specimens with consistent performance. In the Gyratory compaction, ϕ100mm specimen had higher stiffness than that of ϕ150mm specimen when subjected to same gyrations. This reflected the fact that the bigger specimens need more energy to be compacted to the same density level as the smaller specimens.

6.3.3 ITSM vs. specimen density

Figure 6-8 and Figure 6-9 presented a good correlation between the stiffness and the specimen density, higher density is related to higher ITSM value. Kweir and Fordyce (2001) suggested that '*the dynamic stiffness modulus loses 50% of its value with a 7% drop in dry mass density.*' In this experiment, the ITSM loses 75% of its value with a 7% drop in dry mass density.

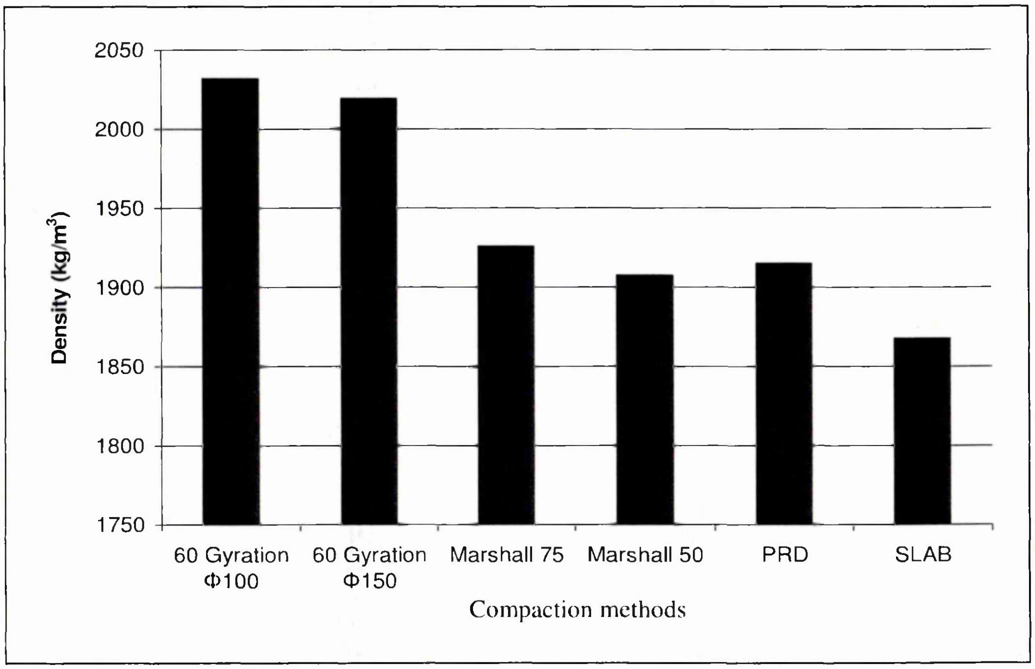


Figure 6-8: Compaction methods vs. specimen density

6.3.4 Comparing lab compaction with field compaction

Ideally, the properties of the laboratory compacted specimens should be related to field cores. Unfortunately, it was not possible to obtain field cores for this project. Kweir and Fordyce (2001) conducted research on different aspects influencing the performance of cold mixes including compaction methods. They compared field cores with laboratory compacted cores and found that site compaction is influenced by the foundation strength and the thickness of the layer before compaction. They also found that the density of a thick pavement layer may vary over the depth. They compared the density at top, middle and bottom layer of the pavement with Marshall specimens at various blows and concluded that the density of Marshall specimen with 50 - 60 blows is very similar to the density of the top layer of the pavement, and that the density with 30 - 40 blows is very similar to the density of the middle layer as shown in Figure 6-10.

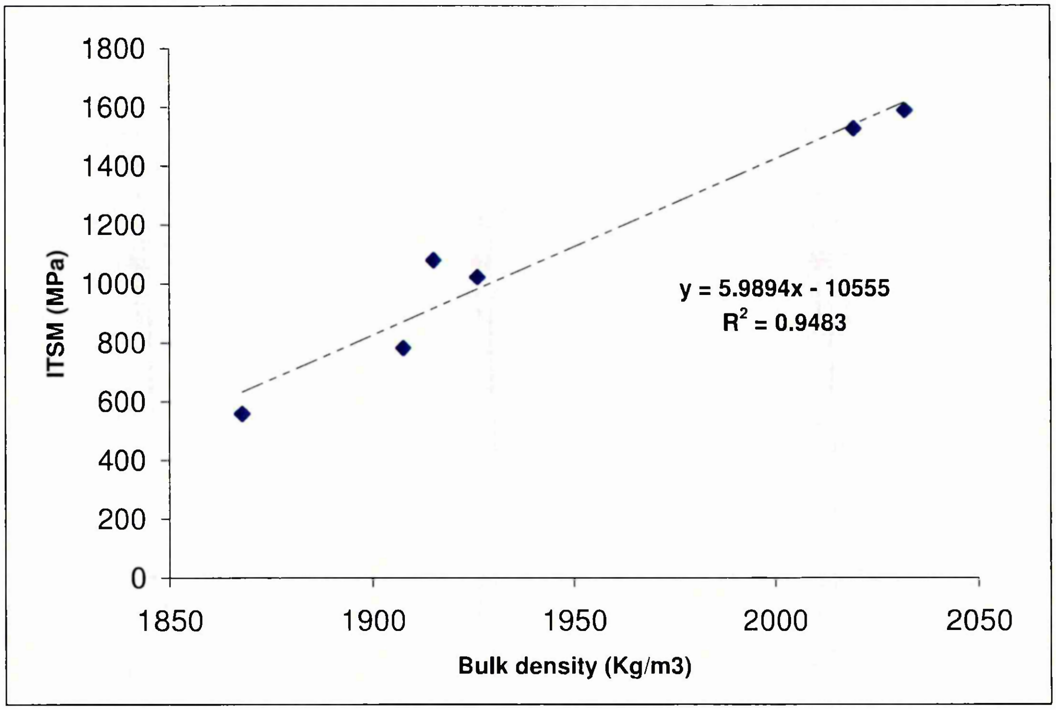


Figure 6-9: Relationship between ITSM and density

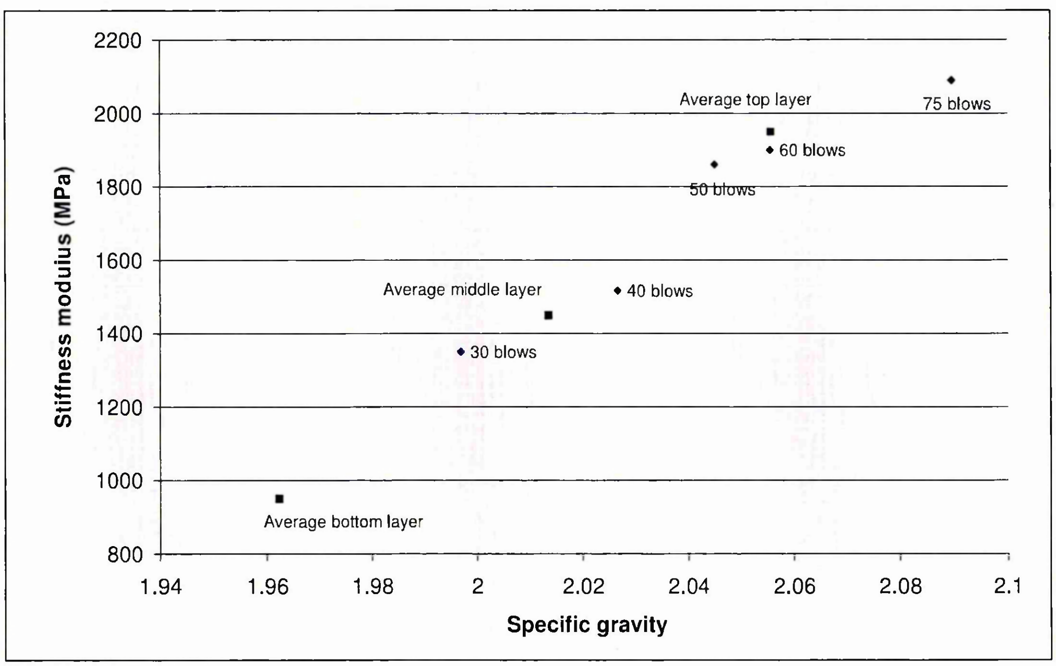


Figure 6-10: Specific gravity versus stiffness modulus after 10 days of curing (Kweir and Fordyce, 2001)

6.4 SUMMARY

Several compaction methods were reviewed in this chapter, namely Gyration compaction, Marshall compaction, Slab compaction and PRD compaction. All these compaction methods are design for hot mixes and have been proved to present no problem in preparing cold mixes specimens.

Gyration compaction proved to be the most effective compaction method. It produced specimens with the highest density and lowest void content, but this compaction method is not easily available for this project.

Marshall compaction produced specimens with the most consistent stiffness (least standard error) and the stiffness is similar to the cores of the pavement top layer as inferred from previous research (Khweir and Fordyce, 2001).

PRD compaction produced specimens with similar density and stiffness to specimens produced by Marshall compaction with 50 or 75 blows, but with higher test variance than Marshall compaction. The PRD compaction process is tedious and noisy, the two surfaces of the specimens produced were not parallel and had to be trimmed in order to conduct NAT test. Considering the number of specimens required for this project, this compaction method was not considered suitable for this project.

Static pressure compression compaction was not considered suitable for recycled aggregate mainly composed of road planings, as road planings may melt down in the compression process and bind all the aggregate together, which is not representative of the field conditions.

Marshall compaction was selected for this research based on following considerations:

- Marshall compaction is simple to operate, which is important considering the number of specimens required for this research.
- The specimens produced showed consistent performance with similar void content to the field cores.
- Marshall compaction has a long history and much research has been conducted using Marshall compaction. Therefore, the results from this research could be compared with the results from other research.

CHAPTER 7 BITUMEN EMULSION COLD MIXES

7.1 INTRODUCTION

The literature review has indicated that cold mixes are mostly suitable for warm and dry areas. On the contrary, the UK is famous for wet and damp weather. The aims of this chapter are to investigate the properties of the bitumen emulsion recycled aggregate mixtures exposed to simulated weather conditions and consider possible improvement methods.

Prolonged curing under high humidity condition and freezing and thawing are the two perceived major factors adversely influencing the performance of the cold mixes. In order to investigate the impact of freezing and thawing, ITSM test was conducted on specimens before and after freezing and thawing.

In order to investigate the moisture sensitivity of the cold mixes, ITSM test was undertaken on specimens cured under the following four conditions:

- High humidity.
- Low humidity.
- High humidity condition then transfer to low humidity condition.
- Low humidity condition then transfer to high humidity condition.

7.2 EXPERIMENTAL DESIGN

7.2.1 Shelf life and workability

Shelf life and workability are interrelated. Shelf life refers to the ability of the cold mixes to remain workable after mixing and this is a performance only related to cold mixes. Workability is a performance measuring both cold and hot mixes.

No test in this project was undertaken for the sole purpose of measuring the shelf life or the workability. Shelf life was assessed by the duration during which the cold mixes remained suitable for fabricating specimens after mixing; workability was measured by the void content of the specimens, specimens with low void content indicating that the related cold mixed materials has a good workability.

7.2.2 Moisture sensitivity

Relative humidity (RH) is often employed to indicate the relative exposure to moisture at various temperatures. Relative humidity is defined as the ratio of the water vapour pressure or water vapour content to the saturation vapour pressure or the maximum vapour content of air or gas at the relevant temperature. The saturation vapour pressure in the air varies with air temperature, the higher the temperature, the more water vapour it can hold. When saturated, the relative humidity in the air is 100% (VAISALA, 2005). In the Sheffield area, the relative humidity on average was 80.7% and 80.2% in 2002 and 2003 respectively, and the most humid month is November which averaged 90.8% and 90.3% in 2002 and 2003 respectively (Figure 7-1). The high humidity stops the moisture in the cold mix from evaporating, '*the complete loss of moisture within an emulsified bitumen mixture layer is unlikely, rather, some equilibrium value of contained moisture content will be reached which will be environment dependent (Robinson et al, 1996).'* Such high and variable humidity conditions as those experienced in the UK makes moisture sensitivity testing inevitable.

AASHTO DESIGNATION: T-283-99 outlines the moisture sensitivity test procedure for hot mixed materials. In which, the specimen is partially vacuum saturated in the test process before conducting Indirect Tensile Strength (ITS) test, but as the cold mix specimens are actually too soft to sustain the vacuum saturation process (Ibrahim, 1998), the moisture sensitivity test procedure designed for hot mixes is thus not applicable here. It is more feasible to investigate moisture sensitivity by monitoring the stiffness development of the specimens cured at various humidity conditions. The following three curing regimes were proceeded with the project:

- Laboratory curing: in this case, the specimens after extrusion were simply labelled and left to cure in the bituminous laboratory. The relative humidity and temperature of the laboratory are shown in Figure 7-2 and Figure 7-3 respectively. The relatively humidity in the laboratory was around 50% in the summer and 30% in the winter. Such a difference results from the ventilation system. In winter, in order to keep the laboratory above 20^{0}C, the air in the ventilation system has to be warmed up and moisture is driven off in this process. In summer, the warming process is switched off, hence the moisture in the lab is high.
- Mist room: the mist room was built for curing concrete specimens. According to BS 1881-111 (1983), the mist room should be kept within a temperature range of 20±2^{o}C and a relative humidity of over 90%, which is similar to or higher than the humidity in November and December in Sheffield.

- Covering with cling film: the specimens were covered with cling film after compaction, with the aim of stopping the moisture evaporating. Kweir (2001) claimed such curing condition is similar to the curing condition when a pavement layer is covered or sealed by other pavement layers.

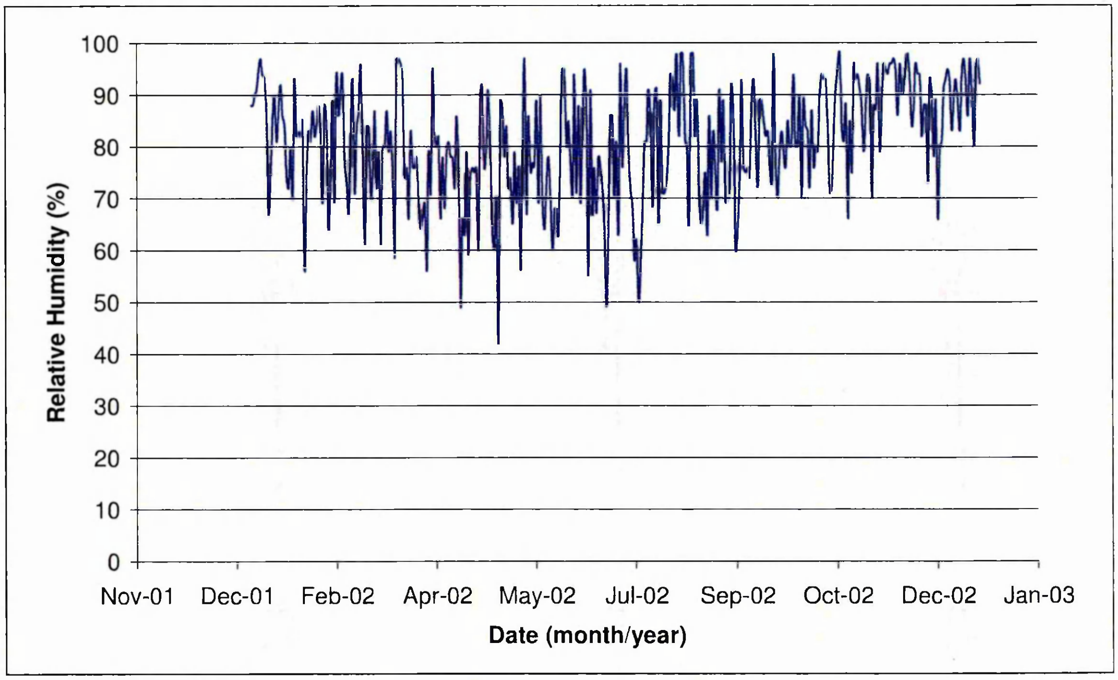


Figure 7-1: Seasonal relative humidity variation in Sheffield area

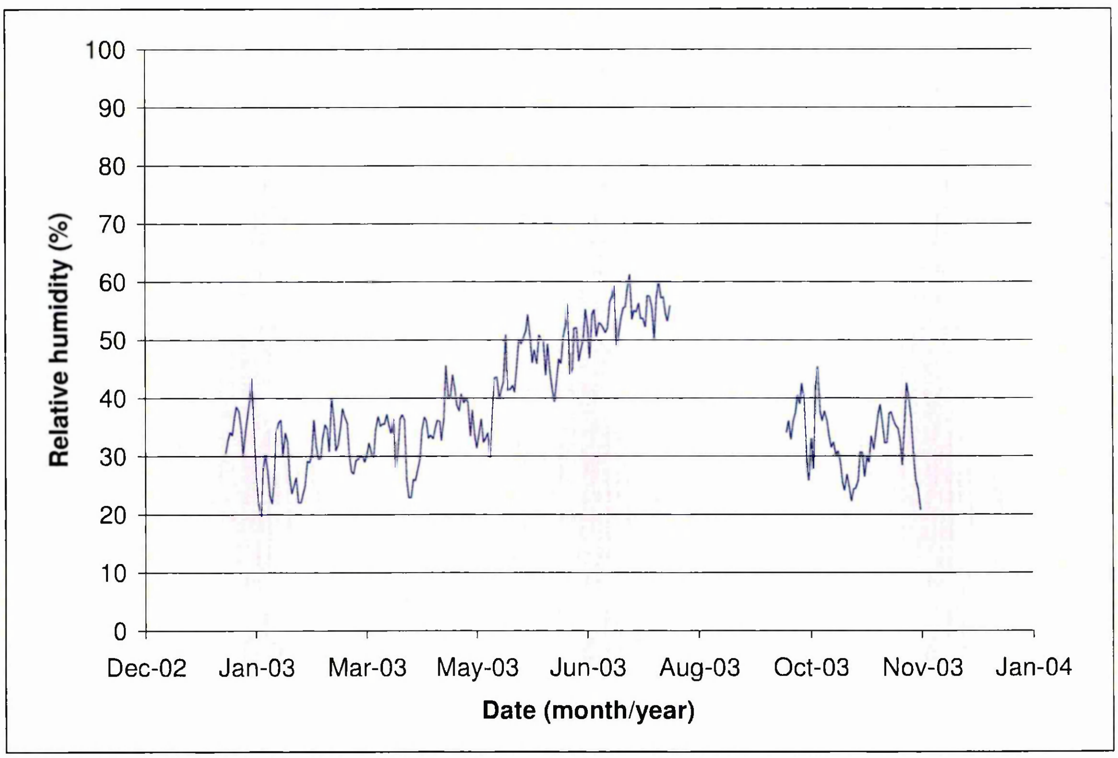


Figure 7-2: Relative humidity in the laboratory

In the experimental process, it was found that the curing method of covering with cling film had some practical problems. It was difficult to monitor the stiffness development because the slab was wrapped with cling film. Unwrapping the cling film to conduct the ITSM test could cause moisture evaporation and the representativeness of the result could be questionable. As a result, this curing method was dropped from the experimental process.

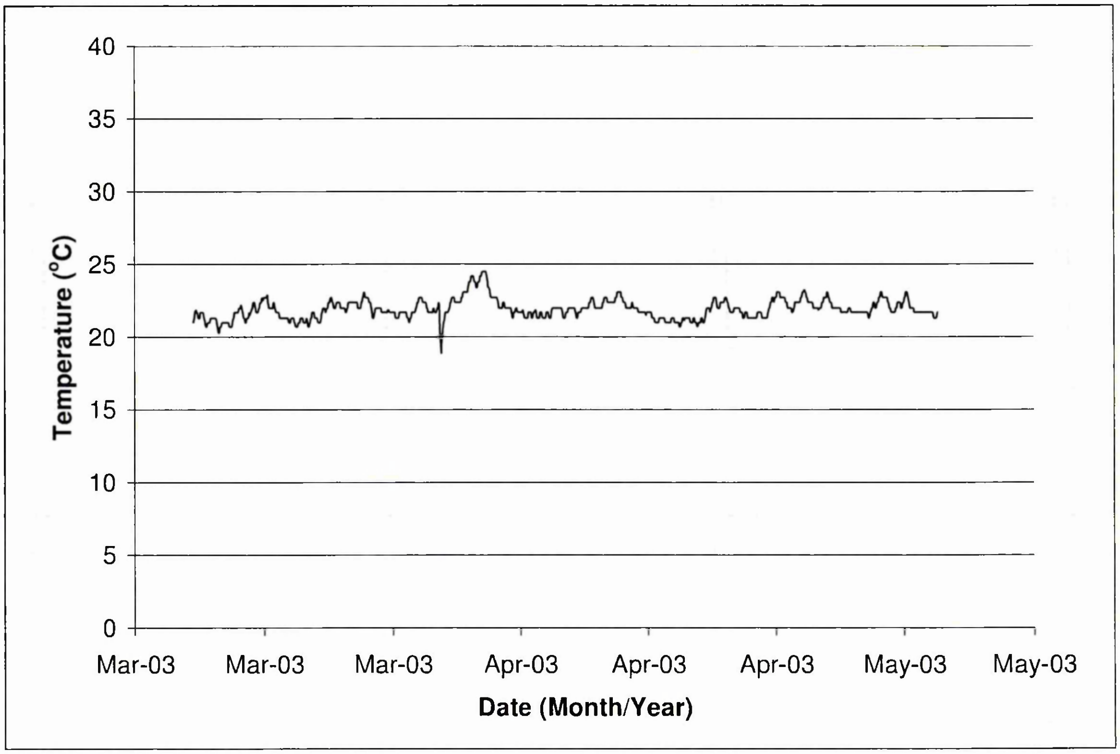


Figure 7-3: Temperature in the laboratory

7.2.3 Freeze-thaw (F-T) resistance

Material with high fine content potentially tends to absorb water. This water will expand in the freezing process, and such expansion could damage the road structure. In the UK, the materials within 450mm from the road surface are required to be not susceptible to frost (Sherwood and Roe, 1986). The cold mixes in this project were intended for base and subbase, which is within the 450mm limit, therefore F-T resistance is necessary. The following experimental work was undertaken to study the F-T resistance of cold mixes.

Two British Standard F-T tests were found relating to materials freeze resistance properties. One is masonry F-T test, where the test temperature is set from -15°C to 20°C (BS EN 772-18: 2000), the other is an aggregate F-T test, in which the lower end is set at -17°C (BS 812-124: 1989). Both -15°C and -17°C are very low compared with the prevalent winter weather. In 2002 and 2003, the lowest recorded temperature was

-6.6°C and -5.6°C respectively in the Sheffield area as shown in Figure 7-4 (Sheffield University, 2003). Taking cognizance that recent Sheffield temperature records are more representative of the temperature encountered in typical English weather, a more attenuated temperature range of -5°C to 5°C was employed. The F-T temperature regime was set at 24 hours per F-T cycle, at -5°C for 10 hours and at 5°C for 8 hours and the rest of the time in transition.

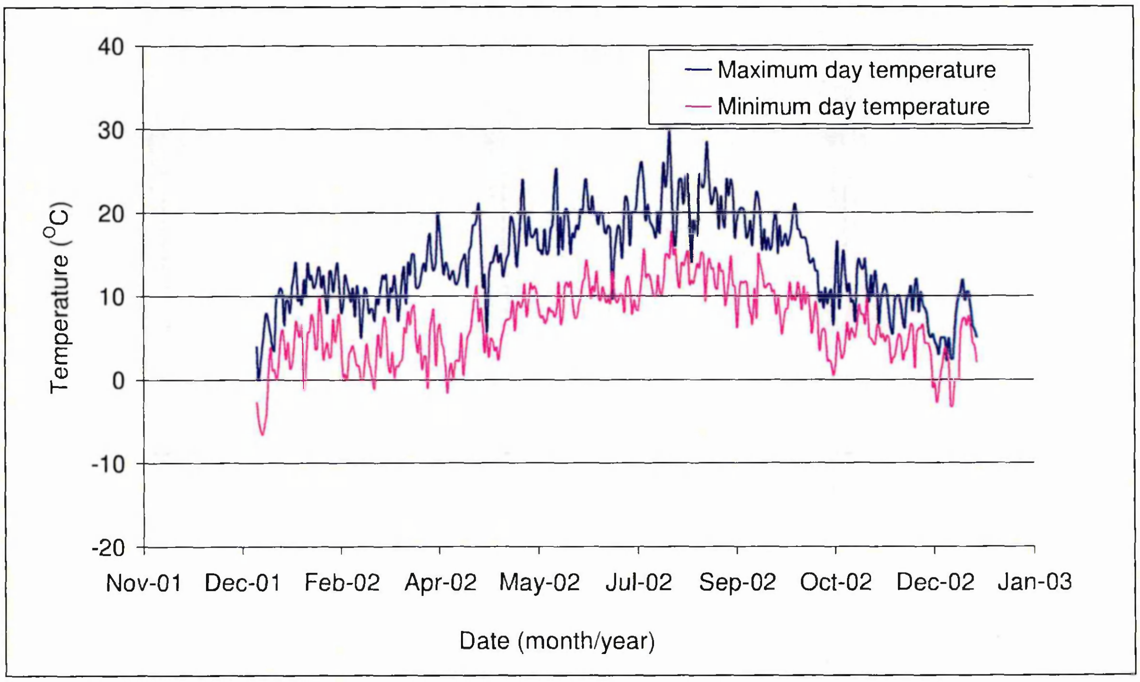


Figure 7-4: Seasonal temperature variation in Sheffield area

7.3 EXPERIMENTAL PROCESS

7.3.1 Moisture sensitivity

The cold mixes used in this part of the research were mixed in the laboratory with the following components:

- Aggregate

The recycled aggregates were composed of bricks, concrete demolitions and RAP. The proportions of the three types of the recycled aggregates were not apparently controlled by any criteria apart from availability and could not be ascertained precisely even by post analysis. In practice, the proportions depended on the incoming aggregates to the recycling plant. The aggregate were crushed into two sizes, coarse (10/20mm) and fine (0/10mm). The coarse and fine parts of the aggregate were mixed following the gradation of 20mm dense bitumen macadam (DBM) (BS 4987, 2003).

- Bitumen emulsion

The bitumen emulsion used in this project is a slow set cationic bitumen emulsion. 4.5% bitumen emulsion by weight of aggregate was added. The bitumen emulsion contains 40% water and 60% bitumen; therefore, a 4.5% bitumen emulsion by weight of the aggregate is equivalent to a 2.7% of residual bitumen by weight of aggregate.

- Water

In the mixing process, some extra water is normally added to surface wet the aggregate and therefore facilitate the coating of the aggregate by bitumen emulsion, but over- addition of water may induce bitumen emulsion drainage and could stop the cold mixes being compacted properly. The water content in the cold mixes comes from:

- The residual water within the aggregate: the aggregates in the field were mixed without drying off. To keep consistency, the aggregates were similarly mixed without drying off, so the water within the aggregates was retained.
- The added water: '*An emulsified asphalt's ability to coat an aggregate is usually sensitive to the pre-mix water content of the aggregate, this is especially true for aggregates containing a high percentage of material passing a 75 μm sieve, where insufficient pre-mixing water results in balling of the asphalt with the fines and insufficient coating. It is common practice to add above 3% water to surface wet the aggregate (Asphalt Institute, 1990).*' Various percentages of water were added in this project depending upon the aggregate employed.
- The water from bitumen emulsion: the bitumen emulsion used in this project contained around 40% water.

The optimum moisture content was found to be around 7.5% for this batch of aggregate. '*As a rule of thumb, it is recommended to keep the total moisture content after mixing within the following limits: OMC[6] to OMC - (0.5 × residual binder content) (Jostein and Roar, 2000).*' Phillips (1997) suggested that the water addition should be from 1.5 to 3.5%. In this project, extra water of 3 to 3.5% by mass of the aggregate was added, which was the least required content to surface wet the coarse aggregates. At this content, the water could properly surface wet the aggregates without causing bleeding problems.

The mixing was conducted by a 'Creteangle' multi-flow pan type mixer (Figure 7-5). In the mixing process, the materials were added into the pan following the sequence (Table 7-1):

1. Coarse aggregates were first added into the mixing pan.
2. Free water added to surface wet the coarse aggregates (around 3% by mass of the aggregate).
3. 2/3 of the bitumen emulsion.
4. Fine aggregate.
5. The remaining 1/3 bitumen emulsion.

Figure 7-5: 'Creteangle' multi-flow pan type mixes

Table 7-1: Composition of cold mixture

Composition	Mass (%)	Note
Aggregate	100	
Bitumen emulsion	4.5	2.7% residual bitumen[7]
Added water	3.0	

The whole mixing process was timed at 3 minutes. The recycled concrete and road planings were properly coated while the coarse bricks were only half coated owing to their smooth surface. The cold mixes were then bagged, sealed and stockpiled in the laboratory, monitoring the shelf life and manufacturing compacted specimens. Specimens were compacted with Marshall compactor at 50 blows on each side. The

[6] OMC: optimum moisture content

specimens were divided into four groups for curing as shown in Table 7-2. The relative humidity in the field lies in the middle of that used for curing Groups 1 and 2 and changes with the season, which is represented by Groups 3 and 4.

Table 7-2: The relative humidity of various curing condition

Cold material	Curing condition	Note
Group 1	Laboratory	RH lower than ambient
Group 2	Mist room	RH higher than ambient
Group 3	Mist room → laboratory	Simulating the changing temperature/humidity in ambient condition
Group 4	Laboratory → mist room	

Note: RH denotes relative humidity, Ambient refers to the field condition here.

As illustrated in Figure 5-3, the stiffness is a major performance indicator reflecting the capability of the pavement layer(s) to distribute the traffic load to the underlying layers. The ITSM test was conducted to monitor the stiffness development over time.

Bituminous materials are visco-elastic materials and tend to deform under repeated load, especially at high temperatures. An assessment of potential rut resistance therefore became necessary. As introduced in Chapter 5, two test methods are normally employed to assess rut resistance, the Repeated Load Axle test (RLAT) and Wheel Track test. An Unconfined RLAT was undertaken in this research and Wheel Track test results from other researchers were referenced.

7.3.2 F-T resistance

The materials used in the F-T test was another batch of mixed materials collected from the recycling plant, which was different from the batch of the cold mixes used in the investigation of the compaction method, described in Chapter 6. The material had a residual bitumen content of 4.5% and the recycled aggregates were composed of RAP, demolished concrete and bricks.

F-T test was conducted on the loose materials as well as on the compacted materials. The aim of conducting F-T test on loose materials was to investigate the F-T resistance of the cold mixes in the stockpile process and on compacted materials was to investigate the F-T resistance of the cold mixes after paving on the road.

- Compacted specimens F-T test

Another batch of specimens was compacted with a Marshall compactor and then divided into three groups and tested as follows:

[7] The bitumen emulsion employed in this project contains 60% 100Pen bitumen and 40% water.

- Group 1 was placed in a fridge set at 5°C. This group of specimens acted as the control group. 5°C is commonly employed as a storage temperature for bituminous materials and Group 1 was curing at this temperature.
- Group 2 was placed into the climatic chamber experiencing F-T conditioning for 13 cycles, and the temperature of the weather chamber was pre-set from -5°C to 5°C as detailed in Section 7.2.3.
- Group 3 was placed into the climatic chamber experiencing F-T conditioning for 16 cycles, and the temperature of the weather chamber was pre-set from -5°C to 5°C as detailed in Section 7.2.3.

After being removed from the weather chamber, all three groups of specimens were left in the laboratory and ITSM test was conducted after the specimens had stabilised at room temperature.

- Loose cold mixes F-T test

A batch of cold mixes collected from the recycling plant was put into three bags around 10kg each. The three bags were conditioned under the same F-T regime as stated in 7.2.3, which was from -5°C to 5°C:

- Bag 1 was conditioned for 3 cycles.
- Bag 2 was conditioned for 8 cycles.
- Bag 3 was conditioned for 16 cycles.

Specimens were compacted several days later, when the materials had reversed back to the laboratory temperature of around 22°C. ITSM test was conducted on these specimens at intervals to monitor the stiffness evolvement.

7.4 RESULTS

7.4.1 Moisture sensitivity

- Group 1

As stated in Table 7-2, Group 1 specimens were cured in the laboratory, with the temperature and relative humidity presented in Figure 7-2 and Figure 7-3 respectively. The ITSM test was conducted at intervals to monitor the stiffness development and the results are presented graphically in Figure 7-6. The stiffness developed rapidly in the first few weeks and then almost levelled off.

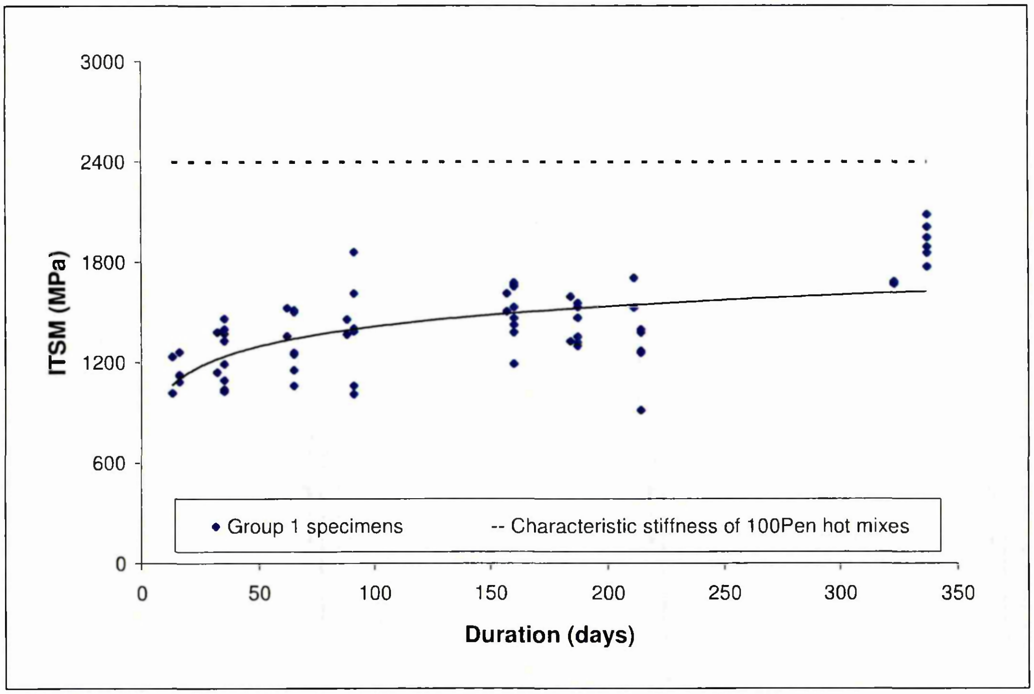


Figure 7-6: ITSM development over time

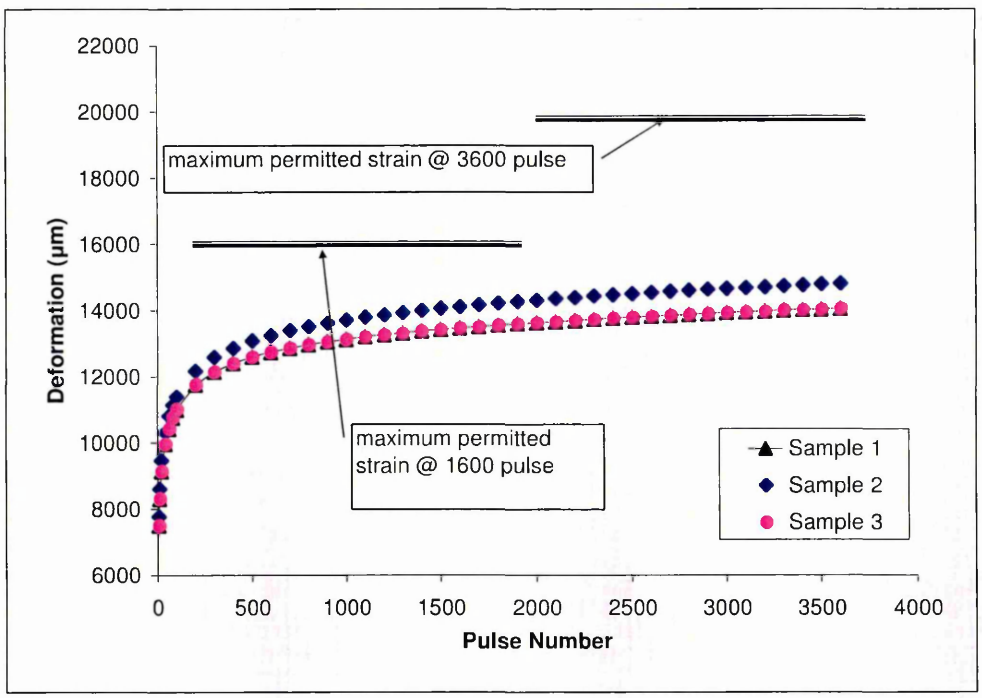


Figure 7-7: RLAT test for cold mixed materials

RLAT was conducted on three samples and the results were presented in Figure 7-7. The three specimens were cured in the laboratory for 4 months before conducting the RLAT test and they had reached a stable condition. The strain levels are less than 20,000µm after 3600 load pulses.

- Group 2

This group of specimens was cured in the mist room where the relative humidity was over 90% at a temperature of 20±2°C. Because of the high humidity, the moisture within the cold mixes could not evaporate. The bitumen emulsion particles could not coalesce or adhere onto aggregate and develop bonds properly. The ITSM test was attempted several times during the curing process and the specimens remained too weak to sustain the load, resulting in some specimens failing in the test process.

- Group 3

Group 3 is designed to investigate the influence of changing relative humidity (RH) on cold mixes. In this curing regime, eight specimens were compacted and divided into two sub-groups of 4 specimens as follows:

- ♦ Control Group: cured in the laboratory before ITSM test.
- ♦ Conditioned Group: cured in the mist room for 99 days and then transferred into the laboratory for 219 days before ITSM test.

The ITSM test was conducted on these specimens 318 days after compaction and results are summarized in Figure 7-8. The average ITSM of control specimens is 1863MPa and the average ITSM of the conditioned group is 1193MPa. The conditioned group had a significantly lower ITSM value than the control group with 95% confidence.

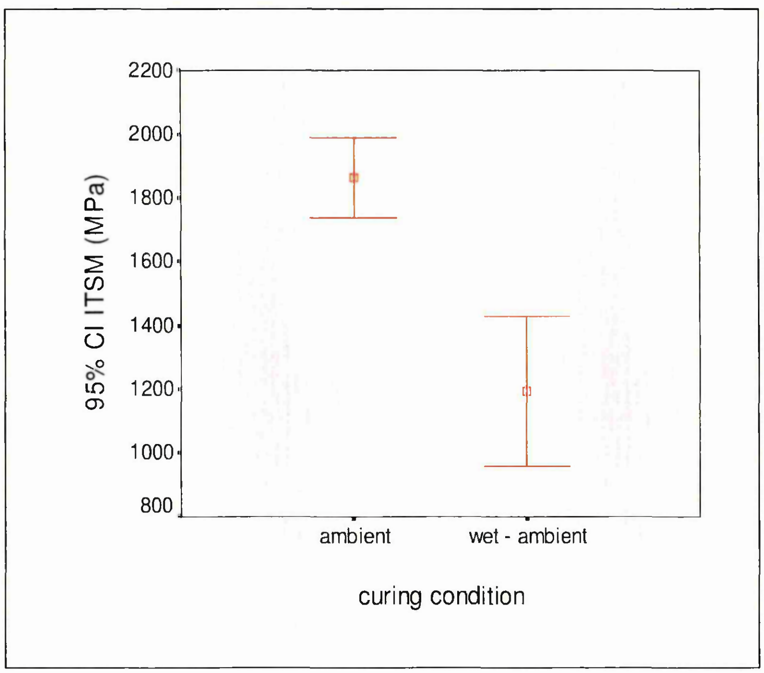


Figure 7-8: The effect of changing RH (high RH to Low RH)

The void content test is a destructive test, hence conducted at the end of the ITSM monitoring process. The results are presented in Table 7-3.

Table 7-3: Void content for specimens with and without conditioning

	Number of specimen	Mean void content (%)	Standard deviation (%)	Coefficient of Variation	Standard Error of Mean (%)
Control Group	4	11.48	0.10	0.01	0.05
Conditioned Group	4	11.68	0.67	0.06	0.34

- Group 4

Group 4 was also designed to measure the impact of changing relative humidity (RH). Contrary to the procedure for Group 3, this group of specimens was cured in the laboratory first before being moved into the mist room. The cold mixes were collected from the recycling plant with a residual bitumen content of 4.5%. The specimens were compacted on 25 September 2002 and were left to cure in the laboratory until 5 February 2003 for 130 days. The specimens were assumed to have reached a stable condition after curing in the laboratory for such a long time. In order to test the influence of changing RH, 4 specimens were put into the mist room. The ITSM test was conducted to monitor the stiffness development and the results are shown in Figure 7-9.

The ITSM was almost halved after the specimens were transferred into the mist room in the first 20 days and then levelled off.

7.4.2 F-T resistance

7.4.2.1 ITSM test

- Compacted specimen F-T test

The first ITSM test was conducted on 02/09/2002, which was 10 days after the F-T test, in order to allow specimens ample time to stabilise. The specimens were further tested on 20/09/2002 and 15/05/2003, which was 28 days and 263 days after freezing and thawing, to assess the possibility of self-healing of the damage caused by F-T test and results are presented in Figure 7-10.

Although Figure 7-10 shows ITSM rising in all specimen groups steadily over time, at 95% confidence level the specimens after F-T have a significantly lower stiffness than the specimens which did not experience F-T. There is a positive relationship between the stiffness loss and the F-T experienced.

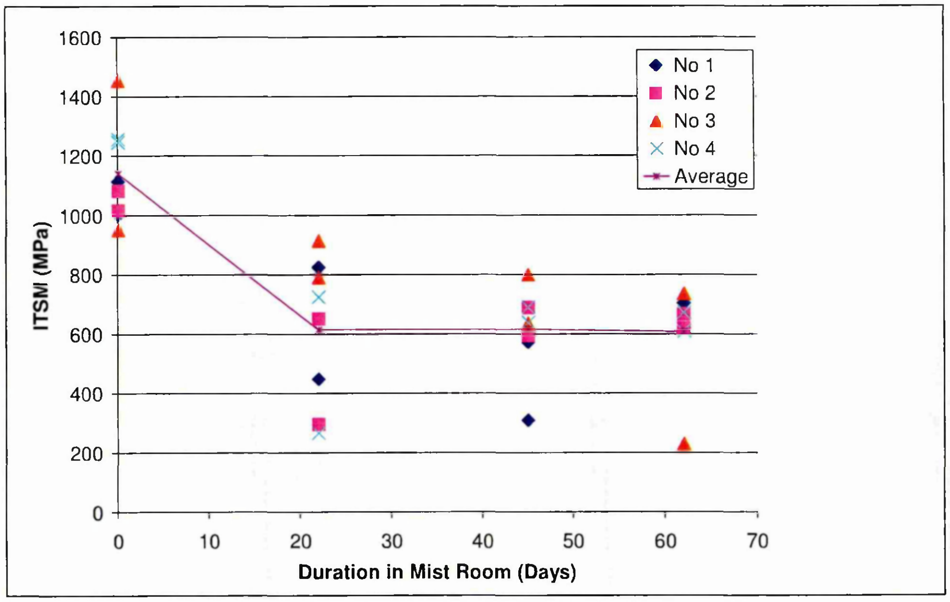
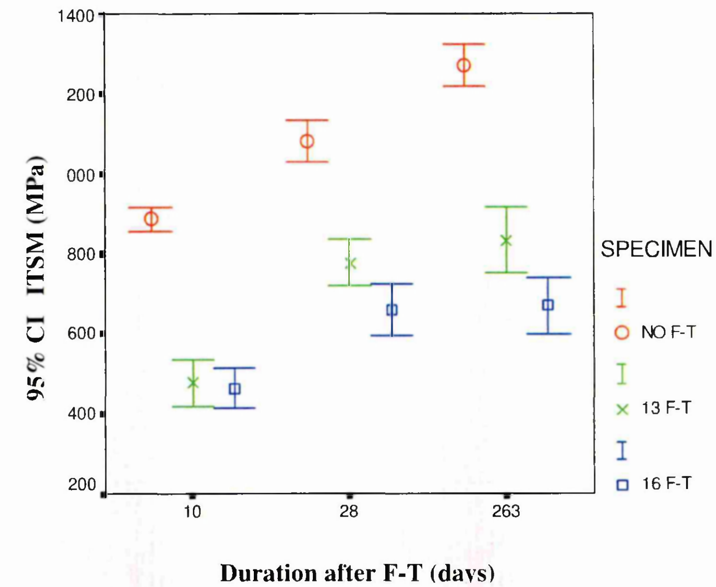


Figure 7-9: The influence of changing RH (Low RH to high RH)



Note: NO F-T: specimens without experiencing F-T
13 F-T: specimens experienced 13 F-T cycles
16 F-T: specimens experienced 16 F-T cycles

Figure 7-10: ITSM test result for specimens experienced F-T conditioning

- Loose cold mixed materials F-T test

The ITSM test was conducted at intervals and the results are shown in Figure 7-11. In Figure 7-11, NO F-T refers to the control specimens which did not experience

F-T; 3d F-T, 8d F-T and 16d F-T refer to specimens compacted from loose materials that experienced 3, 8 and 16 F-T cycles respectively.

Figure 7-11 shows that F-T has a damaging effect on the loose cold mixed materials. The longer the F-T process lasts, the lower the stiffness of the specimens compacted from such materials, although the 16-day F-T specimens behaved somewhat erratically.

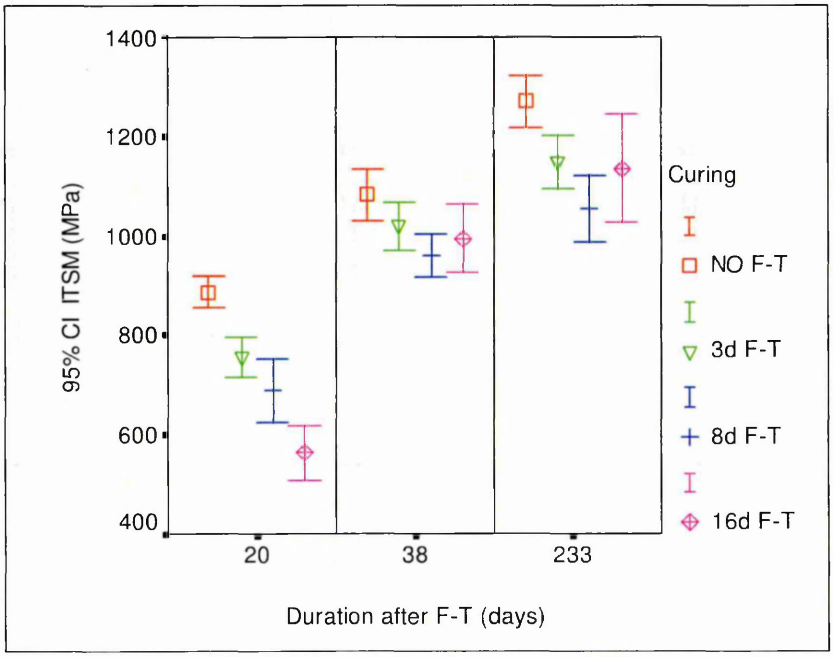


Figure 7-11: ITSM results for specimens compacted from cold mixes experienced F-T conditioning

7.4.2.2 Void content

After monitoring the stiffness development for a prolonged time, a void content test was undertaken in order to obtain maximum information from the specimens. The formula used to calculate void content is presented in Equation 5-4. The specimens were first warmed to 45°C for three consecutive days, aiming at driving off the moisture within the specimens. The maximum density was tested following BS DD 228 (1996), which is detailed in Chapter 5. Detailed results are summarized in Table 7-4 and graphically presented in Figure 7-12. The results show that the bulk densities are very similar, irrespective of the conditioning process. The decreased ITSM was not due to the changing void content but most probably owing to the fact that the bond between bitumen and aggregate was greatly weakened.

In Table 7-4 and Figure 7-12:

- No F-T refers to specimens which did not experience F-T conditioning. These specimens worked as control specimens.
- The 13 F-T and 16 F-T refer to specimens experienced 13 and 16 F-T conditioning cycles.
- The 3d F-T, 8d F-T and 16d F-T refer to specimens compacted from the loose cold mixed materials which had experienced 3, 8 and 16 freeze thaw conditioning cycles.

Table 7-4: Bulk density summary

	Specimen	Number of specimens	Mean bulk density (kg/m^3)	Standard Deviation (kg/m^3)	Standard Error of Mean (kg/m^3)	Void Content (%)
Specimens compacted before F-T	No F-T	6	1912.4	16.4	6.7	19.6
	13 F-T	5	1946.6	11.1	5.0	18.1
	16 F-T	5	1919.4	21.4	9.6	19.3
Specimens compacted after F-T	3d F-T	9	1944.8	12.1	4.0	18.2
	8d F-T	5	1905.1	40.3	18.0	19.9
	16d F-T	4	1911.6	11.3	5.6	19.6

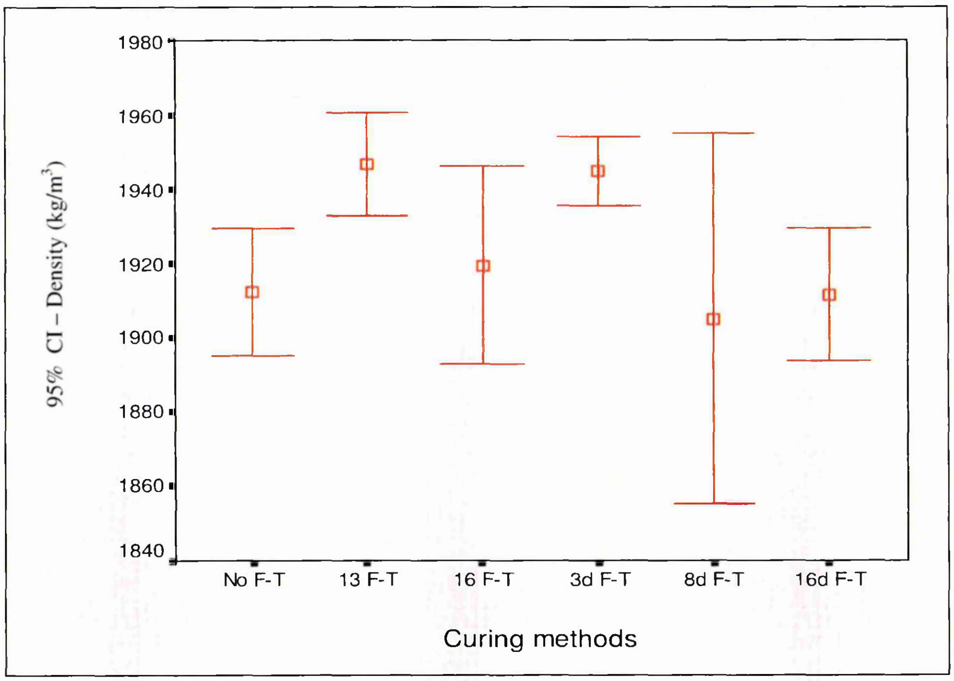


Figure 7-12: F-T specimen density comparison

7.5 DISCUSSION

7.5.1 Shelf life

The bitumen emulsion can be designed to break at various rates, fast or slow. For the slow set bitumen emulsion employed in this project, the shelf life was generally satisfactory as long as the residual bitumen content was less than 4.5%. In this test, even the loose materials, which experienced F-T conditioning, were still workable for quite a long time.

The evaporation of water, chemical reactions and continued aggregate absorption may reduce the shelf life.

7.5.2 Workability

The workability of the cold mixed materials is less than that of the hot mixed materials. As presented in Table 7-4, the void content of cold mixes has been over 10%, which is far higher than the hot mixed materials with similar aggregate gradation and bitumen content. This conclusion is consistent with findings revealed using Nynas Workability tester, presented in Figure 5-11, Figure 5-12 and Figure 5-13 (Jostein, 1999).

7.5.3 Moisture sensitivity

The experiment conducted has proved that the cold mixes are moisture sensitive. High stiffness developed at lower relative humidity conditions.

Specimens of Group 1 were cured in the laboratory, where the relative humidity is far lower than that of the prevalent ambient condition, therefore the developed stiffness is expected to be higher than the stiffness of such materials on the road. The average stiffness of the specimens cured in the laboratory is 1800MPa at 330 days after compaction as presented in Figure 7-6. This is higher than the nominal stiffness of the 200Pen 20mm hot mixed Dense Bitumen Macadam (DBM) but lower than the nominal stiffness of the 100Pen 20mm hot mixed DBM. The nominal stiffness figures of various materials are presented in Table 5-2. The specimens of Group 2 were cured in the mist room where the relative humidity was high and unable to generate enough stiffness to sustain the ITSM test. Group 3 and 4 also revealed a negative effect to the stiffness from high humidity curing condition.

The void content test was conducted on Group 3 specimens. The results showed that the specimens which experienced mist room curing had a similar void content to the control specimens, which had been curing in the laboratory. The reduced stiffness was not attributed to the void content.

Compared with the requirements related to deformation resistance, listed in Table 5-2, cold mixed materials could be considered acceptable, based on the RLAT (BS DD 226: 1998) test result presented in Figure 7-7. The RLAT test was not conducted on other groups of materials because of their low stiffness.

Widyatmoko (1998) investigated the properties of similar cold mixes and found cold mixes passed the RLAT test but failed in the Wheel Track test as reported in Table 7-5.

Table 7-5: Wheel Track results (Widyatmoko, 1998)

Mixture	Air Void BBA/HAPAS Between 2-12%	Rut rate at 45°C (mm/h) Clause 943 Maximum = 2	Rut depth at 45°C (mm) Clause 943 Maximum = 4
Cold Mixture	***17.8%***	***2.20***	***5.11***
100 pen Hotmix	6.85%	1.04	2.48

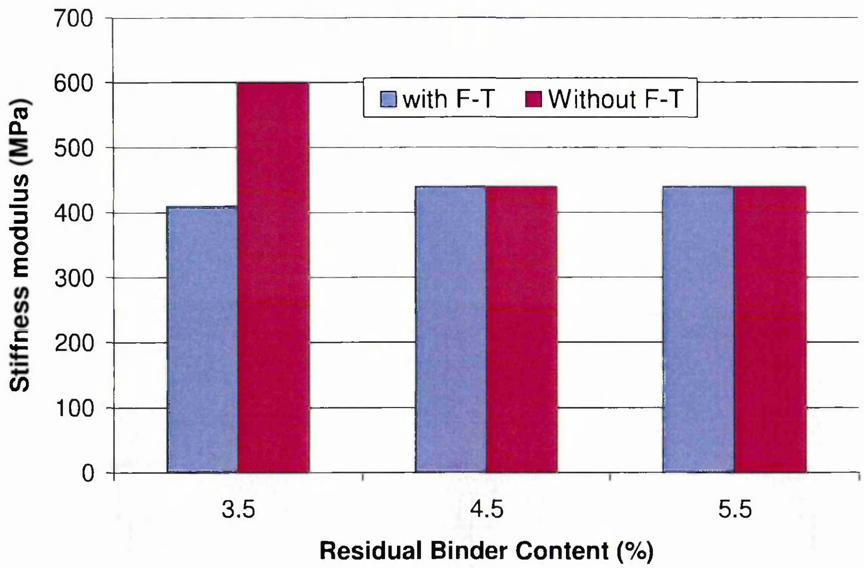
Note: bold italic figures indicate failure to comply with BBA/HAPAS (2001) advice and Manual of Contract for Highway Work Clause 943 (MCHW1, 2001).

Ibrahim (1998) conducted similar research investigating the performance of the cold mixed materials, looking into the stiffness modulus, the rut resistance and the moisture sensitivity of the cold mixes. His findings were similar to those of the author, i.e. the cold mixes have a lower stiffness than that of the hot mixes, the rut resistance is acceptable after full curing and the cold mixes are moisture sensitive.

Ibrahim also found that factors include the water content, curing time, bitumen content and aggregate gradation all influencing the deformation performance. He claimed the fully cured specimens were acceptable in terms of deformation resistance, which corresponds to the findings of the author, and the cold mixes exhibit less permanent deformation resistance compared to corresponding hot mixes. He also stated that the internal friction between the aggregate particles is the most significant factor contributing to rut resistance and increasing the bitumen binder content actually reduces rut resistance.

7.5.4 F-T resistance

Bitumen emulsion cold mixes were found not to be F-T resistant. Both loose materials and compacted specimens are sensitive to F-T conditioning. It is suggested by TRL Report 45 (Sherwood and Roe, 1986) that all the materials within 450mm of pavement surface should be freeze thaw resistant, therefore, cold mixes are not suitable as a paving material. This finding is in stark contrast to the result from Jostein and Roar (2000), who conducted research on cold mixes and claimed '*For binder contents of 4.5% and 5.5%, there is practically no difference between samples cured with and without freeze/thaw cycles. However, for a binder content of 3.5% there is a pronounced effect of freeze/thaw cycles.*' Their results are presented graphically in Figure 7-13.



Note: the stiffness values indicated in the above figure are not from ITSM test.

Figure 7-13: The effect of the F-T on the stiffness modulus[8] (Jostein M and Roar T, 2000)

The different conclusions could be attributable to the following reasons:

- In the work conducted by Jostein and Roar (2000), the specimens were first cured at 40°C for 7 days or at 60°C for 3 days before conducting F-T. Pre-curing under high temperature could drive off the moisture within the specimens, making the specimens less susceptible to F-T damage. In this project, the specimens were compacted and put through F-T without oven curing, which means high moisture content existed within the specimens. The moisture could expand in the freezing process and have a damaging effect on the specimens.
- Furthermore, in their work, the F-T experiment lasted for two days and eight F-T cycles, which was 5 hours freezing and 1 hour thawing in water, but no detailed freezing temperature was given in their paper. In this project, the F-T lasted much longer i.e. at intervals ranging from 3 to 16 day cycles.

In the UK, there will always be high moisture content within the cold mixes. The author believes that this research is more relevant to UK conditions considering the temperature and F-T regime employed.

7.6 SUMMARY

One of the objectives of this research project is to assess the suitability of the bitumen emulsion recycled aggregate for use as reinstatement materials. The above

[8] The stiffness modulus was not tested with NAT tester, hence it is not equivalent to ITSM.

investigation has revealed that the cold mixes were very moisture sensitive and could not generate enough stiffness to sustain ITSM test when cured under high moisture conditions. Furthermore, the developed stiffness could be lost after re-wetting and such loss is irrecoverable. The cold mixes are also prone to F-T damage, which prevent them from being paved within 450mm of the pavement surface. The findings here are in agreement with the findings of Ibrahim (1998) who claimed '*it is apparent that emulsion mixtures behave in a similar way to granular materials rather than hot asphaltic mixtures, even after significant curing*'.

Cold mixes made with quick set bitumen emulsion were claimed to be able to achieve high stiffness, but such mixtures have only a short shelf life while the long shelf life is the selling point of cold mixes. In order to maintain the shelf life while attaining higher stiffness and overcoming the moisture and F-T sensitivity, the addition of hydraulic binder is considered in the next stage. In practice, hydraulic binders such as cement, lime and PFA are frequently added to cold mixtures.

CHAPTER 8 GGBS BITUMEN EMULSION COLD MIXES

8.1 INTRODUCTION

It has been established that recycled aggregate with bitumen emulsion as binder was unable to deliver the required stiffness and it was not possible to achieve this objective by changing the bitumen emulsion, mainly because of the shelf life requirements. The addition of hydraulic binder was clearly an alternative. The objective of this part of the research was to investigate the effectiveness of the addition of hydraulic binder with the presence of bitumen emulsion and, if the addition proved effective, to establish the optimum binder contents.

'It is now fairly standard practice to add small amounts of hydraulic or pozzolanic binder to improve the performance of this material. These binders contribute to performance by acting as a binder in their own right, by improving bitumen/aggregate adhesion and by hydrating with and soaking up surplus moisture (Nunn and Thom, 2002).' Hydraulic and non-hydraulic binders include Portland cement, ground granulated blastfurnace slag (GGBS), pulverised fuel ash (PFA) and lime, of which Portland cement was thought to be the most widely added hydraulic binder. Successful cases of PFA, a pozzolana activated by lime or cement, have also been reported together with GGBS in alleviating the swelling sometimes encountered when sulphate containing clays are stabilised with lime (Tasong et al, 1999; Kennedy J, 1996).

The typical chemical composition for Portland cement, GGBS and PFA is shown in Table 8-1 (Jackon and Dhir, 1996), particularly with respect to the proportions of CaO present. In the UK, PFA is produced at power stations by the combustion of bituminous coal, which consists of carbonaceous matter and a mixture of various minerals. It is extracted by electrostatic and mechanical means from the resulting flue gases. PFA contains only limited amounts of CaO and, when it is mixed with water, no hydration will occur unless lime or cement is added. The pozzolanic reaction of PFA is a reaction of alkali-soluble silica and alumina from its glassy phase with a solution of calcium hydroxide to form calcium silicate and calcium aluminate hydrates possessing cementitious properties. The calcium hydroxide is provided by lime or cement. The pozzolanicity of PFA depends on several factors, including the fineness and composition of PFA. Because of the reliance on cement or lime for the pozzolanic reaction and the effect of lime and cement on shelf life, PFA was not considered in this project.

Table 8-1: Typical chemical composition of cementitious materials (Jackon and Dhir, 1996)

	Cementitious Materials (% by weight)		
	Portland cement	GGBS	PFA
SiO_2	20	37	48
Al_2O_3	5	11	26
Fe_2O_3	6.5	40	3
CaO	***65***	***40***	***3***
MgO	1.1	7	2
SO_3	2.4	0.3	0.7
S^{2-}	-	1.0	-
Na_2O	0.2	0.4	1.0
K_2O	0.9	0.7	3.0
Other oxides	1.4	2.3	1.3
LOI	1	-	5

Note: LOI represents loss on ignition.

GGBS is a by-product of iron manufacture and is widely available in the UK. It is most widely used as a partial replacement of Portland cement in the concrete industry; the resulting concrete has been found to have a lower initial strength but higher long term strength (Concrete Society, 1991). Unlike PFA, which has to be activated by cement or lime, GGBS is intrinsically hydraulic. It reacts with water, increasing pH, generating heat and developing a particle to particle cementitious bond (Concrete Society, 1991). When the molten slag from a blast furnace is subjected either to fine water jets, or otherwise rapidly cooled, it quickly transfers into a glassy granulate or pellet form with a consistent particle size range, chemical composition and degree of vitrification. With further grinding, a fine powder is produced, which is termed GGBS. Hooton and Emery (1983) observed that the properties of GGBS influencing its reactivity were the glass content, chemical composition, mineralogical composition, fineness of grinding and type of activation provided. Finely ground GGBS possesses a higher surface area and higher reactivity; the hydraulicity was also found to increase with rising CaO, MgO or Al_2O_3 and decrease with rising SiO_2, thus, in BS 6699 (1992), the $(CaO+MgO)/SiO_2$ has to be over 1.0 for GGBS in concrete. At ambient temperature, the hydration of GGBS occurs more slowly than that of Portland cement, but, in the presence of sufficient moisture, may continue over a long period, thus higher stiffness can be achieved at the same concentration as Portland cement. The ideal cold mixes should have a long stockpile life and be able to develop high stiffness once paved, which is beneficial for earlier opening to traffic. To increase the hydration rate, accelerators such as cement, lime, water glass, NaOH etc are commonly used to increase the hydration rate. The effect of lime or NaOH solution as an accelerator is further discussed in Chapter 9.

Considering the performance requirements, the availability and the cost, GGBS is a suitable choice for addition to bitumen emulsion. However, no previous research on this could be found. The GGBS in this project was supplied by Appleby Frodingham Ltd and the slag conformed to BS6699: 1992. The chemical and physical properties of the slag are listed in Table 8-2 and Table 8-3.

Table 8-2. Chemical composition of GGBS

Major oxide (% weight)						
CaO	Fe_2O_3	Al_2O_3	SiO_2	MgO	SO_3	LOI
39.51	0.66	12.60	35.70	8.41	0.83	0.49

Note: LOI refers to loss on ignition.

Table 8-3. Physical properties of GGBS

Density	Specific Surface Area (SSA)	Compressive strength (50% GGBS, 50% Portland cement)			Glass Content
		3 days	7 days	28 days	
(kg/m^3)	(m^2/kg)	(N/mm^2)	(N/mm^2)	(N/mm^2)	(%)
2930	436	16.9	25.0	50.9	98

The research was conducted in the following sequence:

- Pilot scale trial: investigates and analyses the effectiveness of GGBS at a low content with the presence of bitumen emulsion.
- In-depth investigation analysis: investigates the optimum binder content.

8.2 PILOT SCALE TRIAL

The aim of the pilot scale trial was to determine the effectiveness of the addition of GGBS at a low concentration in the presence of bitumen emulsion.

8.2.1 Experimental programme

- Curing condition

The strength and stiffness development mechanism of bitumen emulsion cold mixes is complex and the addition of GGBS makes it even more complicated. The GGBS hydration is a slow process and highly dependent upon temperature and moisture. According to Finn et al (1968), Santucci (1977), and Marais and Tait (1989), the curing period for cold mixes may be as much as six months in dry climatic regions and two years in wet climatic regions. In order to accelerate the curing process, different curing regimes have been proposed in the cold mixes design process. Milton and Earland (1999) suggested 72 hours oven dry at 60°C, Nunn and Thom (2002) suggested 7 days oven dry at 45°C. Merrill et al (2004) suggested 40°C over 28 days for slow hydraulic binder and multiplied by a factor (Table 8-4) to predict the long-term performance and they further suggested that this stiffness could be utilised, in the analytical pavement design process.

Table 8-4: Laboratory conditioning regimes and factors to relate to long-term performance (Merrill, 2004)

Family	Temperature (°C)	Duration (days)	Long-term factor
Quick hydraulic	20	28	1.2
Slow hydraulic	40	28	1.0
Quick visco-elastic	20	28	1.2
Quick visco-elastic*	40	28	1.0
Slow visco-elastic	-	-	-
Slow visco-elastic*	40	28	1.0

*: For material containing a pozzolanic binder

However, a curing regime with elevated temperature was not adopted in the pilot scale trial with an aim of simulating field conditions as far as possible. The specimens were divided into two groups, cured in the laboratory and in the mist room respectively. The detailed temperature and humidity of the laboratory is presented in Figure 7-2 and Figure 7-3 respectively. The temperature of the mist room was controlled at 20±2°C and the relative humidity of the mist room was controlled at over 90%. Data introduced in the previous chapter shows the relative humidity in the Sheffield area was over 80% and 90% on a yearly average in November and December respectively, while the highest temperatures were 29.3°C and 33°C in 2002 and 2003 respectively (Sheffield University, 2004). In terms of temperature and humidity, the chosen laboratory curing condition represents fairly typical field conditions over a large proportion of the year.

- Bitumen content

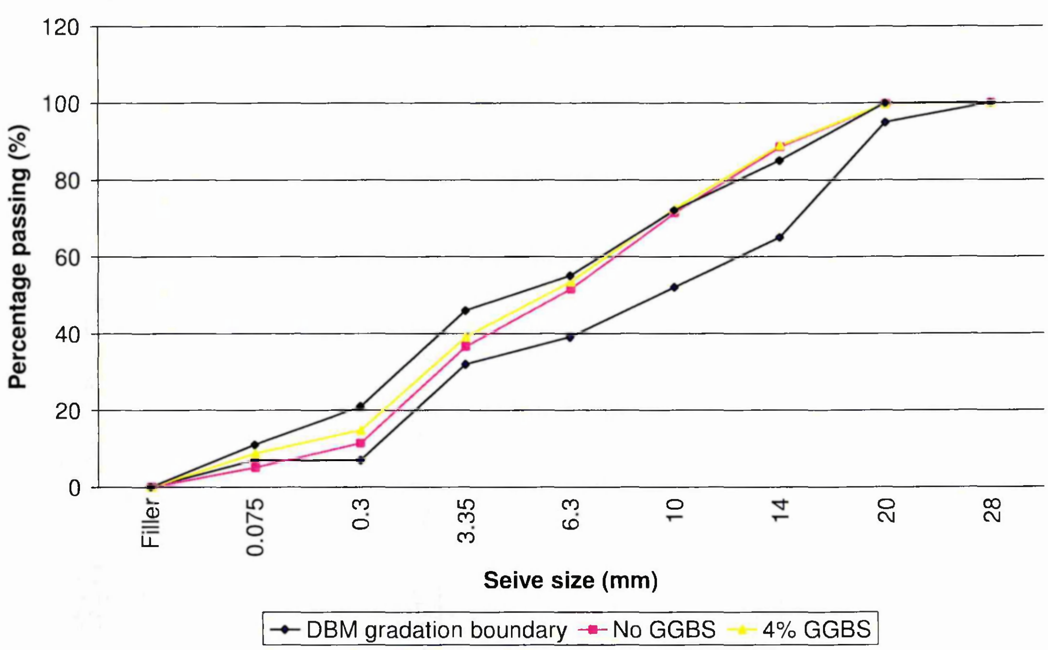
The residual bitumen content suggested by the bitumen emulsion supplier and employed in the recycling plant was 4.5%, taking into account the shelf life and field performance. However, only 2.7% was employed in the pilot trial. It was deemed easier to identify and assess the role of GGBS hydration in the presence of relatively low bitumen content.

- Aggregate gradation

Coarse (10/20mm) and fine (0/10mm) aggregates were collected from the recycling aggregate depot. The two portions were mixed with a gradation as shown in Figure 8-1, following a typical gradation of dense bitumen macadam (DBM) with nominal 20mm aggregate.

- GGBS content

The initial GGBS content was based on the experience of Portland cement as binder in cement bound material or stabilisation. In the case of cement, 2% was suggested for stabilisation in the UK (Nunn and Thom, 2002) and in the US (McKeen, 1999). The low concentration is thought to reduce adverse workability and thermal cracking problems. Since GGBS hydrates at a slow rate, both workability and thermal crack problems associated with cement as binder can be avoided and higher GGBS content can be accommodated. Based on the above consideration, 4% GGBS was added. The aggregate gradation with the addition of GGBS is also presented in Figure 8-1.



Note: Filler refers to the pat of aggregate passing through 0.075mm sieve.

Figure 8-1: Gradation with and without GGBS comparing with nominal 20mm DBM

- Water content

In the mixing process, extra water has to be added in order to surface wet the aggregate and facilitate the even distribution of bitumen emulsion in the cold mixes. 3.5% free water by weight of aggregate was added in the mixing process and no bleeding problem was experienced.

Two batches of cold mixes were made with the "Creteangle" Multi-flow Pan Type Mixer, which is presented in Figure 7-5, one batch with GGBS and the other batch without GGBS. The mixing was conducted in the following sequence:

1. Coarse aggregates were added into the mixing pan.
2. GGBS was added into the pan (if added).
3. Start rotation of the pan.
4. Free water was added to surface wet the coarse aggregates.
5. 2/3 of the bitumen emulsion was added.
6. Fine aggregate was added.
7. The remaining 1/3 of bitumen emulsion was added.
8. Stop rotation of the pan.

The whole mixing process took no more than 3 minutes until the aggregates were properly coated. After mixing, the cold mixed materials were sealed in a polyethylene bag. Two batches of specimens, with and without GGBS respectively, were compacted from the mixed cold mixes, with Marshall compactor at 50 blows on each side. Each batch of specimens was then further split into two groups, one group cured in the mist room and the other group cured in the laboratory. The normal operating condition of the mist room was controlled at a temperature ranging from 18 to 22^{o}C with relative humidity (RH) over 90%. The temperature of the laboratory varied from 20 to 25^{o}C and relative humidity ranged from 20 to 60%, as presented in Figure 7-2 and Figure 7-3 respectively.

ITSM test was conducted at intervals to monitor the stiffness development of the specimens cured in laboratory and in the mist room.

8.2.2 Results

- Shelf life and workability

Both groups of specimens were compacted two months after initial mixing without any workability problem. Furthermore, the addition of 4% GGBS did not apparently shorten the shelf life.

- Laboratory curing

Specimens with and without GGBS were cured in the laboratory side by side. The

ITSM test was conducted at intervals and the results are presented in Table 8-5 and Figure 8-2.

Table 8-5: The ITSM of specimens with and without GGBS cured in laboratory

Duration (days)	No GGBS			With GGBS		
	Number of specimens	Mean ITSM (MPa)	Standard Deviation (MPa)	Number of specimens	Mean ITSM (MPa)	Standard Deviation (MPa)
16	6	1141	92	6	1128	211
35	10	1244	163	8	1257	357
65	8	1327	176	8	1327	188
91	8	1395	275	8	1285	297
160	10	1511	152	8	1328	94
187	8	1431	119	8	1282	315
214	8	1339	231	8	1253	300

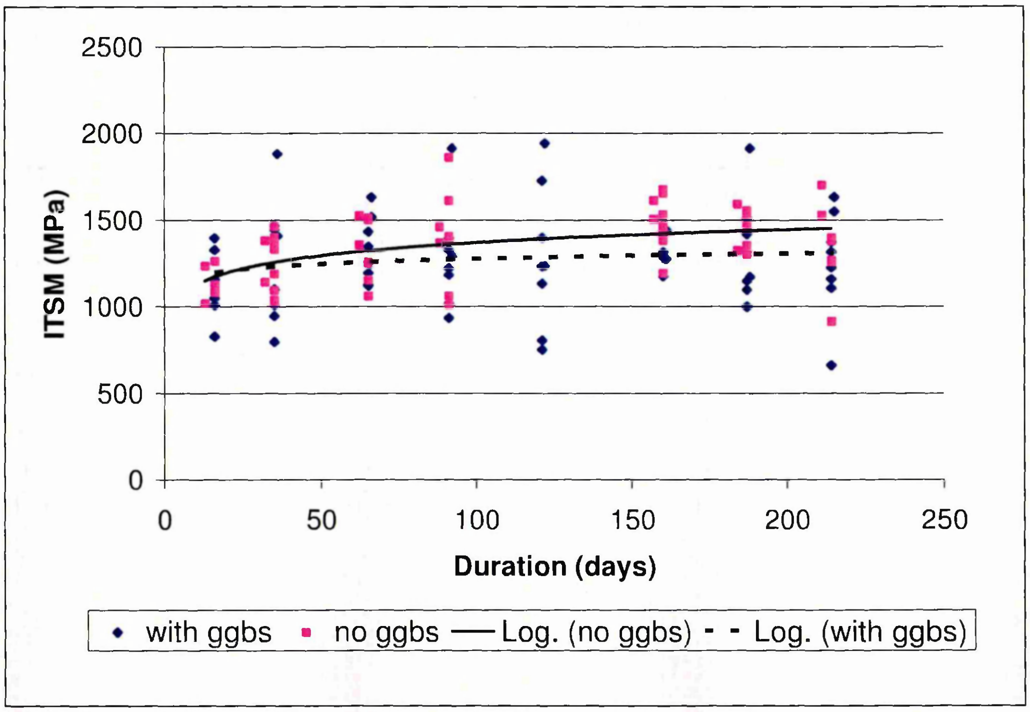


Figure 8-2: Comparison of specimens with and without GGBS cured in laboratory

- Mist room curing

The specimens with GGBS curing in the mist room (high RH) developed very high stiffness. Ultimately, the stiffness exceeded the capability of 5kN NAT tester, as presented in Table 8-6 (Widyatmoko, 2002). The stiffness evolvements process for specimens with GGBS cured in the mist room and in the laboratory is presented in Figure 8-3.

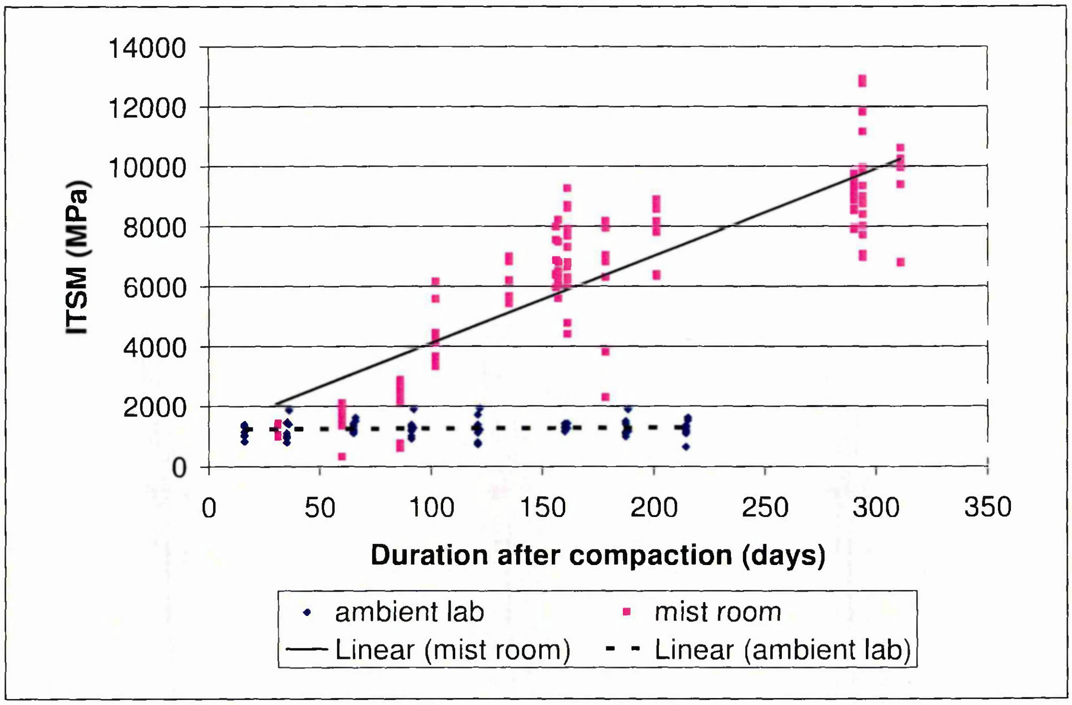


Figure 8-3: ITSM of specimens with GGBS in high and low RH environment

Table 8-6: Maximum stiffness for various specimen dimensions (5kN NAT)

Specimen Diameter (mm)	Specimen Thickness (mm)	Maximum stiffness in MPa at Target Horizontal Deformation
150	30	11,800
150	80	4,400
100	30	16,500
100	80	6,200

Note: the target deformation for 150mm diameter specimen is 7μm and 5μm for 100 mm diameter specimen.

8.2.3 Discussion

The pilot scale trial experiment showed that the specimens with or without GGBS developed similar stiffness by curing in the laboratory. While curing in the mist room, the specimens with GGBS developed much higher stiffness. The most plausible reason for the results is that the relative humidity was low in the laboratory and the water within the bitumen evaporated; therefore, there was not enough water to sustain the GGBS hydration. GGBS was only acting as an inert filler. In the mist room, there was sufficient water supply to sustain the GGBS hydration, therefore GGBS worked as a hydraulic binder and very high stiffness was developed over time.

The pilot scale trial demonstrated that GGBS was a binder deserving further investigation. With the addition of GGBS, higher stiffness was achieved without shortening the shelf life. The findings from the pilot trial were the basis for further experiments.

8.3 IN-DEPTH ANALYSIS

Having found that GGBS is successful in enhancing the specimen stiffness in a high humidity environment, further work was undertaken with the aim of assessing the effects of different concentrations of GGBS and bitumen emulsion together with their interactions, with an intention of optimizing bitumen emulsion and GGBS content.

8.3.1 Experimental programme

Several batches of cold mixes were made consecutively with binder contents chosen to include the predicted optimum binder content, and all the compacted specimens were cured in the mist room. The aim of mixing several batches with similar binder content was to repeat and verify the possible findings.

- Series 1

Series 1 comprised a further batch of recycled aggregates from the depot of the recycling plant. This batch of aggregates appeared to contain a high RAP content as revealed by the colour, which was clearly darker than the previous batches. However, based on the information from the producers, it was not possible either to obtain a predetermined evaluation of the ratios of recycled constituents or to devise an objective method of determining the exact source composition of different recycled aggregate. The recycled aggregates were mixed with bitumen emulsion and GGBS following the same mixing procedure using the same apparatus as previous batches. The binder content and the number of specimens are presented in Table 8-7.

The above specimens were compacted without encountering any workability problem and cured in the mist room. Series 1 was mixed on 1^{st} and 2^{nd} August 2002 and compacted on 20^{th} August 2002. These specimens were initially intended for ITSM, RLAT and ITFT test. The plan was to conduct ITSM test periodically to monitor the stiffness development and to perform ITFT and RLAT test after the stiffness reached a stable condition.

In the experimental process, the ITSM test was conducted at regular intervals after the specimens had generated enough stiffness to sustain the ITSM test. After one year, the stiffness was still increasing and many specimens became too stiff to conduct RLAT or ITFT test. At this stage, the specimens were found to behave more like hydraulic bound than bituminous bound material. A compressive strength test was conducted on some of the specimens and the remaining specimens were retained in the mist room for further monitoring.

Table 8-7: Number and binders composition of Series 1

Number of specimens	Bitumen content (%)	GGBS content (%)
4	0	0
9	2.7	0
10	4.5	0
9	4.5	1
10	0	2
10*	2.7	2
9*	4.5	2
10	4.5	3
10	0	4
10*	2.7	4
10*	4.5	4

Note: * indicating the binder concentrations that were repeated in Series 2, which is detailed in Table 8-8.

- Series 2

Series 2 was manufactured as a partial repeat of Series 1, with the aim of repeating and broadening the possible findings from Series 1. The batching of Series 2 was slightly different from Series 1 as shown in Table 8-8, mixes with no bitumen or no GGBS were excluded and mixes with 6% GGBS and 3.6% residual bitumen were included. The aggregates used were the recycled aggregate from previous projects (not necessarily left over from this research), some of which had been left in the storage room over a year. The aggregate gradation still followed the gradation for DBM as before (Figure 8-1). The materials were mixed on 24th September 2002 and compacted on 18th October 2002. The compacted specimens were cured in the mist room next to Series 1 specimens.

Table 8-8: Number of specimens and binders composition of Series 2

Number of specimens	Bitumen content (%)	GGBS content (%)
22*	2.7	2
27*	2.7	4
30	2.7	6
28	3.6	2
28	3.6	4
30	3.6	6
29*	4.5	2
29*	4.5	4
30	4.5	6

Note: * Indicating repeated binder concentration from Series 1.

The specimens were compacted with a plan of conducting ITSM, RLAT and ITFT tests, but unfortunately, this batch of specimens was not strong enough to sustain the ITSM test even after 4 months' mist room curing. The reason for its lack of sufficient hydration was assumed to be attributable to the aggregate employed. The aggregate employed in this batch of cold mixes was the recycled aggregate that had been left over from various previous tests and had been stored for from several months to over one year. To promote sufficient hydration in a reasonable timescale, the specimens were moved into a chamber where the temperature was set at 26°C, after which some of the specimens developed enough stiffness to sustain the ITSM test. The results are presented and discussed in Chapter 9.

- Series 3

Series 3 was compacted with the intention of using them as controlling specimens for Series 1 and 2. Series 3 was mixed at the end of November 2002. At that point, Series 1 had been curing in the mist room for three months and Series 2 had been curing for one month, neither of them having developed sufficient stiffness to sustain ITSM test. At a similar duration, Pilot scale trial specimens had developed reasonable stiffness for ITSM test (Figure 8-3). Assuming that the different recycled aggregate employed was the main reason for such a big difference, limestone was used in this batch instead of recycled aggregate. Limestone is often used in road construction and possesses more stable properties than recycled aggregate; therefore, it was chosen as a reference aggregate, with the intention of producing specimens with repeatable properties.

The limestone was collected from the depot of a hot bituminous mix producing plant with gradation following DBM as presented in Figure 8-1. The binders content were 4% GGBS and 0, 2.7 and 3.6% residual bitumen, partially replicating binder content in Series 1 and 2. The specimens were compacted without difficulty using the Marshall Compactor at 50 blows on each side and cured in the mist room together with Series 1 and 2.

Series 3 was monitored continuously in the following months but none of the specimens were able to sustain the ITSM test. To promote hydration, this batch of specimens was also put into the chamber where the temperature was controlled at 26°C. Finally, the specimens containing 0% bitumen and 4% GGBS developed very high strength/stiffness and a compressive strength test was conducted on these specimens. The specimens containing 4% GGBS with 2.7 and 3.6% residual bitumen developed no stiffness even after being moved into the chamber controlled at 26°C. The experimental process and results of specimens curing at elevated temperature are presented in Chapter 9. As the ITSM test could not be conducted, Series 3 could therefore not function as control specimens for Series 1 and 2.

8.3.2 Results

Both Series 2 and 3 failed to develop enough stiffness to sustain the ITSM test, and therefore were unsuitable as control specimens. Only Series 1 developed measurable stiffness, and ITSM test results are reported in Table 8-9 and Figure 8-4.

Figure 8-4 revealed that, at the same bitumen emulsion content, specimens with 4% GGBS has much higher stiffness than the specimens with 2% GGBS; at the same GGBS content, the stiffness actually decreased with increasing bitumen content.

The ITSM test conducted on 7th October 2003, 288 days after the specimens had been curing in the mist room, is presented in Table 8-9 and Figure 8-5.

Table 8-9: Series 1 ITSM result summary (at 288 days mist room curing)

GGBS (%)	Bitumen content (%)	Number of specimens	ITSM (MPa)	Standard Deviation (MPa)
2	0	9	2279	884
2	2.7	14	1311	493
2	4.5	14	805	122
3	0	No samples conducted		
3	2.7			
3	4.5	10	945	370
4	0	18	13937*	2402
4	2.7	20	4188*	1367
4	4.5	16	1166	227

8.3.3 Discussion

The ITSM test results of Series 1, which have been presented in Table 8-9, Figure 8-4 and Figure 8-5, demonstrate that GGBS makes a positive contribution towards stiffness whilst bitumen emulsion contributes negatively.

A comparison between Series 1 and Pilot scale trial is presented in Figure 8-6. The section of Series 1 specimens employed for comparison has the same binder content as Pilot scale trial, which is 4% GGBS and 2.7% residual bitumen. Series 1 specimens developed stiffness at a much slower rate. For example, Pilot scale trial specimens attained 9310MPa at 280 days while specimens from Series 1 attained 4188MPa at 289 days, curing in the same mist room (Figure 8-6). The possible reasons for the difference between the Pilot scale trial and Series 1 was identified as being either one or a combination of two factors:

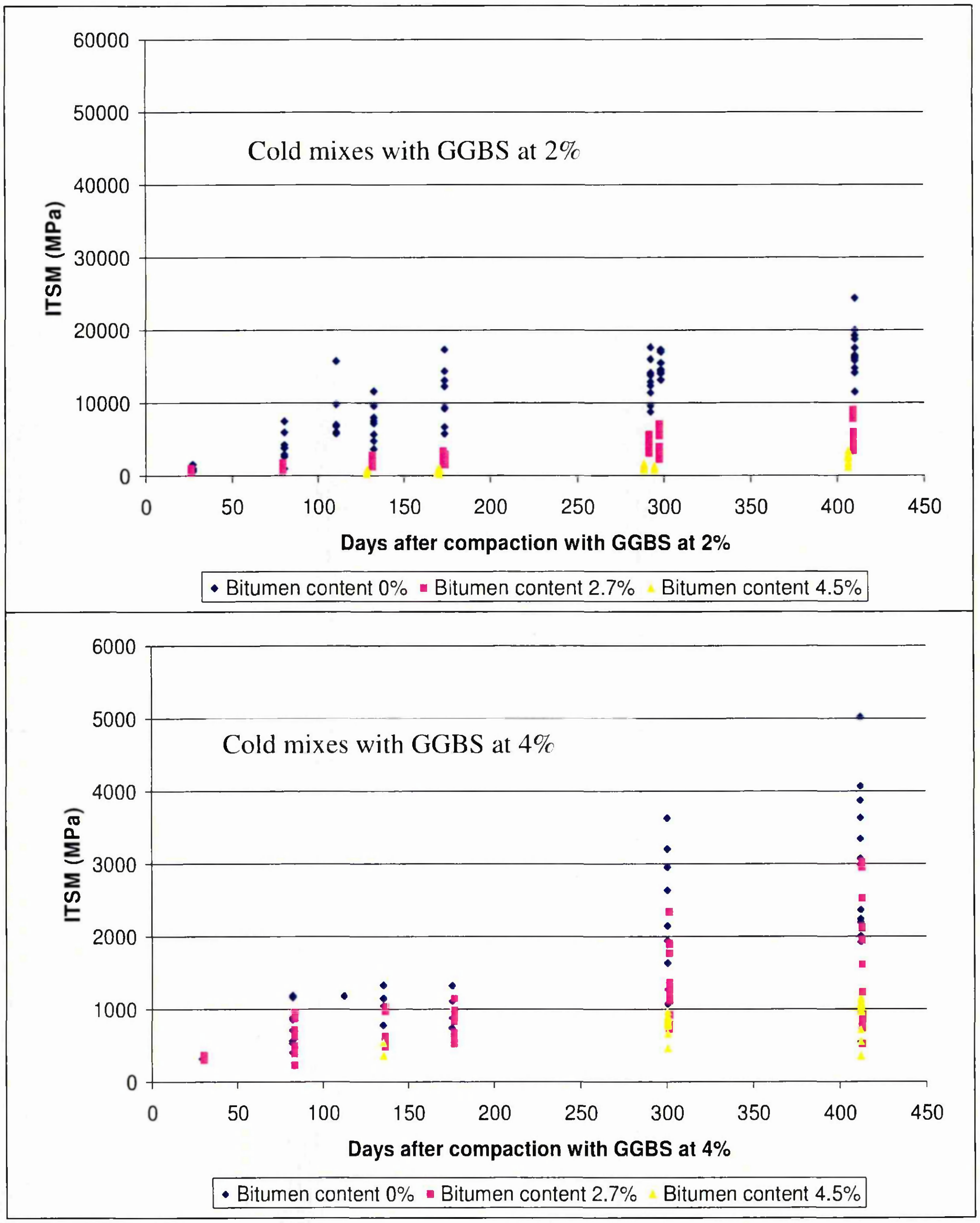


Figure 8-4: The ITSM development of Series 1 specimens

- Different recycled aggregate: Pilot scale trial employed freshly crushed aggregate while Series 1 employed aggregates that had been weathered for some time in the recycling plant.

- Different curing temperature: the Pilot scale trial specimens were compacted at the end of June 2002 and the Series 1 specimens were compacted at the end of August 2002. Although the mist room temperature was controlled at 20±2°C, the room temperature was slightly higher in summer than that in winter, therefore, the Pilot scale trial specimens experienced slightly higher initial curing temperature than Series 1 specimens. The hydration is very temperature-sensitive and the sensitivity was systematically investigated and reported by Wimpenny and Ellis (1989).

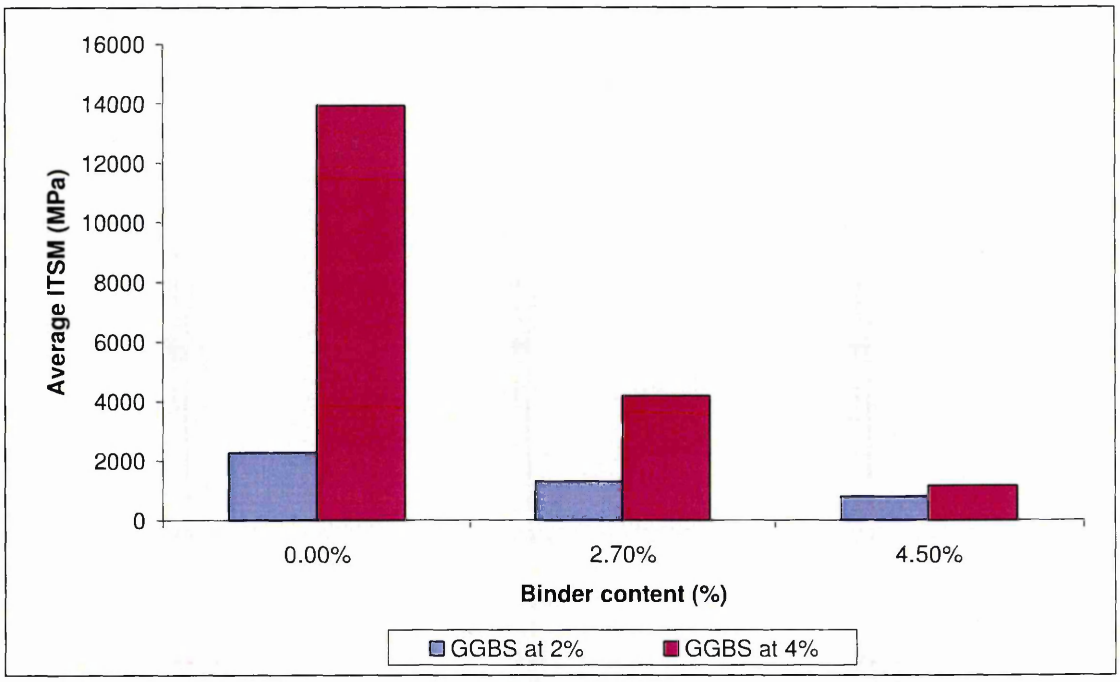


Figure 8-5: The ITSM for Series 1 specimens at 288 days

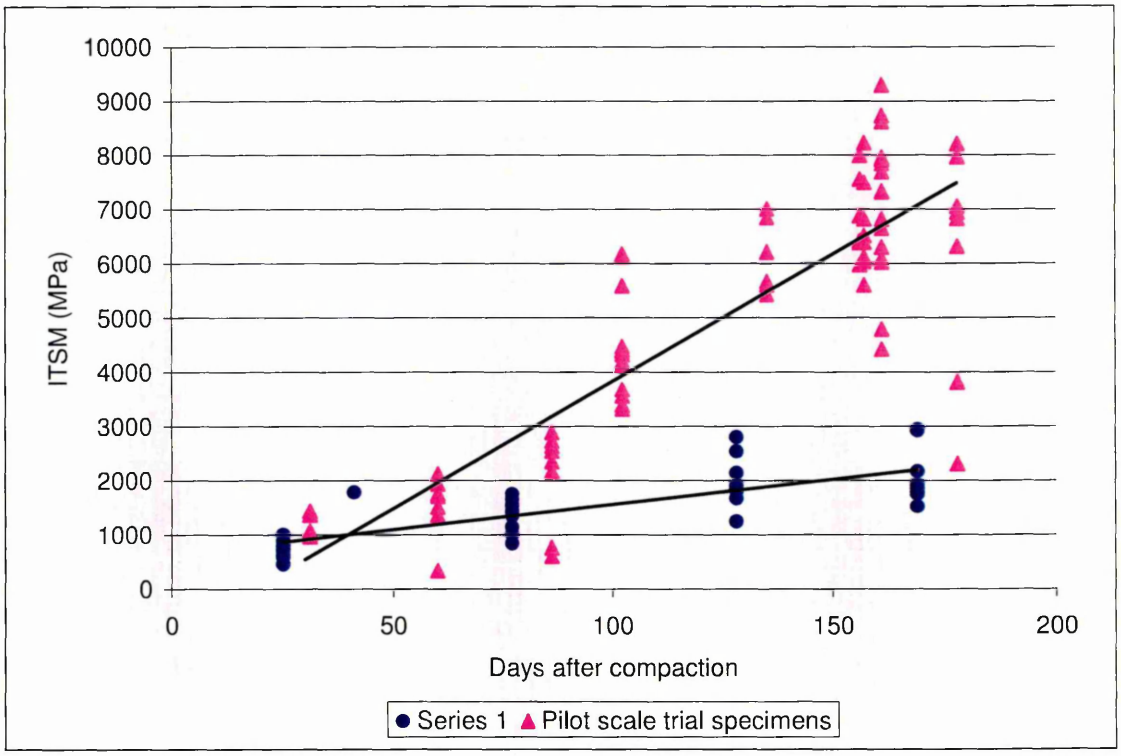


Figure 8-6: Comparing the stiffness of Pilot scale trial and Series 1 specimens

A stiffness modulus of 2400MPa is normally regarded as the characteristic stiffness of DBM100 with 20mm aggregate (Table 5-2) (BBA/HAPAS, 2003). Table 8-9 presented the three mixes which attained stiffness of over 2000MPa, namely GGBS 2% with no bitumen, GGBS 4% with no bitumen, GGBS 4% with bitumen content of 2.7%. The specimens without bitumen are simply latent hydraulically bound materials which tend to be friable and are thus prone to disintegration owing to lack of inter-particle cohesion as revealed in Figure 8-7. It is suggested that the addition of bitumen can mitigate this tendency, thus a binder content of 2.7% residual bitumen with 4% GGBS appears to be the most appropriate binder content out of the three binder combinations listed above.

Specimen with 2.7% bitumen and 4% GGBS bitumen as binder

Specimen with 4% GGBS as binder

Figure 8-7: Comparing specimens with and without bitumen

8.4 SUMMARY

GGBS effectively improves the stiffness of cold mixed materials whilst bitumen emulsion prolongs and attenuates the hydration process. Specimens with 4% GGBS and 2.7% residual bitumen developed sufficient stiffness to meet the relevant BBA/HAPAS (2001) requirements (Table 5-2).

The three series of specimens from the in-depth analysis stage with similar binder content produced specimens with widely differing results, indicating that there are further key factors influencing the performance of cold mixture yet to be differentiated and quantified, namely aggregate (composition and/or state) and temperature. The in-depth experimental results have revealed that aggregate type, composition and duration after aggregate processing (crushing) may have a profound impact upon the activation of the GGBS and the GGBS hydration is very sensitive to even minor changes in temperature.

Based upon the above findings, further research work was focused on the effect of the aggregate type and curing temperature towards GGBS hydration, which is described in Chapter 9.

CHAPTER 9 FACTORS INFLUENCING COLD MIXES WITH BITUMEN EMULSION AND GGBS AS BINDER

9.1 INTRODUCTION

The pilot scale trial has revealed the effectiveness of GGBS as a binder, but in the following in-depth analysis experimental work, only one out of three batches of specimens developed enough stiffness to sustain the ITSM test after curing in the mist room for several months, which was not considered enough. The composition of the recycled aggregate and curing temperature were found to be fundamentally influencing the performance of the cold mixes. The objectives of this chapter are to confirm the findings of Chapter 8, investigate the influence of the composition of the recycled aggregate and the curing temperature upon the performance of the cold mixes. In order to achieve the objectives, the following experimental work was undertaken:

- Types and combinations of recycled aggregate

The influence of the recycled aggregate on the performance of the cold mixes has been demonstrated in Chapter 8. In order to understand this influence further, recycled aggregates including road planings, demolished concrete and bricks were combined at various percentages and then mixed with bitumen emulsion. Specimens from such mixtures were compacted and performance was investigated.

- Temperature

The influence of temperature was investigated by curing specimens at three temperature levels (26, 20 and 10°C) and their performances were compared.

- Freezing and thawing (F-T) test

A freezing and thawing test was conducted on cold mixtures with bitumen emulsion as binder, as described in Chapter 7, which proved that the cold mixtures without hydraulic binder are sensitive to F-T, therefore unsuitable to be laid within 450mm of the road surface. An F-T test was undertaken on cold mixture with GGBS as binder to find out if the addition of GGBS improved the F-T resistance. It is required that materials within 450 mm of road surface must be F-T resistant (Sherwood and Roe, 1986).

- Rewetting

It has been described in Chapter 7 that the specimens with bitumen emulsion as binder (no hydraulic binder), which had reached a steady stiffness level by curing in the laboratory, lost half of their stiffness only two weeks after transferring into the mist room (High RH) from the bituminous laboratory (Low RH). The material is not deemed to be appropriate for paving in the structural layer because it is so sensitive to moisture condition. A similar test was also conducted on specimens with bitumen emulsion and GGBS as binder as a comparison in this chapter.

- Acceleration

It has been demonstrated in Chapter 8 that GGBS hydration is a slow process, often taking months before the specimens develop enough stiffness to sustain the ITSM test, indicating that the cold mixes will have a very low early strength after paving. In the industry, GGBS is often used together with cement or lime, which can act as an accelerator for GGBS, but the addition of active hydraulic binder will greatly reduce the shelf life.

In this chapter, some acceleration methods are considered.

9.2 EXPERIMENTAL DESIGN

9.2.1 Type and composition of recycled aggregate

The influence of aggregate type and composition on the performance of cold mixes has been demonstrated in previous chapters. In order to rigorously explore the effects of the recycled materials (primarily road planings and concrete demolition with some crushed bricks) for use in highway applications, comparison has to be made between how the aggregates behave individually and how they perform in combination.

The experimental work on this particular aspect was conducted by two undergraduate students Northwood (2004) and Langley (2004), based on the programme devised by the author. In their experimental work, the different types of recycled aggregate, namely concrete demolition, road planings and brick waste, were crushed separately in the laboratory by a recently acquired and dedicated jaw crusher. These were blended together at different compositions as shown in Table 9-1.

The binder content was the same as that adopted in the Pilot scale trial, 4% GGBS and 2.7% residual bitumen. Most of the specimens were cured in the mist room (>90% RH). The ITSM test was conducted at intervals to monitor the stiffness development.

Table 9-1: Batch plan for cold mixes

Mixes	Composition
Mix 1	100% crushed concrete
Mix 2	50% crushed concrete and 50% road planings
Mix 3	50% crushed concrete and 50% crushed bricks
Mix 4	100% crushed bricks
Mix 5	100% road planings
Mix 6	50% bricks and 50% road planings
Mix 7	1/3 crushed concrete, 1/3 road planings, 1/3 bricks

9.2.2 Curing temperatures

- Elevated temperature level

In order to investigate the influence of the curing temperature, three temperature levels, elevated, normal and low temperature were considered. The elevated temperature chosen had to be high enough to accelerate the hydration while not exaggerating the final stiffness to a level unachievable in the field. The temperature chosen should also consider the presence of bitumen, as overly high temperatures for a prolonged time can soften the bitumen, and the softened bitumen could bind the aggregate in a way similar to hot mixes and render the specimens with an unrealistic high performance after cooling. In the years 2002 and 2003, the highest temperatures in the Sheffield area were 29.7 and 33°C respectively. Research was conducted by Nottingham University to investigate the temperature range of the pavement road base, in which a sensor was buried 30mm under the pavement and the temperature was monitored. The temperatures throughout July/August are presented in Figure 9-1, which represents the highest monthly temperature in that year.

Based on the information from Nottingham University, 26°C was chosen as being representative of the elevated temperature based on an average temperature in July and August. At this temperature level, the bitumen is not likely to melt down which potentially could render the cold mixes with unrealistic high stiffness.

- Normal temperature level

The laboratory mist room was normally controlled at 20°C and readily available for curing large numbers of specimens. This temperature level also suggested in TRL report 386 for curing cold mixes (Milton and Earland, 1999). As presented in Figure 9-1, 20°C is a common average temperature occurring in the summer so some of the specimens were cured at this temperature.

- Low temperature level

A database from the Department of Meteorology at Reading University was freely supplied, which included soil temperature monitored at 20, 30, 50 and 100mm below the soil surface from 2nd January 1990 to 4th February 2003. The temperature levels at 20, 30, 50 and 100mm from the soil surface are graphically presented in Figure 9-2 and summarised in Table 9-2.

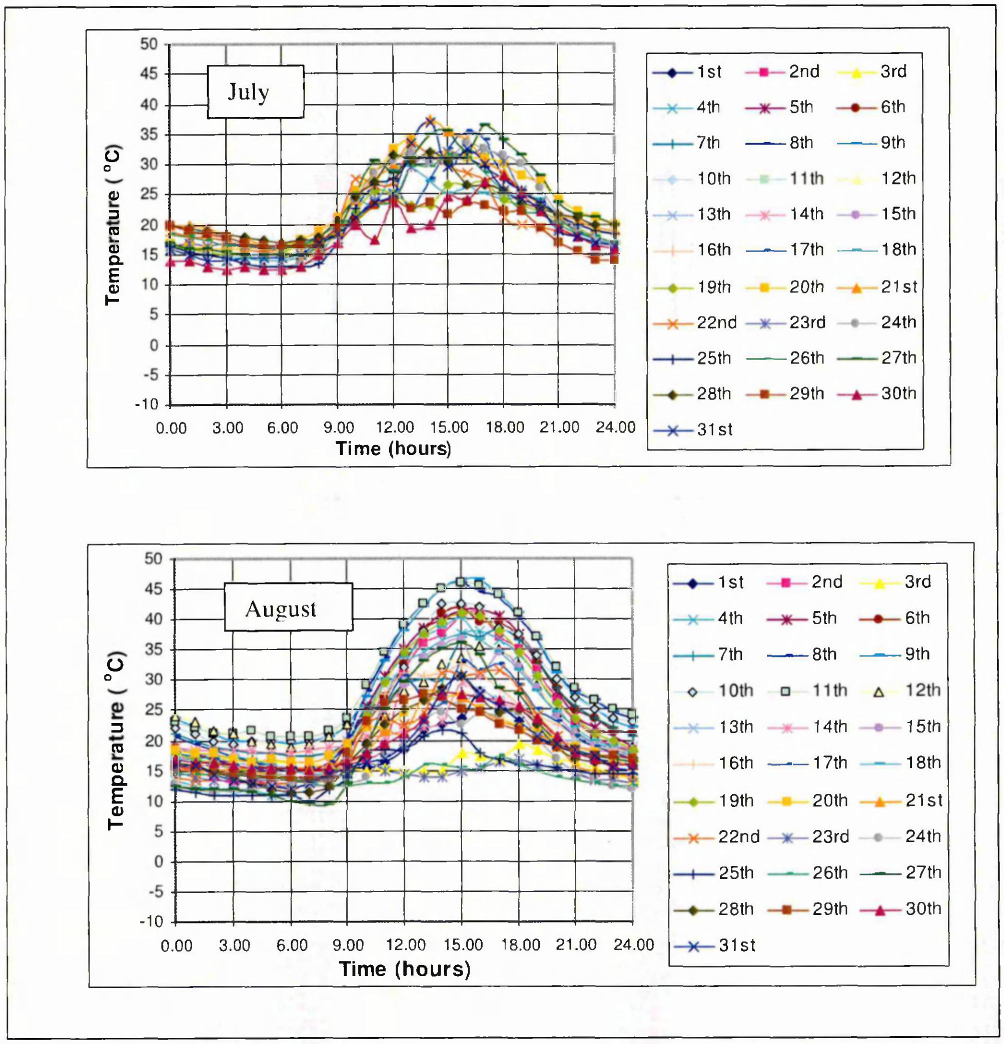


Figure 9-1: Temperature 30mm below the pavement surface in July/August (Nottingham University unpublished information)

Table 9-2: Temperature distribution below soil surface from 02/01/1990 to 04/02/2003 (from Reading University)

Depth from surface (mm)	Count (day)	Mean (°C)	Median (°C)	25th Percentile (°C)	75th Percentile (°C)
20	4745	10.7	10.1	6.4	15.2
30	4745	11.2	10.9	7.2	15.4
50	4745	11.4	11	7.5	15.3
100	4745	11.4	11.1	8	14.8

The concept of 'median' and 'percentile' are employed in analysing the raw temperature data in Table 9-2. A percentile is a value on a scale of 100 which indicates the percentage of a distribution that is equal to or below it. For example, 25th percentile value is equal to 25th percent of the data in the sample less than it. Normally, less than 25th percentile is believed to be below normal and greater than 75th percentile is considered above normal. Median is a number dividing the higher half of a sample from the lower half. The median of a finite list of numbers can be found by arranging all the observations from the lowest value to the highest value and selecting the middle one. If there are an even number of observations, one often takes the mean of the two middle values. The median temperature levels at 20mm to 100mm from the soil surface vary from 10 to 11°C. The Nottingham University unpublished information suggested that the temperature at 30 mm from the pavement surface could reach over 40°C in summer (Figure 9-1) and down to -6°C (Figure 9-3) in winter. The yearly average pavement temperature is around 10°C.

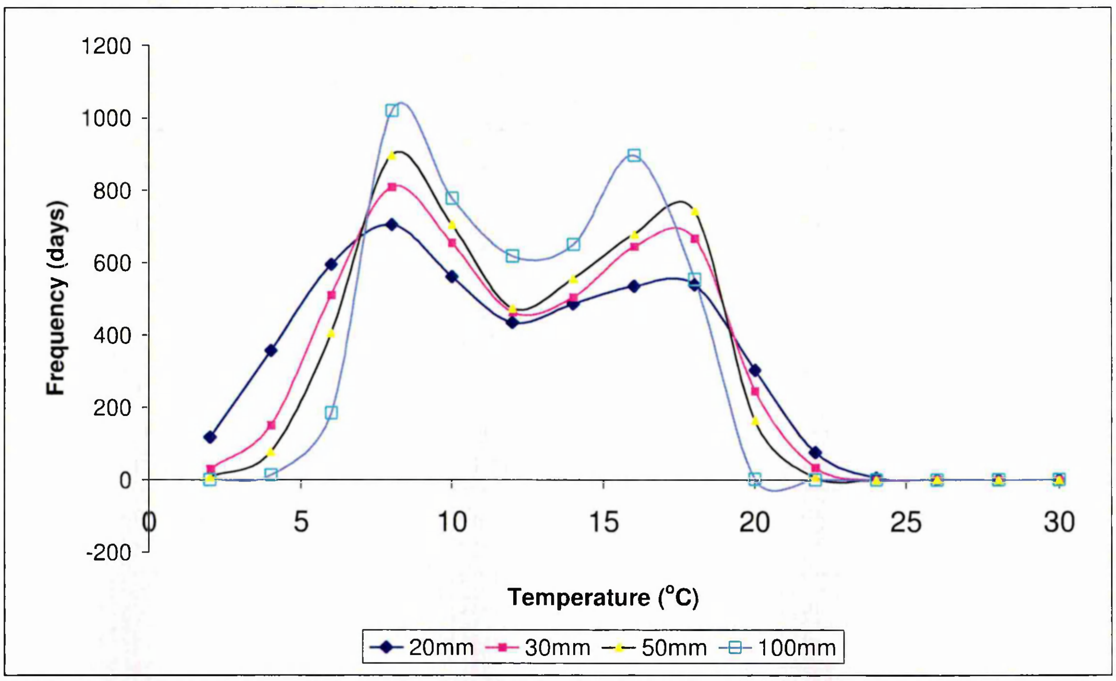


Figure 9-2: Temperature distribution for Reading area from 02/01/1990 to 31/12/2002

'Cold mix construction should not be done when ambient temperatures under 10°C are expected, or when generally poor weather is predicted. As the aggregate is not heated, its maximum temperature is limited to that of the atmosphere, plus that attributable to solar radiation. Upon application the asphalt quickly reaches the temperature of the aggregate. If it is too cool, mixing is difficult. ...... (Asphalt Institute, MS-14, 1990).'

Since the cold mixes are not recommended for use when the ambient temperature is lower than 10°C, by choosing 10°C as the testing temperature, the worst- case scenario for the cold mixes application was investigated. The 10°C temperature was also employed by Atkinson et al (1996) in their investigation of the performance of secondary aggregate in pavement foundation.

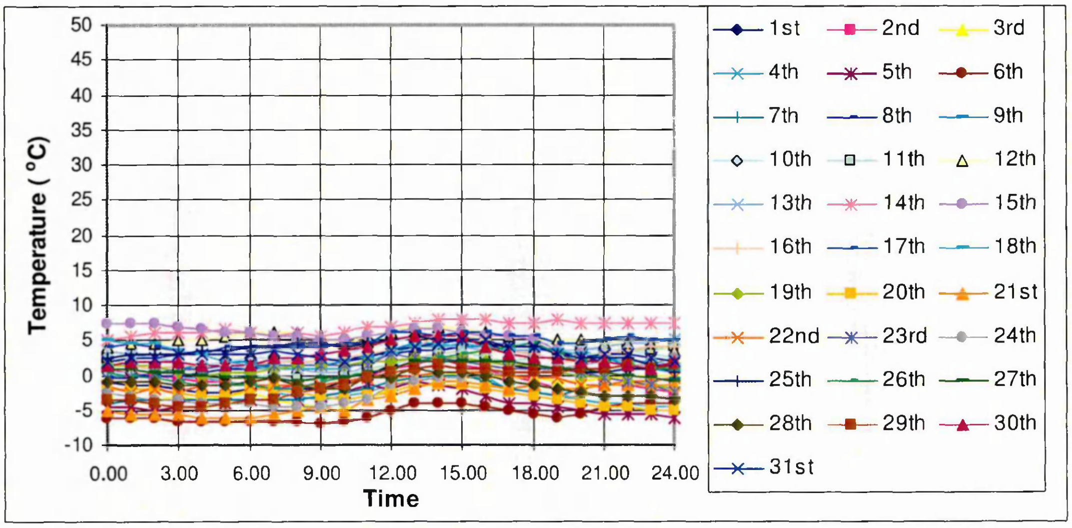


Figure 9-3: 30mm below pavement surface temperature in December (Nottingham University Pavement Group unpublished information)

- Freezing and thawing (F-T)

As indicated in Figure 9-3 and Figure 7-9, pavement temperature can fall below zero in winter so a F-T test has been conducted on specimens without hydraulic binder, with the lower limit set at -5°C, as detailed in Paragraph 7.2.3. An F-T test was conducted in this part of the research as a comparison.

9.2.3 Changing humidity

As indicated in Figure 7-1, the ambient relative humidity is variable with the season and an investigation into the effect of changing curing humidity condition has been conducted on specimens without hydraulic binder. It was found that the stiffness of the specimens was halved by moving them from high humidity conditions to low humidity conditions. An investigation into the effect of changing humidity was undertaken in this section as a comparison.

9.2.4 Accelerators

The ideal cold mixes should have a long stockpile life and will generate high stiffness soon after compaction. As indicated in this project, cold mixes always struggle with early strength, which means cold mixes are vulnerable to early traffic in the field. In order to secure some early strength, acceleration is necessary. According to Hewlett (1998), '*the slag activators can be either alkaline activators, such as sodium hydroxide, lime, sodium carbonate and sodium silicate, or sulphate activators like calcium sulphates or phosphogypsum.*' Hewlett (1998) further claimed that water glass is the best activator. After balancing all these options, NaOH solution and lime were chosen as accelerators for further research. NaOH solution was chosen as a solution, because it could be added with ease in the paving process without shortening the shelf life. Lime was chosen considering it is the most commonly employed accelerator for GGBS.

- lime

Lime as an accelerator to GGBS is widely reported and actually GGBS is seldom used alone in real applications and is always applied together with lime or cement. Lime was not used in the previous mixes because of its impact on shelf life. Merrill (2004) summarised the function of lime as:

'*Lime can create weak bonds between the recycled aggregate particles and can be used to reduce the plasticity of aggregate containing plastic or organic elements. Lime can improve the adhesion properties of the aggregate with bituminous binder. For some slow curing hydraulic binders, lime is an activator for the other binding agents.*'

Hydrated lime was added in the mixing process. The lime was added at a very low content (less than 1%) because of its impact on the shelf life.

- NaOH solution

Mixing solid Na_2O with water gives an exothermic reaction and water was heated in the process of adding granular Na_2O. As NaOH solution is very caustic, making NaOH solution over 4 mol/litre was inconvenient. As a result, only 2 mol/litre and 4 mol/litre NaOH solution were produced in this project. Here the mol/litre means moles of dissolved substance per litre, for example, 4 mol/litre NaOH solution represents a solution containing 4 moles NaOH per litre. The addition of up to 10ml NaOH solution was found to present no bleeding problems, therefore 10ml NaOH solution was added in the project.

9.2.5 Compressive strength test

Cold mixes bound with bitumen emulsion and latent hydraulic binder behave more like unbound granular materials initially, with only limited cohesive force from bitumen emulsion and evolve into hydraulically bound materials with very high strength. A compressive strength test is often employed to characterise hydraulically bound materials. Hot mixed bituminous materials are unsuitable for the compressive strength test, because the materials will continuously deform under compressive load and no peak load can be obtained under test conditions. GGBS is intrinsically a hydraulic binder and cold mixes with hydrated GGBS perform as a hydraulically bound material, therefore a compressive strength test can be conducted. The compressive strength test was also reported in analysing the properties of foamed bitumen mixes with cement addition (Nunn and Thom, 2002).

The standard compressive strength test is detailed in BS EN 12390-1 (2000), where the specimens are prepared as cubes or cylinders. In the case of a cube, the height and depth are equal and in the case of a cylinder, the diameter is equal to half of the height as illustrated in Figure 9-4. However, in this project, the specimens were not designed for the normal compressive strength test and they were Marshall specimens with 100mm in diameter and around 60mm in thickness. The compressive strength was therefore estimated with Equation 9-2, which is presented in BS 6089 (1981).

$$Estimated\ \ strength = D/(1.5 + 1/\lambda) \times Core\ \ Strength \qquad \text{Equation 9-2}$$

Where:

D = 2.5 for cores drilled horizontally.

D = 2.3 for cores drilled vertically.

λ = l/d is the length (l) /diameter (d).

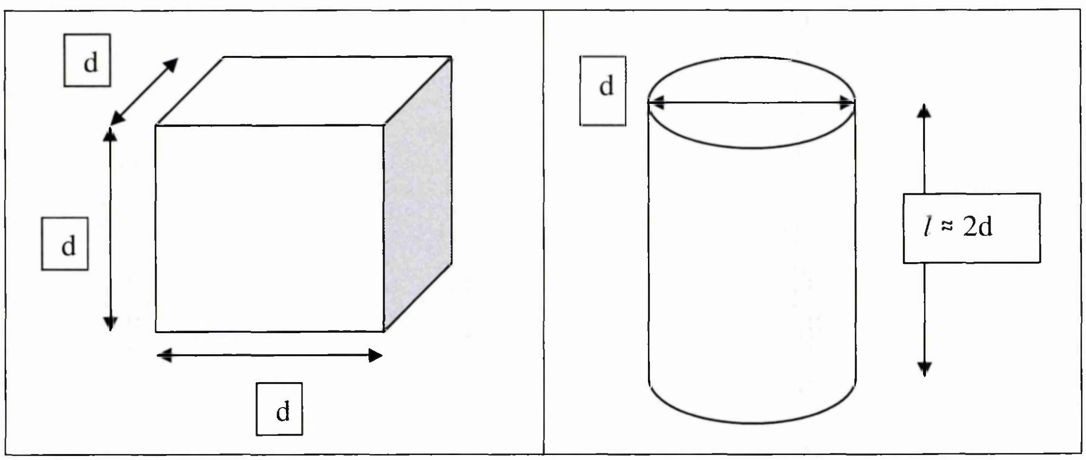


Figure 9-4: Illustration of specimen preparation for compressive strength test

The specimens were compacted by Marshall compactor, more like vertically drilled cores, therefore 2.3 was chosen as the D value. In BS 6089 (1981), the core thickness was required to be at least 95% of its diameter and not more than 1.2 times its diameter. Although the Marshall specimens did not fully comply with this requirement, the results were for comparison purposes only and Equation 9-2 was employed in this instance.

9.3 EXPERIMENTAL PROCESS

9.3.1 Type and composition of recycled aggregate

The experimental work investigating the influence of the type and composition of recycled aggregate upon the performance of the cold mixes was conducted by two undergraduate students. The detailed test process is not recorded here.

9.3.2 Elevated curing temperature (26°C)

A high relative humidity curing environment was created utilising a slab compaction chamber, which could be controlled at a set temperature, as illustrated in Figure 9-5 and Figure 9-6, where specimens were placed on damp towelling in trays and the trays were further enveloped in plastic bags. The trays with specimens were then put onto a shelf in the slab compaction chamber, which was controlled at 26°C. In the curing process, water was added periodically to keep the towels wet, which made sure the moisture was saturated in the bag, ensuring a relative humidity close to 100%.

Two sets of specimens, Series 4 and 5, were initially cured in the chamber. The binder content of Series 4 and 5 are presented in Table 9-3 and Table 9-4 respectively. In order to accelerate the hydration process, lime was added in Series 5. The cold mixes with lime were found to be desiccated only days after mixing, the higher the lime content, the shorter the shelf life. As a result, lime was added at a very low content of 0.25, 0.5 and 1.0% only. The ITSM test was conducted on two series of specimens at intervals to monitor the stiffness development over time and the results are reported in Section 9.4.

Another two sets of specimens, Series 2 and 3 were also moved into the chamber later, after being found unable to develop enough stiffness to sustain ITSM test by curing in the mist room over months, as detailed in Chapter 8.

After curing in the slab compaction chamber for over 100 days, the specimens developed very high strength. A compressive strength test was therefore conducted on the Series 4, 5 and part of Series 3 specimens. The compressive strength test was conducted following BS EN 12390-3 (2001). The loading rate was 75kN/min, equivalent to 0.59MPa/s. The test was conducted on 11th June 2003 using a compression testing machine from Denison Mayes Group (Figure 9-7).

Table 9-3: Series 4 specimen composition and numbers

GGBS (%)	Number of specimens		
	2.7% bitumen	3.6% bitumen	4.5% bitumen
1	3	4	2
2	9	8	9
3	4	4	2
4	4	8	11

Figure 9-5: Specimen prepared for high humidity conditioning

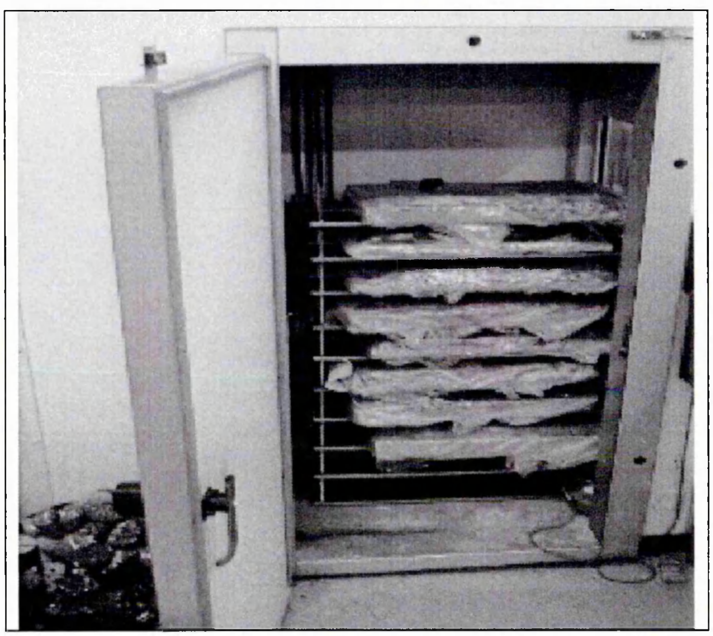


Figure 9-6: Specimens curing in a rack in slab compaction chamber controlled at 26°C

Table 9-4: Series 5 cold mixes composition and specimen numbers

GGBS (%)	Bitumen content (%)	Number of specimens		
		0.25% lime	0.5% lime	1.0% lime
4	2.7	2	2	-
4	3.6	-	2	2

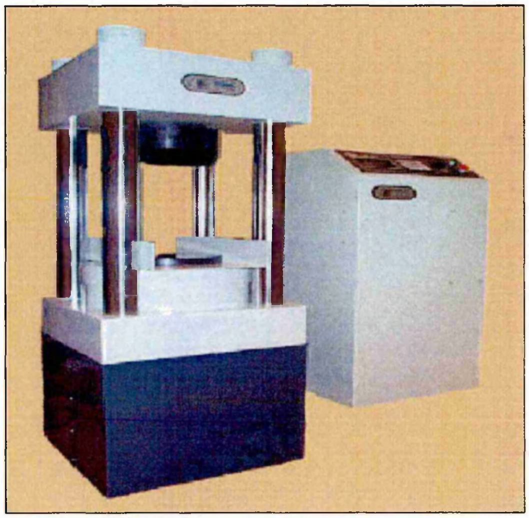


Figure 9-7: Denison Mayer compressive machine

9.3.3 With accelerators curing at 10 and 20°C

Three batches of specimens, Series 6, 7 and 8, were mixed to investigate the effectiveness of accelerators and curing temperature. Series 6 and 7 were mixed to investigate the effect of NaOH solution as an accelerator and Series 8 was mixed to study the effectiveness of Lime as accelerator. The specimens were further divided into two comparison groups, cured at 10°C and 20°C respectively, to investigate the effectiveness of the accelerators at low temperature.

- Series 6

Series 6 were mixed with another batch of recycled aggregate collected from the recycling plant with a high concentration of road planings (the colour is darker than other batches). The cold mixes were produced with 4% GGBS and 2.7% residual bitumen. The NaOH solutions were manufactured at three concentration levels, pure water with no NaOH, 2 mol/liter NaOH solution and 4 mol/liter NaOH solution. NaOH solution was added into the cold mixes hold in the mould before compaction. There was little water bleeding in the compaction process. The specimens were compacted and cured as listed in Table 9-5. The specimens at each NaOH concentration level were further divided into two groups, one group wrapped as Figure 9-8 and then cured in a fridge at 10°C and the other group cured in the mist room where the temperature was controlled at 20°C.

Table 9-5: Series 6 specimen mixing plan and curing temperature

Number of specimen	NaOH content (mol/litre)	Curing temperature (°C)
4	0	20
4	4	20
4	0	10
4	2	10
4	4	10

Figure 9-8: Placing specimens on wet towel and wrap up for curing

It took nearly 160 days before most of the specimens in Series 6 generated enough stiffness to sustain the ITSM test. While waiting for the result from Series 6, another batch of specimens was compacted to investigate the acceleration effect of NaOH solution.

- Series 7

Series 7 was composed entirely of laboratory crushed recycled concrete demolitions, with the aim of accelerating the experimental process. Freshly crushed concrete demolitions tend to have higher residual lime content. The residual lime contained in the freshly crushed concrete waste is potentially able to accelerate the GGBS hydration and shorten the experimental process. It was found that, with freshly crushed concrete, more water was needed to surface wet the aggregate and more bitumen emulsion was needed to coat the concrete particles. Ultimately, 7.3% water

(3.5% in Series 6) was added to surface wet the aggregates and 3.6% bitumen emulsion (2.7% in Series 6) was added to coat the aggregate. The mixes dried out only two days after mixing. 10ml NaOH solution at two concentration levels of 0 mol/litre and 4 mol/litre was added. The specimens were compacted soon after mixing and cured at 10°C and 20°C as shown in Table 9-6.

Table 9-6: Series 7 mixing plan and curing temperature

Number of specimens	NaOH solution Concentration (mol/lire)	Curing Temperature (°C)
3	0	20
4	4	20
3	0	10
4	4	10

- Series 8

Series 8 was mixed to investigate the effect of lime as an accelerator at low temperature. The aggregate employed was the same batch of aggregate used in Series 6, composed of recycled aggregate with a high concentration of road planings. The binder was composed of 4% GGBS, 2.7% bitumen and various lime concentrations as shown in Table 9-7. All the specimens were compacted and cured at 10°C and ITSM test was conducted to monitor the stiffness development.

Table 9-7: Series 8 batching plan for specimens with lime

Number of specimen	Lime content (%)	GGBS (%)	Bitumen (%)
4	0	4	2.7
4	0.25	4	2.7
5	1.00	4	2.7

9.3.4 Freeze and thaw (F-T) test

An F-T test had previously been conducted on specimens with bitumen emulsion as binder (containing no GGBS) and those specimens were found to be F-T sensitive as reported in Chapter 7. Correspondingly, an F-T test was conducted here with specimens containing bitumen emulsions well as GGBS. Sixteen fully cured specimens were employed in the F-T test process. The specimens were those left over from the 26°C slab compaction chamber curing. F-T test was not conducted on loose materials or freshly compacted specimens, based on the assumption that before

GGBS hydration, the cold mixes would behave similarly to the materials without GGBS, which had been proved to be F-T sensitive (Chapter 7).

The F-T regime employed was the same as that employed for the cold mixes without GGBS, with 24 hours a cycle, 10 hours at -5°C and 6 hours at 5°C, the rest of the time in transition between the two. In the test process, the fully hydrated specimens were put on the damp towels in a pan and the pans were further wrapped in plastic bags as shown in Figure 9-8. The pans were then put into the weather chamber for F-T test. The ITSM test was conducted before the specimens were put into the chamber, at 9 F-T cycles and at 16 F-T cycles respectively.

9.3.5 Re-wetting

While the prevalent weather conditions in the UK are wet and damp, there is no guarantee that the road base and foundation are always wet and some sections of the road at some seasons of the year may dry out. It has been proved that GGBS will behave as inert filler if cured in a relatively low humidity environment. It is important to investigate whether the GGBS will resume hydration after re-wetting. The ability to recommence hydration for GGBS cement after drying out and rewetting was reported by the Concrete Society (1991), '*as with all concretes, if GGBS concrete is kept continuously dry, the long term properties are therefore affected. If the concrete is subsequently exposed to moisture, hydration recommences, and for both GGBS and OPC concrete, the long term strength is little altered by the initial poor curing. For GGBS concrete, it has been reported that curing in water recovers the strength, even after demoulding at six hours and drying in a wind tunnel.*'

To discover whether the GGBS will resume hydration after re-wetting, specimens were transferred from the laboratory (low RH, Figure 7-2) into the mist room (high RH, RH > 90%). Two specimens were put into the mist room on 1st November 2002 and further four specimens were put into the mist room on 5th February 2003. The specimens employed were the specimens left over from the Pilot scale trial. They had been left in the laboratory for 100 days and 216 days respectively before being put into the mist room and had reached a stable condition.

9.4 RESULTS

9.4.1 Type and composition of the recycled aggregate

Two undergraduate students under the supervision of the author conducted the experimental work on the type and composition of the recycled aggregates upon the performance of the cold mixes. Several batches of specimens were compacted and tested, with the ITSM results of one batch of specimens presented in Table 9-8.

Table 9-8: The interrelation between recycled aggregate composition and stiffness

No.	Mix proportions	ITSM (MPa)	Mean ITSM (MPa)
1	50% brick, 50% RAP	257	221
2	50% brick, 50% RAP	218	
3	50% brick, 50% RAP	220	
4	50% brick, 50% RAP	187.5	
5	50% brick, 50% concrete	405	328
6	50% brick, 50% concrete	254	
7	33% brick, 33% concrete, 33% RAP	367	409
8	33% brick, 33% concrete, 33% RAP	385.5	
9	33% brick, 33% concrete, 33% RAP	476.5	
10	100% RAP	300.5	291
11	100% RAP	281.5	
12	100% concrete	686	1185
13	100% concrete	1268.5	
14	100% concrete	1442	
15	100% concrete	1344	
16	100% brick	121.5	121.5
17	100% brick	No result	
18	50% concrete, 50% RAP	996	782
19	50% concrete, 50% RAP	566.5	

9.4.2 Elevated curing temperature (26°C)

Three sets of specimens, Series 3, 4 and 5, were cured at elevated temperature (26°C). The ITSM test was conducted at intervals to monitor the stiffness development and the compressive strength test was conducted at the end of the monitoring process to get maximum information from the specimens.

- Series 4

The detailed compositions and test results of Series 4 are presented in Table 9-9. Despite the care taken to ensure consistency, this batch of specimens ended up with the highest stiffness variation when compared with all other specimens in this project, perhaps because the temperature in the chamber was uneven, or because some trays became inadvertently uncovered.

Table 9-9: ITSM results for specimens at 26oC after 110 days (Series 4)

GGBS content (%)	Bitumen content (%)	Number of specimens	Mean ITSM (MPa)	Standard Deviation (MPa)
1	2.7	3	1672	501
	3.6	4	1593	313
	4.5	2	1322	787
2	2.7	9	4518	2030
	3.6	8	4482	1949
	4.5	9	3655	783
3	2.7	4	7333	2868
	3.6	4	9325	5014
	4.5	2	3773	240
4	2.7	4	19637	3241
	3.6	8	9833	9618
	4.5	11	7160	3332

The stiffness development of Series 4 was monitored over time and the results at 110 days after compaction are presented in Table 9-9. The experiment further demonstrated that ITSM increases with the increasing GGBS and decreasing bitumen emulsion.

Table 9-10 presents the compressive strength results of series 4 specimens and the test result again reflected the fact that the compressive strength increases with increasing GGBS content but the contribution from bitumen emulsion appears to be negligible. Blue tints were found on the specimen's breaking surface, which is a characteristic of GGBS hydration (Figure 9-9).

Table 9-10: Compressive strength of Series 4 cured at 26oC for 110 days

GGBS (%)	Bitumen content (%)	Number of test results	Mean compressive strength (MPa)	Standard deviation (MPa)
1	2.7	2	1.69	-
	3.6	4	1.76	0.24
	4.5	2	1.41	-
2	2.7	8	2.59	0.42
	3.6	8	2.49	0.53
	4.5	9	2.56	0.37
3	2.7	4	3.46	0.41
	3.6	3	3.36	-
	4.5	2	2.40	-
4	2.7	*	*	*
	3.6	6	3.56	1.78
	4.5	6	2.89	0.46

Note: * the corresponding specimens were employed for F-T test, therefore, no compressive strength was conducted. -: the specimen number is too small to valid standard deviation.

- Series 5

The cold mixes with lime were found to be desiccated only days after mixing, the higher the lime content, the shorter the shelf life. Specimens were compacted soon after mixing, then wrapped in trays with plastic films and cured in the slab compaction chamber controlled at 26°C (Figure 9-5 and Figure 9-6). The surface of the specimens with lime looked much smoother than those without. The ITSM test was conducted at intervals to monitor the stiffness development and the results are presented in Table 9-11.

A compressive strength test was also conducted on this batch of specimens and the results are presented in Table 9-12 and graphically presented in Figure 9-10. It can be seen that the addition of lime does not increase the compressive strength but helps to reduce the variance between specimens.

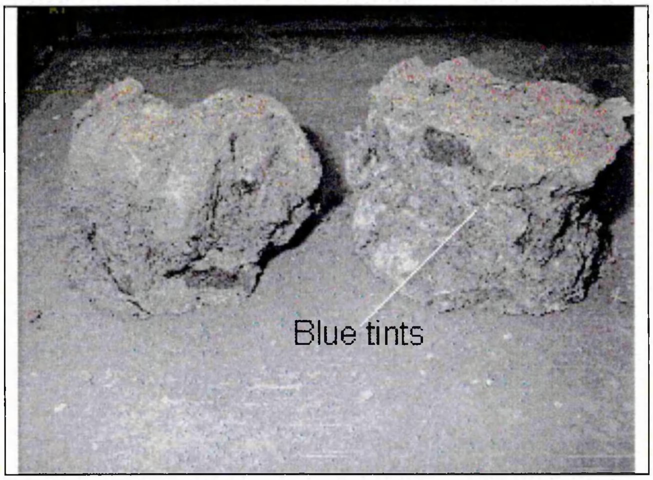


Figure 9-9: Specimen with GGBS after crushing

Table 9-11: Series 5 ITSM test results

Duration (days)	Lime content (%)	Mean ITSM (MPa)	
		Bitumen 2.7%	Bitumen 3.6%
12	0.25	4718	-
	0.5	7661	6999
	1	-	8166
28	0.25	6202	-
	0.5	10369	9827
	1	-	11162
56	0.25	12328	-
	0.5	14230	13593
	1	-	13863
98	0.25	13007	-
	0.5	15906	15217
	1	-	14561

-: no specimen compacted at this binder content

Table 9-12: Series 5 compressive strength test results

Number of specimens	GGBS content (%)	Bitumen content (%)	Lime content (%)	Mean Compressive strength (MPa)	Standard Deviation (MPa)	Coefficient of Variation (%)
6	4	3.60	0	3.56	1.78	50.0
4	4	3.60	0.5	3.81	0.53	13.9
4	4	3.60	1	3.76	0.24	6.4

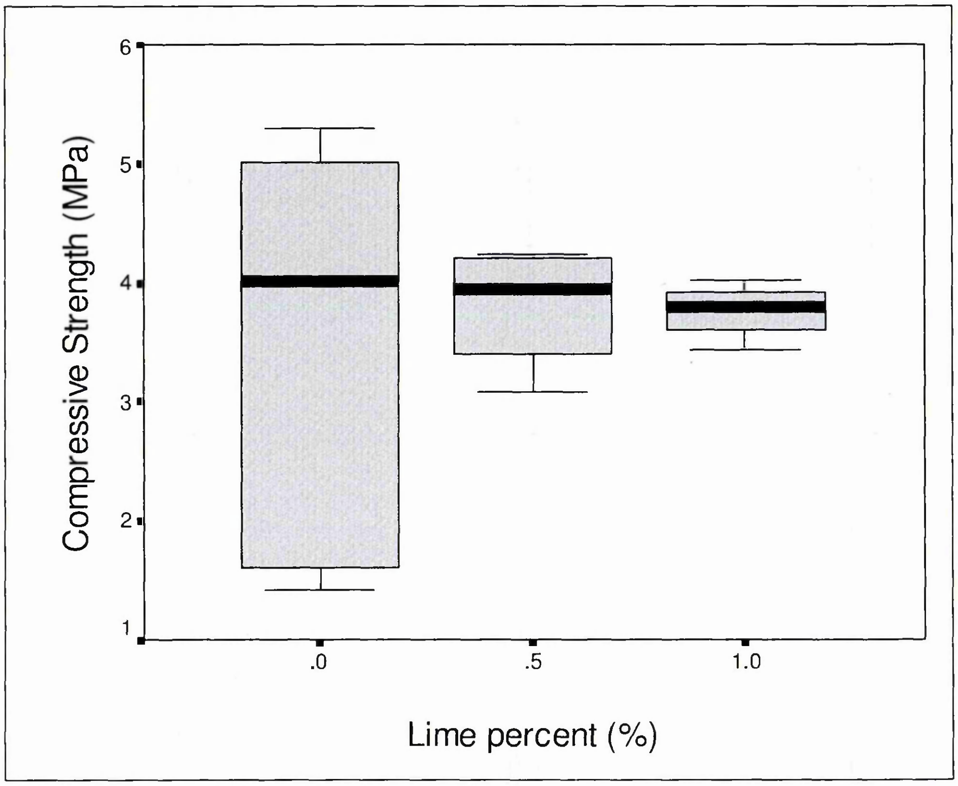


Figure 9-10: The influence of lime on compressive strength (Series 5)

- Series 2 and 3

Series 2 was compacted in October 2002 and Series 3 was compacted in December 2002 as introduced in Chapter 8. The two batches of specimens had been curing in the mist room until February 2003, none of the specimens developed enough stiffness for the ITSM test, and as a result, the specimens were moved into the slab compaction chamber, which was controlled at 26°C. The ITSM test was conducted on these specimens at intervals, monitoring the stiffness development from February until May 2003. Some of the Series 2 specimens developed high stiffness and others failed to do so (detailed in Section 8.3, in-depth analysis). Some of the Series 3 specimens containing no bitumen developed very high stiffness over time, therefore a compressive strength test was conducted with the test results presented in Table 9-13.

Table 9-13: Compressive strength of part of Series 3 specimens

Number of specimens	GGBS content (%)	Compressive strength (MPa)
3	2.3	7.4
2	4.0	14.3
3	4.6	23.9

9.4.3 With accelerators curing at 10°C and 20°C

Three batches of specimens, Series 6, 7 and 8, were compacted to investigate the effectiveness of accelerators, including NaOH solution and Lime. The specimens were divided into two groups curing at 10°C and 20°C respectively; the results are presented as follows:

- Series 6

The ITSM test results of Series 6 are presented in Table 9-14 and the results at 218 days after compaction are further graphically presented in Figure 9-11. The influence of temperature and NaOH solution on stiffness is evident from the figure and the table. Although Dr. Widyatmoko (2001) suggested that the lower limit of NAT ITSM test is 500MPa, stiffness less than 500MPa was included in Table 9-14. At this level, the exact ITSM value may be too low to be trustworthy but the mere fact that the specimens were able to sustain the ITSM test indicated increased stiffness.

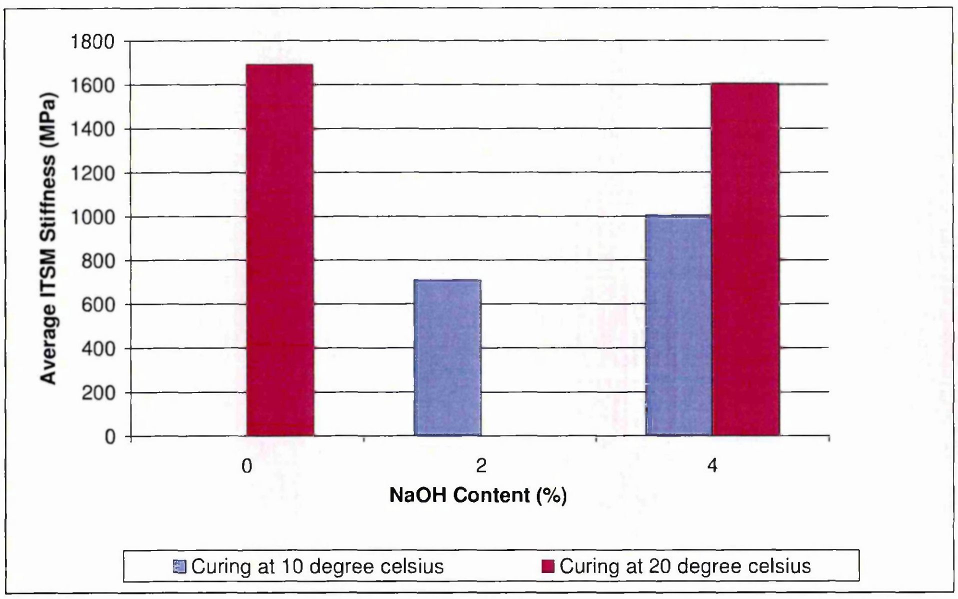


Figure 9-11: Mean ITSM of Series 6 specimens at 218 days after compaction

- Series 7

Series 7 developed very high stiffness in a short time, probably owing to the high content of lime and un-hydrated cement within the freshly crushed concrete demolitions. The ITSM results are presented in Table 9-15 and Figure 9-12. The stiffness values are high and variable with 95% confidence interval presented in Table 9-16. At 8 days, the specimens curing at 20°C developed prominently higher stiffness then specimens curing at 10°C. At the same temperature levels, for example at 10°C or 20°C, the specimens with or without NaOH solution developed similar stiffness. At 125 days, the specimens curing at 20°C or 10°C, with or without NaOH solution developed similar stiffness. This could be attributable to the high un-hydrated cement content within the freshly crushed concrete demolitions employed in this batch of specimens. With the high level of lime and un-hydrated cement content in the crushed concrete, the GGBS can hydrate properly even at low temperature without the addition of NaOH solution.

Table 9-14: The effect of NaOH on ITSM values

Sample Number	Curing condition (°C)	NaOH (mol/liter)	ITSM (MPa)			
			12 days	27 days	160 days	218 days
1	20	0	-	-	1509	1338
2	20	0	-	-	1623	1409
3	20	0	-	-	2090	2503
4	20	0	-	-	2252	1515
5	20	4	214	881	1261	1465
6	20	4	576	997	1826	1568
7	20	4	315	949	1173	1395
8	20	4	549	624	1866	1990
9	10	0	-	-	-	392*
10	10	0	-	-	-	-
11	10	0	-	-	-	-
12	10	0	-	-	-	-
13	10	2	-	-	-	-
14	10	2	-	-	855	859
15	10	2	-	-	852	759
16	10	2	-	-	812	514
17	10	4	224	-	1324	1085
18	10	4	-	321	807	892
19	10	4	287	413	1387	1207
20	10	4	-	-	-	836

Note: -: too soft to test. *: not included in Figure 9-11.

Table 9-15: The ITSM test results from Series 7

Duration (days)	Curing condition (°C)	With NaOH addition			Without NaOH addition		
		Number of tests	Mean ITSM (MPa)	Standard Deviation (MPa)	Number of tests	Mean ITSM (MPa)	Standard Deviation (MPa)
8	20	6	2301	665	8	2668	418
	10	6	1508	497	8	1832	436
125	20	6	10504	1369	6	8903	2661
	10	6	10247	402	8	9637	1514

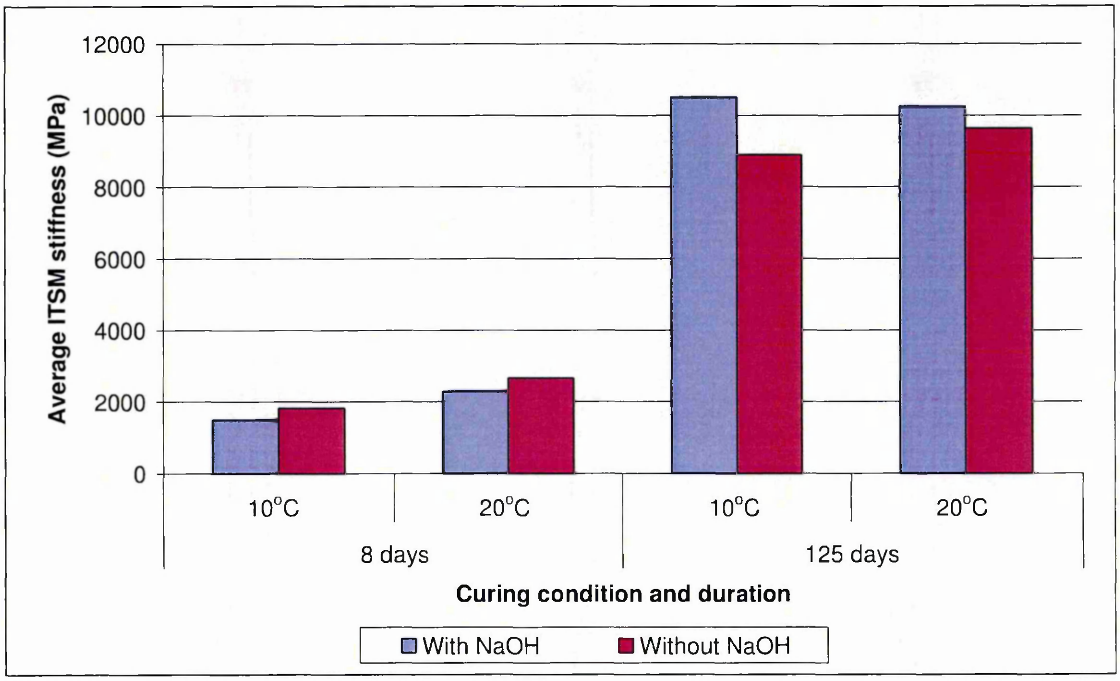


Figure 9-12: The ITSM test result of Series 7

Table 9-16: 95% confidence interval of Series 7 ITSM test

Duration (days)	Curing temperature (°C)	With NaOH		Without NaOH	
		lower 95% confidence interval (MPa)	Upper 95% confidence interval (MPa)	lower 95% confidence interval (MPa)	Upper 95% confidence interval (MPa)
8	20	1603	2999	2319	3017
	10	986	2030	1467	2197
125	20	9067	11941	6334	11472
	10	9825	10669	8371	10903

- Series 8

Series 8 was composed of the same aggregates as those of Series 6, which by visual inspection had a high content of road planings. Lime was added and all the specimens were cured at 10°C. The ITSM test was conducted at intervals and the results are presented in Table 9-17. The experimental results in Table 9-17 and their analysis in Table 9-18 showed that the lime dosage and the duration of curing are highly significant factors influencing the performance of the cold mixes.

Table 9-17: ITSM test for Series 8

Lime content (%)	0			0.25			1.00		
	No. of tests	Mean ITSM (MPa)	Standard. deviation (MPa)	No. of tests	Mean ITSM (MPa)	Standard. deviation (MPa)	No. of tests	Mean ITSM (MPa)	Standard. deviation (MPa)
ITSM @ 26 days (MPa)	-	-	-	-	-	-	10	3562	490
ITSM @ 129 days (MPa)	-	-	-	12	938	286	20	7356	1218

Note: - the specimens were too weak to sustain the ITSM test.

Table 9-18: ITSM 95% confidence interval for specimens with lime

Lime content (%)	0		0.25		1.00	
	Lower bound	Higher bound	Lower bound	Higher bound	Lower bound	Higher bound
95% confidence interval for ITSM @26 days (MPa)	-	-	-	-	3211	3913
95% confidence interval for ITSM @129 days (MPa)	-	-	756	1120	6786	7926

Note: - the specimens were too weak to sustain the ITSM test

9.4.4 F-T test

Sixteen specimens were involved in F-T test. The ITSM test results are summarised in Table 9-19. Clearly, specimens without F-T, with 9 F-T cycles and with 16 F-T cycles have very similar stiffness.

Table 9-19: Comparing ITSM stiffness before and after F-T test

Specimen number	ITSM (MPa)		
	No F-T	9 F-T cycles	16 F-T cycles
1	6216	6664	6848
2	2527	3753	3389
3	5831	4215	4926
4	4473	2243	2984
5	11062	12839	-
6	12065	11823	11068
7	10515	11309	12324
8	8830	8758	9071
9	7976	8789	7166
10	9085	10132	9355
11	7320	8401	7337
12	8868	9158	9460
13	9431	9246	9147
14	5082	4647	6002
15	7528	7983	7688
16	-	7614	8461
No. of specimens	15	16	15
Mean of ITSM (MPa)	7787	7997	7625

Specimen number	ITSM (MPa)		
	No F-T	9 F-T cycles	16 F-T cycles
Standard deviation (MPa)	2615	3016	2614

Note: no ITSM test was undertaken

9.4.5 Re-wetting

After re-wetting, the specimens quickly started hydrating and stiffness reached 2500MPa at around 130 days as shown in Figure 9-13.

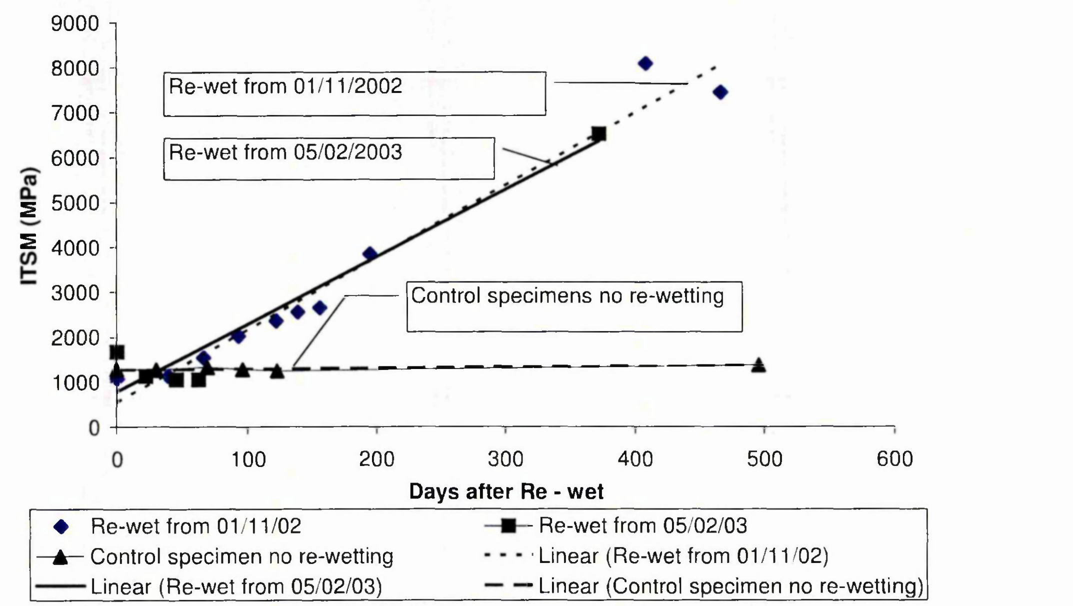


Figure 9-13: Comparing specimens containing GGBS with or without re-wetting

9.5 DISCUSSION

9.5.1 Types and composition of the recycled aggregate

The experimental work up to this stage, including the work conducted by the two undergraduate students (Northwood, 2004) and (Langley, 2006), has demonstrated that the stiffness of the cold mixes comes mainly from the GGBS hydration and the GGBS hydration appears to be closely related to the recycled aggregate type. The GGBS hydration appears to be activated by the residual lime and cement within the concrete demolition. For example Series 6, which is mostly composed of road planings, developed much lower stiffness than Series 7, which is mainly composed of freshly crushed concrete, even with the addition of NaOH solution. Series 2 was composed of recycled aggregate left over from various previous projects which had differing performance, some of the specimens developed enough stiffness for ITSM test to be undertaken after moving into slab compaction chamber controlled at 26°C whilst others failed to develop enough stiffness for ITSM test to be undertaken.

Although freshly crushed concrete will hydrate and produce high stiffness, it can significantly reduce shelf life, as demonstrated by Series 7 and the work conducted by the two undergraduate students. Norwegian researchers Jostein and Telle (2000) suggested stockpiling the aggregate for at least 1 to 2 months before production of the cold mix and the mix design being carried out using aggregates which had been stored for the same period of time. The Methylene Blue Test (BS EN 933-9, 1993) was conducted to characterise the activity of the recycled aggregate in their research and showed that aggregate surface reactivity will stabilise after 1-2 months (Figure 9-14). In the UK, the weather is wet and rainy, and therefore the crushed aggregate could be expected to stabilise in less time than that in Norway where the weather is drier and colder. The crushed bricks have no clear contribution towards the performance of the cold mixes and can only be regarded as a finely graded balancing constituent.

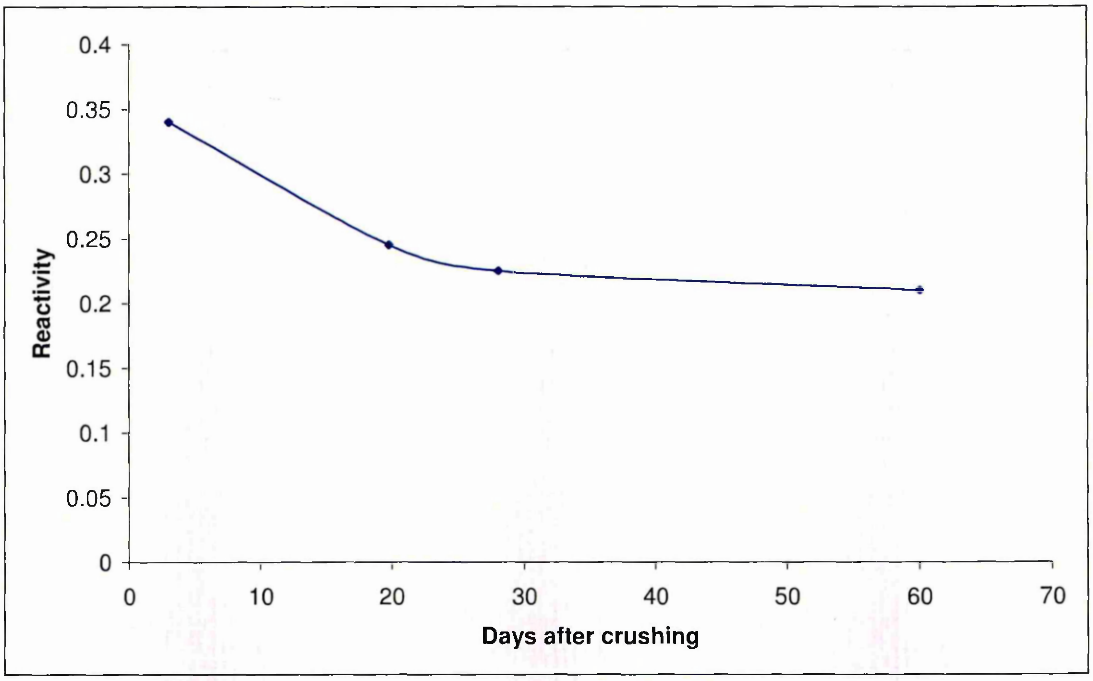


Figure 9-14: Aggregate surface activity diminishes over time (Jostein and Telle, 2000)

9.5.2 Curing temperature

Three temperature levels had been employed up to this stage, 10°C, 20°C and 26°C. The temperature appeared to be a factor fundamentally influencing the performance of the cold mixes.

Figure 9-15 compared the stiffness development of three groups of specimens with the same binder content, 4% GGBS and 2.7% bitumen, from the Pilot scale trial (Chapter 7), Series 1 (Chapter 8) and Series 4 respectively. All three batches of specimens were cured in a condition with relative humidity close to 100%. Series 4 developed the highest stiffness at the same duration, mainly because of the high curing temperature.

In Series 6 and 7, specimens with the same binder and accelerator content were split into two groups curing at 10°C and 20°C respectively. The specimens cured at 20°C developed much higher stiffness than those curing at 10°C, as revealed in Figure 9-11 and Figure 9-12, confirmed the significant influence of temperature.

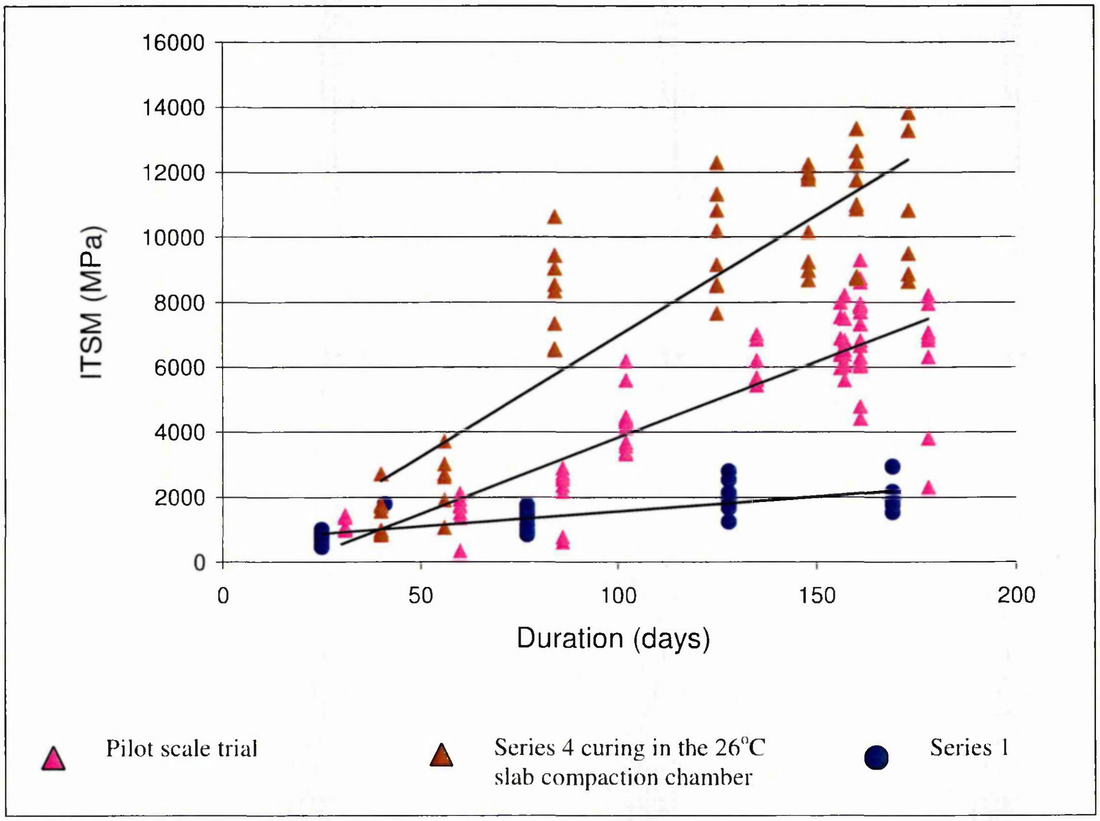


Figure 9-15: Comparing stiffness development of Pilot scale trial, Series 1 and specimens cured at 26oC

9.5.3 Lime and sodium hydroxide solution as accelerators

Both lime and NaOH solution were shown to be effective as accelerators. Two batches of specimens, Series 5 and 8, were compacted with the addition of lime. As revealed in Table 9-11 and Table 9-17, the addition of lime increased the GGBS hydration without contributing to the final stiffness and reduced the variability between specimens as revealed in Figure 9-10. The specimens with lime also had a smoother surface texture and reduced shelf life than those without. NaOH solution proved to be an effective accelerator judging by the test results from Series 6 and 7. NaOH solution was made by mixing Na_2O with water, which was found to be an exothermic reaction and only 2mol/litre and 4mol/litre solution were produced.

In the industry, lime is commonly employed as an accelerator for latent hydraulic binders, such as GGBS and PFA, but the addition of lime will reduce shelf life. In order to maintain shelf life, lime could only be added in the application process when shelf life is not a requirement. The additional process of mixing lime with cold mixes in the application process will increase the cost of the cold mixes application. NaOH solution as a liquid can be sprayed on the cold mixes in the application process, hence reducing the potential mixing cost.

9.5.4 Contribution and interrelation between bitumen emulsion and GGBS

Up to this stage, all the experimental work revealed that the main contribution to the stiffness/strength of the cold mixture comes from the GGBS hydration, whilst the bitumen emulsion only delays the strength/stiffness development. The contribution from bitumen emulsion, GGBS and the interrelation between the two factors is illustrated in Figure 9-16, with data from the ITSM test results from Series 4 as an example.

9.5.5 F-T test

An F-T test was conducted, employing the same weathering chamber and same temperature regime as F-T test for the cold mixes with bitumen emulsion as binder (no hydraulic binder), 24 hours per F-T cycles, at -5°C for 10 hours and 5°C for 8 hours and the rest of the time in transition. The test proved that the specimens containing hydrated GGBS were F-T resistant, which represents an improvement compared with the specimens with bitumen emulsion as binder, which proved to be sensitive to F-T as concluded in Paragraph 7.3.2.

9.5.6 Re-wetting

The re-wetting test of specimens containing GGBS has shown that specimens with GGBS are capable of resuming hydration after re-wetting. Even after prolonged curing at a low relative humidity condition, the final strength is similar to those specimens which had been curing in wet conditions.

9.5.7 Compressive strength

For cement bound road base materials, the characteristic compressive strength at

7 days is normally required to be over 10MPa, which corresponds to CBM3 and above as detailed in Table 9-20. As indicated in Table 9-13, the long term compressive strength of specimens containing 4% and 4.6% GGBS reached 14.3MPa and 23.9MPa. According to TRL report 615 (Nunn, 2004), the compressive strength at 7, 28 and 360 days can be estimated from the data in Table 9-21. Therefore, the long term compressive strength of CBM3 materials can be estimated at 15MPa. This figure is close to the compressive strength of specimens with 4% and 4.6% GGBS. Therefore, cold mixes with over 4% GGBS are regarded as suitable paving materials for road base.

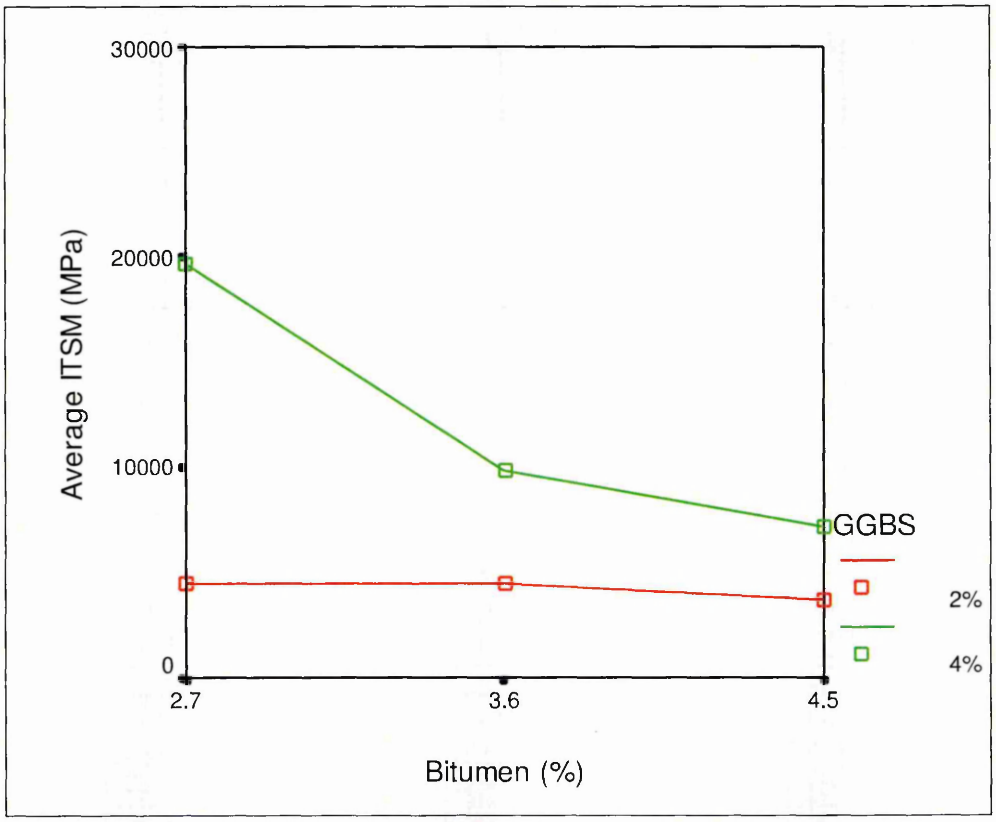


Figure 9-16: The effect of GGBS and bitumen content on ITSM

Table 9-20: Compressive strength of cement bound materials (Table 10/9, MCHW1)

Category	Minimum 7 days compressive strength (N/mm^2)	
	Mean	Minimum
CBM1	4.5	2.5
CBM1A	10.0	6.5
CBM2	7.0	4.5
CBM2A	10.0	6.5
CBM3	10.0	6.5
CBM4	15.0	10.0
CBM5	20.0	13.0

Table 9-21: The relation between curing period and compressive strenth

Curing period (days)	Relative compressive strength of CBM
7	0.67
28	0.8
360	1

9.6 SUMMARY

Various factors influencing the cold mixes performance were investigated in this part of the research, including:

- Aggregate type and composition: road planings, concrete demolitions, bricks.
- Curing temperature: 10°C, 20°C and 26°C.
- Freezing and thawing: -5°C to 5°C.
- Accelerators: NaOH solution and Lime.
- Re-wetting: from low RH condition to high RH condition.

In addition to the ITSM test, a compressive strength test was also employed to investigate the effect of lime, residual bitumen and the GGBS.

The experimental work has demonstrated that the cold mixes composed of a balanced composition of asphalt planings and demolished concrete with bitumen emulsion and GGBS as binder can achieve the required shelf life and stiffness as reinstatement materials. GGBS hydration is activated by the residual lime or cement within the demolished concrete. Without demolished concrete, the GGBS cannot hydrate. With freshly crushed concrete demolition as aggregate, the cold mixes have very limited shelf life. The recycled aggregate should therefore ideally be a mixture of asphalt planings and crushed concrete demolition.

With the addition of GGBS, the cold mixes are F-T resistant and can regain hydration when the curing conditions are favourable.

Up to this stage, most of the experimental work demonstrated that the stiffness/strength mainly attributes to the GGBS hydration while bitumen emulsion only slows down such hydration process. No positive contribution from bitumen emulsion towards the development of stiffness could be identified at this stage. The following chapter focuses on the contribution of bitumen emulsion to the mechanical properties.

CHAPTER 10 FATIGUE AND INDIRECT TENSILE STRENGTH

10.1 INTRODUCTION

The ITSM test had been the primary performance test conducted up to this juncture and the benefits and contributions from the addition of bitumen emulsion were not well reflected. For example, in the cold mixes, the fines are coated with bitumen and less likely to segregate from the coarse aggregate in the stockpiling, transportation and paving process. The bitumen also contributes some initial stiffness to the mixture after compaction.

The objective of this chapter is to ascertain the contribution from bitumen emulsion by other test methods. In order to consider the contribution from bitumen emulsion, the following tests were conducted:

- Indirect Tensile Fatigue test (ITFT)

Fatigue cracking, originating from the bottom of the pavement slab, has been regarded as one of the major reasons for pavement failure. As introduced in Chapter 5, there are several test methods relating to fatigue property. The ITFT is conducted with the Nottingham Asphalt tester, which is the same facility as for ITSM and RLAT test.

- Indirect Tensile Strength test (ITS)

The ITS test is normally employed to assess the moisture sensitivity of the hot mixed materials. This test can be conducted with adapted Marshall test equipment.

The test procedure and application of ITFT and ITS test have been broadly introduced in Chapter 5 and are further detailed in the following sections.

10.2 INDIRECT TENSILE FATIGUE TEST (ITFT)

10.2.1 Experimental design

Unlike the ITSM test, where the same specimens can be monitored over time, the specimens in the ITFT test are loaded to failure. The final result of the ITFT test is a plotted line representing the strain versus cycles to failure at that strain/stress level. Typically, 12 specimens are required to plot a regression line.

GGBS is a latent hydraulic binder. The cold mixes with latent hydraulic binder behave more like unbound granular materials at first and transform into hydraulically bound materials gradually. During the transition process, the physical performance including fatigue resistance varies over time. A large number of specimens could be required to monitor the fatigue properties over this transition period. Considering this practical problem, the ITFT test was conducted on cold mixes with bitumen emulsion and cement as binder first, with the intention of going on to cold mixes with bitumen emulsion and GGBS as binder if successful. The reason for testing Portland cement instead of GGBS at this stage was because both Portland cement and GGBS are hydraulic binders, but Portland cement hydrates much quicker than GGBS, thus making it easier to repeat the experiment within a reasonable time scale.

In the NAT fatigue test, repeated loads are exerted on 100mm diameter specimens until failure, either using a constant applied load or stress (termed controlled stress), or constant deflection or strain (termed controlled strain). In the controlled stress ITFT test, the maximum load or stress is held constant during the test and the resultant strain or deformation increases until failure occurs. In contrast, for tests of the constant strain type, the strain level is maintained constant until fracture is reached. The NAT tester used in this project can only conduct controlled stress fatigue tests. A group of typical fatigue lines for SMA and HRA with 40/60 pen bitumen as binder are presented in Figure 10-1 as an example.

In the ITFT test, the strain level is calculated using Equation 10-1.

$$Maximum\ tensile\ strain = \frac{Tensile\ Stress}{Stiffness\ Modulus}(1 + 3 \times Poisson's\ ratio) \times 1000$$ Equation 10–1

The Poisson's ratio is the strain at right angles to the load divided by the strain in the direction of the load. A Poisson's ratio of 0.35 is used in this project. The Poisson's ratios suggested by the Highways Agency for pavement design are summarised in Table 10-1 (HD29/94, DMRB7).

Table 10-1: Poisson's ratio for use in back-analysis

Material	Poisson's ratio
Bituminous material	0.35
Cement bound	0.20
Crushed stone	0.40
Soils	0.45

The maximum tensile strain is the tensile strain at the centre of the specimen, induced by the tensile stress calculated as Equation 10–2:

$$Tensile\ \ Stress = \frac{2 \times vertical\ \ load\ \ applied\ \ (kN)}{\pi \times diameter\ \ of\ \ the\ \ specimen\ \ (mm) \times thickness\ \ of\ \ the\ \ specimen\ \ (mm)}$$

Equation 10-2

The tensile stress in Equation 10-2 is the maximum horizontal tensile stress at the centre along the line of the specimens (kPa).

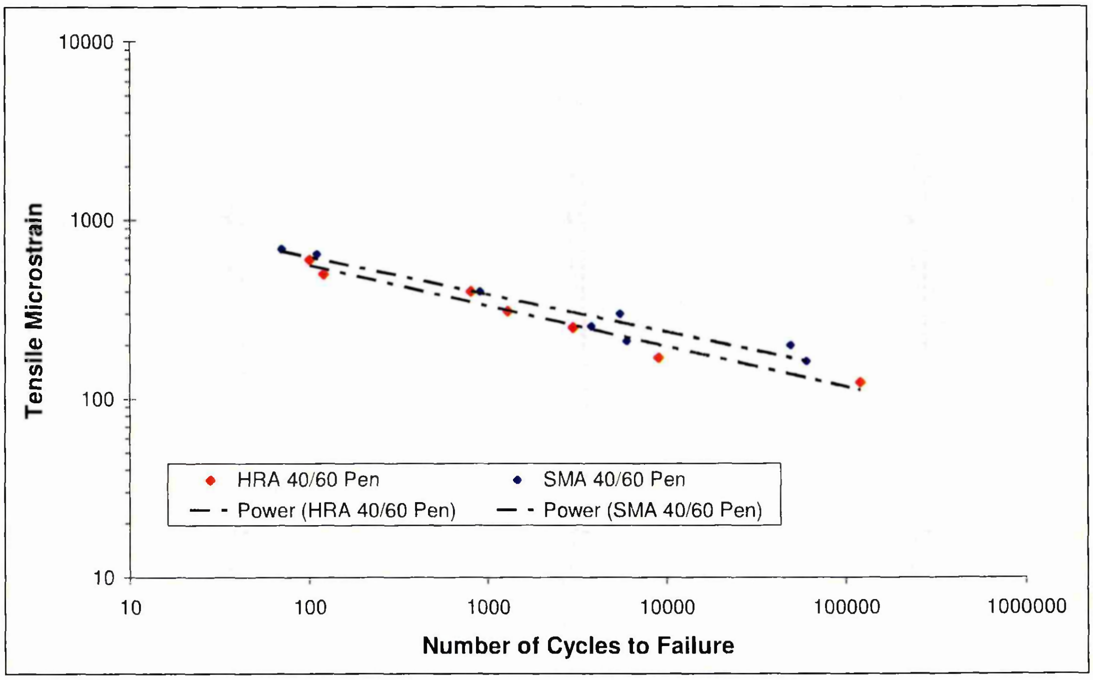


Figure 10-1: Strain - Cycles to failure relationship (Elliot et al, 2005)

For visco-elastic materials, the stiffness modulus varies with the stress applied. In controlled stress ITFT tests, specimens are loaded at different stress levels until failure, and therefore the ITSM corresponding to that specific stress level has to be applied in Equation 10-1 to calculate the strain level. A relationship between the stress and the stiffness modulus is therefore necessary. *'If stiffness is plotted against stress using the ITSM results, a relationship of the form shown below should be obtained (Figure 10-2). As stress increases, stiffness modulus should decrease. The relationship may be curved or linear depending on the materials under test (Nottingham Asphalt Tester Manual, 1994).'* This would also apply under normal stiffness testing.

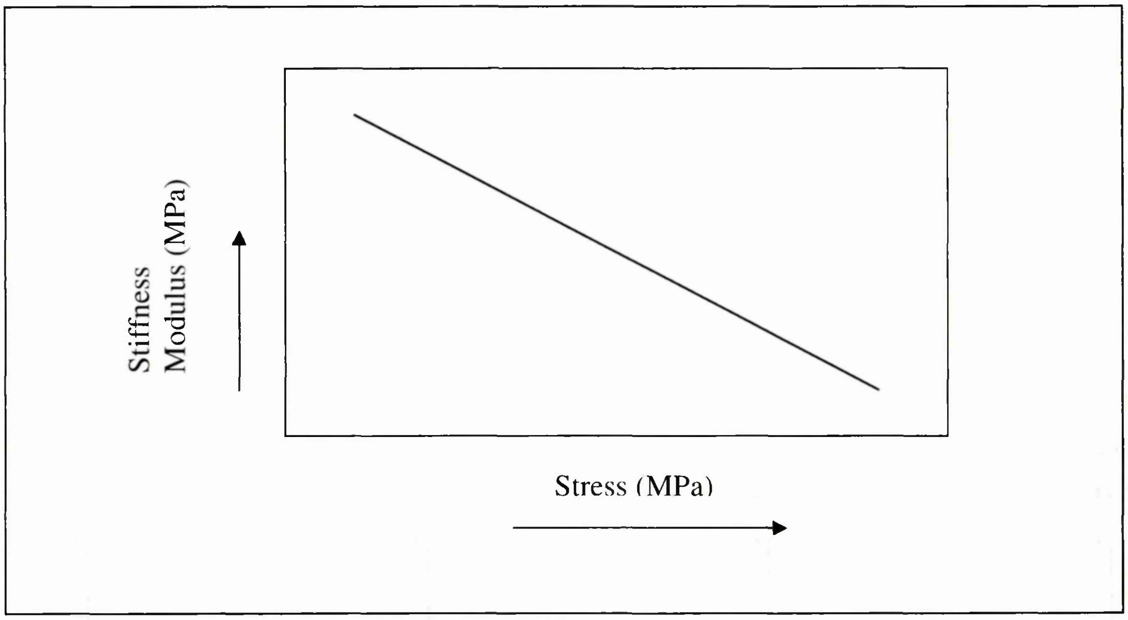


Figure 10-2: The relationship between stiffness and stress

10.2.2 Experimental process

Three batches of specimens were compacted for the fatigue test, two batches with bitumen emulsion as binder and one batch with bitumen emulsion and Portland cement as binder, labelled as Batch 1, Batch 2 and Batch 3 in Table 10-3. Firstly, the ITSM test was conducted at various stress levels in order to find out the relation between stress and stiffness modulus.

The BS DD ABF (1999) suggested that it was preferable to set horizontal deformation at 5, 9, 13 and 17μm respectively. Following this suggestion, the ITSM test was conducted with horizontal deformation set at the above stated levels and the tensile stress was calculated following Equation 10-2.

10.2.3 Results

Figure 10-3 presents the relation between the horizontal stress and the stiffness modulus. The correlation coefficient is too low for the linear relation between stress and stiffness modulus to be viable. This is different from the claim made by the NAT test manual. It may be that the claim in the NAT test manual is based on hot bitumen specimens and such a claim is not necessarily suitable for cold mixes. Since the relation between stiffness modulus and stress is the foundation for the calculation of tensile strain, without a reliable linear relation between stiffness modulus and stress, the tensile strain at the centre of the specimens cannot be calculated accurately or reliably based on Equation 10-1.

Nevertheless, the ITFT test was conducted on various specimens. Firstly, the test was conducted on specimens with bitumen emulsion as binder. The starting stress level of the ITFT test was around 180MPa. At this stress level, the specimens with bitumen emulsion as binder could not endure many load cycles before breaking; increasing the stress level only shortening the breaking process. The bitumen emulsion specimens were too weak to sustain the ITFT test.

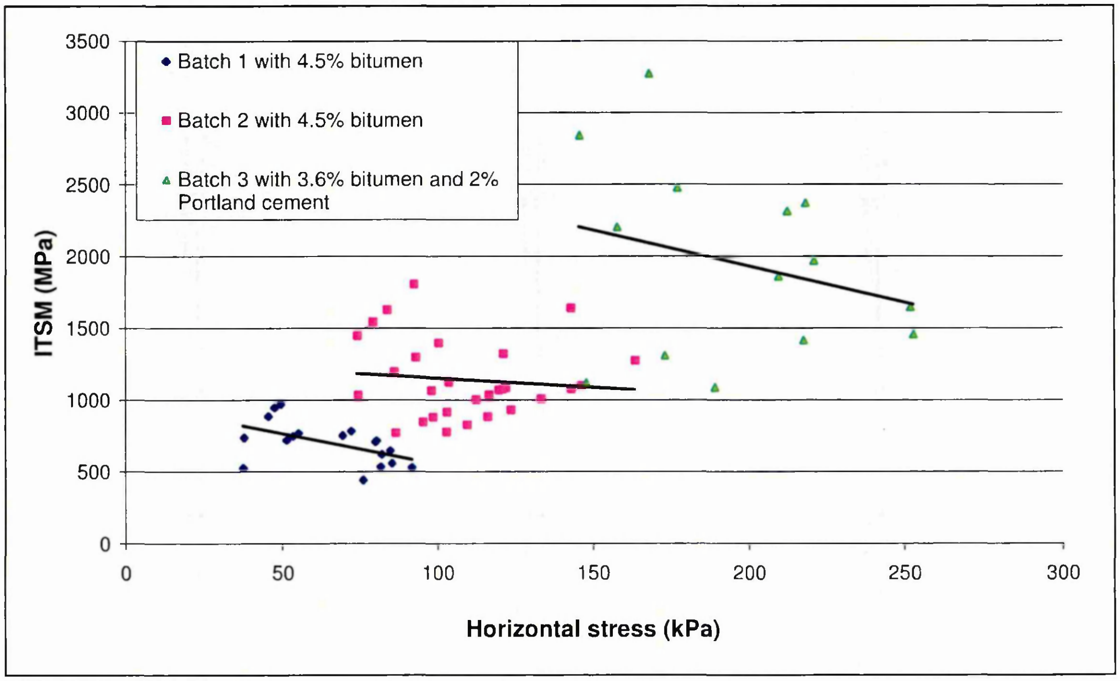


Figure 10-3: Relation between horizontal stress and stiffness modulus

10.2.4 Discussion

Considering there is no linear relation between stiffness modulus and stress, and the specimens with bitumen emulsion as binder were too weak to conduct the ITFT test, the ITFT test was not pursued further.

Having failed to reveal the contribution from bitumen emulsion upon the mechanical properties by the ITFT test, an ITS test was conducted in an attempt to assess the ductility or brittleness of the cold mixes, as a surrogate assessment of fracture toughness.

10.3 INDIRECT TENSILE STRENGTH (ITS) TEST

10.3.1 Experimental design

The ITS test was employed after both ITSM and ITFT tests had failed to reveal the contribution towards the mechanical performance from bitumen emulsion. It was proposed that the ITS test might reveal the ductility/toughness characteristics of cold mixes, including tensile strength. The ITS test is mostly used to test the resistance of compacted bituminous mixture to moisture induced damage, as described in AASHTO DESIGNATION: T283-99.

The ITS test facility was adapted from the Marshall Test facility (Figure 10-4). The two half-circled jaws in the Marshall Test were replaced by two loading strips. The specimen, cored or lab-compacted, is placed vertically within the two loading strips. According to the BS EN 12697-23 (1999), the diametric load should be applied continuously and without shock, at a constant rate of deformation of 50±2mm/min until the peak load is reached. In this project, instead of 50±2mm/min, the rate of the Marshall test, a deformation rate of 1mm/min, the rate for California Bearing Ratio (CBR) test, was eventually adopted as the most suitable to enable a logger to be used to record the load and deformation during the test process. At Marshall speed, the break would be instant and the logger used would be unable to record enough data to reflect the whole breaking process.

With the logger, both vertical load and deformation was monitored. The tensile stress at the centre of the specimen was calculated using Equation 10-2, assuming the specimen was broken by the tensile stress at the centre of the specimen.

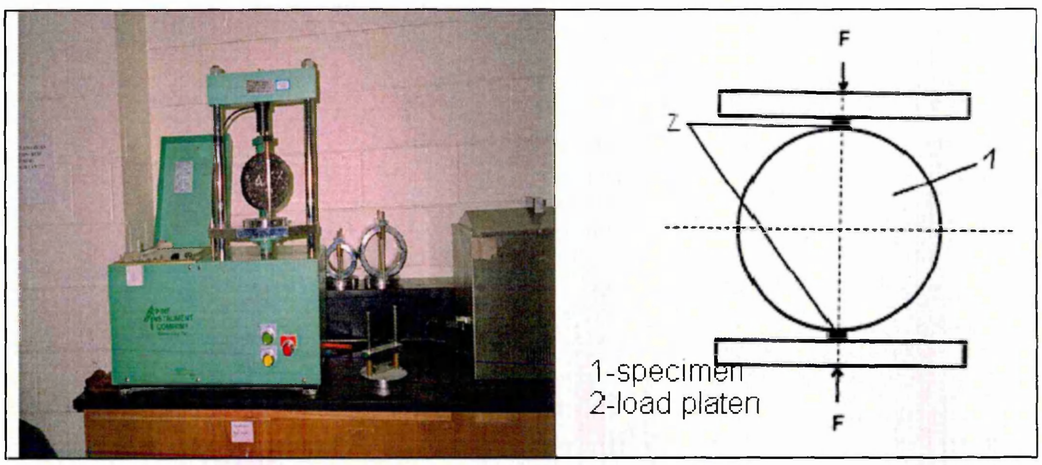


Figure 10-4: ITS test facility

10.3.2 Experimental process

The ITS test was conducted on specimens with bitumen emulsion, bitumen emulsion/GGBS, bitumen emulsion/cement and hot bitumen as binder(s). The purpose of including specimens with bitumen emulsion/cement and hot bitumen as binder here was that these materials were already widely accepted and understood by the industry and it was helpful for further understanding the bitumen emulsion/GGBS materials by comparing them with more conventional materials.

- Specimens with cement as binder

Cold mixes with 3.6% residual bitumen and 2, 4 and 6% Portland cement were mixed and specimens were manufactured on 24th September 2002. The cold mixes were compacted soon after mixing with the Marshall Compactor and were divided into two subsets: one set was cured in the mist room (high RH, i.e. RH>90%) and the other in the laboratory (low RH, Figure 7-2).

Since specimens are tested to failure in the ITS test process, an ITSM test was conducted on all the specimens before undertaking the ITS test. The ITS test on this batch of specimens was conducted on 14th March 2003, 136 days after the specimens were manufactured.

- Specimens with GGBS as binder

The ITS test was also conducted on specimens containing GGBS. The specimens employed were the Series 4 specimens, which had cured in the 26°C slab compaction chamber. The specimens were conditioned at 20°C before testing. The detailed composition and ITSM test results of Series 4 specimens are detailed in Chapter 9.

- Specimens with hot bitumen binder

Hot mixed dense bitumen macadam (DBM) is the most widely used base/binder course material in road construction. Comparing the behaviour of cold mixes with hot mixes is helpful for further understanding the properties of the cold mixed materials. Two tins of 100Pen bitumen, the original bitumen used to make the bitumen emulsion, were supplied by one of the collaborating companies in this project to compare the performance of cold and hot mixes, based on the same binder.

The hot mixes specimens were compacted with the Marshall compactor at 50 blows on each side in the same way as cold mixes. It was found that the aggregate could not be coated properly with less than 3.6% bitumen, possibly because the hot bitumen coat on the aggregate was thicker than that of cold mixes.

The ITSM test was conducted upon the specimens before undertaking the ITS test.

10.3.3 Results

The ITSM test was conducted on all the specimens before the ITS test.

- Specimens with Portland cement as binder

The ITSM test results are summarised in Table 10-2 and graphically presented in Figure 10-5. The results of ITS test are presented in Table 10-3.

- Specimens with GGBS as binder

The ITS test results for specimens with GGBS are presented in Table 10-4.

Table 10-2: ITSM for specimens with bitumen emulsion and cement as binder

Duration (days)	Cement content (%)	Laboratory Curing (Low RH)		Mist Room Curing (High RH)	
		Mean ITSM (MPa)	Number of tests	Mean ITSM (MPa)	Number of tests
43	2	2248	5	2587	6
	4	3656	4	3979	14
	6	4880	4	5286	10
75	2	2335	5	2904	6
	4	3020	4	5118	6
	6	5445	5	6396	8
136	2	2126	7	3065	6
	4	3344	5	5176	8
	6	5374	7	6539	8

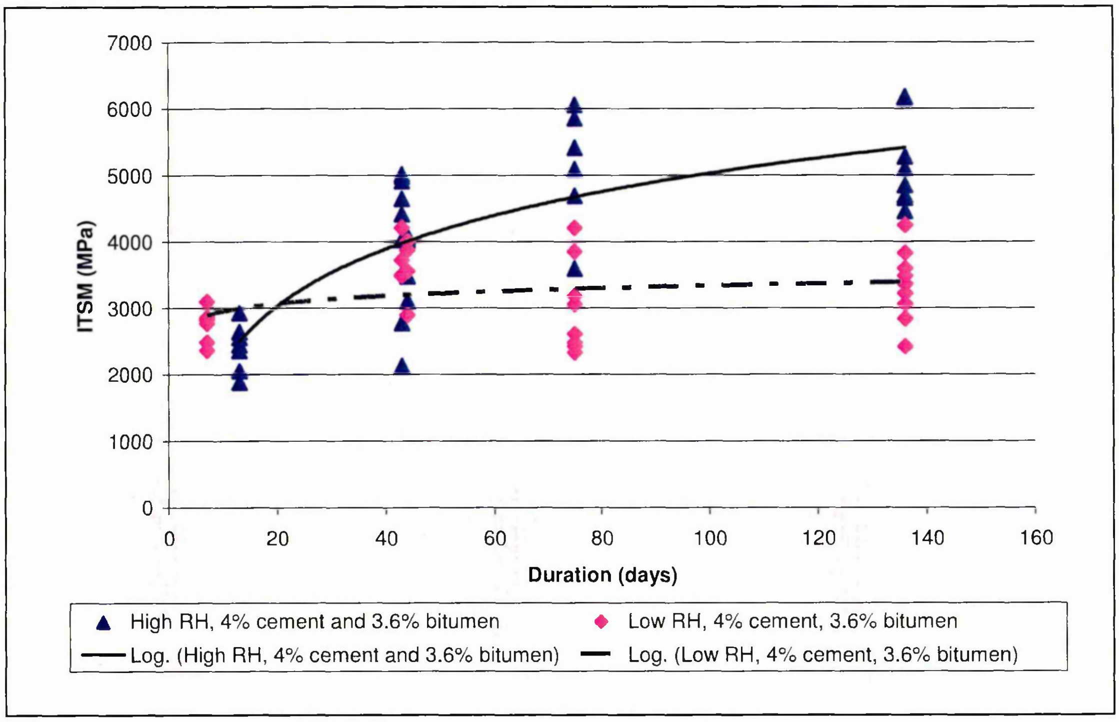


Figure 10-5: ITSM evolution comparison for specimens curing at high and low RH conditions

Table 10-3: Summary of ITS test results (bitumen at 3.6% for all specimens)

Cement Content (%)	Bitumen Content (%)	Number of specimens	Curing Condition	Strain at failure x 10^{-3}	Peak Tensile Stress (kPa)
2	3.6	2	Mist room	5.88	248.2
		3	Laboratory	6.94	196.9
4	3.6	3	Mist room	5.75	332.1
		3	Laboratory	7.10	343.9
6	3.6	3	Mist room	6.00	537.7
		3	Laboratory	7.03	448.3

Note: Mist room – RH over 90%, Laboratory – low relative humidity, refers to Figure 7-2.

Table 10-4: ITS test for specimens with bitumen emulsion and GGBS as binders

	Number of specimens	Bitumen (%)	GGBS (%)	Strain at Failure x 10^{-3}	Peak Tensile Stress (kPa)	ITSM (MPa)
Bitumen effect	2	0	4	5.4	733.1	13937
	3	2.7	4	6.6	240.6	4188
	3	4.5	4	9.5	111.0	1166
GGBS effect	2	2.7	2	8.7	89.4	1311
	3	2.7	4	6.6	240.6	4188

- Specimens with hot bitumen binder

ITSM test, ITS and void content test were conducted on the hot mixes specimens and the results are presented in Table 10-5.

Table 10-5: Comparing hot and cold bitumen specimens by ITS and ITSM test

Binder	Number of specimen	Bitumen content (%)	Mean strain at failure (10^{-3})	Mean peak tensile stress (kPa)	Mean ITSM (MPa)	Mean voids (%)
Hot bitumen	3	2.7	7	501	2903	16
	3	3.6	11	627	3636	13
	3	4.5	12	683	3109	10
	3	5.5	15	597	2728	5
Cold bitumen	3	4.5	10	148	1232	>10

10.3.4 Discussion

- Specimens with Portland cement as binder

The ITSM test revealed that the specimens in the mist room developed a higher ITSM value than the specimens cured in the laboratory (Table 10-2) and the ITS test indicated that the specimens cured in the laboratory broke at lower peak tensile stress and higher strain levels (Table 10-3). The most plausible explanation for the result was that, in the mist room with relative humidity over 90%, the there was sufficient water supply to sustain the cement hydration and therefore high hydraulic bonds were developed. However, the bitumen globules were unable to break and connect with each other at high humidity conditions, hence limited bituminous bonds could be developed. With high hydraulic bonds from cement and low bituminous bonds from bitumen emulsion, the specimens broke at high tensile stress but at a low tensile strain level in the ITS test. On the other hand, in the laboratory, the relative humidity was low as presented in Figure 7-2 and there was not enough water supply to sustain the continuous cement hydration, therefore the hydraulic bond could not be fully developed whereas the bitumen globules could set and work as a cohesive binder after the evaporation of the water. With low hydraulic bond and some cohesive bond from bitumen, the specimens cured in the laboratory broke at lower tensile stress but higher tensile strain level in the ITS test than the specimens cured in the mist room.

Based on the above test result analysis, the bitumen emulsion contributed towards the ductility of the specimens as follows:

- Specimens with GGBS as binder

As indicated in Table 10-4, at the same GGBS content, for example with 4% GGBS, the tensile strain levels at break increased with increasing bitumen content and the tensile stress levels at break decreased with increasing bitumen content. It also indicated that at the same bitumen content, for example, 2.7% residual bitumen, the higher the GGBS content, the lower the strain level and the higher the stress level at break. The test results again suggested that bitumen contributes to ductility and GGBS contributes to stiffness/strength.

- Specimens with hot bitumen binder

The hot mixes had better workability hence produced specimens with lower void content. The void content of the hot bitumen specimens decreased with increasing bitumen content, with a minimum of 5% and a maximum of 16%, whilst the void content for cold mixes was always over 10% for bitumen contents ranging from 2.7% to 5.5% under similar compaction force (Table 10-5).

- Hot vs. cold vs. hydraulic binder

The whole ITS test process, from exerting load until the specimens broke into two parts, was monitored for all of the specimens tested and only three were selected as representatives presented in Figure 10-6. Comparing specimens with hot and cold bituminous binder, at the same bitumen content, the specimens with hot bitumen as binder failed at higher stress and strain levels than specimens with cold bitumen. In the ITSM test, specimens with hot bitumen developed higher ITSM value. In general, specimens with hot bituminous binder performed much better than their cold mixes counterparts at comparable binder contents (Table 10-5, Figure 10-6).

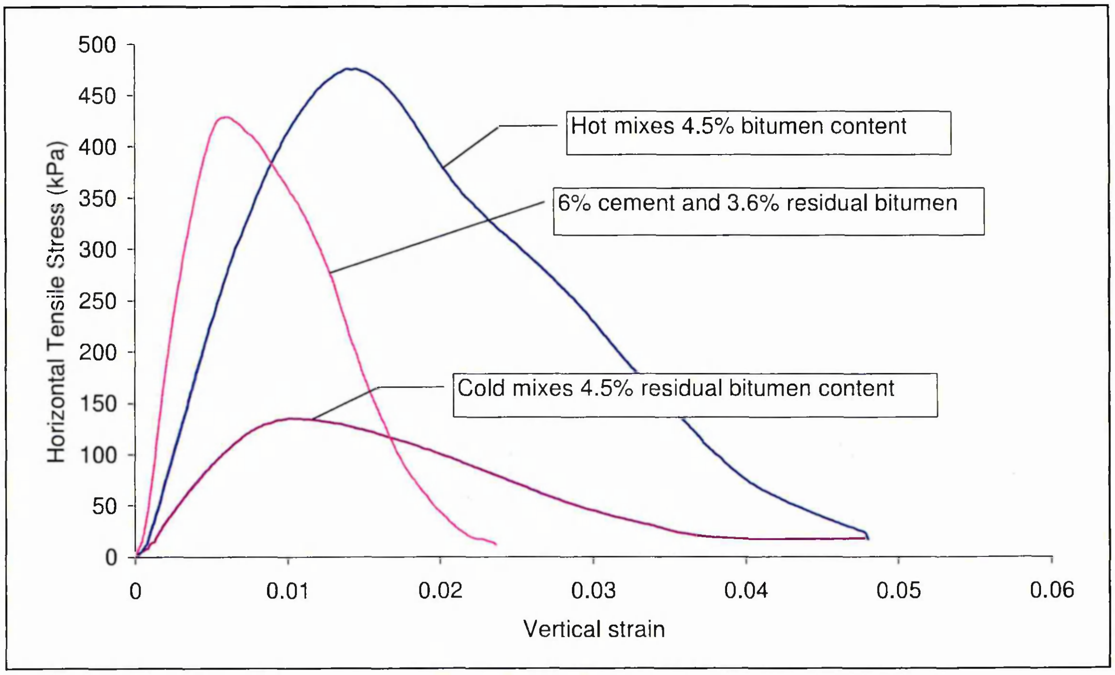


Figure 10-6: Comparison of the bitumen emulsion, bitumen emulsion/cement and hot bitumen specimens with ITS test

The area under the stress-strain curve in Figure 10-6 is representative indicative of the energy expended to break the specimen, which can be calculated with Equation 10-3 as follows:

$$S_{area} = \sum_{i=1}^{N} strain_i \times stress_i \qquad \text{Equation 10–3}$$

Where:

S_{area}: the area under the curve.

$Strain_i$: strain level corresponding to i_{th} log point in the ITS test.

$Stress_i$: stress level corresponding to i_{th} log point in the ITS test.

N: total log points in the ITS test.

The area under the stress-strain curve can be further divided into two parts as illustrated as Figure 10-7:

- S1: area from test start to peak stress;
- S2: area from peak stress to the specimen fell into two parts;

The total area of S1+S2 represents the energy expended to break the specimen, and may be otherwise known as the work to fracture or rupture.

As shown in Table 10-6, the total area under the curve for hot mix specimen is much larger than that for specimens with bitumen emulsion or with bitumen emulsion/cement as binder, indicating that specimens with hot bituminous binder have superior properties than the other two. Also indicated in Table 10-6, the energy consumed by the cold mixes specimens with and without cement were in a similar order of magnitude. The specimens with hydraulic binder are stiffer and more brittle, although the breaking stress of the specimens with hydraulic binder approached that of the hot mixes, the specimen broke at a much lower strain level than specimens with hot bituminous binder, which gives an indication of its brittleness.

Table 10-6: Area under Stress – Strain curve

	Hot mixes (kNm/m^3)	Cold bitumen emulsion mixes (kNm/m^3)	Cold cement/bitumen emulsion mixes (kNm/m^3)
S1	249	33	43
S2	1012	295	218
S1 + S2	1261	328	261

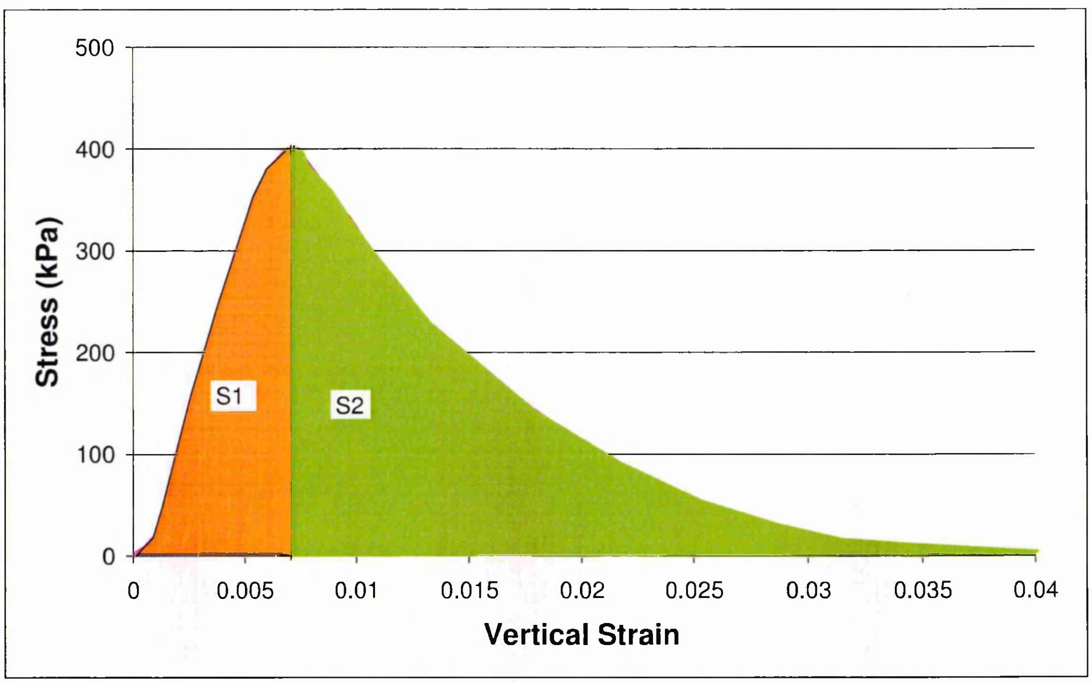


Figure 10-7: Division of area under breaking curve

10.4 DISCUSSION

Needham (1996) and Ibrahim (1998) have successfully conducted fatigue tests on cold mixes with the NAT tester. Both of them adopted a controlled stress method, but their NAT tester was adapted to obtain the ITSM simultaneously with vertical strain at the differing stress levels used in the ITFT test. As a result, they did not have to develop a relationship between the stiffness modulus and stress. Unfortunately, it was not feasible to adapt the NAT tester used in this project, because of the lack of resources and the opportunity for achieving a timely modification.

Ibrahim (1998) concluded that '*generally, the mixtures showed poor fatigue resistance whatever the level of residual bitumen content (2.2 - 5%).*' He blamed the poor adhesion between the bitumen binder and the aggregate particle for the poor fatigue resistance. Needham (1996) successfully compared the fatigue properties of bitumen emulsion specimens with and without cement. He claimed that in the case of an initial strain level less than 200μm, the addition of cement could improve the fatigue resistance. Nunn and Thom (2002) also looked at the fatigue properties and their fatigue test data showed a high degree of variation. They claimed that '*it is possible that either the ITFT is not relevant to foamed bitumen mixes or that the concept of fatigue for this type form of construction needs to be reconsidered.*' Although their research was based on the foamed bitumen mixes, introduced in paragraph 4.3.4, considering the similarity between foamed bitumen and bitumen emulsion mixes, the conclusion may also be applicable to bitumen emulsion bound materials. It can be seen that the views of different researchers, who actually came from same research institute, vary towards the effectiveness of ITFT test on the performance of the cold mixes.

It is proposed that the most important contribution from bitumen emulsion may be to inhibit the crack propagation, which often happens to hydraulic bound materials. Issr et al (2001) investigated the properties of the cold mixes with bitumen emulsion and cement as binders and claimed that '*these composites mitigated the high brittleness of cement and the strength temperature susceptibility of asphalt concrete. The aforementioned properties have been the contributing factors to transverse cracking in flexible pavement.*' The hydraulic bound materials, although providing high stiffness, are liable to crack owing to the thermal expansion/contraction or uneven strength development or excessive strains. When used as base/binder course materials, the developed cracks tend to penetrate up to the top of the road surface under repeated traffic load. These cracks are normally referred to as 'reflective cracks'. Such cracks may form a channel for water to penetrate into the foundation, softening the base and foundation, and eventually inducing pavement damage. Because of this problem, all CBMs that have an average 7 days compressive strength over 10.0 N/mm^2 must have surface cracks induced during construction, normally spaced at 3 metre intervals (HD26/01, DMRB7). The addition of bitumen emulsion is potentially able to alleviate this cracking problem by imparting ductility through the inclusion of visco-elastic binder to the cold mixes.

10.5 SUMMARY

The aim of this section was to ascertain the contribution to the mechanical properties of bitumen emulsion. The ITFT test was conducted first and, in this particular situation, was found to be not very suitable for assessing the performance of the cold mixes. Instead a modified ITS test was employed as an alternative, to assess the ductility of the materials.

The work in this section has demonstrated that hot mixes have superior properties in terms of stiffness and ductility to that of cold mixes with or without hydraulic binder. Cold mixes with or without hydraulic binder consume similar energy to break. However, specimens with hydraulic binder broke at higher stress levels and specimens with bitumen emulsion as binder (no hydraulic binder) broke at higher strain level, suggesting that bitumen emulsion has a contribution towards ductility.

The ITS test is rarely used in the industry for cold mixes design purposes. Research at this stage has revealed some advantages of the ITS test in revealing the cold mixes performance, but further work is needed to explore and interpret the test result and link it to cold mixes design.

CHAPTER 11 PAVEMENT DESIGN WITH COLD MIXES

11.1 INTRODUCTION

The aim of this chapter is to propose a pavement design method based on the performance of cold mixed materials revealed in this research.

When this research project commenced in January 2001, the application of cold mixed recycled materials in highway pavement was in its early stages in the UK. The companies involved in this project were principally focussing on the reinstatement market. With encouragement from the Highways Agency (HA) and the Local Authorities, cold mixes recycling has become a default design option for roads with design traffic less than 30msa, which covers most of the road network controlled by Local Authorities. This is a much larger market than that offered by simple reinstatement. Frequently, the limiting factors for cold recycling are not technical issues but traffic management, manoeuvrability of facilities and pollution, such as the possible spreading of cement or lime powder causing nuisance in densely populated areas. In the UK, cold recycled bitumen bound material most frequently refers to foamed bitumen with the addition of active or latent hydraulic binder. However, considering the similarity between foam bitumen and bitumen emulsion cold mixes (the comparison between the two is summarised in Section 4.3.6), the design method for foamed bitumen can actually be adapted for bitumen emulsion cold mixes.

'*Pavement design is a process of determining the most economical combination of layer thickness and material types to enable a pavement construction to carry the design traffic loading, to sustain climatic conditions and to take into account the properties of the natural subgrade (Werkmeister et al., 2004).*' There are three basic approaches for pavement design, empirical, semi-empirical and analytical design. There has been bitter debate about the relative merits of empirical and analytical pavement design methods. Pavements have traditionally been designed using empirical design methods, i.e. from the results of full-scale experiments and the detailed study of the performance of in-service roads. Although proven successful, empirical methods are in most cases inadequate for untested situations, for example, the ever increasing traffic volumes are well beyond experiment and new materials such as recycled materials do not fall within the traditional categories which have been proven. On the other hand, analytical design is much easier to extrapolate into heavier traffic volumes and adapt to new materials, especially in the pavement maintenance process.

Currently both the pavement and the foundation design curves presented in the relative specifications are calculated based on the versatile pavement design method. The specifications for pavement design (DMRB7, HD26/06) are based on the formulae proposed in TRL Report 615 (Nunn, 2004) and TRL Report 630 (Hassan, 2005). TRL Report 615 details the flexible and flexible composite pavement design method and TRL Report 630 presents the rigid pavement design method. The versatile pavement design method has been adopted for cold mixes design, which is systematically presented in the TRL Report 611. Here the cold mix refers to foamed bitumen with the addition of hydraulic binder. In the previous highway agency's design specification, 7 days' compressive strength is employed to characterise the performance of cement bound materials. In the current highway agency's design specification, one-year performance is employed. The reason for employing one-year performance is to accommodate binders such as GGBS and PFA which take a much longer time to be fully hydrated.

In the versatile design process, the foundation (includes sub-grade and sub-base) is classified into one of four classes based on the foundation surface stiffness:

- Foundation Class 1: surface stiffness over 50MPa
- Foundation Class 2: surface stiffness over 100MPa
- Foundation Class 3: surface stiffness over 200MPa
- Foundation Class 4: surface stiffness over 400MPa

Class1 foundation normally refers to foundation with capping layer only. This type of foundation is only suitable for lightly trafficked roads, where the traffic is less than 20msa. Class 2 foundation refers to the type of foundation with capping and subbase or subbase only. Class 3 and 4 foundation refers to the type of foundation with lean concrete subbase. Class 3 and 4 foundations are suitable for roads with a high traffic volume. Cold mixes are suitable for lightly trafficked roads, where Class 1 and 2 foundations are more appropriate. In the following analytical design process, only Class 1 and 2 foundation are considered.

HAUC[9] (2002) classified road into 5 categories as presented in Table 11-1. The cold recycled materials are suitable for application in road categories 1, 2, 3 and 4, where the design traffic is less than 30msa. TRL Report 611 (Merrill, 2004) adopted the HAUC road categorization and further classified the cold mixes into three

[9] HAUC: Highway Authorities and Utilities Committee

categories as presented in Table 11-2. For a road with traffic less than 5 msa, Potter (1996) proposed designs for pavements containing cold in-situ recycled materials for Type 2, 3 and 4 roads as shown in Table 11-3. The stiffness of the cold mixes presented in Table 11-3 should be over 1900MPa.

This research has identified some key factors affecting the GGBS hydration, including the curing temperature and the composition of the recycled aggregate. There appears to be neither unique optimum binder content nor specific long- term stiffness, because the temperature and composition of recycled aggregate can be highly variable. However, it was demonstrated that a long term stiffness over 2500MPa can be achieved by mixing recycled aggregate supplied from the co-operating companies with 4.5 - 7.5% bitumen emulsion (2.7 - 4.5% residual bitumen content) and over 4% GGBS by weight. In the industry, in order to guarantee design stiffness, much higher hydraulic binder contents (around 10% for latent hydraulic binder) are often employed. Considering all these factors, the stiffness of the cold mixes was assumed at three levels in the analytical design process, 1900MPa, 2500MPa and 3100MPa, which were labelled as B1, B2 and B3.

Roads with traffic above 30msa were not deemed appropriate for cold mixes and roads with traffic less than 5msa could be designed following Table 11-3, therefore, roads with traffic from 5msa to 30msa, including Type 1 and Type 2 road, were considered in the analytical design process presented as follows:

Table 11-1: Road type categories

Road Type Categories	Traffic design standard
0	Roads carrying over 30 to 80msa traffic
1	Roads carrying over 10 to 30msa
2	Roads carrying over 2.5 to 10msa
3	Roads carrying over 0.5 to 2.5msa
4	Roads carrying up to 0.5msa

Table 11-2: Bitumen bound cold recycled materials classification

Bitumen bound cold-recycled zone	Minimum long-term stiffness (MPa)
B1	1900
B2	2500
B3	3100

Table 11-3: The thickness of the pavements using cold recycled materials as the combined structural course and foundation platform in roads up to 5msa (Potter, 1996)

Cold in-situ recycling	Depth of recycling (mm)					
	Type 2 road		Type 3 road		Type 4 road	
Thickness of surfacing layers	100mm	140mm	100mm	140mm	100mm	140mm
With foamed bitumen						
Sub-grade CBR <2	n/r	n/r	n/r	n/r	n/r	n/r
Sub-grade CBR 2 – 4	n/r	280	250	200	280	195
Sub-grade CBR 5 – 7	n/r	260	230	170	260	185
Sub-grade CBR 8 – 14	300	240	215	160	245	160
Sub-grade CBR >15	270	215	185	130	215	130
Thickness of surfacing layers	40mm	100mm	40mm	100mm	40mm	100mm
With cement						
Sub-grade CBR < 2	n/r	n/r	n/r	n/r	n/r	n/r
Sub-grade CBR 2 – 4	300	240	240	180	200	150
Sub-grade CBR 5 – 7	280	220	220	160	180	150
Sub-grade CBR 8 – 14	270	200	200	150	160	150
Sub-grade CBR >15	250	200	200	150	150	150

n/r: not recommended.

11.2 ANALYTICAL DESIGN METHOD

Several versatile design methods have been developed; the most widely employed methods include the methods developed by the Shell Bitumen Ltd. and Transport Research Laboratory (TRL). The model, which was developed by TRL and has been adopted by the Highways Agency in generating pavement design curves presented in HD26/06 of DMRB, is presented here and employed for cold mixes design. The parameters adopted as input for the analytical design program are presented in Figure 11-1. The assumed load in the TRL model is a single 40kN wheel load instead of a dual wheel load of 80kN, which is used in the Shell pavement design process. The stiffness of the hot asphalt surface layer is assumed to be 3100MPa, which is the characteristic stiffness of DBM100. DBM100 (Dense Bitumen Macadam with 100Pen bitumen) is often employed on road with light traffic. Assuming that the pavement failure originates from the underside of the pavement base, the pavement residual life can be calculated using the Equation 11-1 as follows:

$$N/10^6 = (\varepsilon_\gamma / (k_{flex} \times k_{safety} \times 201 \times 10^{-6}))^{-(1/0.24)} \qquad \text{Equation 11-1}$$

Where:

$N/10^6$ Traffic before pavement failure, expressed as million standard axles (80kN).

k_{flex} k_{flex} is a coefficient related to the stiffness of the specimen.

$k_{flex} = 1.089 \times E^{-0.172}$, where E is the stiffness of the cold mixed base materials and $flex$ is the abbreviation of flexible materials.

ε_{γ} Tensile strain at the underside of the bitumen bound materials, which can be calculated by linear elastic software BISAR.

k_{safety} Safety factor, assumed as 1 for materials with known performance.

Equation 11-1 is extracted from TRL Report 615 (Nunn, 2004) and has been employed by the Highways Agency in developing the current pavement design guidance.

For Type 0, 1 and 2 roads, the minimum thickness of the hot mix surfacing placed on top of the bitumen bound cold recycled material is presented in Table 11-4. Cold mixes applications are currently limited to road with traffic less than 30msa. As discussed before, Type 1 and 2 roads refer to roads with traffic between 5msa and 30msa, therefore designed as per analytical design method detailed as follows.

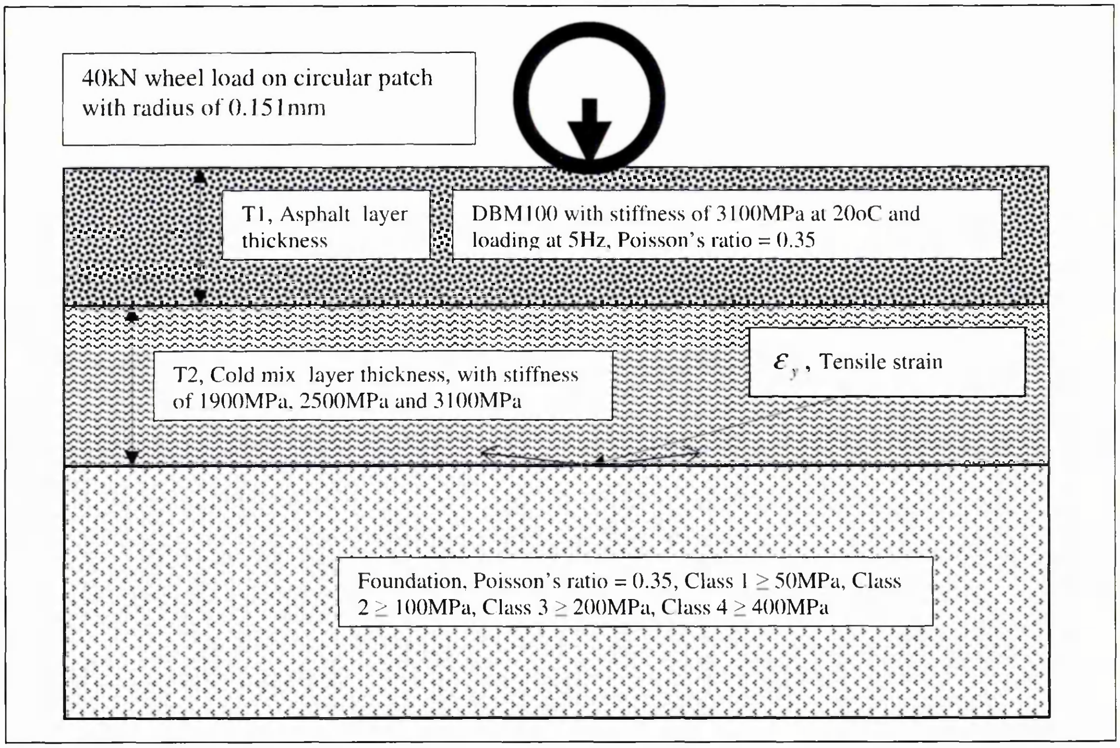


Figure 11-1: Assumptions employed in the pavement analytical design (after TRL 615, Nunn, 2004)

As a rule, the surfacing is the most expensive layer in the pavement. It employs high quality aggregate, often with modified binder. In order to reduce cost, the surfacing is often kept to its minimum requirement in the design process. The minimum surfacing thickness requirements are presented in Table 11-4 and adopted in the following pavement design process.

Table 11-4: Overlay hot mixes thickness requirement

Road type	Minimum thickness of surfacing (mm)
0	100
1	70
2	50

11.3 PROPOSED PAVEMENT DESIGN CURVES

As indicated in Figure 11-2, pavement involving cold mixes can have two thickness levels of hot mix surface layer (depending on the road types), three cold mix subbase stiffness levels and two foundation stiffness levels (depending on the foundation construction), which makes up twelve options in all.

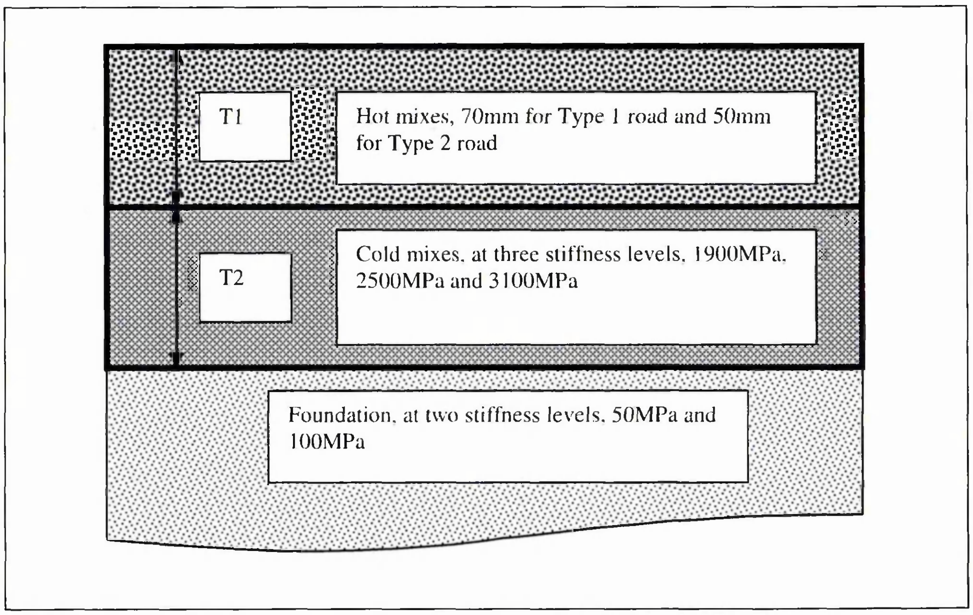


Figure 11-2: Pavement construction options

The minimum pavement thickness for twelve possible construction options at traffic levels from 5msa to 30msa is calculated using BISAR 3.0. BISAR 3.0 is a programme developed by Shell Bitumen, capable of calculating the stresses, strains and displacements in an elastic multilayer system. In the model within BISAR, the pavement is divided into several elastic layers, each layer is assumed to be an elastic material with linear stress-strain relationship, the foundation assumed to be a semi-infinite base. BISAR 3.0 is a programme widely used in the pavement design process.

The design traffic levels corresponding to cold mixes with stiffness of 2500MPa, on Class 1 and 2 foundations with 70mm and 50mm hot mixes as surface (corresponding to Type 1 and Type 2 road) are presented in Table 11-5. The regression relationship between design traffic and cold mix base course thicknesses for Type 1 and Type 2 roads are presented in Figure 11-3 and Figure 11-4.

Table 11-5: Pavement thickness vs. traffic level under Class 1 and 2 foundation

Road Type	Foundation Class	Cold mixes thickness (T2) (mm)	Hot mixes thickness (T1) (mm)	Traffic (msa)
1	Class 1	240	70	5
1	Class 1	260	70	8
1	Class 1	280	70	12
1	Class 1	300	70	18
1	Class 1	320	70	26
1	Class 1	330	70	31
1	Class 2	200	70	4
1	Class 2	220	70	6
1	Class 2	240	70	10
1	Class 2	260	70	15
1	Class 2	280	70	23
1	Class 2	300	70	34
2	Class 1	260	50	5
2	Class 1	280	50	8
2	Class 1	300	50	12
2	Class 1	320	50	17
2	Class 1	340	50	26
2	Class 1	350	50	31
2	Class 2	230	50	5
2	Class 2	240	50	6
2	Class 2	260	50	10
2	Class 2	280	50	15
2	Class 2	300	50	23
2	Class 2	320	50	33

Similar exercises were carried out for cold mixes with stiffness of 1900MPa and 3100MPa respectively. The detailed calculation and the regression curves between design traffic and pavement thickness are not presented here. The correlations between the cold mixes base course thickness and the design traffic are summarised in Table 11-6.

In the regression equations presented in Figure 11-3, Figure 11-4 and Table 11-6, 'Y' represents the thickness of the cold mixes layer and 'X' represents the design traffic.

The total pavement thickness on top of foundation is equal to the cold mixes thickness plus the relevant hot mix thickness, 70mm for Type 1 road and 50mm for Type 2 road.

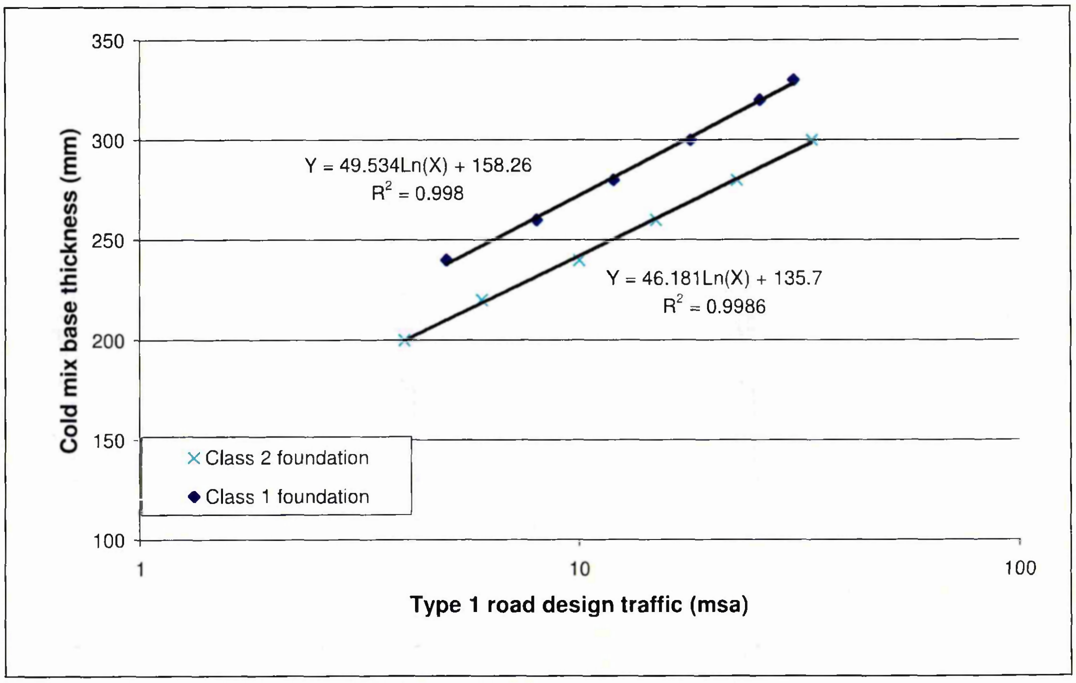


Figure 11-3: Pavement thickness design for Type 1 road

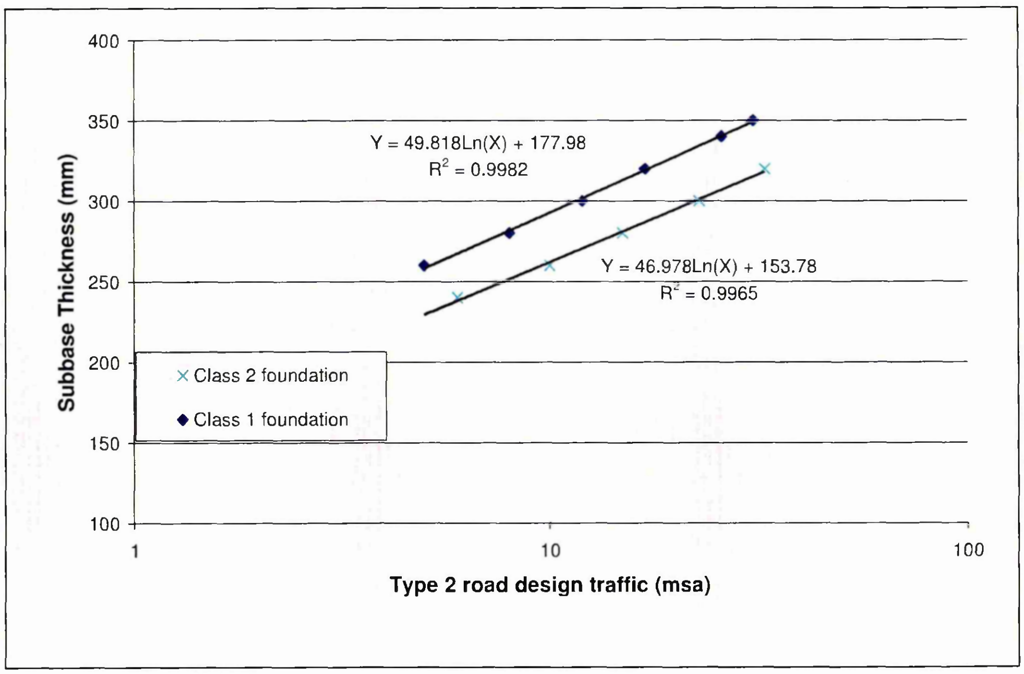


Figure 11-4: Pavement thickness design for Type 2 road

Table 11-6: Cold mixes thickness design formulas

B1 materials, which has a stiffness over 1900MPa	
Type 1, Class 1 foundation	$Y = 54.075 \times Ln(X) + 175.73$, $R^2 = 0.9996$
Type 1, Class 2 foundation	$Y = 51.544 \times Ln(X) + 145.9$, $R^2 = 0.9995$
Type 2, Class 1 foundation	$Y = 50.852 \times Ln(X) + 205.36$, $R^2 = 0.9996$
Type 2, Class 2 foundation	$Y = 47.473 \times Ln(X) + 177.27$, $R^2 = 0.9992$
B2 materials, which has a stiffness over 2500MPa	
Type 1, Class 1 foundation	$Y = 49.534 \times Ln(X) + 158.26$, $R^2 = 0.998$
Type 1, Class 2 foundation	$Y = 46.18 \times Ln(X) + 135.7$, $R^2 = 0.9986$
Type 2, Class 1 foundation	$Y = 49.818 \times Ln(X) + 177.98$, $R^2 = 0.9982$
Type 2, Class 2 foundation	$Y = 46779 \times Ln(X) + 15438$, $R^2 = 0.9977$
B3 materials, which has a stiffness over 3100MPa	
Type 1, Class 1 foundation	$Y = 48.388 \times Ln(X) + 139.11$, $R^2 = 0.9994$
Type 1, Class 2 foundation	$Y = 44.44 \times Ln(X) + 120.61$, $R^2 = 0.9992$
Type 2, Class 1 foundation	$Y = 44.472 \times Ln(X) + 168.87$, $R^2 = 0.9994$
Type 2, Class 2 foundation	$Y = 42.821 \times Ln(X) + 143.61$, $R^2 = 0.9997$

11.4 SUMMARY

This chapter presented pavement designs for road with traffic less than 30msa, based upon the properties of the cold mixed materials identified in this research.

The calculation employed the same model, same formula and the same computer programme as that employed in developing the Highways Agency's pavement design specification. The regression between base course thickness and design traffic showed very high correlation coefficient, with R^2 over 0.99, which suggests the regression curve is effective statistically.

Considering the model adopted and the statistical analysis results, the proposed design formulas are assumed valid for design purposes.

CHAPTER 12 MAJOR FINDINGS AND RECOMMENDATIONS

12.1 BACKGROUND OF THE RESEARCH

With the current emphasis on sustainable development, recycling in the construction industry including highway planning, design, construction and maintenance has become a default option. Traditionally, recycled aggregate has been employed as filling or capping materials. However, the need to replace virgin materials in higher grade applications and reduce landfill has stimulated the need to enhance their performance. As a general rule, aggregate for bound layers is more expensive than that for unbound layers, and utilizing recycling aggregate in the bound layer of the pavement will effectively encourage aggregate recycling.

Recycling can be conducted using cold or hot processes but, except for road planings, which are considered to fulfil the requirements for aggregate in hot mixes, hot recycling is rarely conducted. Cold recycling, which includes mixing recycled aggregate with bituminous and hydraulic binder, is gaining more attention in recent years, and further research is required in order to understand the performance of such mixtures. This research is primarily focussing on the performance of the cold mixes and investigating their suitability as combined subbase, base and binder course materials.

The conclusions listed here are based on the current research results and draw from the summary and conclusions of previous chapters.

12.2 CONCLUSIONS

12.2.1 Compaction method

Various compaction methods, including Marshall compaction, Gyration compaction, Slab compaction and Percentage Refusal Density compaction, were investigated and all the compaction methods investigated were found suitable for preparing cold mixed specimens. The Marshall compaction was selected for this research, because the Marshall compactor was widely available, simple to operate, the specimens produced had consistent performance and the void content of the specimens was similar to cores from the field.

12.2.2 Cold mixes without GGBS

Cold mixes with 7.5% bitumen emulsion by weight of the recycled aggregate, equivalent to 4.5% residual bitumen considering that 40% of the bitumen emulsion is water, were produced by the co-operating companies. Recycled aggregate and bitumen emulsion were supplied to this project with the aim of assessing the performance of the cold mixes. It was found that cold mixes without hydraulic binder possess a long shelf life and the mixture can be compacted weeks and even months after mixing. The performance of pavement materials are normally assessed by their stiffness, rut resistance and fatigue resistance and the cold mixes did not meet the specified criteria laid down by BBA/HAPAS. The cold mix specimens cannot develop enough stiffness by curing in high humidity conditions and the developed stiffnesses were greatly reduced after freezing and thawing.

In summary, cold mixes without hydraulic binder failed to achieve the required performance as paving materials. For use in the weather prevailing in the UK, some improvements in the mixture composition and/or design were warranted.

12.2.3 Cold mixes with the addition of GGBS

This research programme has demonstrated that cold mixes, composed of recycled aggregate, including road planings, concrete demolitions and bricks with bitumen emulsion and GGBS as binder, can develop high long-term strength and stiffness, and therefore are suitable for application as general paving and trench reinstatement materials. The mixture can be stockpiled for months before application, which is an advantage over the traditional hot mixed materials. The mixture develops hydraulic bond over a long period of time, and is therefore less likely to generate the reflective cracking which is a characteristic mode of failure for cement bound materials. Such materials therefore have a clear advantage in the maintenance and construction processes as base and binder course material.

The research work also indicates that the modified Indirect Tensile Strength (ITS) test could be a viable cold mixes design test method to enable more clear differentiation between the attributes of different mixture types in terms of their ductility and fracture toughness. Further improvements may be needed to refine the test procedure and interpret the test results.

The following factors have to be considered in the cold mixes design process.

- The type and composition of recycled aggregate

There are many types of recycled and secondary aggregate. The aggregate employed in this project includes crushed road planings, bricks and concrete demolitions. The experimental work demonstrated that the recycled aggregate composition and state has a profound influence on the cold mixes performance. Specimens at the same binder content mixed with different recycled aggregates, which could comprise different types and composition of recycled aggregate component or even the same batch of recycled aggregate stored for various durations after crushing, showed widely differing performance. The cold mixes with a high proportion of asphalt planings had a long shelf life but developed very low stiffness; the cold mixes with a high proposition of concrete demolitions developed high stiffness with a short shelf life. Cold mixes with long shelf life and consistently high stiffness can only be achieved by controlling the proportions of road planings and concrete demolition, combined with a suitable binders composition.

- The contribution from bitumen emulsion and GGBS

The stiffness and strength of the cold mixes with GGBS and bitumen emulsion as binders mainly comes from GGBS hydration. Comparing the specimens with GGBS and Portland cement as binder, at the same binder content, specimens containing GGBS had a lower initial strength/stiffness but much higher long-term strength/stiffness. The presence of bitumen emulsion appeared to retard the GGBS hydration, the higher the bitumen emulsion content, the slower the stiffness development. A very important contribution from bitumen emulsion towards the cold mixes is to reduce the brittleness and friability of hydraulic bound materials, hence maintaining specimen integrity by reducing crack potential. Another contribution is prolonging the shelf life of the cold mixes.

In most cases, cold mixes with 4% GGBS and 2.7% residual bitumen developed stiffness over 2400MPa after one year of mist room curing, but there seems to be no fixed optimum binder content. Many factors have to be taken into account when deciding the optimum binder content. These include the type, composition, the state of the recycled aggregate used and the curing temperature.

In the pavement construction industry, hydraulic binders including PFA, lime and cement are normally applied at a much higher dosage than that recommended in this research, in order to guarantee the required performance.

- Curing temperature

The curing temperatures employed in this project include 26°C, 20°C and 10°C. The experiments in this research have demonstrated that the rate of the GGBS hydration changes considerably with even slight temperature variations, the higher the temperature, the quicker the GGBS hydration. It is suggested that cold mix recycling should only be undertaken when the ambient temperature is over 10°C. The GGBS hydration is very slow below this temperature.

- F-T resistance

The F-T test revealed that the specimens with hydrated GGBS are F-T resistant. It should be noted that before GGBS hydration, the cold mixes behave more like unbound granular materials than hydraulically bound materials. As such, the cold mixes are F-T sensitive before GGBS hydration, same as the cold mixes without hydraulic binder.

- Re-wetting

The cold mixes with GGBS as binder were able to resume hydration when the temperature and moisture condition were suitable, even after prolonged drying.

- Accelerator

Throughout this project, weak initial strength/stiffness has been found to be a problem for such cold mixed materials and furthermore long shelf life and high initial stiffness/strength are difficult to obtain in one mixture. In order to accelerate the hydration process, lime ($Ca[OH]_2$) and sodium hydroxide (NaOH) solution were added as accelerators.

Even with the accelerators, the weak initial strength/stiffness could still be a problem, depending upon the aggregate employed, the ambient temperature and GGBS/bitumen emulsion content. Concerning the initial strength, the recent version of the MCHW (Manual of Contract for Highway Work) states that the aggregate for the Cement Bound Granular Materials (CBGM) with fly ash, slag as binder should have exposed crushed surface of over 90%. With such a requirement in place, the interlock between aggregates guarantees the early strength/stiffness.

12.2.4 Pavement design

A pavement design based on the performance of the cold mixes investigated in this research project was undertaken using an analytical design method. The stated formulae are suitable for roads with traffic less than 30msa.

12.3 CONTRIBUTIONS, LIMITATIONS AND RECOMMENDATIONS

12.3.1 Contributions

The key findings from this research have:

- Demonstrated that the cold mixes comprising bitumen emulsion bound recycled aggregate mixtures modified with GGBS are capable of achieving the required performance as paving materials under prevalent UK weather conditions and confirmed that the cold mixes may maintain a shelf life of several weeks, which is an advantage over the hot asphalt mixes or cement bound paving materials.
- Shown that the addition of GGBS to the bitumen emulsion bound recycled aggregates enhanced the mixtures' resistance to freeze-thaw exposure,
- Demonstrated that lime and sodium hydroxide are effective in accelerating GGBS hydration at relatively low concentrations,
- Revealed the key environmental factors affecting the performance of the cold mixes, include temperature, humidity and moisture condition.
- Assessed the contribution from various recycled aggregates and suggested that residual lime within the demolished concrete plays a key role in activating the GGBS hydration,
- Demonstrated that the modified Indirect Tensile Strength test is an effective method for assessing the relative ductility and toughness performance to enable a clear differentiation between the behaviour of the various cold and hot bituminous mixtures,
- Proposed a set of pavement thickness design formulae based on the performance of the cold mixes investigated.

12.3.2 Limitations

This research was conducted with the support of two industrial companies, who supplied the bitumen emulsion and recycled aggregates which of necessity limited the scope of this research. The recycled aggregate supplied by the co-operating company comprising road planings, crushed concretes and bricks, was not available as separately crushed and graded components. Therefore the exact composition could not be objectively assessed. The capability of producing recycled aggregate with controlled compositions of RAP, concrete demolitions and bricks was only possible at a late stage in the research programme, following the acquisition of a mechanical crusher by the University. The results from a subsequent complementary investigation confirmed the author's findings regarding the assessment of the influence of the relative proportions of recycled components on mixture performance, including shelf-life. Furthermore, there are many types of recycled and secondary aggregate, and even the same type of recycled aggregate may perform differently. For example, the freshly crushed concrete demolition has a higher residual lime content than the weathered concrete demolition. As a result, the cold mixes with fresh concrete demolition have a shorter shelf life and higher strength/stiffness.

12.3.3 Recommendations

It has been demonstrated that shelf life and GGBS hydration may be profoundly influenced by the source and state of the recycled aggregate employed. Although a relationship between the content of the different recycled aggregate and GGBS hydration has been identified, further work is needed to classify the aggregate and link it to the shelf life and GGBS hydration. Such research could lead to a more systematic approach to the production of cold mixes with balanced proportions of different recycled aggregates, to optimise design for a particular purpose or application.

Cold recycling is frequently employed as a treatment for tar bound materials, because tar is regarded as an environmentally hazardous material and tar bound materials have to be landfilled at a charging rate of £100.00 per tonne in 2007, a rate which is expected to increase over time. It is environmentally unacceptable to hot recycle the tar bound materials. Consequently, every effort is being made to recycle tar bound materials on site. As tar is widespread on old British roads, many such roads are now facing rehabilitation, but no existing research has been found focussing on the cold mixes mainly containing tar bound materials as aggregate. Research in this specific area will offer great economic and environmental advantages.

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