IMPROVING THE PERFORMANCE OF COLD BITUMINOUS EMULSION MIXTURES (CBEMs) INCORPORATING WASTE MATERIALS

by

I Nyoman Arya Thanaya

Submitted in accordance with the requirements for the degree of
Doctor of Philosophy

The University of Leeds
School of Civil Engineering

July 2003

The candidate confirms that the work submitted is his own and that appropriate credit has been given where reference has been made to work of others

ACKNOWLEDGEMENTS

The author wishes to express his gratitude and deep appreciation to all those who have assisted in producing this thesis.

The author would like to thank in particular his main academic supervisor Dr. S.E. Zoorob, and his second supervisor Dr. J.J. Brooks for their valuable assistance, suggestions, advise, continuous guidance and encouragement throughout the research.

The author would also like to express his appreciation and sincere gratitude to:

- The Faculty of Engineering, Udayana University, Denpasar-Bali, Indonesia for supporting the study.
- Engineering Education Development project (EEDP), ADB Loan No. 1432-INO, The Ministry of National Education, Republic of Indonesia for providing the scholarship.
- All the technical and administration staff at the School of Civil Engineering, The University of Leeds for their support and assistance.
- Mr D.Rockliff and Mr A.Moffett from Tarmac Northern Ltd. for their technical input and for supplying materials.
- Research fellows: Dr. M.H.S Alawi, Dr. Latif B. Suparma, Dr. Ary Setyawan, Dr. S.A. Kristiawan, Dr. C.Love, Mr. Ismail Zarouni, Mr. Sultan al-Salem, and Ms. Heather Dyson for their discussion and friendship.

Finally the author would like to express his deep appreciation to all members of his family, in particular his wife Sri, his daughters Yutri and Tini, for their support, patience, care and understanding.

ABSTRACT

Currently, there exists no universally accepted Cold Bituminous Emulsion Mixtures (CBEMs) design procedure. Three of the most popular design procedures, which in essence were based on AASHTO and the Asphalt Institute design guidelines were studied and tested in detail during the course of this investigation. In general, the design procedures investigated were found to be *not user-friendly or simple* to implement. The main obstacles that restrict the adoption of CBEMs as the first choice material as opposed to conventional hot asphalts, for all bound pavement layers are: the high compacted mixture porosity, low early life strength and long curing times. CBEMs are more widely accepted in low to medium trafficked pavements. The key aims of this investigation were to improve and simplify the design procedure of CBEMs, and to investigate ways of improving CBEMs volumetric and mechanical properties.

The main aggregate materials used in this investigation were carboniferous limestone and quartzitic asphalt sand. But in response to environmental conservation campaigns, a range of selected waste materials were also tested as partial and full replacement to the virgin mineral aggregates, including: pulverized fuel ash (PFA), red porphyry sand, synthetic aggregates, steel slag, crumb rubber, and crushed glass. The aggregate gradations were designed using a modified Fuller's curve. The emulsion used in this investigation was a cationic bitumen emulsion with 60% and 62% binder content composed of 100 pen base bitumen.

The mix design procedure initially developed in this investigation was found to be complicated from a practical application point of view, in particular the steps required to determine the *optimum total liquid content at compaction,* which were unlikely to be practicable for site applications.

A more simplified CBEMs design procedure was therefore introduced in this thesis, where the *coating test* was found to play a very essential role. Improvements in all mixture properties were readily accomplished by *increasing the compaction effort* to reduce porosity and by *incorporating cement.* The porosity target of 5-10% and minimum indirect tensile strength (ITSM) value of 2000 MPa at a fully cured condition were more easily achievable. The main emphasis of this modified design procedure was on simplicity and practicality whilst maintaining the key volumetric and mechanical properties of the mixtures. In this investigation, the mechanical performances of the CBEMs at full curing condition were more comprehensively evaluated in terms of fatigue and creep tests.

Attempts to accelerate the curing times of cold asphalt mixtures were made by compacting the CBEM specimens in two layers (two lifts) thus allowing the moisture to escape faster from each layer and hence reducing the overall curing time. The results from these laboratory trials were very encouraging. Additionally, the incorporation of plastic cells was found to significantly reduce shear deformations of CBEMs under loading during their early lives. The inclusion of plastic grids in the upper layer of a two layered cold mixture system appeared to be very promising.

It was concluded that the CBEMs design procedure proposed in this investigation was simpler than the initially adopted procedure. The main advantages of this modified design procedure were that whilst it maintained all the key volumetric and mechanical properties of the mixtures it was simpler and more practical than other existing procedures. Heavier compaction effort and the incorporation of 1 to 2% cementitious materials were found to be essential for improving the performance of CBEMs, and as is well known, CBEMs are most suitable in dry warmer climates. When CBEMs are carefully designed and are allowed to achieve a full curing condition, the performance of CBEMs can be comparable to hot asphalt mixtures with the same penetration grade binder.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER I

INTRODUCTION

1.1 Problem Identification

Environmental conservation campaigners have more recently escalated their efforts and have provided added pressures on the road industry to tackle environmentally unfriendly processes.

The targeted schemes include: reducing wastes from aggregate production processes, minimising land-filling, reducing CO_2 emissions during hot bituminous mixture production and laying, control of dust and other particulate matter, utilisation of bulk inert wastes and or recycled materials, monitoring leaching from bound and unbound materials into the subsurface water, etc.

Additionally, issues of energy saving and safety at work have stimulated efforts to introduce alternative methods of producing bituminous mixtures. One possible and attractive method is using 'Cold Mixture Technology', in which the bituminous mixtures are produced at ambient temperatures by utilizing liquid asphalts (bitumens). Liquid bitumens are obtained by reducing the straight run bitumen viscosity in several different ways. One of which is by emulsifying the bitumen in water in a process known as 'bitumen emulsification'.

The technology of cold bituminous emulsion mixtures for road pavements has been extensively utilised in many countries. The USA and France have been utilising the materials since the 1970's and appear to have considerable experience in the performance of cold bituminous emulsion mixtures. Meanwhile, due to the more wet / cold climatic conditions in the UK, which are not favourable to the emulsion curing process, the development of cold mixture technology is only more recently being brought forward [Leech, 1994].

At the moment, there is no 'universally accepted' mixture design method or specifications for cold bituminous emulsion mixtures. In addition, correlations and methods of assessing test results vary considerably amongst different researchers and institutions. Currently there does not exist a *unified procedure for laboratory evaluation of cold bituminous emulsion mixtures.*

In the UK, attention towards cold mix cold lay materials has been encouraged by the publication in 1992 of 'Specification for Reinstatement of Openings in Highways' by the Highway Authority and Utility Committee (HAUC), which allows the use of Permanent Cold Lay Surfacing Materials (PCSMs) as an alternative to hot mixture materials for reinstatement works in low volume roads and footpaths.

Besides being environmentally more friendly, energy saving, and safer to handle, there are some other advantages in adopting cold bituminous emulsion mixtures. They are easier to mix and handle, the size and the time for the process of installation (or re-installation) of a mixing plant is smaller and shorter respectively compared to a hot mix plant. Cold bituminous mixtures are particularly suitable where hot mix plants, due to distance, cannot deliver mixtures within an acceptable time, or where asphalt production facilities are not available such as in remote areas or in regions that consist of many islands [Nikolaides, 1994].

The incorporation of waste materials into cold mix recycling techniques gives additional incentive, supports environmental conservation and can reduce demand particularly on virgin aggregates.

However, cold bituminous emulsion-aggregate mixtures are still generally classed as inferior to hot mixtures with respect to performance, although the engineering equivalence and practical difficulties in adopting cold mixture formulations have not yet been clearly defined [Biczysco, 1994].

There are some problems with utilising cold bituminous mixtures. The *prime concern* is with their inadequate early strength and the long curing time required to achieve maximum performance [Nikolaides, 1994].

Cold bituminous mixtures possess a low degree of binder coating particularly on the coarser aggregates. The performance of the mixtures is *highly dependent on the availability of a 'superior' emulsion formulation* [Robinson, 1997].

Studies by Chevron Research Company in California reported that full curing of cold bituminous mixtures on site can occur after 2 to 24 months [Santuccy, 1977 and Leech, 1994]. Meanwhile, stiffness data on cores in the UK have indicated that mixtures develop stiffness gradually over time. For example, the indirect tensile stiffness modulus of a 6 mm Dense Macadam (200 pen base emulsion) met the HAUC Specification requirements of 600 MPa after 10 months, and the value increased to almost 800 MPa in 24 months [Robinson, 1997].

Cold bituminous mixtures can have some problems with binder drainage during storage as a result of the low binder viscosity. Binder stripping can also occur due to weak adhesion and in general the compacted mixtures *contain high voids content.*

1.2 Objectives

Taking into consideration the problems mentioned above, the main objectives of this research are as follows:

1. To improve and simplify the design procedure for Cold Bituminous Emulsion Mixtures (CBEMs) making it more practicable for site application.
2. To investigate ways of improving the performance of Cold Bituminous Emulsion Mixtures (CBEMs) in particular mixtures incorporating selected waste materials. Two main properties were targeted, namely:
    - Porosity; acceptable values from 5 to 10% [MPW-RI, 1990] and 6 to 12% [Nikolaides, 1994].
    - Indirect Tensile Stiffness Modulus (ITSM); an acceptable range from 2000 to 2500 MPa [HAUC, 1992 and TRL Report 386, 1999].

1.3 Research Programme

In order to meet the above main objectives, the following research methodology was adopted:

1. Selection of suitable waste materials and virgin aggregates.
2. For each combination of aggregates and/or waste products, an optimum aggregate gradation was selected for the Cold Bituminous Emulsion Mixtures (CBEMs).
3. Investigation and critical analysis of three CBEM design procedures, namely; 1- Asphalt Institute [Asphalt Institute, 1989], 2- Nikolaides, A.F [Nikolaides, A.F., 1994] and 3- The Ministry of Public Work - Republic of Indonesia [MPW-RI, 1990].
4. Formulating a modified CBEMs mixture design procedure, followed by extensive evaluation and modification.
5. Proposing a sound, workable CBEMs Design Procedure.
6. Investigating ways of improving the performance of CBEMs, particularly in terms of; a- Porosity and Stiffness (ITSM), b- rate of strength gain, and c- retained strength after moisture damage.
7. Additional parameters investigated were; susceptibility of CBEMs to permanent deformation (Dynamic Creep testing) and fatigue life (Indirect Tensile Fatigue Testing).

1.4 Thesis Organisation

This thesis covers a theoretical review, methodology, results and analysis from laboratory investigations. The thesis is composed of 11 chapters. A brief description of the contents of each chapter follows:

CHAPTER I is an Introduction that outlines the background of the investigation, problems, objectives, and research program. **CHAPTER II** presents a review of bitumen emulsions covering some essential properties and applications.

CHAPTER III describes aspects of cold mixtures in general and the types of cold mixtures currently available, and then **CHAPTER IV** reviews materials that are suitable for incorporation into CBEMs, in particular waste materials.

The common tests for CBEMs are presented in **CHAPTER V.** These general tests that are commonly used to evaluate the properties of bituminous mixtures. **CHAPTER VI** reviews selected CBEM design procedures and Specifications with emphasis on U.S. design procedures. The chapter puts forward the argument regarding the absence of a universal design procedure. Following this **CHAPTER VII** describes in detail the CBEMs design procedure initially adopted in this investigation, principally based on U.S. design procedures.

Problems with the design procedure initially developed are given in **CHAPTER VIII** which also covers a proposed CBEMs design procedure that simplifies and improves the existing design procedures.

CHAPTER IX describes the methodology developed for this investigation and covers selection of materials, adoption of the proposed CBEM design procedure, experiences gained on the performance of bitumen emulsions, and various experiments for improving the performance of CBEMs. **CHAPTER X** presents an overall summary to high light the findings of laboratory experiments. Finally **CHAPTER XI** presents the conclusions and recommendations for further work on CBEMs.

CHAPTER II

REVIEW ON BITUMEN EMULSIONS

2.1 Liquid Bitumen and Bitumen Emulsions

There are four ways to reduce bitumen viscosity or in other words to produce a more **'liquid bitumen'**, thus enhancing its workability at ambient temperatures. The first method is by heating the bitumen at elevated temperatures (hot mixtures), secondly: by blending the bitumen with a petroleum solvent (cutback), thirdly: by emulsifying the bitumen in water to form bitumen emulsion and finally: by foaming the bitumen (for more details on foamed bitumen, please refer to Chapter III, Section 3.2.3).

Hot mixtures are normally used for medium and large sized jobs where equipment is available for heating, storing, and applying the bitumen. However, this may not be suitable for small scale jobs or where equipment is not available, such as in remote areas. Meanwhile using a cut back bitumen can be more expensive than a hot mixture as it requires a petroleum solvent which is generally expensive. Additionally, the solvents can pollute the environment and they are fire hazardous [Akzo Nobel, 2002, URL-1].

The option of using **bitumen emulsions** has increasingly become an attractive choice. An **emulsion** is a two-phase system consisting of two immiscible liquids, the one being dispersed as minute globules in the other. In the case of bitumen emulsion, the bitumen is the dispersed phase and water is the continuous phase. The size of the dispersed bitumen globules is in the range of 0.001 to 0.005mm (1 to 5 microns) in diameter. These globules are held in suspension due to the presence of electrostatic charges given by an emulsifier and stabilised by a stabiliser [Whiteoak, 1991].

The bitumen content within the emulsion depends on the intended application of the emulsion. Generally, this figure is between 40% and 70%. There is an upper limit of bitumen content which depends on the relative volume of the two phases. Close to the upper limit, there is insufficient room for more bitumen droplets without deforming them. The droplets will become tightly packed and may coalesce to each other. Eventually the water entrapped between the bitumen droplets develop into water droplets and the result is an inverted emulsion, i.e. water in bitumen emulsion.

'Multi phase' emulsions also exist, in which the dispersed droplets themselves contains smaller droplets of a third phase, as in the case of a *breaking agent* described Section 2.3.5 and Fig. 2.2.

2.2 The Nature of Bitumen Emulsions

At ambient temperatures bitumen emulsions are in a very fluid condition. They are normally applied cold or slightly heated. The advantages of utilising aqueous bitumen emulsions are that they can be applied at low ambient temperatures, can be adjusted to greater fluidity, are non flammable, can easily penetrate porous materials, and can have good coating ability. One major application of bitumen emulsions is in cold bitumen–aggregate mixtures.

2.3 Components of Bitumen Emulsions

2.3.1 Bitumen

Depending on the climate, the grade of base bitumen normally used for manufacturing emulsions can be anything from 50pen to 200pen. Harder binders are more suitable in hotter climates. The quality of the bitumen can affect the properties of the emulsion.

2.3.2 Emulsifiers

In the production of bitumen emulsions, a small portion of *surface active agent / surfactant / emulsifier* which will provide each bitumen globule with an *electrical charge* has to be added. The emulsifier also facilitates the dispersion of bitumen and keeps the bitumen globules in permanent suspension, so that a homogenous emulsion can be obtained. Without an emulsifier, the two immiscible liquids will separate into two layers rapidly. The emulsifier usually consists of long hydrocarbon chains that terminate in either anionic or cationic functional groups. Due to the presence of an emulsifier, an adsorbed film of emulsifier is formed round each bitumen globule in the emulsion. This film becomes a protective coating and can resist the coalescence of the dispersed bitumen globules.

The emulsifier molecules have both **hydrophobic** or *lipophilic* (oil loving) and **hydrophilic** or *oleophobic* (water loving) portions. The hydrophobic portion orientates in the bituminous phase and the hydrophilic portion in the water phase. The emulsifier molecules are much smaller than the bitumen droplets. Each droplet is stabilized by a considerable amount of emulsifier molecules [Akzo Nobel, 2002, URL-1].

The type of electrical charge of the emulsifier depends on the pH, which indicates acidity or alkalinity of the system. Emulsions that are produced with anionic and cationic emulsifiers are not compatible when they are mixed, causing coagulation of the bitumen globules. There are four groups of emulsifiers, namely:

i. Anionic Emulsifiers
ii. Cationic Emulsifiers
iii. Nonionic Emulsifiers
iv. Colloidal Emulsifiers

The first two of these emulsifiers are most widely used. *Anionic emulsions* are *usually alkaline with pH value higher than 7, with a negative electrical charge.* Meanwhile *Cationic Emulsions* are *acid emulsions with pH < 7, with a positive electrical charge.* Bitumen emulsions generally contain 0.5 to 1% emulsifier to prevent coagulation of the bitumen droplets.

Anionic Emulsifiers

These types of emulsifiers are principally *alkaline solutions.* They consist of 'large anionic ions', a type of this is sodium stearate $CH_3(CH_2)_{16}COONa$. In water this emulsifier dissociates into (negative) stearate anions $CH_3(CH_2)_{16}COO^-$ and (positive) sodium cations Na^+ [Dept. of Scientific and Industrial Research RRL, 1962].

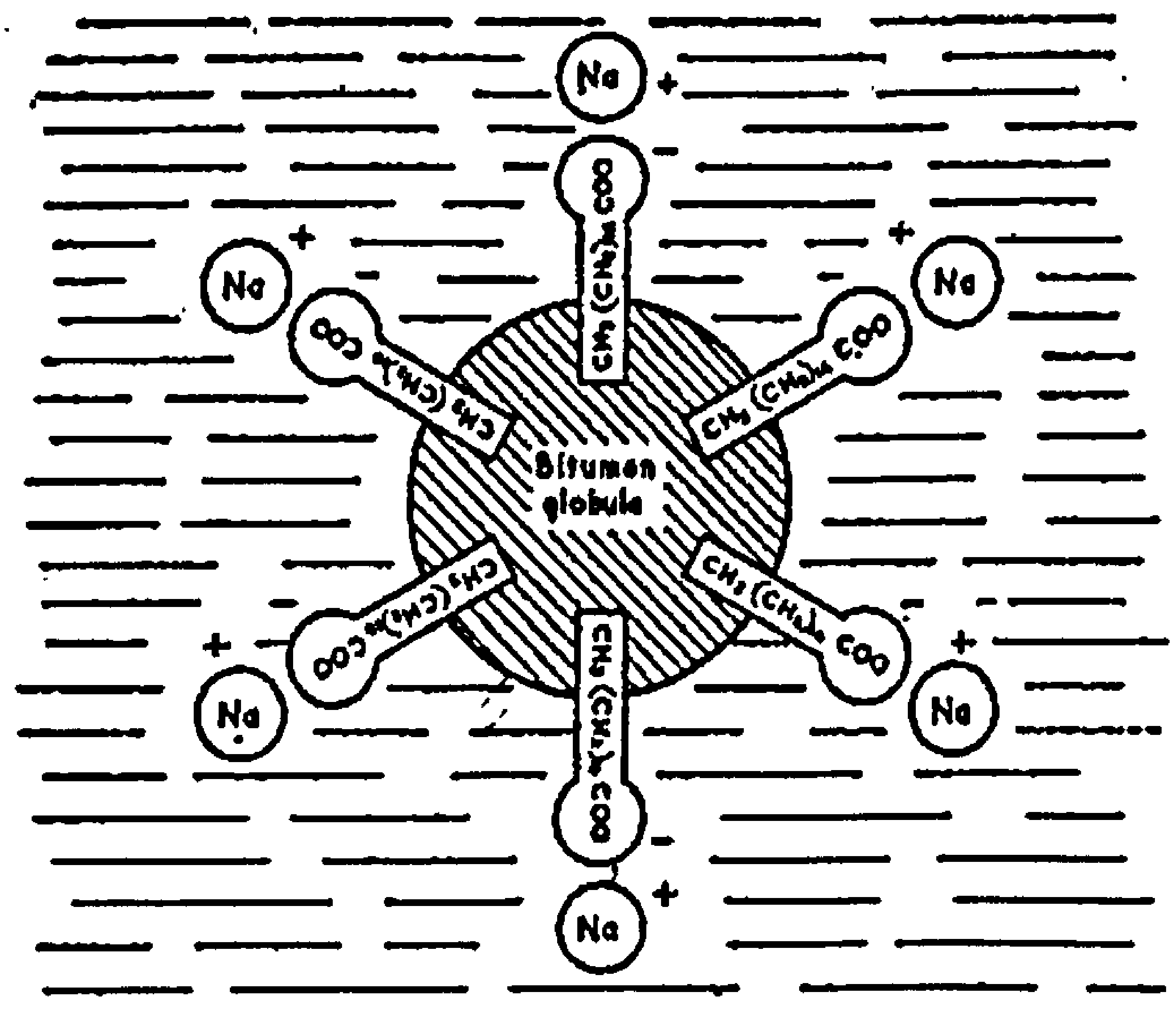
The stearate anion is soluble in bitumen with the carboxylic group (COO^-) that carries the negative charge being '*the least soluble part*' and forms films of ions with negative charges on the surface of the bitumen globules. The positively charged ions are then attracted to the negatively charged ions to form '*electrical double layers*'. This kind of emulsion is illustrated in Fig. 2.1a. Like charges on the globules repel each other, hence an emulsion system is formed.

Cationic Emulsifiers

These emulsifiers are basically *acidic solutions*, characterised by 'large cation organic fractions'. A typical example is cetyl trimethyl-ammonium bromide $C_{16}H_{33}(CH_3)_3NBr$. This dissociates in water into the (positive) cetyl trimethylammonium cation $C_{16}H_{33}(CH_3)N^+$, and the (negative) bromine anion Br^-. A cationic emulsion system is presented in Fig. 2.1b.

Colloidal Emulsifiers

This kind of emulsifiers are more commonly used for industrial purposes rather than for road emulsions. Casein, gelatin, and fine powders such as clays and bentonites are examples of colloidal emulsifiers.



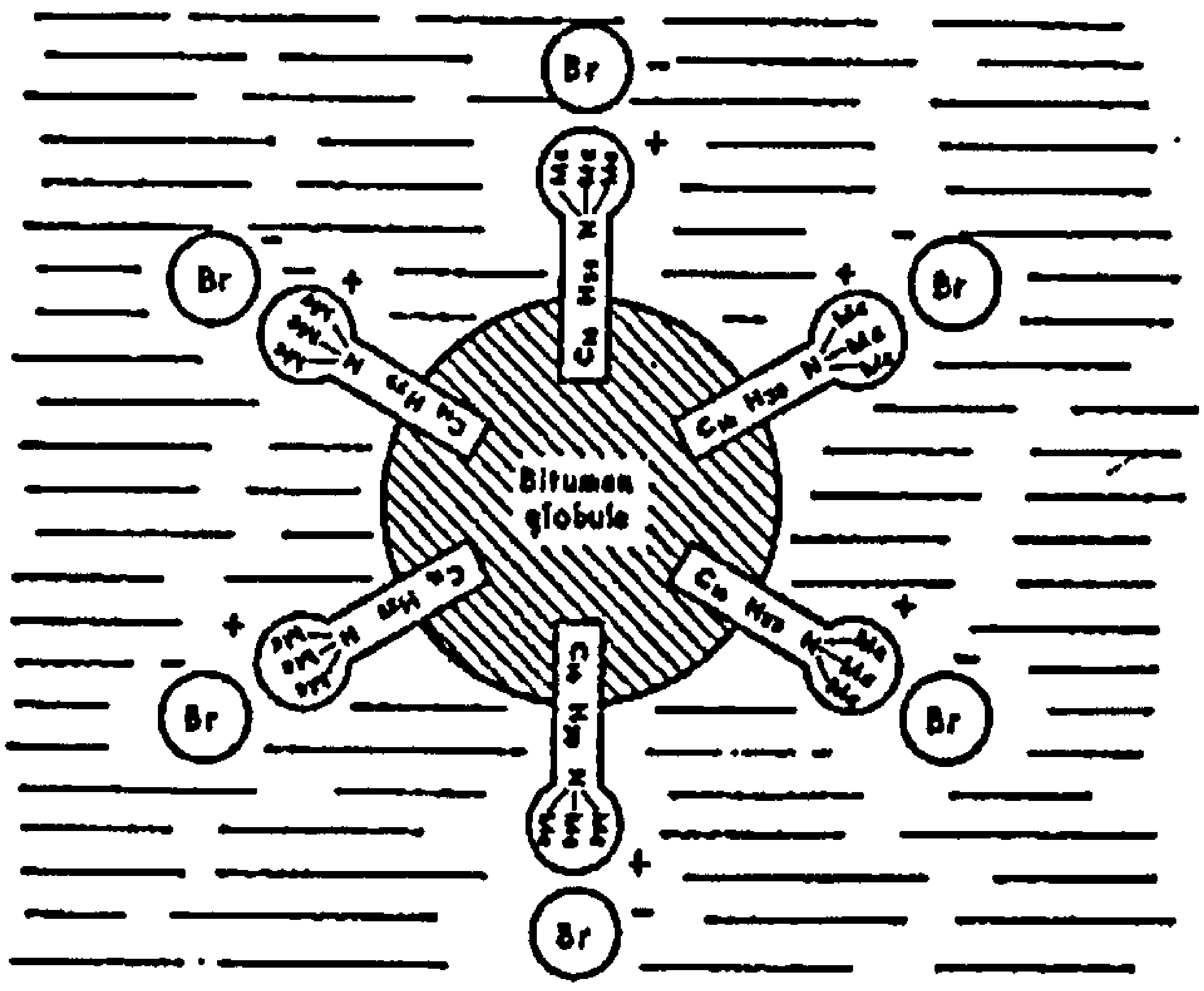
Anionic Emulsifier :

Sodium Stearate :
$CH_3(CH_2)_{16}COO\,Na$,

Dissociates in water becomes :

- **Sodium Cation, Na^+,**
  (in water / continuous phase).
- **Stearate Anion (large portion),**
  $CH_3(CH_2)_{16}COO^-$,
  (in the bitumen globules / discontinuous phase).

(a) Anionic Emulsion



Cationic Emulsifier :

Cetyl Trimethyl – Ammonium Bromide : $C_{16}H_{33}(CH_3)_3NBr$**,**

Dissociates in water becomes:

- **Bromide Anion , Br^- ,**
  (in water / continuous phase).
- **Cetyl Trimethylammonium Cation, (large portion),**
  $C_{16}H_{33}(CH_3)_3N^+$ **,**
  (in the bitumen globules / discontinuous phase).

(b) Cationic Emulsion

Fig. 2.1 Diagrammatic representation of the arrangement of emulsifier around dispersed bitumen globules in typical emulsion, [Dept. of Scientific and Industrial Research RRL, 1962].

Non-ionic Emulsifiers

These emulsifiers do not ionise, and they are rarely used for road emulsions. Non-ionic emulsifiers can be used in making *ethers* and *esters that are required* for producing fatty acids and alcohol.

In general term, the typical emulsifiers used and their levels are shown in Table 2.1.

Table 2.1 Typical emulsifier used and level.

Emulsion Type	Emulsifier level (%)	Emulsion pH	Acid / alkaline	Typical emulsifier
Cationic Rapid-Setting	0.15-0.25	2.0 - 4.0	acid	tallow diamine
Cationic Medium-Setting	0.30-0.60	1.5 - 4.0	acid	tallow diamine
Cationic Slow-Setting	0.80-2.00	2.0 - 5.0	acid	quarternary amine
Anionic Rapid-Setting	0.20-0.40	10.5 - 12	alkaline	tallow acid
Anionic Medium-Setting	0.40-0.80	10.5 - 12	alkaline	tallow acid
Anionic Slow-Setting	1.20-2.50	7.5 - 12	alkaline	Non-ionic + lignosulphonate

Source: [Akzo Nobel, 2002, URL-1].

2.3.3 Stabilisers

Bitumen may contain salt left over from inadequate desalting of the crude oil. This salt can cause *swelling* of the bitumen droplets through **osmosis**, i.e. when there is a high content of salt in the bitumen and water is pulled into the bitumen droplets. This can increase emulsion viscosity and affect stability.

In a cationic system, about 0.05-0.2% **calcium chloride** ($CaCl_2$) or other soluble salts can be added into the water phase of the emulsion (as a stabiliser) to help in reducing osmosis of water into the bitumen. $CaCl_2$ can also reduce the settlement of emulsions by increasing the density of water, and in many cases improves the performance of emulsifiers [Akzo Nobel, 2002, URL-1].

In an anionic system, 'trisodium polyphosphate' can be used. There are some other types of stabilisers such as casein, liquid resin, or vinsol resin [Whitoeak, 1991].

2.3.4 Acids

Acid is used to form salt from the emulsifier and for adjusting the pH to a predetermined level. **Hydrochloric acid** is used for **cationic** systems, and *sodium hydroxides* for *anionic* systems.

2.3.5 Bitumen Emulsion Additives

Production of bitumen emulsion involves ***complicated chemical aspects*** in order to obtain the desired quality of emulsion. Below are some types of additives which can be used for improving the quality of bitumen emulsion.

- **Solvent**

Solvent (non volatile fractions of petroleum) can be added into the bitumen and evenly mixed. This will provide temporary softening to the binder, hence improving the coating ability. White spirit is the most widely used solvent.

- **Sodium Tripolyphosphate**

This additive can be used as a water softener to improve emulsion quality, particularly for anionic emulsions which are sensitive to hard water.

- **Thickener**

Soluble types of these additives can increase emulsion viscosity in order to reduce emulsion run off in open graded mixtures.

- **Adhesion Promoter**

This material can be added into the bitumen before emulsification or to the finished emulsion. It can increase adhesion of the bitumen to the aggregates, hence reduce risk of stripping.

- **Asphalt Peptizer**

This material improves the emulsifiability of the bitumen to enable the production of smaller droplets. It can also reduce settlement, increase viscosity, and improve adhesion.

- **Latex**

Latex is a natural or synthetic dispersion of rubber particles in water. It is usually based on natural rubber, styrene butadiene styrene (SBS), styrene butadiene rubber (SBR), polychloroprene or acrylates. Basically, Latex is *a water based dispersion of polymer that can be used for modifying emulsion.* It can be added into the water phase, bitumen phase, or to the emulsion. Addition into the water phase usually gives the best result. Polymer modification generally gives improvements in cohesion, resistance to cracking at low temperatures and deformation at high temperatures.

- **Breaking Agent**

The Swedish company, NYNAS developed a breaking agent system for cationic emulsion (as it is more widely used than the anionic emulsion). The breaking agent is added into the emulsion immediately before use. It is based on a double emulsion system, which consists of an aqueous solution of a weak base (basic salt solution) emulsified in light mineral oil. This system itself is emulsified in water (Fig. 2.2). The base is initially kept away from the emulsifier by the mineral oil. Soon after mixing the base migrates through the oil and *neutralises the emulsifier.* Hence breaking the emulsion [Needam, 1996].

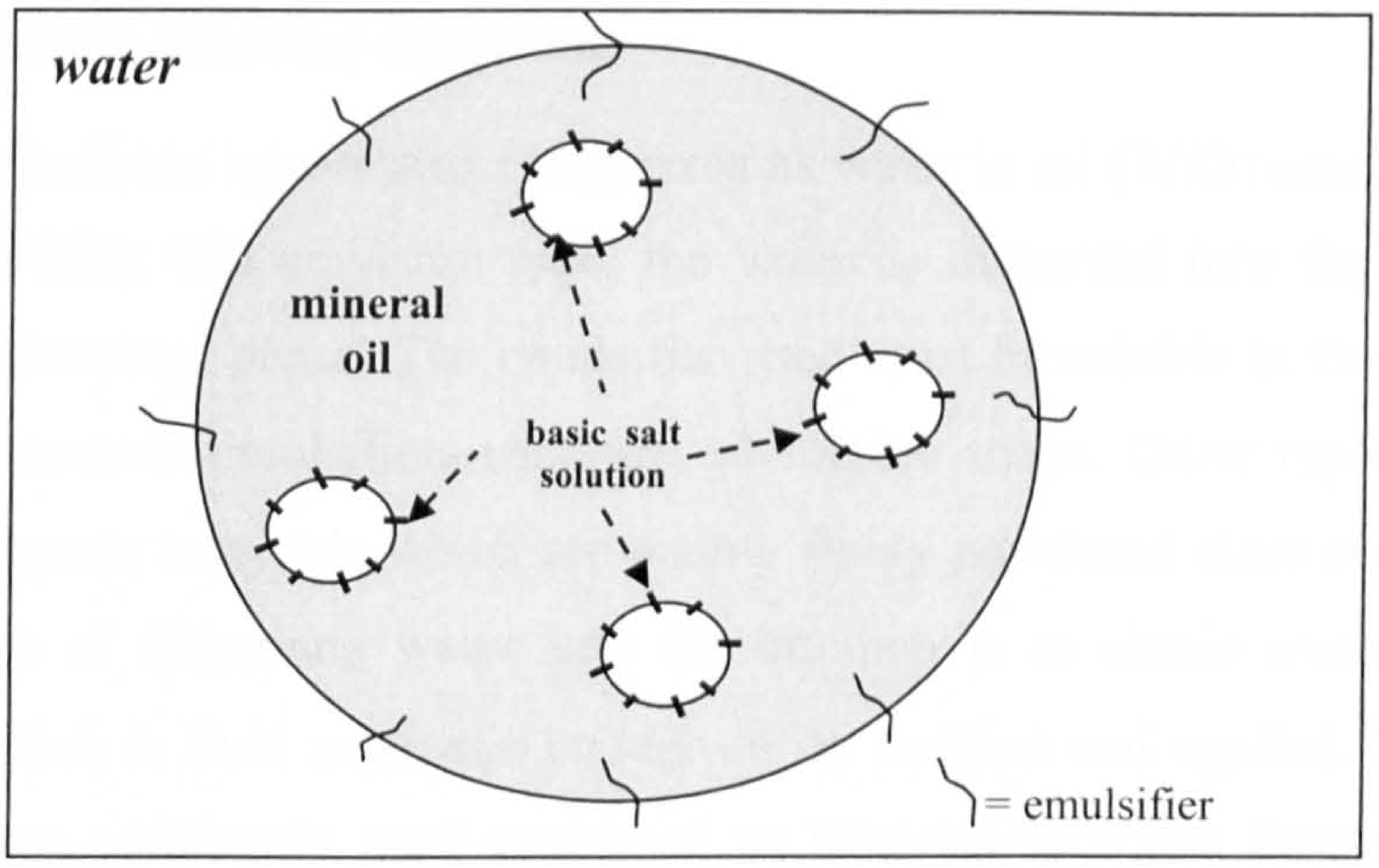


Fig. 2.2 Breaking Agent Double Emulsion.

2.4 Category of Bitumen Emulsions

In broad categories, there are two types of emulsions incorporating bitumen and water, i.e. *bitumen-in-water emulsions* and *water-in-bitumen emulsions*. Another category is based on *stability* when in contact with aggregates or pavement surfaces, i.e. *rapid, medium, and slow setting* bitumen emulsions.

2.4.1 Bitumen-in-Water Emulsions

Bitumen emulsions are categorised as bitumen-in-water emulsions. This emulsion type is also known as oil in water (O/W) emulsion. Depending upon the electrical charges surrounding the bitumen globules, bitumen emulsions can be grouped into four groups in which the first two groups are mostly used in road construction:

1) Anionic Emulsions
2) Cationic Emulsions
3) Non-ionic Emulsions
4) Clay Stabilised Emulsions

Anionic emulsions were first developed in the early 1900s. The growth in their use was relatively slow. Cationic emulsions were introduced in the mid 1940s with a major technical improvement [Akzo Nobel, 2002, URL-1].

2.4.2 Water-in-Bitumen Emulsions

This emulsion type is also categorized as water in oil (W/O) emulsion or *inverted emulsion.* Within this emulsion type, the water is dispersed into the bitumen as the continuous /external phase. The emulsifier used must be soluble in the external phase. The most common emulsifiers used are oil-soluble soaps. Other types of emulsifiers include inorganic materials which are mainly finely powdered clays such as bentonite. The purpose of dispersing water into the bitumen is to obtain material with lower viscosity which is fluid or plastic enough to be handled and applied. These emulsion types are less commonly used compared to Bitumen-in-Water Emulsions. They are mostly used in the roofing industry and for special products. One of their main characteristics, besides their cementing and water proofing properties, is their plasticity.

2.4.3 Bitumen Emulsions Based on Reactivity

Bitumen emulsions can also be classified based on their reactivity when in contact with aggregates or pavement surfaces, i.e. **rapid, medium, and slow setting.** *Rapid setting* emulsions have little or no ability to mix with aggregates. They are mainly used for *spray applications. Medium setting* emulsions are *suitable for mixtures that mostly contain coarse aggregates with little or no fines. Slow setting* emulsions are designed for mixtures with both *coarse and significant amount of fines (high surface area).* Slow setting emulsions set slowly, remaining workable up to a certain time during mixing, thus enabling the production of mixtures with sufficient workability and degree of coating.

2.5 The Manufacture of Bitumen Emulsions

Bitumen Emulsions are produced by means of **colloid mills** that have rotors revolving at about 1000 to 6000 rpm in a stator. The clearance between the rotor and stator is adjustable and is typically set to between 0.25 to 0.50 mm. Fig. 2.3 shows a schematic diagram of an asphalt emulsion manufacturing plant [Kett Irving, 1999].

Into the colloid mill, concurrent streams from separated feeders of hot bitumen and preheated water containing an emulsifying agent, are pumped. As the bitumen and water enter the colloid mill, they are subjected to intense shearing forces that break the bitumen into minute droplets which are then coated by the emulsifier. The emulsifier gives similar electrical charges to the surface of the bitumen globules that cause the presence of electrostatic forces to prevent the coalescence of the globules.

The emulsions thus formed are pumped through a heat exchanger which extracts excess heat, they are then stored in a storage tank with a stirring device for maintaining homogeneity. The temperature of the bitumen and water when entering the mill vary depending on the grade and percentage of bitumen in the emulsion, and type of emulsifier used. The viscosity of the bitumen should not exceed 0.2 Pa.s, with temperatures ranging from 100 to 140°C.

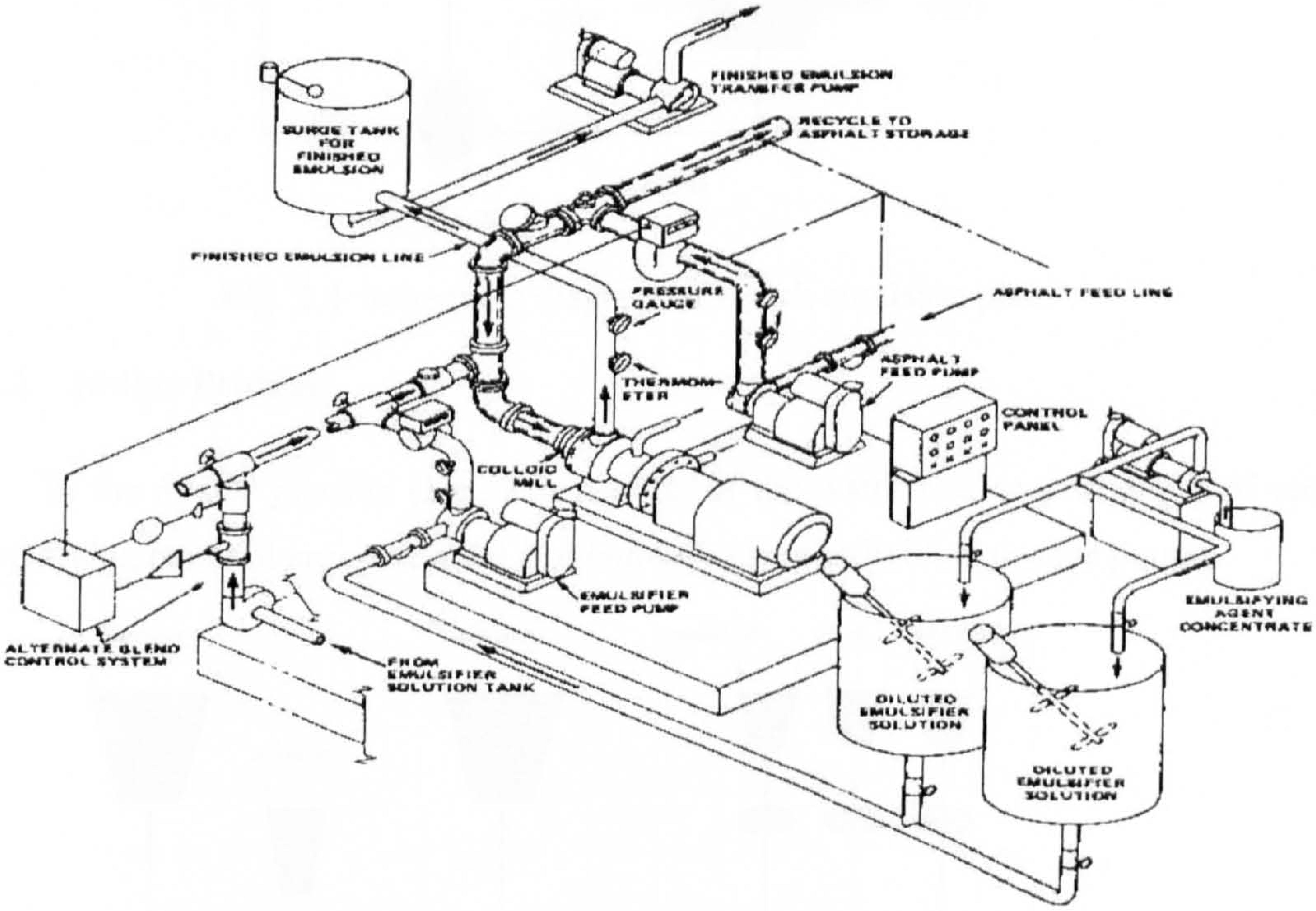


Fig. 2.3 Schematic of a Bitumen Emulsion Manufacturing Plant, [Kett Irving, 1999].

Bitumen emulsions can be produced either in a **batch or an in-line process** plant.

2.5.1 Batch process

This consists of at least two steps, i.e. water phase (***soap***) preparation and the actual production of emulsion. A tank is used to prepare the water phase. Heated water, emulsifier, other emulsion chemicals such as a stabilizer or acid are metered and properly mixed in the tank as the water phase of the system. The bitumen is heated to make it flowable and occasionally a solvent may be added for temporary softening the binder. The bitumen and the ready-made water phase are dosed into the colloid mill. This batch plant process allows manual control as it only involves the flow of few materials. However, manual control can be less accurate and less efficient as a result of human error. Additionally some chemicals may be hazardous. A schematic *batch emulsion plant* is shown in Fig. 2.4.

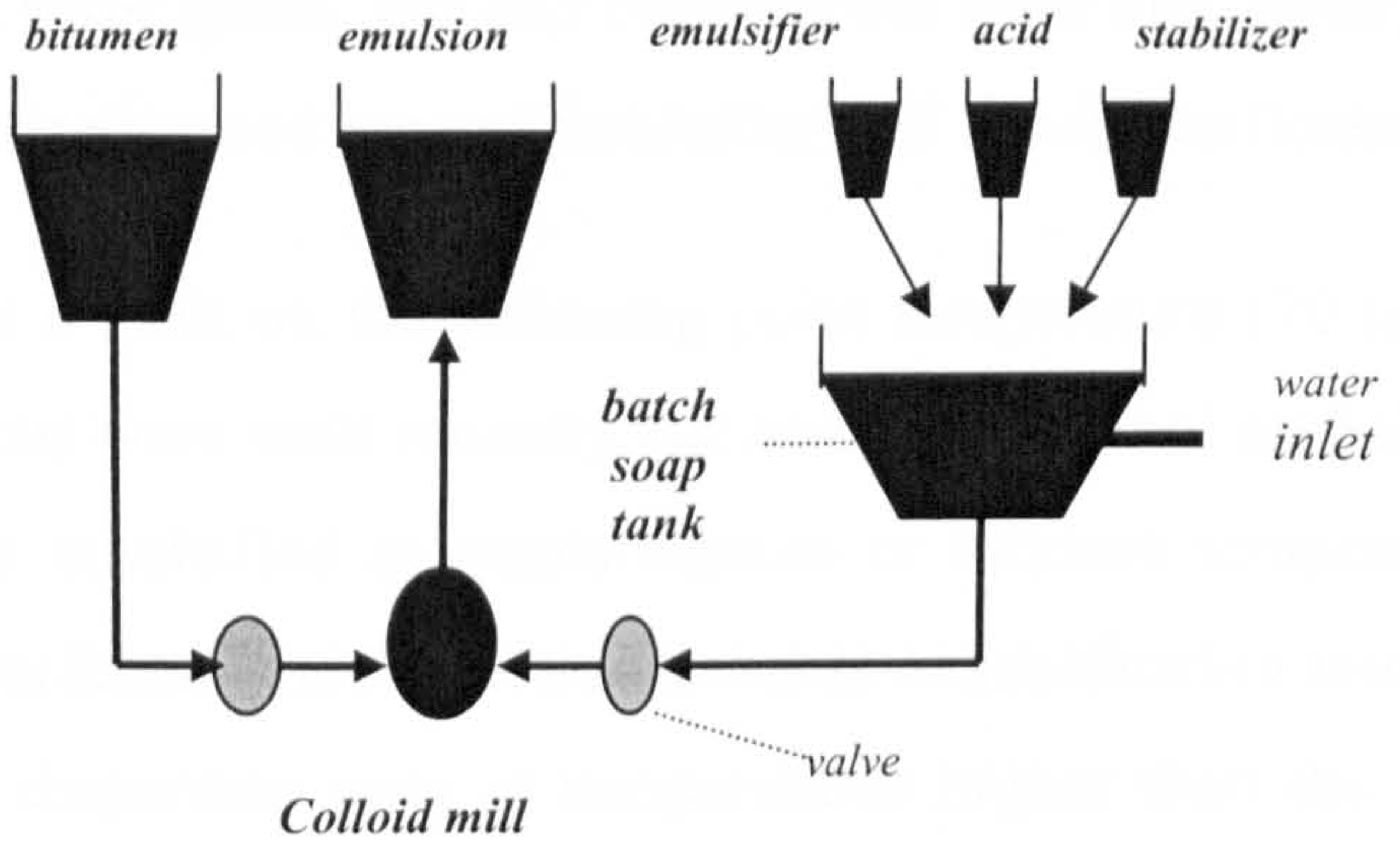


Fig. 2.4 Schematic diagram of 'batch emulsion plant'.

2.5.2 In-line Process

In the *in-line process* (Fig. 2.5) heating of the water and proportioning of each emulsifying material are performed *continuously* using individual dosage pumps.

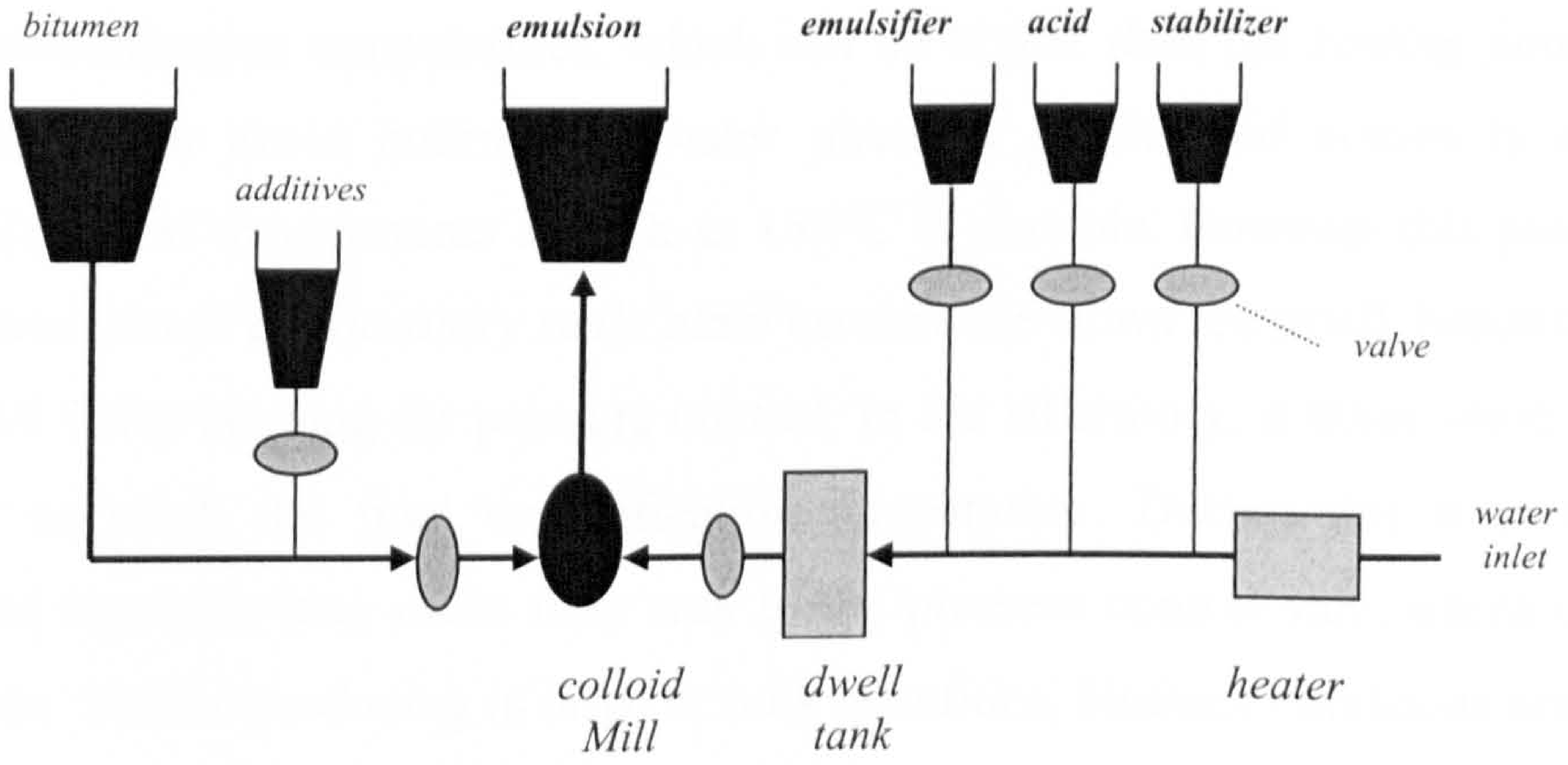


Fig. 2.5 Schematic diagram of 'in-line emulsion plant'.

There should be sufficient reaction time for the chemicals in the water phase before meeting the bitumen. This process is carried out automatically, therefore the flow meters should be carefully calibrated to ensure good control, except for the acid which should be controlled by the pH in the water phase.

2.5.3 Process of Emulsification

In this process, the bitumen is broken up into small droplets. The process is resisted by the cohesion, the viscosity, and surface tension of the bitumen, as the bitumen droplets tend to re-join or coalesce. In order to obtain an emulsion with the required size of bitumen droplets, the application of adequate mechanical energy is required to prevent their coalescence. The particle size of the emulsion can be related to several factors including: mill rotor speed, the gap between the rotor and stator, the dwell time in the mill, the concentration and type of emulsifier and the emulsification temperature.

There is a limit on the softening point temperature (70 to 80°C), above which, stable dispersions were until recently not possible. Normal asphalt with softening point around 50°C is emulsified at emulsification or mixture temperature of around 85 to 95°C. Not all surfactants are effective towards emulsification temperature and can give stability to the dispersion even at temperatures higher than the normal emulsification temperature. In a normal process, the temperature of the water phase system is limited to 90-95°C. This is in order to avoid boiling the water. Heating the water phase is needed for dissolving the emulsifiers and for maintaining the emulsification temperature after mixing with the bitumen [Logaraj et all and Decanio et all, 2000]

Emulsification of harder grade bitumens (normal or modified bitumen) requires higher emulsification temperatures, which can be higher than the *boiling point* of the water phase. To avoid boiling the water phase, a *pressurised system* is required. Emulsification at temperatures as high as 165°C is possible. However this process has some *limitations in a laboratory scale plant* because the flows are small, hence require a very small valve opening for pressure control. In the laboratory, it takes some time for the mill to reach the final emulsification temperature. During this time any un-emulsified materials may make their way to the pressure control vale, which can clog the system. Before producing in large or bulk quantities, bitumen emulsions are usually manufactured on a smaller scale in the laboratory using a *laboratory emulsion mill*, as shown in Fig. 2.6 to ensure the production of the desired emulsion.

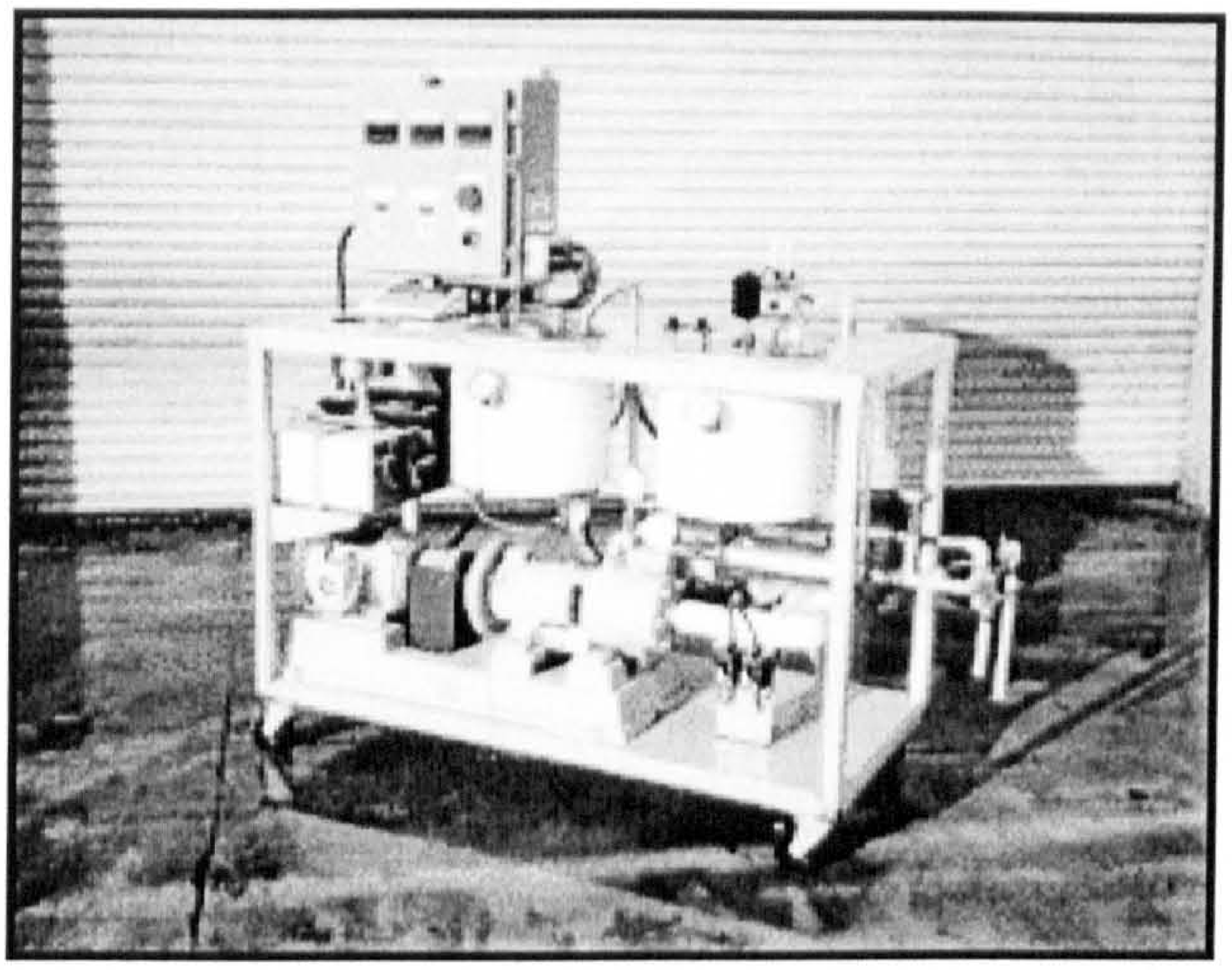


Fig. 2.6 A laboratory emulsion mill,
[Valley Slurry Seal Co., West Sacramento, Calif.-USA]

2.6 Stability of Bitumen Emulsions

2.6.1 Storage Stability and Shake Stability

Bitumen emulsions should possess sufficient storage stability during storage time and shake stability during transportation without setting, and they should also break quickly soon after application. Bitumen emulsions have complex formulations that affect their stability. Besides stability, adhesivity and viscosity are two other important properties of bitumen emulsions.

Emulsions in storage can be said to be *stable* when there is *no indication of* ***settlement***. Settlement occurs as the bitumen has a slightly higher density than water. Due to gravitational pull, the bitumen droplets (in particular the larger sizes) tend to orientate themselves to the bottom of the container. *Slow breaking* type emulsions *can remain stable* for periods from *3 to 6 months*, provided there is no evaporation of water, no possibility of electrolyte contamination, the bitumen particle sizes are mostly at optimum, and the emulsifier is not undergoing putrefaction (process towards deterioration). *Stability can* still *be considered satisfactory* when a slight amount of sedimentation can again be made uniform by mechanical agitation or by pump circulation.

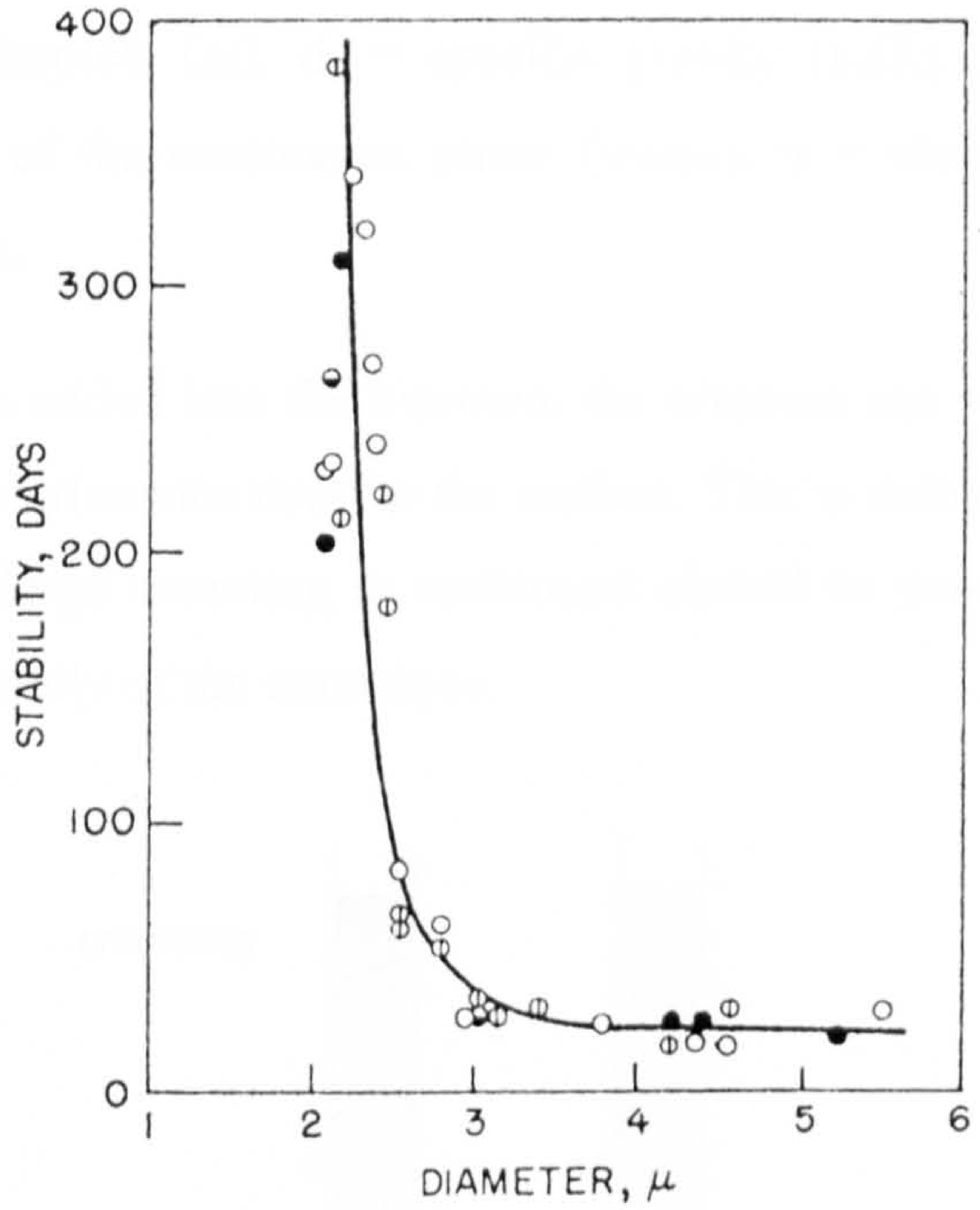


Fig . 2.7 Stability of emulsions during storage as a function of particle size, [Barth, 1962].

The effect of initial particle size on storage stability is presented in Fig. 2.7 [Barth, 1962]. Three different kinds of bitumen emulsions were tested. Particle sizes of about 3 microns or greater showed a sharp break in stability which was equivalent to one month stability. Good stability was obtained with particle diameters of about 2.5 microns. The presence of particle sizes considerably larger than this size was shown to result in settlement. Sedimentation was initially indicated by an increase in viscosity towards the bottom of the stored material. An indication of settlement can therefore be obtained by testing and comparing the viscosities of emulsion samples taken from the bottom from the top of an emulsion container.

According to Stoke's law, larger particles are more prone to settlement, which can lead to a further increase in particle size while emulsions are in storage [Needam,1998]. However, Stoke's Law does not consider the electrical charges on the surfaces of the bitumen droplets.

Stoke's equation: $$v = \frac{2gr^2(d_1 - d_2)}{9\eta} \tag{2.1}$$

where: ν = velocity of settlement (ms^{-1}), g = acceleration due to gravity (ms^{-1}), r = radius of bitumen droplets (m), d_1 = specific gravity (S.G.) of the dispersed phase (bitumen), d_2 = SG of the continuous phase (water), η = viscosity of the dispersing phase (water) in Pa.s.

If a solvent is added into the bitumen, the bitumen can acquire a lower density than water and hence orientates itself to the surface. This is called '***creaming***'. Bitumen emulsions which undergo creaming or settlement should be gently agitated in order to maintain the homogeneity of the emulsions.

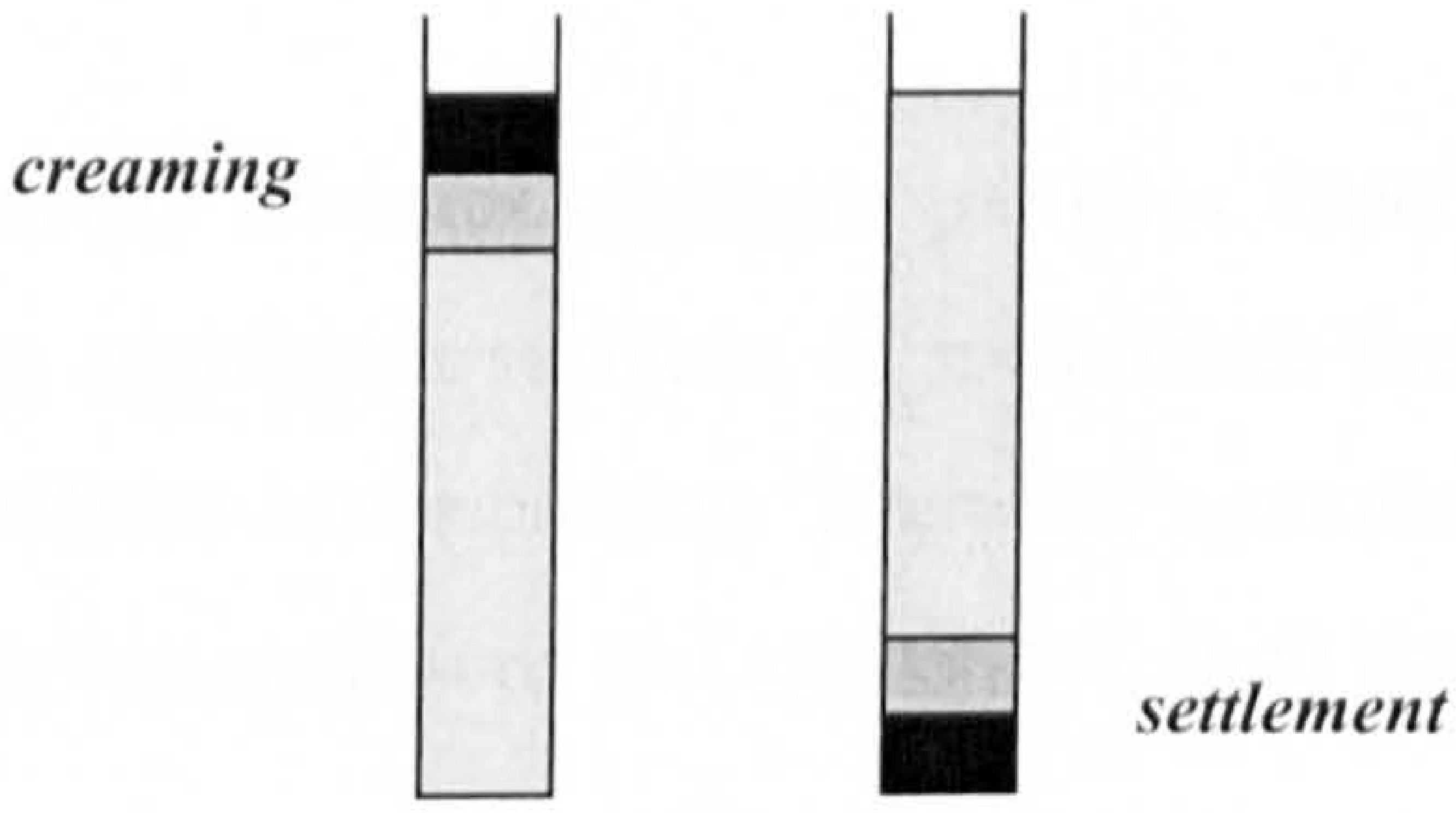


Fig. 2.8 Creaming and settlement of bitumen emulsions.

Holleran [2002] concluded that particle size (PS) and particle size distribution (PSD), which are both controllable variables, play an important role. PS is a function of mill diameter, gap, peripheral speed, and mill factor that depends on the internal configuration of the mill. This can be expressed in the following equation:

$$\text{Shear rate} = \left(\frac{2\pi RV}{60E}\right)Mf \tag{2.2}$$

where: R = colloid mill radius, V = rotational speed (revolutions/minute), E = gap dimension, Mf = mill factor.

It was concluded ***that the finer the PS and the narrower the PSD, the better the performance*** of the bitumen emulsion [Holleran, 2002, URL-2].

2.6.2 Storage of Bitumen Emulsions

Bitumen emulsions should not be stored at around 2°C or below, as such conditions can produce ice crystals and cause coagulation of the bitumen globules.

Warm temperatures of 60 to 70ºC, even up to 90ºC usually do not affect emulsions provided there *is no aeration*. This means that emulsions should be stored in a sealed condition. Loss of water by evaporation can be overcome by adding a certain amount of water into the emulsion (to keep the ratio of water to bitumen constant) followed by gentle agitation to maintain homogeneity [Barth, 1962]. To avoid evaporation of the volatiles, *a film of heavy kerosene* can be placed on the surface of the emulsions in storage as a sealant.

2.7 Adhesion of Bitumen Emulsions

Bitumen emulsion adhesivity towards aggregate particles depends upon the ability of the emulsions to coat the aggregates surfaces. In most cases the aggregates need to be slightly dampened to facilitate better coating. There are a number of factors that affect the bond or adhesion between emulsions and aggregates, such as:

- The type and the amount of emulsifier
- The bitumen grade and chemical composition
- The pH of emulsifier solution
- The particle size distribution of the emulsions
- The aggregate type
- The affinity between bitumen emulsion and aggregates

Theoretically, an emulsion adheres best to aggregates when they have opposite charges. This means that *anionic emulsions* should be used *with positively charged aggregates* such as *limestone or basalt*, whilst cationic emulsions are more suitable for negatively charged aggregates such as granite or quartzite. However, experience indicates that cationic emulsions work well with all types of aggregates. Moreover an *emulsifier* of this type can actively act as an *anti stripping agent*, i.e. is less affected by weather conditions compared to anionic emulsifiers [Leech, 1994].

Aggregates can be arranged according to their silica content (Fig. 2.9). Those with a high silica content are acidic aggregates with a lower range of pH. They tend to adopt a negatively charged surface in water. Aggregates with lower silica content are alkaline aggregates with tendency to adopt positively charged surfaces in water. However aggregates rarely consist of a pure mineral. Even a pure mineral can have both acidic and alkaline components.

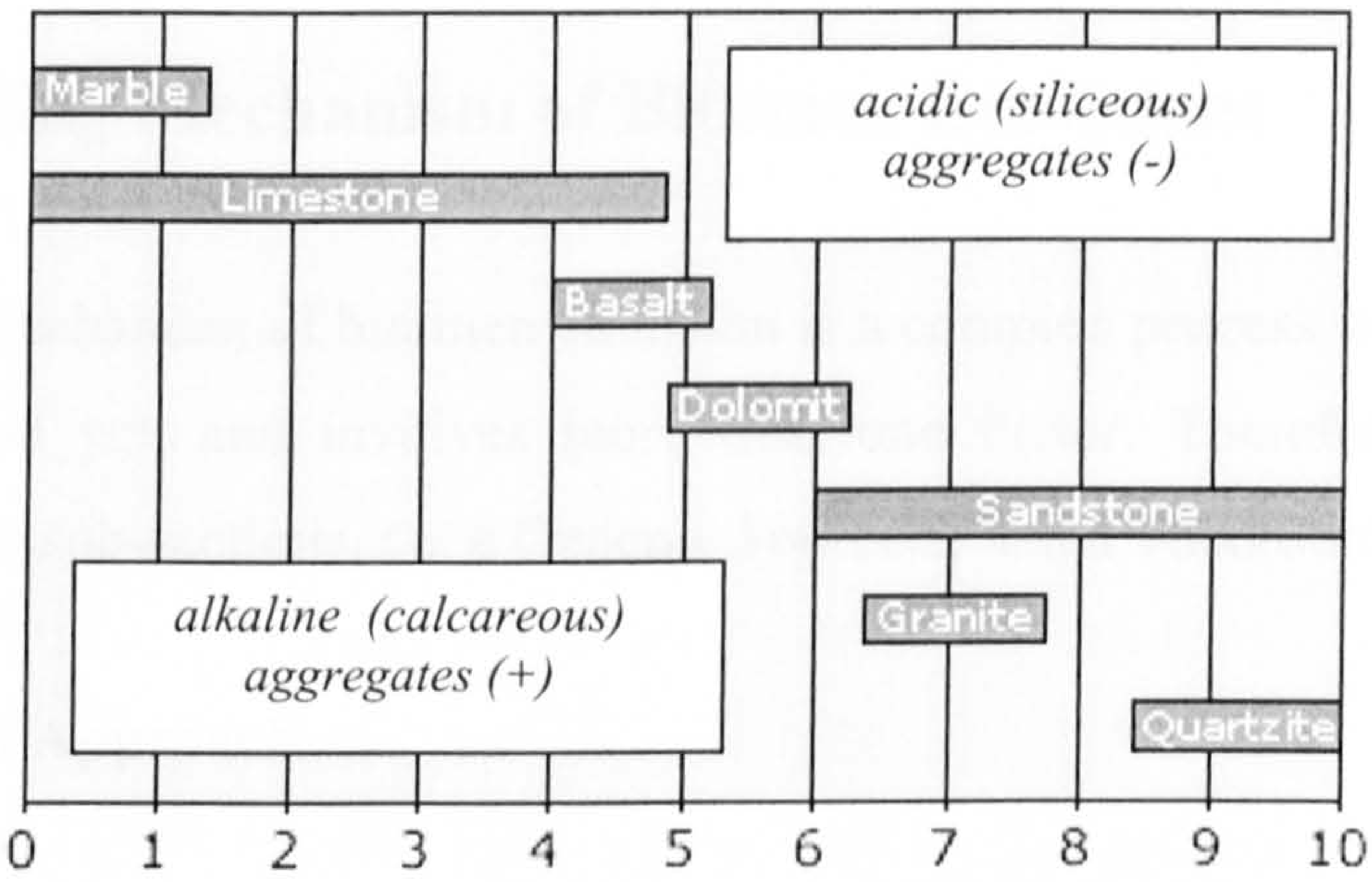


Fig. 2.9 Estimates of aggregate silica content, [Akzo Nobel, 2002, URL-1]

2.8 Viscosity of Bitumen Emulsions

The objective of bitumen emulsification is to reduce the viscosity of binder thus enabling the bitumen emulsion to coat the aggregates at ambient temperatures as well as for spray applications. Viscosity is mainly controlled by the bitumen content of the emulsion. Viscosity strongly increases as the bitumen content increases over 65%. Other factors that influence viscosity are: particle size, particle size distribution, temperature and emulsifier type. Bitumens from different sources and different refining processes can give different viscosities. Some bitumen types give higher viscosity emulsions than others. This is usually due to osmosis that can be adjusted by adding Calcium Chloride ($CaCl_2$) (see section 2.3.3). Emulsion viscosity can be corrected in the following ways:

- adjusting the level of bitumen content.

- adjusting the amount of $CaCl_2$ incorporated.
- changing the type of emulsifier.

Bitumen with a high salt content tends to give a higher viscosity. This can lead to the formation of multiple emulsion in which some water is trapped within the bitumen droplets [Akzo Nobel, 2002, URL-1].

2.9 Breaking Mechanism of Bitumen Emulsions

The breaking mechanism of bitumen emulsion is a complex process which has not been fully understood yet, and involves more than one factor. Therefore this section is divided into two sub-sections, i.e. a General Approach and a Theoretical Approach.

2.9.1 General Approach

A stable emulsion can be transported and applied without coagulation. After application, it is essential that the emulsions should break down as soon as possible. This matter can be easily achieved with *cationic emulsions* which react and *chemically break* when in contact with aggregates. In addition the cationic emulsifiers can work as an *adhesion or anti stripping agent.* The breaking mechanism can be controlled by the choice of emulsifier and emulsifier content.

Anionic emulsions do not break chemically. They *mainly break due to the evaporation of volatiles* within the system. This results in a *much slower process* with weak adhesion. Generally, the breaking rate of the emulsion is influenced by some factors, such as:

i. Evaporation of water that is dependent upon temperature, humidity, wind, and method of application.

ii. The porosity of the compacted mixture, and removal of water due to capillarity.

iii. The physical and chemical condition of the aggregates.

iv. Mechanical disturbance during compaction and due to vehicular traffic.

Some of the processes affecting the breaking mechanism may include:

- **Adsorption of emulsifier onto the aggregate surfaces.**

  This mechanism is due to opposing surface charges between the emulsifier and the aggregate that can cause droplet instability leading to coalescence.

- **Movement of the bitumen droplets towards the aggregate surfaces.**

  In this case the bitumen droplets (carrying a small charge) that are surrounded by emulsifiers, move towards aggregate surfaces with opposite charge (*electrophoresis*). The concentration of the droplets on the aggregate surfaces brings the particles together leading to flocculation, coalescence and spreading over the surface.

- **Changes in pH.**

  Some types of aggregates such as limestone, lime filler or cement can neutralize the acid in a cationic emulsion and raise the pH value. This can cause destabilization of the emulsion. In other cases the aggregates may adsorb hydrogen ions leading to a slower rise in the pH, but sufficient enough to destabilize. Some soluble aggregates like limestone can provide calcium or magnesium ions to the solution which tend to neutralize the charges of anionic emulsions.

- **Evaporation of water.**

  The bitumen droplets become concentrated as the water phase leaves the solution, thus causing coalescence of the bitumen globules. Evaporation can be the main breaking mechanism for the very slow-setting emulsions. Molecules of emulsifiers exist in the emulsion system both in the water phase and on the surface of the bitumen droplets. In an equilibrium condition, some emulsifier ions form '*miscelles*' of ions that act as a reservoir of emulsifiers. When equilibrium is disturbed such as due to evaporation of water content or electrolyte contamination, to some extent the balance can be restored by the ions from the micelles or from the surface of the droplets [Whitoeak, 1991].

The setting of bitumen droplets involves *flocculation, settlement, and coalescence*. Flocculation is a process where the droplets start adhering together. Usually there is a large central droplet with smaller droplets surrounding it. The droplets can be separated by agitation. Without agitation the droplets can settle down due to gravity. This may be followed by coalescence when the droplets merge to form larger particles.

The particles of the bitumen become closely packed together as water evaporates. Mechanical disturbances during compaction and further evaporation of volatiles (water and the emulsifier) cause disturbance of the equilibrium state of the emulsion resulting in coalescence of bitumen globules resulting in weakening of the like charges on the surfaces of the bitumen droplets. This occurs when the binder proportion in the emulsion approaches 80 to 90%. In addition, opposite charges of the aggregates rapidly absorb some of the ions on the droplet surfaces. The bitumen droplets move towards the aggregates surfaces (electrophoresis). At this stage the emulsion breaks down and adheres to the aggregate surfaces. This can be accelerated by compaction caused by passing traffic. This mechanism is presented in Figs. 2.10 and 2.11.

The *breakdown of emulsion* at the *initial stage* due to water evaporation is *reversible, i.e.* the evaporated volatiles can be replaced by rain that re-dilutes the emulsion, thus causing damage to the compacted mixture. When the weather is cold / wet and the environment is humid, the emulsions can only be partly broken down and cannot hold the aggregates satisfactorily.

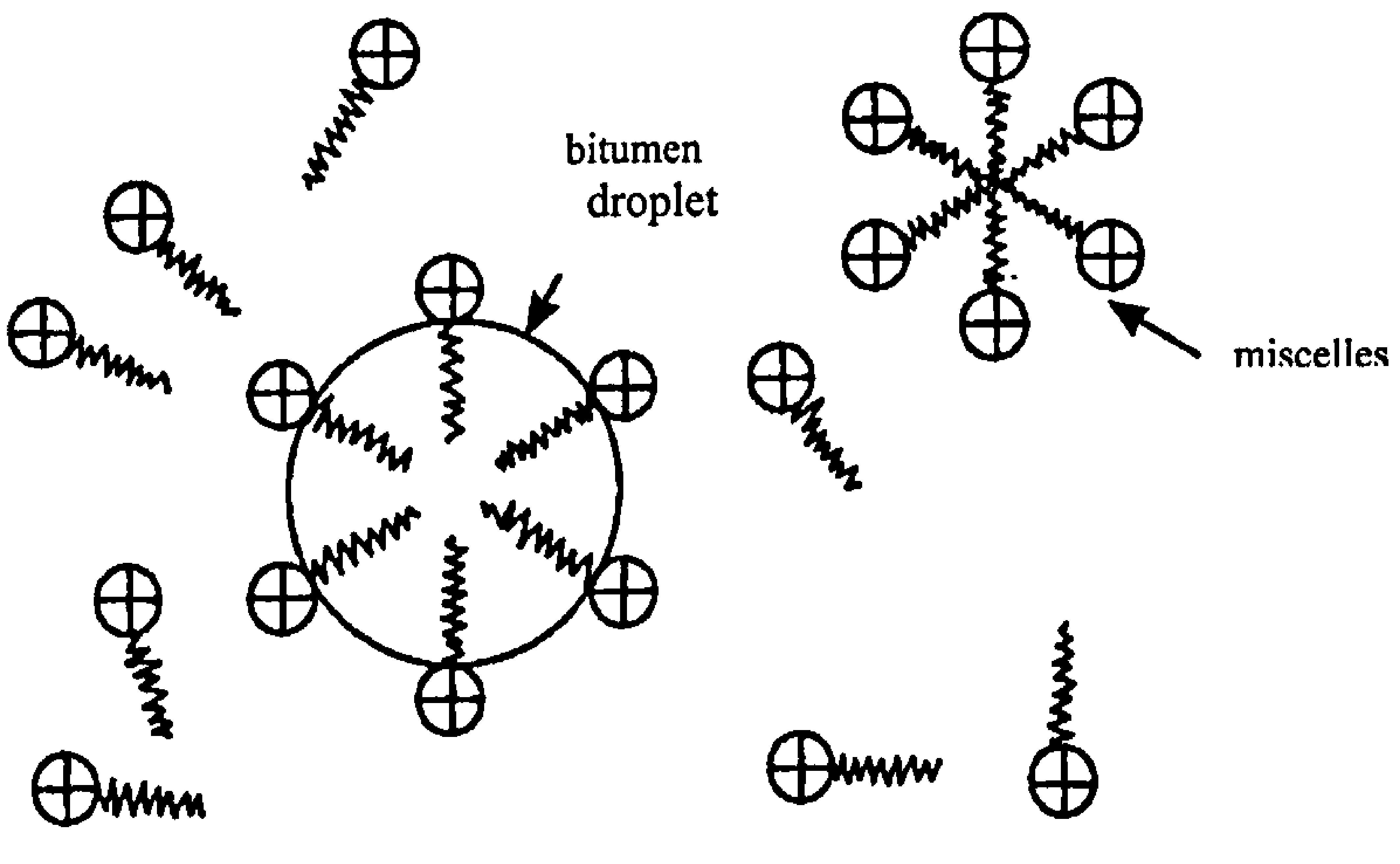


Fig. 2.10 'Miscelles' of ions in a stable emulsion solution, [Whitoeak, 1991].

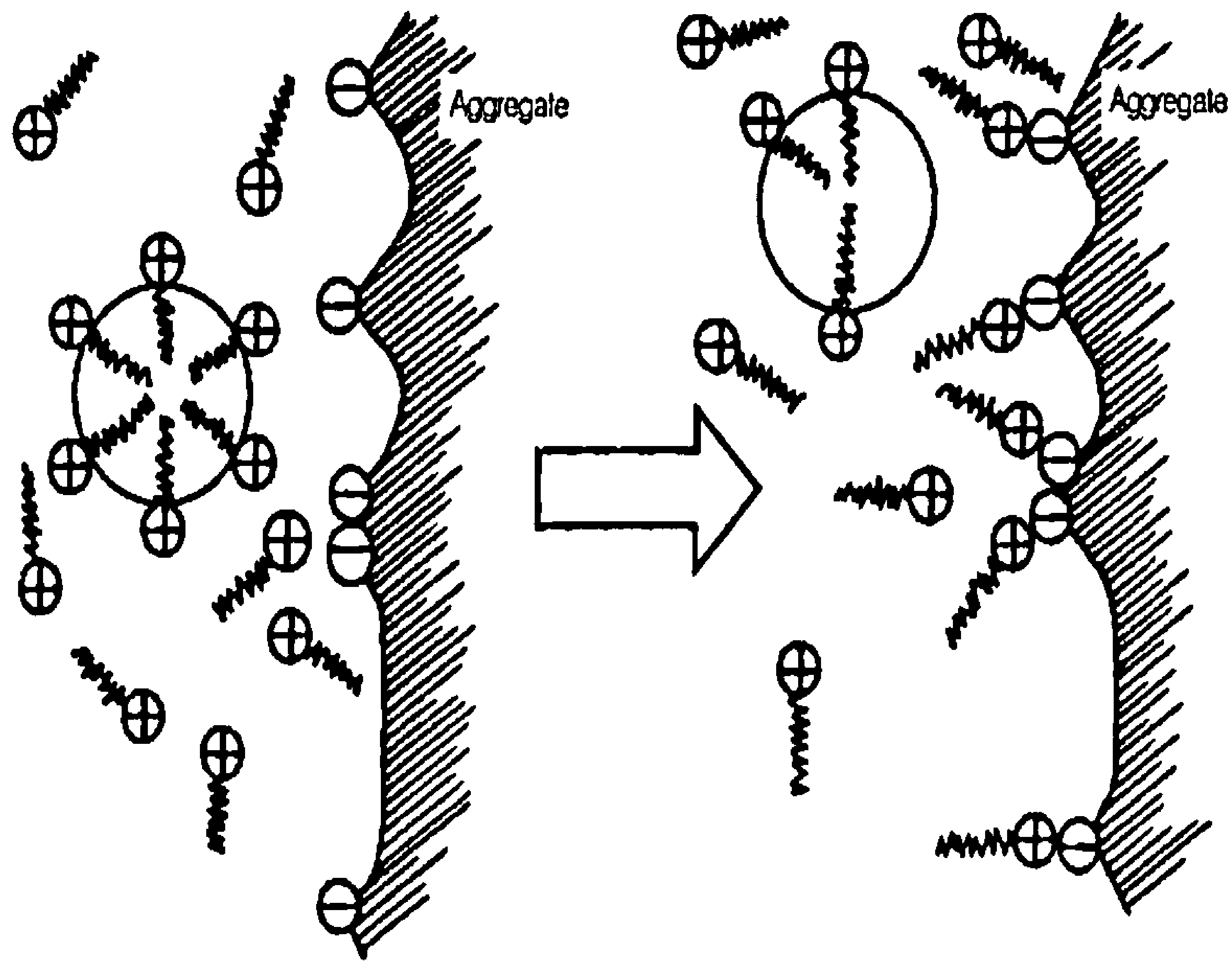


Fig. 2.11 Diagram of an emulsion breaking process, [Whitoeak, 1991].

2.9.2 Theoretical Approach

In the following section, the differences in the breaking mechanisms between anionic and cationic emulsions are explained.

2.9.2.1 Breaking Mechanism of anionic emulsions

According to Gaestel [1988], there are two breaking mechanisms in anionic emulsions.

1. With positively charged aggregates, e.g. calcareous aggregates such as limestone and basalt, the emulsifier forms a salt with the calcium ions contained in the aggregates. Due to the evaporation process, the emulsifier is removed and this causes the emulsion to break.

2. With negatively charged aggregates, e.g. siliceous aggregates such as granite and quartzite, the inorganic cation is adsorbed onto the aggregate surfaces. The emulsifier is de-activated through the loss of the counter-ions, causing the emulsion to become unstable and break. The breaking rate in this mechanism is proportional to the concentration of inorganic cations.

2.9.2.2 Breaking Mechanism of cationic emulsions

Wates and James [1993] introduced three methods to explain the breaking of *cationic emulsions onto negatively charged aggregates (siliceous aggregates) , e.g. granite and quartzite.*

1. Emulsifier Abstraction

Here the emulsifier is abstracted (taken away) or withdrawn from the bitumen/water interface by the aggregate surfaces. The emulsion becomes unstable due to the loss of emulsifier, causing coalescence of bitumen droplets (Fig. 2.12).

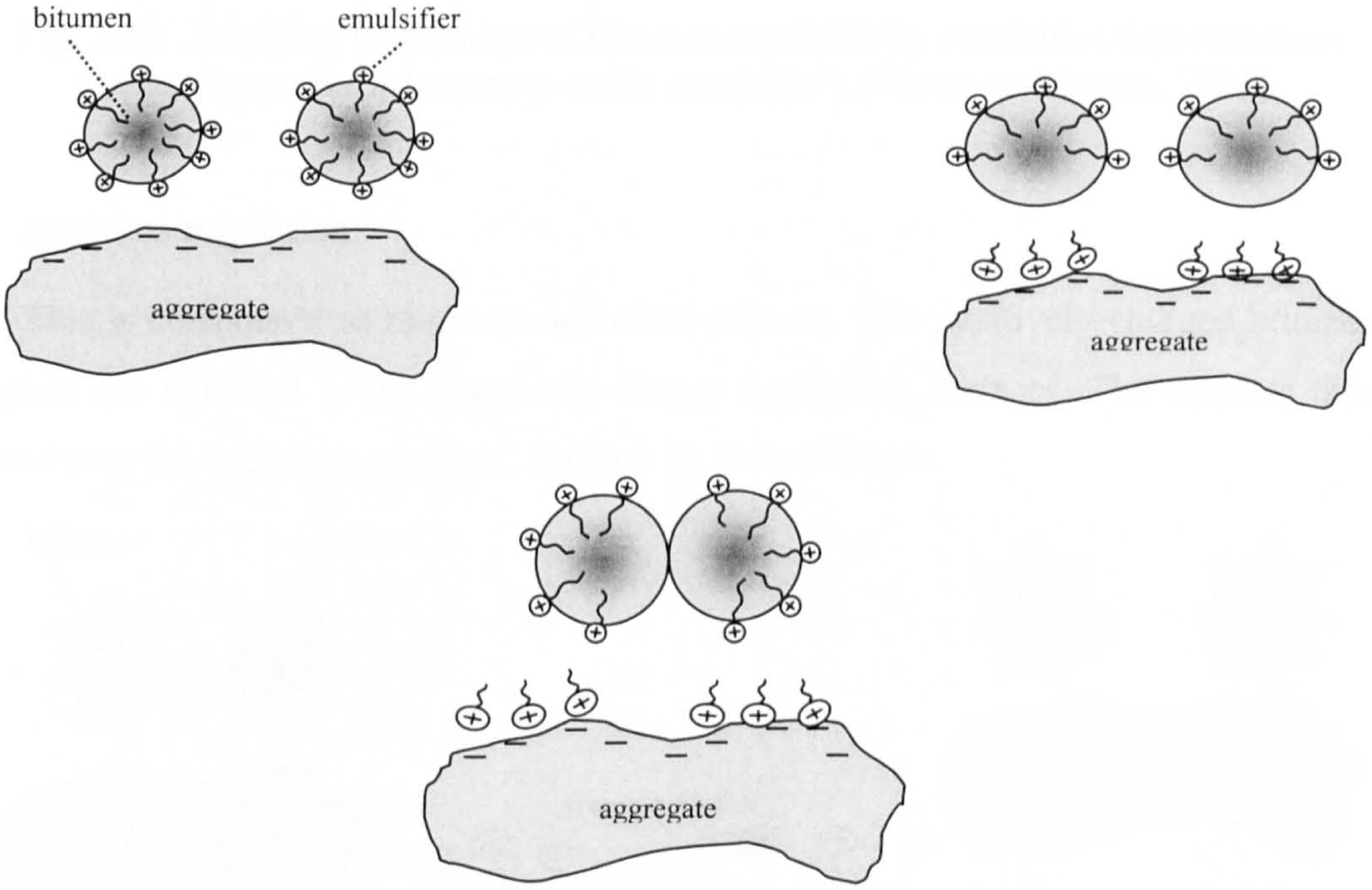


Fig. 2.12 Breaking mechanism of bitumen emulsion by emulsifier abstraction, [Wates and James, 1993].

2. Emulsifier Deprotonation

In this process the acidified emulsifiers are deprotonized in which the protons are adsorbed onto the aggregate surfaces, deactivating the emulsifier hence causing the emulsion to break. (Fig. 2.13)

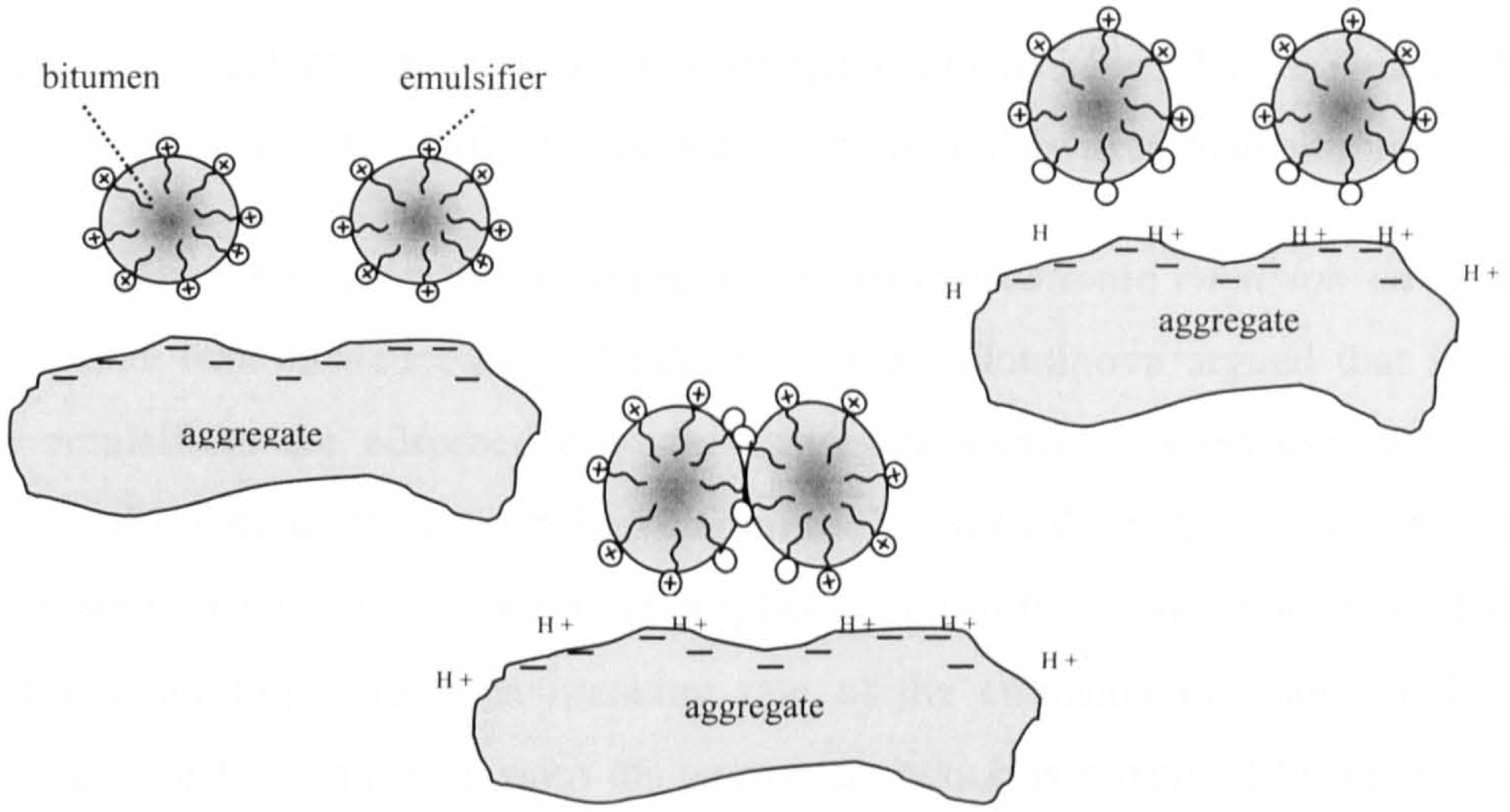


Fig. 2.13 Breaking mechanism of bitumen emulsion by emulsifier deprotonation (*generally not for quartenary amine emulsifier*), [Wates and James, 1993].

3. Droplets migration

This is considered as *the most important process*. The positively charged bitumen droplets are attracted to the negatively charge aggregates surfaces. The droplets then spread over the aggregate surfaces, assisted by the surfactant.

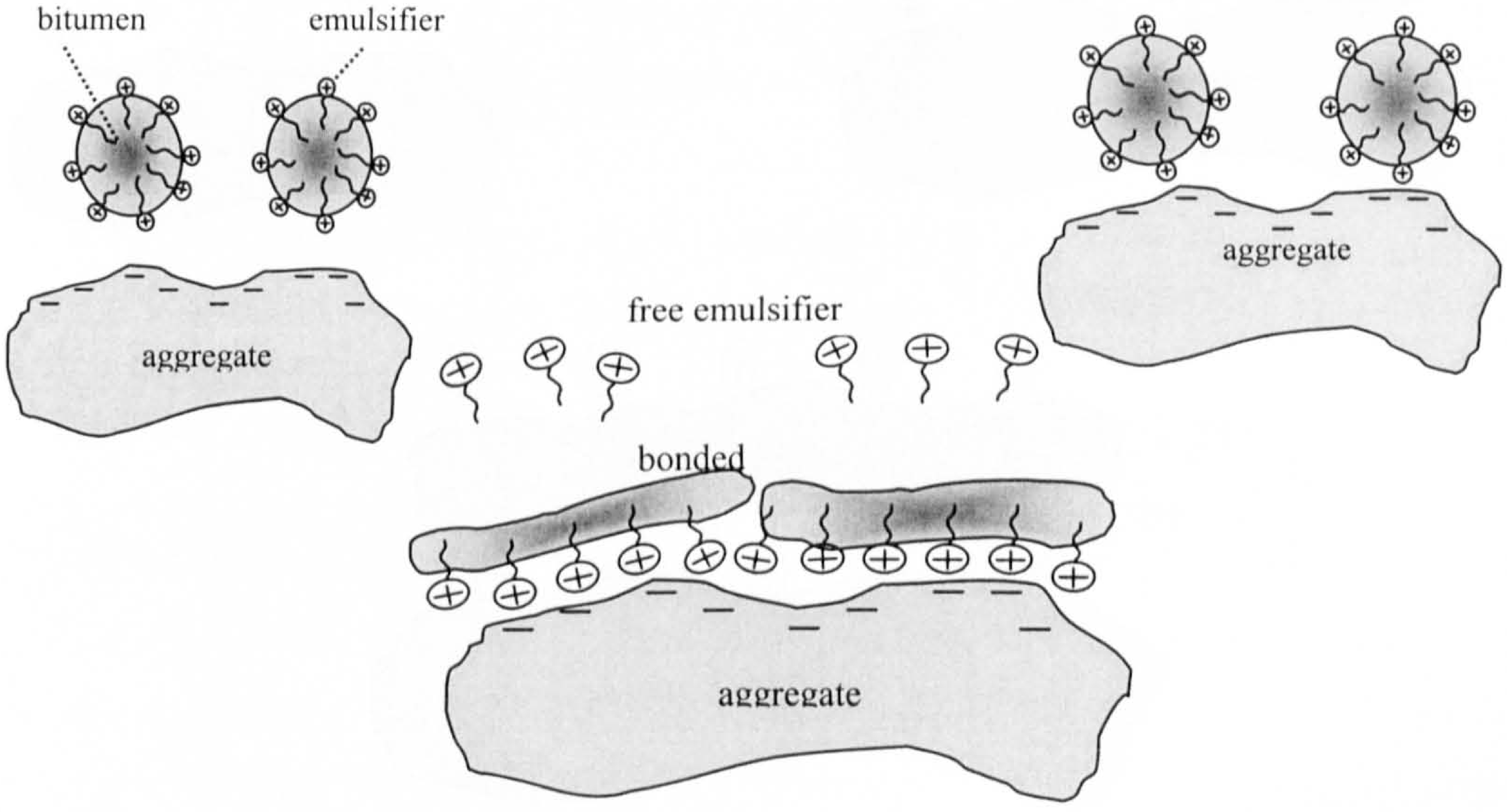


Fig. 2.14 Breaking mechanism of bitumen emulsion by droplets migration, [Wates and James, 1993].

After this the bitumen is bound to the surface by the surfactant molecules which subsequently act as *adhesion or anti stripping agents* (Fig. 2.14). This is the main reason that cationic emulsifiers have become more widely used than anionic emulsifiers.

Another theory on the breaking mechanism of *cationic emulsion on to siliceous aggregates* was introduced by Plotnikova [1993]. Plotnikova argued that initially the free emulsifiers are adsorbed onto the aggregate surfaces. Then the emulsifiers are abstracted (taken away) or withdrawn from the bitumen droplets and adsorbed onto the aggregate surfaces which become hydrophobic due to the adsorption of emulsifiers, as illustrated on Fig. 2.15. The breaking rate of the emulsion depends on the rate of adsorption of the emulsifier onto the aggregate, which is governed by the surface area and chemical nature of the aggregate and the bitumen emulsion.

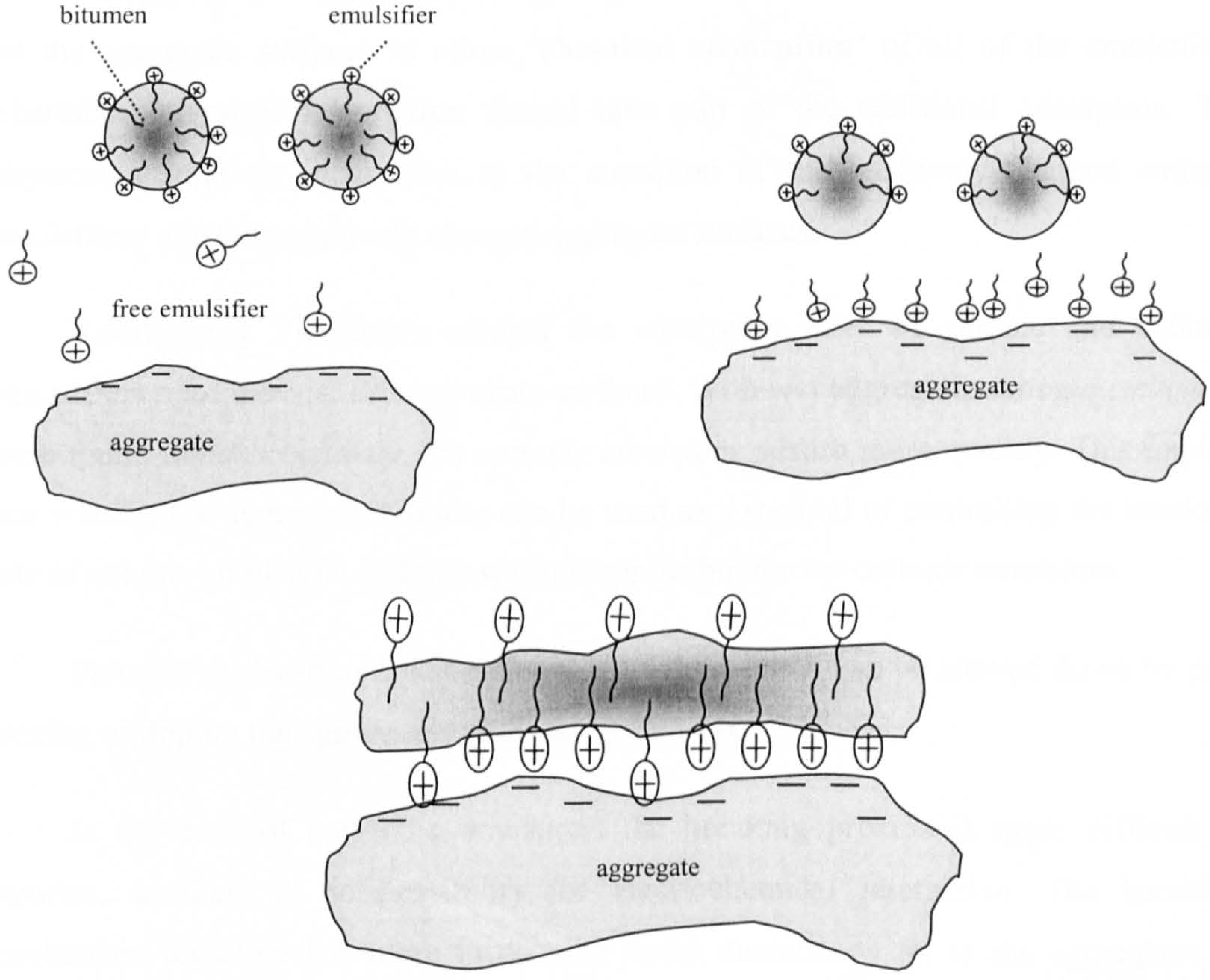


Fig. 2. 15 Breaking mechanism of bitumen emulsion, by emulsifier adsorption, [Plotnikova, 1993].

The above mechanism is thought to be *partly chemical and partly physical.* Chemi-sorption involves ion exchange between emulsifier and aggregate. As an example, in the case of cationic emulsions using a *primary amine hydrochloride* as *emulsifier* $(^{+})$ and *siliceous aggregates* $(^{-})$, the exchange of ions will be as follows:

$$\mathrm{SiOH}^{(-)} + \mathrm{RNH_3Cl}^{(+)} \rightarrow \mathrm{SiONRH_3} + \mathrm{HCl} \tag{2.3}$$

- **The case of cationic emulsions $^{(+)}$ with calcareous aggregates $^{(+)}$** [Plotnikova, 1993].

In this case ***ionic exchange capacity*** in fact is ***ten times larger*** than that of siliceous aggregates $^{(-)}$. If the aggregates are limestones ($\mathrm{CaCO_3}$), the reaction would be:

$$\mathrm{CaCO_3}^{(+)} + 2\,\mathrm{RNH_3Cl}^{(+)} \rightarrow \mathrm{(RNH_3)_2CO_3} + \mathrm{CaCl_2} \tag{2.4}$$

According to Plotnikova [1993], in general there are insufficient sites available on the aggregate surfaces to allow **'chemical adsorption'** of all of the emulsifiers. Therefore, **physical adsorption** should take part of the additional adsorption. The physical adsorption occurs due to the attraction of the positively charged cationic emulsifiers and the negatively charged aggregate surfaces.

Additionally Plotnikova studied the adsorption rates of anionic and cationic emulsifiers onto wet and dry aggregate surfaces. With wet aggregates *anionic emulsions* were found to *adsorb slowly*, but *cationic emulsions* adsorb more quickly. This implies that wetting the aggregate surfaces can be used as a method of controlling the breaking rate of anionic emulsions, but this was not applicable for the cationic emulsions.

Plotnikova also found that cationic emulsions break can be slowed down by pre-wetting or doping the aggregates with *quaternary amine.*

In the case of non-ionic emulsions the breaking process is more difficult to theorise, as there is no possibility for electrochemical interaction. The breaking mechanism therefore has more to do with water absorbency on to the aggregates or water evaporation. This is also part of the mechanism in ionic emulsions.

2.10 Specifications and Main Properties of Bitumen Emulsions

Specifications of bitumen emulsions may vary nationally. In this section, the guidelines covered within BS 434:1984 are presented. The properties of cationic road emulsions to BS434 are given in Table 2.2.

2.10.1 Particle Charge

The particle charge is determined by conducting the particle charge test in which a positive electrode (anode), and a negative electrode (cathode) are placed in the emulsion. If the *cathode* becomes coated with bitumen, this indicates that the emulsion is *cationic* with positive electric charge on its surface, and vice versa.

Table 2.2 Properties of Cationic Road Emulsions, [BS 434: Part 1: 1984].

Property	**Class of Cationic Road Emulsions**				
	K1-70	K1-60	K1-40	K2	K3
Particle Charge	Pos.	Pos.	Pos.	Pos.	Pos.
Residue on 710 μm BS Sieve (% (m/m), maximum)	-	0.05	0.05	0.05	0.05
Residue on 150 μm BS Sieve (g per 100 mL, maximum)	-	0.15	0.15	0.15	0.15
Binder Content (% (m/m), minimum)	67	57	38	57	56
Viscosity (degree Engler (°E) at 20°C)- in Pascal Second (Pa.s)	-	6 to 9	4 max	10 max	10 max
Viscosity Redwood No. 11 (s at 85°C)- in Pascal Second (Pa.s)	25 to 35	-	-	-	-
Coagulation of emulsion at low temperature	-	nil	nil	nil	nil
Storage Stability (Short Period Test) (No of inversion to clear sediment, max)	-	60	60	60	60
Storage Stability (Long Period Test) (% water content difference, maximum)	-	2	-	2	2

2.10.2 Residue on Sieving

Bitumen is slightly heavier than water, therefore the bitumen particles in the water-bitumen emulsion system tend to sediment, especially the particles with larger sizes. The residue on sieving is required as a *control on the oversized particles* in the emulsion to avoid settlement that affects stability. The percentage by weight of the bitumen residue retained on a standard sieve should meet the specifications.

2.10.3 Particle size distribution

This property can affect viscosity, stability and the overall performance of the emulsions. There are two main types of equipment for measuring particle size distribution, namely: Electrozone Method developed by Coulter, and Laser Light Scattering. The Electrozone Method is based on monitoring the flow of electric current between two electrodes. The electrodes are placed in an electrolyte which contains a sample of the particles (e.g. bitumen emulsion). As the emulsion passes through an orifice positioned between the electrodes, the electrolyte is displaced causing an increase in resistance in the circuit. The size of this increase is proportional to the volume of the particles and thus the particle dimensions of the emulsion can be calculated. Meanwhile, the Laser Light Scattering uses the diffraction pattern of laser light emerging from a particle dispersion. It involves complicated calculations which utilize the optical properties of the particles tested for the determination of particles size distribution [Needam, 1996].

2.10.4 Binder Content

Bitumen emulsions may contain up to 70% binder. This data is essential when designing the residual bitumen content of a bituminous mixture. The bitumen content as well as the penetration grade of the base bitumen is often varied to suit the particular application. The binder content can be determined by simple evaporation or distillation.

2.10.5 Viscosity

Viscosity of bitumen emulsions is usually determined as 'the efflux time from a flow cup with a standard orifice at a temperature relevant to application'. Viscosity should be low enough so that the emulsion can be sprayed or coat aggregates at ambient temperature. On the other hand, the viscosity should be sufficiently high, so that the binder does not drain during application.

2.10.6 Coagulation at Low Temperature

The test on Coagulation at Low Temperature gives an indication of emulsion reaction to freezing and thaw under standardised conditions. An example of the procedure is as follows; the emulsion is contained in a glass tube and exposed to testing temperatures from 30°C down to -4°C (BS 4434:Part1:1984). After achieving -4°C, the sample is immediately exposed to room temperature (above 15.5°C). The sample is then sieved on a 710μm sieve. The coagulated bitumen, if any, will be retained on the sieve.

2.10.7 Storage, Transport and Shear Stability

When stored under normal atmospheric conditions (other than frost), the emulsions should not separate and should re-disperse into a homogenous condition by agitation. When emulsions are *supplied in drums*, the drums should be rolled or inverted at least once a month to agitate the emulsions. The drums should be rolled on their sides for 2 to 3 minutes in opposite directions with at least 5 revolutions in each direction. Each drum is then placed in the normal standing position and then inverted (upside down) twice.

Emulsions may appear stable in the normal stability test but can show coalescence during transport or pumping. Therefore in a more realistic test, the emulsion is shaken or recycled through a pump, and the ***sieved residue*** is re-determined.

2.10.8 Reactivity

This property indicates the speed of setting of the bitumen emulsion. This property can be measured by dipping aggregates into the emulsion and the weight of bitumen deposited on the aggregate surfaces is determined. Alternatively, a type of filler such as fine quartz sand or cement can be added into the emulsion in the so-called *filler index* or *cement mix test*. In this test, the reactivity is also indicated by the weight of bitumen deposited on the aggregate surfaces. Additionally, this test can give data on the amount of filler required to achieve a particular speed of setting (reactivity) of the bitumen emulsion. This case applies when the reactivity of the available emulsion needs to be accelerated [Akzo Nobel, 2002, URL-1].

2.10.9 Properties of the Residual Bitumen

The residual bitumen obtained as a result of the binder content test, can be tested for its softening point, penetration grade, ductility etc. The properties should meet the properties of the base bitumen of the emulsion.

2.11 Zeta Potential

2.11.1 General

There are three states of matter, namely gas, liquid and solid. If one of these states is finely dispersed into another the result is a *colloidal system.* A bitumen emulsion is a dispersion of minute bitumen droplets in water assisted by a surfactant. In order to maintain stability of the colloidal system, there should be sufficient repulsive forces among the particles. The forces exist due to the presence of electrical surface charges.

Most particles in a colloidal dispersion carry electric charges on their surfaces. There are a number of origins of these surface charges. One of them is due to 'ionization' of the surface groups. Dissociation of any *acidic groups* on a particle surface will give a *negatively charged surface, e.g.* COOH is ionised to COO^- and H^+. Dissociation of any *basic groups* will give *positive charged,* such when OH is ionised to ($^+$ ion) and OH^- [Silver Colloids, 2002, URL-3].

2.11.2 Slipping Plane and the Zeta Potential

In the colloid system, an electric charge develops at the particle surface. This affects the distribution of ions in the neighbouring interfacial region, resulting in an increase of counter ions (ions of opposite charge to that of the particle) close to the surface, that forms an *electrical double layer* (Fig. 2.16). The *electrical double layer is a liquid layer* surrounding the particle that exists as two parts, i.e. the *inner region*

(Stern Layer) where the ions are strongly bound and the *outer region* (Diffuse Layer). In the diffuse layer the ions are less firmly attached. Within the diffuse layer, *close to the stern layer* exists a *notional boundary* which is called the ***'slipping plane'*** or the ***'surface of hydrodynamic shear'.*** The electric potential at this boundary is called the **Zeta Potential** [BIC, 2002, URL-5].

Zeta potential is a function of *the electric charge of the stern layer*, *the thickness of the double layer*, and the *dielectric constant.* Dielectric constant is the property of a material that determines the relative speed that an electrical signal will travel in that material. A low dielectric constant will result in a high signal propagation speed, and a high dielectric constant will result in a much slower signal propagation speed [Arlon, 2002, URL-7].

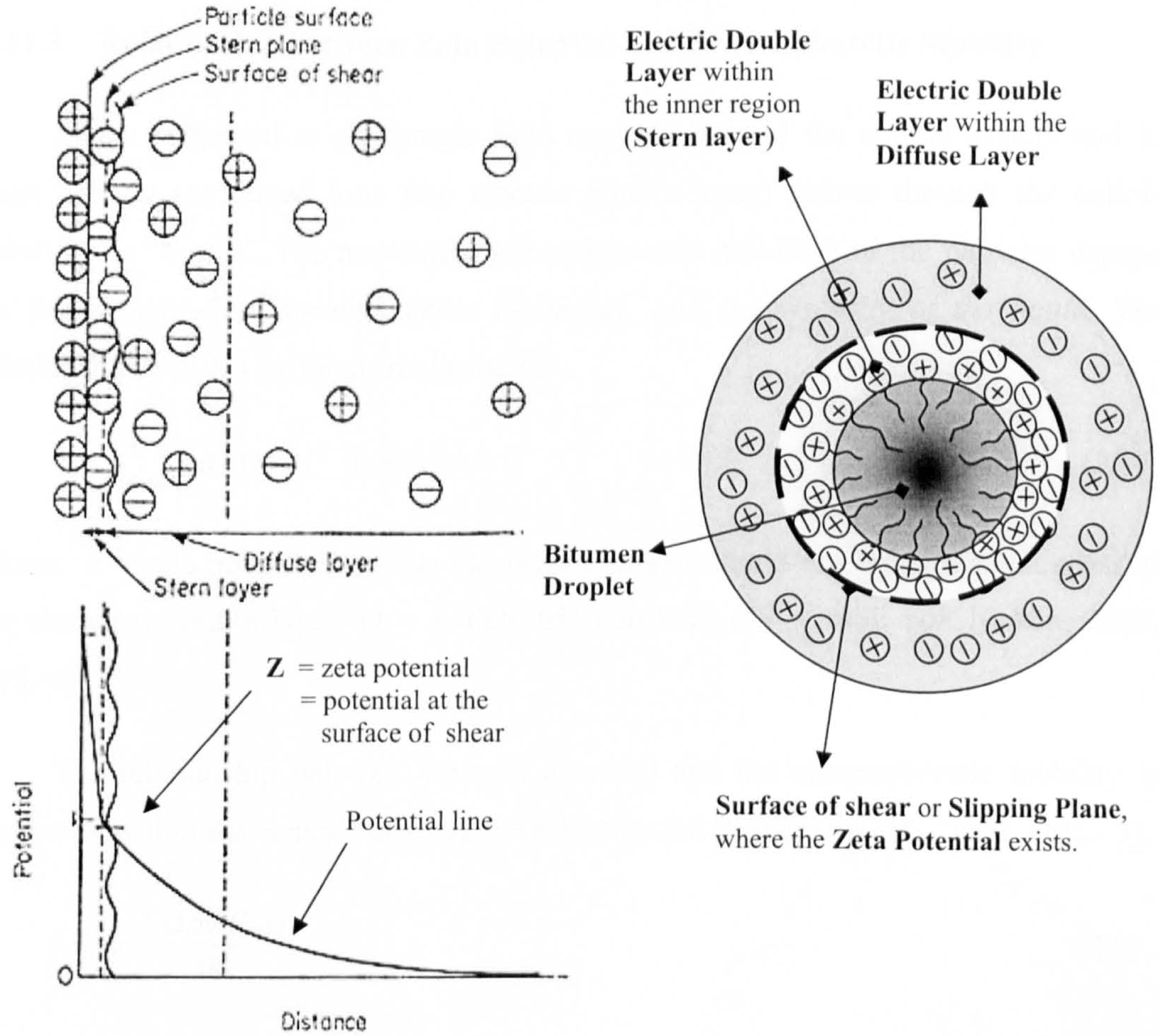


Fig. 2.16 Schematic Illustration of Zeta Potential, [Arlon, 2002, URL-7].

This constant is not easy to measure or to specify, because it depends not only on the intrinsic properties of the materials, but also on the test method, test frequency, conditioning of the samples before and during the test. Dielectric constants tend to shift with temperature.

Zeta potential is usually but not necessarily of the same sign as the potential at the particle surface. It reflects the ***effective charge*** on the particle and is therefore related to the electrostatic repulsion among the particles. This is a prime variable that controls the stability of a colloidal system and process of flocculation.

The interaction of particles in polar liquids is not governed by the electrical potential at the surface of the particle, but by the Zeta Potential [Malvern, 2002, URL-6].

2.11.3 Relationship between Zeta Potential and Electrophoretic Mobility

When subjected to an electric field *each particle* of the colloid system and its most closely associated ions (the electric double layer) moves through the colloid solutions as 'a unit'. The movement (electrophoretic mobility) of the particles depend on the *electric field strength (Zeta Potential), and the viscosity of the liquid.* The relationship is given by the formulas below.

$$z = 4\sum(e.d/D) \qquad (mV) \tag{2.5}$$

where: z = zeta potential, e = the electrical charge in the Stern layer, d = thickness of the electrical double layer, D = a dielectric constant, mV = milli volt [Orhan, 2002, URL-4].

The relationship between the zeta potential and the electrophoretic mobility is expressed within the Henry Equation, i.e. Equation 2.6 [Silver Colloids, 2002, URL- 3].

$$U_E = \frac{2.D.z.f(ka)}{3K} \tag{2.6}$$

where: U_E = electrophoretic mobility, z = zeta potential, D = dielectric constant, K = viscosity, f(ka) = Henry's function, ka = the ratio of particle radius to double layer thickness. For particles in polar media the max value of f (ka) is 1.5 (Smoluchowski approximation). For particles in non-polar media the max value of f (ka) is 1 (Huckel approximation).

The U_E can be measured using an instrument. The D (see Table 2.3) and K variables are obtained from the literature or they can be measured, hence the Zeta Potential Z can be calculated using the above equations.

Zeta Potential is a very good index of the magnitude of the interaction between colloidal particles, and it is used to assess the stability of the system. A dividing line between a stable and an unstable aqueous dispersion is generally taken either +30 or – 30mV, which means particles with zeta potentials more than +30 mV or less than (-30mV) are considered stable [Silver Colloids, 2002, URL-4]. Zeta Potential is also a measure of the magnitude of the repulsion or attraction among particles.

Table 2.3 Typical Dielectric Constant of Materials.*

Materials	Dielectric Constant
Hard Vacuum	1.0
Pure Teflon	2.1
Type GY Teflon - Glass	2.2 - 2.3
Type GX Teflon - Glass	2.55
Cyanate Ester Glass	3.2 - 3.6
Cyanate Ester-Quartz	2.8 - 3.4
Polyimide Quartz	3.5 - 3.8
Polyimide-Glass	4.0 - 4.6
Epoxy-Glass (FR4)	4.4 - 5.2
Non-woven Aramid Epoxy	3.8 - 4.1
Woven-Aramid Epoxy	3.8 - 4.1
Ceramic-Filled Teflon	6.0 - 10.2
Water	70.0

*measured at 1 MHz frequency. Source : [Arlon, 2002, URL-7].

2.11.4 Effect of pH level on Zeta Potential

The pH of the system affects the Zeta Potentials *significantly* as shown in *an example* in Fig. 2.17. From this figure it can be seen that at a certain pH level the Zeta

Potential is zero, i.e. at pH levels slightly less than 6. This point is called 'isoelectric point' (IEP). At pH levels below 4, or above 8, there will be sufficient charges to confer stability. *Different materials can have different pH and Zeta Potential relationship.*

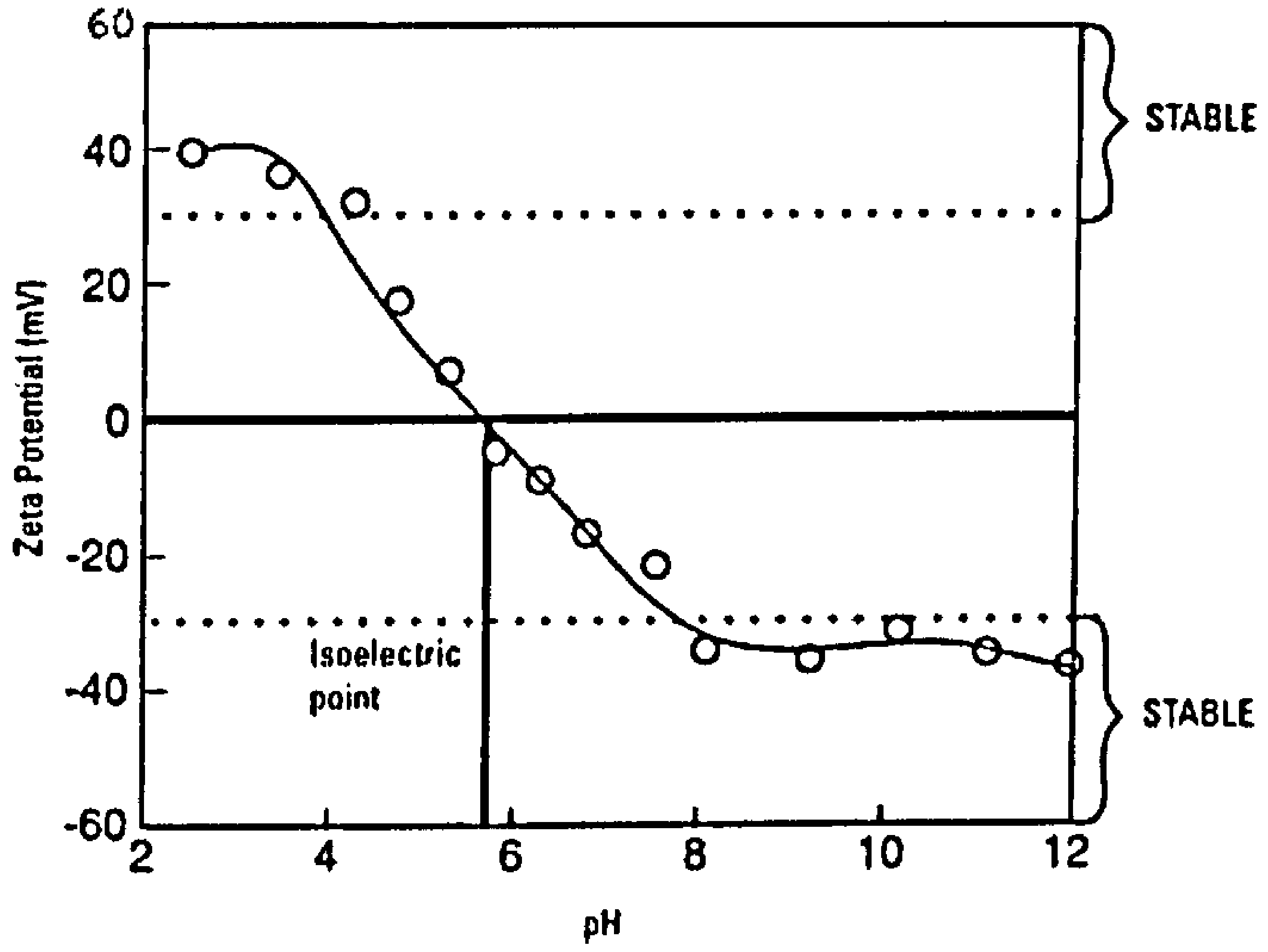


Fig. 2.17 An example of a relationship between pH level and the Zeta Potential, [Silver Colloids, 2002, URL-3].

Isoelectric point (IEP) values of some common oxides are shown in Table 2.4 [BIC, 2002, URL-5].

Table 2.4 Isoelectric Point (IEP) values of some common oxides.

Material	pH of IEP
Silica (SiO_2)	2
Anatase (TiO_2)	4
Zirconia (ZrO_2)	4
Stannic Oxide (SnO)	6
Hematite (Fe_2O_3)	8
Alumina (Al_2O_3)	9
Lead Oxide (PbO)	10
Magnesia (MgO)	12

2.12 Emulsion Inversion

An emulsion inversion is dispersion of an aqueous phase in a non-aqueous phase. This can occur within the breaking process of bitumen emulsion. This means that the system inverts from being bitumen dispersed in water, to water dispersed in bitumen. This mechanism occurs as water loses (evaporates) from oil in the water emulsion. At some point there will be not enough water to separate the bitumen globules. Further water losses will end up in a situation where the water will be dispersed in a continuous bitumen phase, hence forming an emulsion inversion. It will be very logical that at this condition, water will find it difficult to escape from the mixture as it is sealed off. This situation can affect the softening of the binder causing strength reduction of the mixture [Needam, 1996].

2.13 Modified Bitumen Emulsion Binders

Referring to the inherent problems of CBEMs, it would be an advantage if bitumen emulsions with *harder base bitumens* either from normal bitumens or modified bitumens can be made more widely available sooner. This will improve the overall performance of the CBEMs. Harder, normal or modified, bitumens are already available, however the emulsification of these harder base bitumens appear to present *some practical limitations on a lab scale plant* (see section 2.5.3). Considering that Hot Mixture products can meet the demand for harder binders sufficiently, emulsification of harder base bitumens may not yet be urgent. Additionally, CBEMs have traditionally been accepted to be more suitable for low to medium trafficked roads, which require emulsification of softer bitumen (100 pen grade bitumen or lower).

Modified Bitumen Emulsions are mostly *proprietary products.* Currently *Modified Rapid Setting* bitumen emulsions for single or double surface dressing or microsurfacing applications are available for road maintenance purposes. Those products are generally latex or polymer modified bitumen emulsions which set quickly (within 15 to 30 minutes).

Polymers can be grouped into two major families, namely (Nicholls, 1998):

- **thermoplastic, crystalline polymers or 'plastomers'** such as: polyethylene, polypropylene, polyvinyl chloride (PVC), polystyrene, ethylene vinyl acetate (EVA) and ethylene methyl acrylate (EMA).

- **thermoplastic rubbers or 'elastomers'** such as: natural rubber, styrene-butadiene- rubber (SBR), styrene-butadiene-styrene (SBS), styrene-isoprene-styrene (SIS), polybutadiene (PBD), and polyisoprene.

Provided the types of base bitumen and polymer have been carefully chosen, the modified binder will significantly reduce temperature susceptibility. The polymers will increase stiffness of the bitumen at higher temperatures hence reducing deformations, and reduce stiffness at lower temperatures thus minimising brittleness and cracking.

2.13.1 Types of Bitumen Additives in General

A bitumen additive is a material that can normally be added to, or mixed with the bitumen before or during mixture production in order to improve the properties of the binder / mixture. The overall objective of incorporating additives into bituminous mixtures is to increase tensile strength and flexibility. This is a contradiction and not simple to achieve. One specific aim of adding additives is to reduce the pavement potential to cracking and plastic deformations. Although no single additive has been found to satisfy all the requirements, some additives may provide a compromise solution. The main types of additives currently available are presented in Table 2.5

In general polymer type additives can reduce temperature susceptibility and brittleness whilst increasing ductility, toughness and tenacity. In terms of stability, additives give moderate improvements as well as in tensile properties [Button & Little, 1987].

An increase in binder viscosity can be expected at temperatures above 5°C, but there are no appreciable changes below this temperature. There is a disadvantage in increased binder viscosity caused by the addition of fillers. In this case higher temperatures are required for mixing which need more fuel for the operation of a mixing plant. Incorporation of additives (which increase viscosity) can increase the in-place cost of hot bituminous mixtures by about 10 to 15%.

Table 2.5 Types of bitumen additives, [Button & Little, 1987].

No	Category	Generic Name	Method of Addition	Suggested Conc. in bitumen (%)
I	Synthetic Rubber Type Copolymer	SBR (Styrene-Butadiene-Rubber)	Added in a mix plant as a separate stream after addition of bitumen.	3 - 5
		SBS (Styrene-Butadiene- Styrene)	Pre-blended with bitumen using high shear	3 – 5
		SBS (vulcanized)	Pre-blended with bitumen using high shear	3 - 5
		Neoprene Latex	-------------	------
		SEBS (Styrene Ethylene-Butylene-Styrene)	Pre-blended with bitumen using high shear	
		SEPS (Styrene-Ethylene-Propylene-Styrene)	-------------	------
II	Polymers	Polyolefins		
		a. Polyethylene	Pre-blended with bitumen using high shear	5
		b. Polyprophylene	-------------	-------
		Polysulfides Polyisoprenes Polybutenes Polybutylenes	-------------	-------
		Nylon and polymer resin by-products	--------------	-------
III	Copolymers	EVA (Ethylene-Vinyl – Acetate)	Pre-blended with bitumen using high shear	3 – 5
		Polyisobutylene & Polyvinyl Acetate		
		EPDM (Ethylene-Propylene-Diene-Monomer)	--------------	-------
IV	Dry Powder	Carbon Black	Batch Plant: Pre-weigh polyethylene bags Drum Plant: High shear blended in bitumen with dispersing agent.	10 – 15
		Hydrated Lime	Slurry on aggregate	1% by weight of mix
V	Organic Metalic Complex	Manganese (chemkrete-lubrisol)	Pre-blended with bitumen using low shear	
VI	Acrylic	------------	Not yet marketed (1987)	--------
VII	Anti - Oxidants	Lead and Zinc diethylditio carbonate	Pre-blended with bitumen using low shear	1 - 2
		Lead diamylditho carbonate	Pre-blended with bitumen using low shear	1 - 2
		Lead and Zinc dialkyl dithiocarbonate	Pre-blended with bitumen using low shear	1 - 2

One aspect in using additives is the '*compatibility*' between the additives and the bitumen. They can be referred to as compatible when the additives can be easily dissolved into the bitumen to form a homogeneous blend that shows no phase separation (caused by differences in the S.G.) under static conditions at high temperatures. At the end, *the utilization of additives require careful selection, proportioning, and proper application for maximum engineering and economical benefits* [Button and Little, 1987].

2.14 Utilisation of Bitumen Emulsions

Bitumen Emulsions have been used since the 1920's in the USA and some European countries. In the UK, Bitumen Emulsions are classified as shown in Table 2.6 [BS 434: Part 1: 1984].

Table 2.6 Types of Anionic and Cationic Bitumen Emulsions.

No	Type of	Class	Stability	Description
1	Anionic	A1	Labile	Rapid breakdown . Usually not suitable to be mixed with aggregates. Used cold. Subdivided based on % binder.
		A2	Semi-Stable	Sufficient stability before breakdown. Can be mixed with most aggregates. Used cold
		A3	Stable	Sufficient mechanical and chemical stab. Can be mixed with coarse and fine aggregates. Used cold.
		A4	Slurry Seal A4 slow setting A4 rapid setting	Formulated for slurry seal application. Used cold. Can be mixed using simple or mobile mixers. Suitable for special mobile mixing only.
2	Cationic	K1	Rapid Acting	Rapid setting ; Early resistance to rain ; Incompatible with agg. ; for spray application Used hot (75 - 85°C) for K1 – 70; Used cold (other than K1 – 70)
		K2	Medium Acting	Sufficient delay setting. Can be mixed with certain coarse agg. Used cold.
		K3	Slow Acting	Sufficient delay setting. Can be mixed with most agg. Used cold; slurry application.

In line with the development of CBEMs technology and demand on bitumen emulsions for various applications, the consumption of bitumen emulsion increases from time to time. Recent bitumen emulsion consumption figures are presented in Table 2.7 [EAPA, 2002, URL-8]. As can be seen in Table 2.7, amongst European countries, France produces the largest amount of emulsions. The reason is believed that France has a large network of secondary and rural roads with medium to low traffic volumes. Cold bitumen emulsion technology had provided a non-expensive way to pave those roads.

Table 2.7 Bitumen Emulsions Consumption in Major Producing Countries (in the last 30 years).

No	Country	Consumption (tons)	% of emulsions for road application
1	USA	2,300,000	5
2	France	1,010,000	25
3	Mexico	515,000	34
4	Brazil	400,000	5
5	Spain	350,000	18
6	Japan	316,000	5
7	Thailand	300,000	24
8	United Kingdom	160,000	5
9	Germany	130,000	3
10	Italy	100,000	3

Source: [EAPA, 2002].

In addition to utilization in CBEMs, bitumen emulsions have been utilised in other areas such as:

Surface Dressing (Chip Seal)

The binder is sprayed onto the road surface and chippings are spread over and rolled before the binder has cured. The process can be repeated to build up thicker layers. This work is mainly for maintenance purposes. Best results are achieved with hot-sprayed (50 to 85°C) using a rapid setting cationic emulsion with a high binder content (65 to 80%).

Slurry Surfacing

A slurry is composed of a dense-graded fine aggregate that is mixed with emulsion and water to form a slurry. Using a specially made mix-paver, and depending on the maximum aggregate size, the mix is spread onto the road surface at a thickness ranging from 3 to 30mm. For best results the mix should be produced with the required or suitable reactivity. Slurry surfacings that are laid in thick layers with quick setting polymer modified slurries are called '*micro-surfacings*'.

Plant Mixes

Structural materials can be produced from CBEMs in a plant mix. With added cement the properties of the mixtures are significantly improved.

Cold In-place Recycling

Wearing courses or even the full depth of the road can be recycled in place using special equipment. Bitumen emulsions either alone or in combination with lime or cement can be used. Typically, a cationic slow setting emulsion is used.

Soil Stabilisation

Bitumen emulsions can be used to stabilise soil by spraying them onto the soil surface to provide the binding effect, minimise erosion and protect evaporation of water from agricultural land (**mulching**). They can also assist in the germination of seeds by retaining moisture, thermal insulation, and protect the seeds from birds.

Slip Coats

Bitumen emulsions can be utilised as an impermeable membrane layer between concrete layers, in which bitumen emulsions can prevent water seepage that influence the concrete strength. Slip coats also allow concrete layers to slip (move) to each other and mechanically separate the slab layers.

Protective Coats

Buried pipes, concrete structures, and other utility facilities can be protected against environmental conditions by coating them with bitumen emulsions.

Grouting

The viscosity of bitumen emulsions are relatively low, therefore they can be used for grouting work to fill cracks resulting in impermeable structures such as grouting cracks on roads, footpaths, retaining walls, roof structures, etc.

Cement Curing

Cement concrete layers can be cured by spraying them with bitumen emulsions, which retards the evaporation of water and ensures the development of maximum strength.

Fog Seal

A light application of diluted bitumen emulsion restores bitumen on weathered surfaces and extends the roadway life at low cost.

Prime Coats

Are used on unbound and lime or cement stabilized base materials to provide good adhesion for a hot mix (the upper) layer. The emulsion can be of the rapid or medium setting types, it may be diluted with water (1 portion water, and 1 portion emulsion). The prime coat should be evenly sprayed onto the base surfaces.

Tack Coats

Tack coats are a light application of bitumen between layers of hot mix *to prevent slippage.* There are different types of emulsions that can be used, but mostly either slow or rapid setting types. Similar to prime coats, the emulsion may be diluted with water (50 to 50% ratio), with typical application of 0.40 to 1.00 litre per square meter [MPW-RI, 1990].

Penetration Macadams

In this application, a rapid or medium setting cationic emulsion is applied (sprayed) onto a compacted *open-graded* layer. The emulsion penetrates through the open surface texture thus binding the roadway [Akzo Nobel, 2002, URL-1].

CHAPTER III

REVIEW ON COLD BITUMINOUS MIXTURES

3.1 General

Cold bituminous mixtures are primarily composed of liquid bitumen and graded mineral aggregates that can be mixed and compacted at ambient temperature, with no requirement for heating. At room temperature, straight run penetration grade bitumens suitable for pavement applications generally have high viscosity (i.e. are relatively hard semisolids). There are several ways of reducing bitumen viscosity, namely: by mixing with flux oil (cut back bitumen), by foaming (foamed bitumen), and by emulsification (bitumen emulsion).

It has generally been recognised that cold bituminous mixtures have advantages in terms of simplicity in production, energy savings and environmental friendliness, as they almost produce no gas emissions to the atmosphere during production.

3.2 Types of Cold Bituminous Mixtures

The most commonly used types of cold bituminous mixtures are: Cold Lay Macadams, Grave Emulsion, Foamed Bituminous Mixtures, and Cold Bituminous Emulsion Mixtures (CBEMs).

3.2.1 Cold Lay Macadams

Cold Lay Macadams are mixtures of aggregate and low viscosity bitumens obtained by adding a solvent or flux oil to the bitumen (cutback bitumen). Flux oil is a relatively non-volatile fraction of petroleum used as a diluent to soften bitumen to a desired consistency. The performance of cutback bitumens relies on the evaporation of flux oils during application and service, which in turn depends on the volatility of the flux oils and the climatic conditions. Viscosity of a cutback depends on the amount of flux present.

There are many different types of flux oils available, namely: white spirit, kerosene, gas oil, coal tar oils (creosotes) and combinations of them (Nicholls, 1998).

Cutback bitumens are normally used for surface dressings and for macadam mixtures. These mixtures can also be used as temporary fill material in reinstatement work. Temporary reinstatements are mainly carried out to provide convenience for road users by reducing potential delays and possible accidents. These cold lay materials are subsequently dug out and replaced by hot bituminous mixtures as permanent reinstatement materials. This is because tests carried out on cored samples from Cold Lay bitumen macadams were found to have low stiffness values due to the presence of flux oil [Robinson, 1997]. Additionally, the *solvents can pollute the environment, are flammable, and are generally expensive.*

The utilization of cold lay macadams as temporary fill materials have been considered to be uneconomic, and as a result the road authorities in the UK introduced specifications in 1992 to allow the use of permanent Cold Lay Surfacing Materials (PCSMs).

In line with developments in asphalt technology, various types of permanent cold lay macadams have recently been made available. These products utilise low viscosity modified bitumens and flexibilisers which reduce brittleness of the bitumen [Instarmac, 2002, URL-9].

3.2.2 Grave Emulsion

Grave Emulsion (emulsion stabilized aggregates) was originally developed in France and has been specified in 1974 [Needam, 1996]. The binder used is *bitumen emulsion*, which is mixed with aggregates that had been *pre-wetted with water.* Grave emulsion in France is mainly used in the warmer and drier southern regions due to the water sensitivity of the mixture. It can be an economical alternative to traditional hot bituminous mixtures, especially in areas where there is a long distance to the nearest hot mix plant. The process can be used to produce mixtures suitable for base courses or wearing courses using *continuously graded virgin aggregate* mixtures normally containing between 5 and 10% passing 75μm fines. It is also commonly used for strengthening and reprofiling light traffic roads. The mixture can also be used for overlaying cement bound base courses to prevent crack propagation. Mixtures may be used shortly after manufacture. When a small amount of flux oil is added, the mixtures can be stored or stockpiled for several days before use, but these mixtures can only be used for patching and reprofiling. This is because the flux oil will soften the binder hence reducing the stiffness of the compacted mixtures and thus taking a longer time to cure.

Grave emulsion normally has *a lower binder content* than other types of cold mixtures. This leads to a mixture which is not fully coated but consists of a coated fines mastic which holds the larger aggregate fractions together. The use of a lower binder content was initially intended to promote higher aggregate contact to maximise internal friction and improve resistance to rutting. In recent years specifications had been revised towards increasing the binder content. Traditionally, Grave Emulsion is protected with a surface dressing some days after laying to seal the surface and prevent ravelling [Akzo Nobel, 2002, URL-10].

Grave emulsion requires aggregates with angular faces that can give sufficient mechanical interlock. The aggregates nominal size is either 20mm or 14mm typically with 4% residual bitumen content for *base courses* and 4.5% for re-profiling old roads. The voids content should be less than 15% (typically 5-10%). The mixture is left exposed for several months (depending on the weather conditions) to allow sufficient curing before an overlay is applied. Grave emulsion has been used in Ireland with satisfactory results, however core samples could only be taken after almost 2 years from laying [Leech, 1994]. Emulsions for Grave are normally prepared using cationic slow-setting or medium-setting emulsions with 60-65% bitumen content, which may contain some solvent.

Grave emulsion mixtures can be trafficked soon after compaction with steady speed. The full strength of the mixtures is achieved after the entire water content within the compacted mixture has evaporated. This can be a slow process depending on the weather conditions.

3.2.3 Foamed Bituminous Mixtures

3.2.3.1 General

The production technique and selected uses of foamed bitumens were first described by Prof. Ladis H. Csanyi at Iowa State University, USA in 1956. Initially foamed bitumen was produced by injecting steam into hot bitumen. The steam or vapour is trapped in thousands of tiny bitumen bubbles. The foam dissipates in less than one minute, and the bitumen regains its original properties. The bitumen has to be incorporated with the aggregates while still in its foamed state. This process was convenient for plant operations as steam is readily available. But it was found to be impractical for in situ operations that require special equipment such as steam boilers.

Improvements in the production method were developed by Mobil Oil Australia Ltd. in 1968. The modification was carried out by adding cold water instead of steam into the hot bitumen and that made the process much simpler and more economic [Muthen, 1998].

Foamed bitumen enables easy mixing and coating of *cold damp aggregates,* in particular the fine fraction. With one part of water and 50 parts of hot bitumen, foamed bitumen can be produced with about 10 to 15 times increase in volume [Ruckel et all, 1979]. Figures 3.1 and 3.2 show a schematic illustration of foamed bitumen production in which a small flow of water containing foaming agent and hot bitumen are combined in a mixing chamber for producing foamed bitumen that is then discharged through a spray header on to aggregates in the pug mill. Figure 3.3 illustrates the application of foamed bitumen in cold mixture pavements.

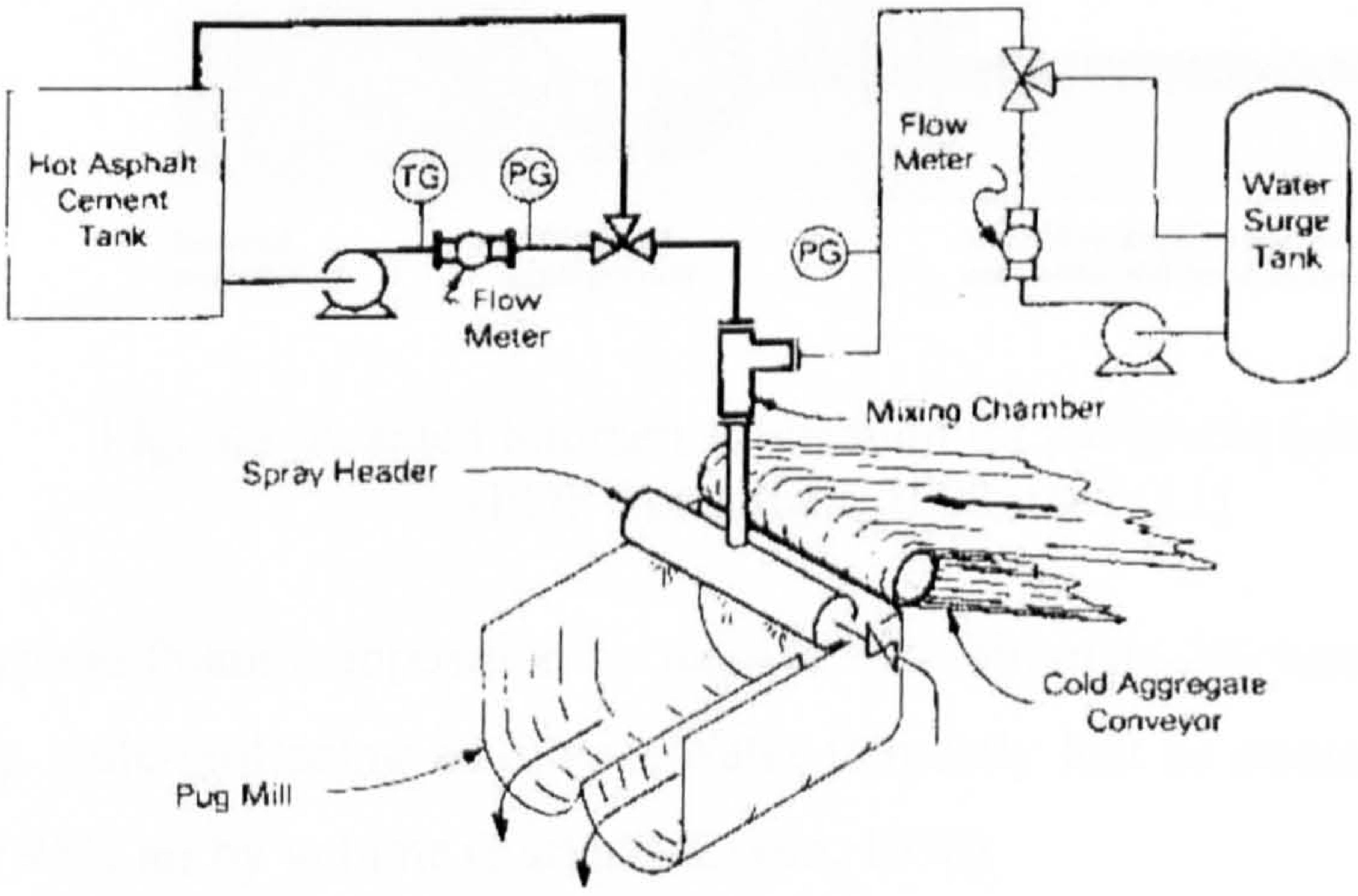


Fig. 3.1 Foamed Bitumen Cold Mixture Schematic Flow.

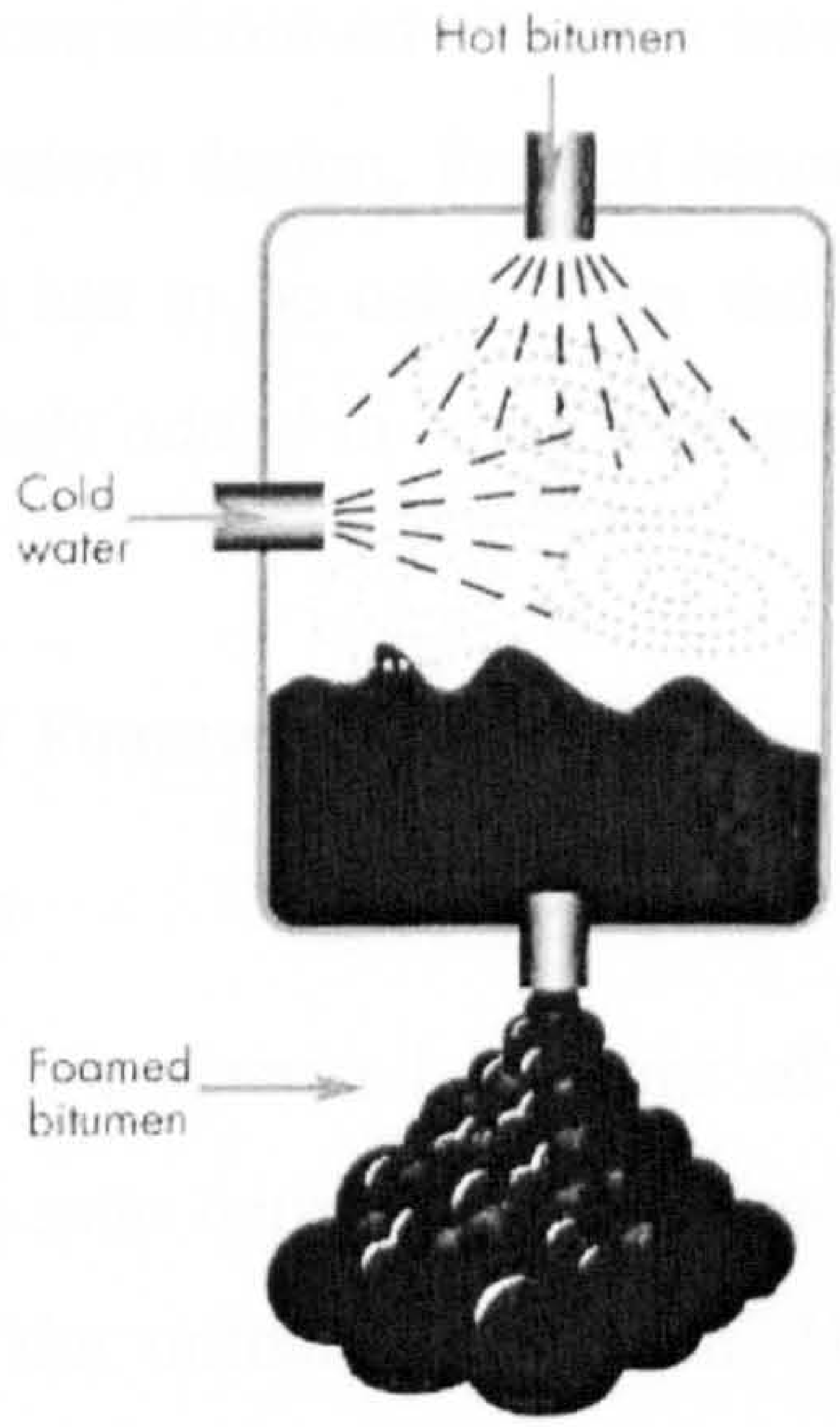


Fig. 3.2 Schematic of Foamed Bitumen Expansion Chamber.

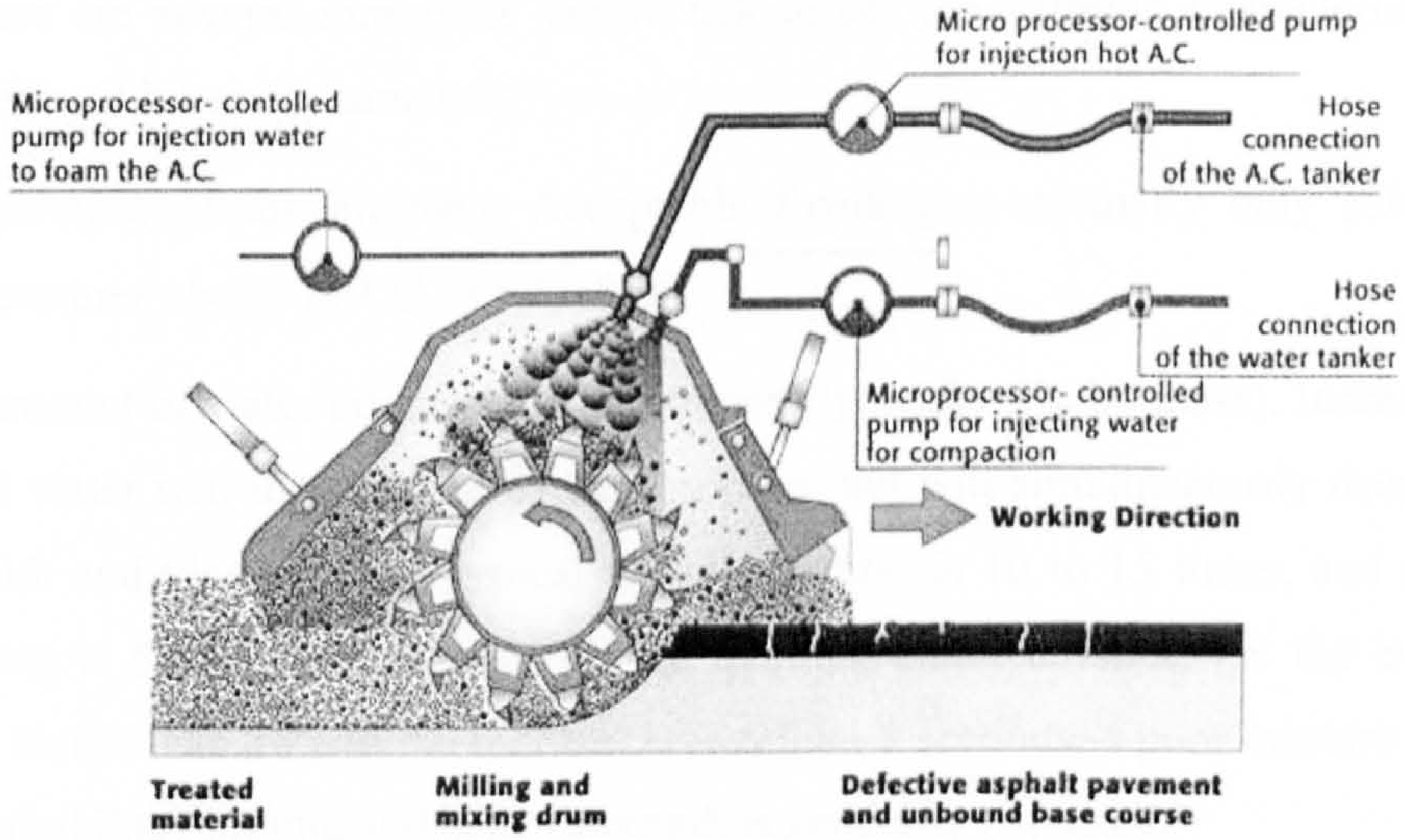


Fig. 3.3 Foamed bitumen application for pavement cold recycling, [EDP Consultant, 2002, URL-11].

A typical Foam composition by mass is 97% bitumen, 2% water and 1% foaming agent (e.g. water softening additive). Water is mostly lost as steam and the foam can consist of 95% air by volume (Earland & Nun, 1998).

The foam collapses as the air escapes, leaving a bitumen residue with similar properties to the original bitumen to coat the aggregates. *The main difficulty* is in the *development of spraying systems* that can give even foamed bitumen distribution in the mixture. Foaming can be carried out on site (in a travel or moveable plant) as part of the mixing process. For laboratory design, foamed bitumen can be stored for short periods in a sealed container and has to be used when the bitumen is still in a foamed state. *Cement or lime is commonly added* in small amounts for improving the strength of the mixture.

3.2.3.2 Characteristics of Foamed Bitumen

a. Foaming Properties

Foamed Bitumen is characterised in terms of “expansion ratio” and “half life”. The expansion ratio is the ratio between the maximum volume achieved in the foamed state and the volume of the unfoamed bitumen. Half life is the life time taken, in seconds, for the foam to settle to a half of the maximum volume attained [Wirtgen, 1998].

There are several important factors that affect the foaming characteristics of a specific type of bitumen, namely:

- Temperature of the bitumen. Acceptable foaming is normally only achieved at temperatures above 149°C [Abel, 1978].
- The amount of water added into the bitumen (typically 2% by mass). Increasing the added water will increase the expansion ratio, but will simultaneously decrease the half life and vice versa. A typical expansion ratio is 10 to 15 times, and a typical half life is 5 to 10 seconds. When the foaming characteristics, i.e. the expansion ratio, is less than 10 and the half life is less than 5 seconds, a poor mixture in terms of workability, coating and hence strength is generally expected.
- Pressure for injecting the bitumen into the expansion chamber. Pressures below 3 bars negatively affect the expansion ratio and the half life.
- The presence of anti foaming agents, such as silicon compounds in the bitumen.
- A suitable type of pressure nozzle.

Ruckel et al (1982) recommended expansion ratios between 8 to 15, and a half life of at least 20 seconds. It is possible to produce foamed bitumen with expansion ratios greater than 15 and a half life greater than 60 seconds, by incorporating “surface additives” [Maccarrone et al, 1995]. Polymers and micro-bubbles are examples of surface additives which can reduce the aero or hydrodynamic drag of the bitumen, hence are easier to expand during foaming process [Filippone, 2002, URL-13].

b. Bitumen Grade and Rheology

Better foaming characteristics are usually obtained from softer bitumens, but foaming harder bitumens can give better coating. Selection of bitumen type depends on ambient temperature. Harder bitumens (less than 100 pen.) are more suitable in hot weather. Bitumens that contain high asphaltenes and silicone content have poorer foaming characteristics.

c. Bitumen Dispersion

During the production of foamed bitumen mixtures, when the foamed bitumen is in contact with the aggregates, the bitumen bursts into very small spots which seek out and adhere to the fine particles, particularly those passing 0.075 mm (filler fractions). This creates a bitumen-bound filler that acts as mortar between the coarse aggregate particles which are thus generally less well coated with bitumen.

3.2.3.3 Design Considerations for Foamed Bituminous Mixtures

Design procedures for foamed-bituminous mixtures may vary amongst road authorities. However, the following general guidelines are common.

a. Material Suitability

Aggregate gradation for foamed bitumen should typically contain a minimum of 5% fines passing 0.075 mm. This is in order to assist the dispersion of the foamed bitumen onto the aggregate surfaces. Aggregate gradation bands that have been suggested for foamed bitumen mixtures are shown in Figure 3.4 [Muthen, 1998]. When there are less fines, the foamed bitumen can form what is called "stringers", or bitumen-rich agglomerations of the fines. Stringers tend to act as lubricant in the mixture and can reduce strength and stability. Materials which have a Plasticity Index (P.I.) > 12 should be treated with lime to reduce their (P.I.). Lime or cement can be used to achieve the minimum fines content. *However, the use of more than 2% cement by mass can have negative effects on the fatigue properties of the mixtures.*

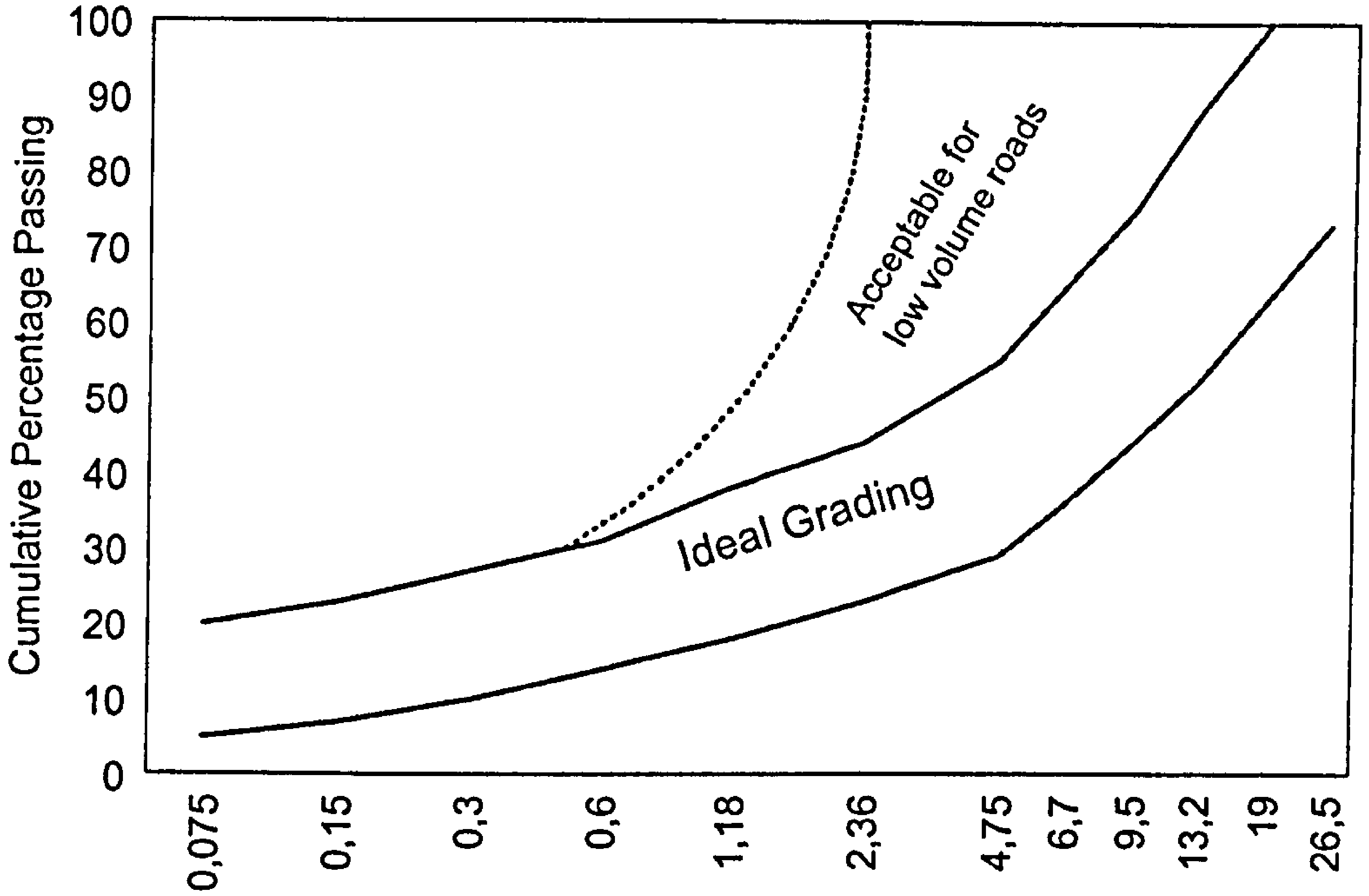


Fig. 3.4 Gradation of aggregates suitable for Foamed Bitumen Mixtures.

The stability of foamed bituminous mixtures is more dependent on aggregate interlock than the viscosity of the bitumen, hence foamed bituminous mixtures are less susceptible to temperature compared to hot mixtures.

b. Foamed Bitumen Content

The optimum binder content for foamed-bitumen mixtures largely depends on the ratio of binder content to fines content and the absorption properties of the materials. Table 3.1 can be used as a guideline to select the appropriate binder content for optimising mixture performance based on the fines content of the mixture [Muthen, 1998].

Table 3.1 Estimated Binder Content for Foamed Bitumen Mixtures.

% 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

c. Moisture Condition

Moisture (pre-wetting water) content is a significant factor influencing the performance of foamed-bituminous mixtures. The *optimum moisture content for mixing* is suggested on the *"fluff point"* of the aggregates, i.e. the moisture content at which the aggregates have maximum loose bulk volume (65% to 85% Modified/Heavy AASHTO Optimum Moisture Content-OMC) [Lee, 1981].

The concept of optimum liquid content of bituminous emulsion-aggregate mixtures may also be applicable. The total liquid content (moisture + bitumen) - TLC for *mixing* is somewhat greater than OMC. It was recommended that the best TLC for compaction (TLCc) occurs when the TLCc equals to the OMC. As the TLC is generally somewhat greater than OMC, the mixtures usually require air drying. [Castedo & Franco, 1983].

Sakr and Manke (1985) recommended the use of a Maximum Moisture Content (MMC) *for both mixing and compaction* of Foamed-Asphalt Mixtures as shown below:

$$\text{MMC}(\%) = 8.92 + 1.48\,\text{OMC} + 0.4\,\text{PF} - 0.39\,\text{BC} \qquad (3.1)$$

Where: OMC = Modified (heavy) AASHTO Optimum Moisture Content for the aggregates (alone), PF = percentage of fines (passing 0.075 mm), BC = percentage of bitumen content.

It was found by Sakr and Manke (1985) that there were no significant differences in mixture volumetric and mechanical properties when the same amount of water (MMC) was used for both mixing and compaction when compared with the commonly used procedure in which mixing is conducted at TLC and compaction carried out after the loose mixture had achieved Optimum Moisture Content (OMC) by air drying.

d. Mixing and Compaction

For laboratory mixture design, the foamed bitumen can be stored (for a relatively short time) in a sealed container to prevent moisture loss. The aggregates are dried to a constant mass. Five batches of aggregates, 10 kg each for example (plus cement or lime if required) are prepared. A suitable mixing temperature of the aggregates for foamed bituminous mixtures lies in the range of 13°C to 23°C depending on the aggregate type.

Below this temperature poor quality mixtures can result. Warmer aggregate temperatures are more desired as this will increase binder dispersion.

The aggregates, the required moisture content and the estimated foamed bitumen content are mixed in a mechanical mixer. Each sample requires about 1200 grams of loose mixture. The foamed bitumen content for each batch of aggregates shall be varied in 1% increments. The samples are compacted using heavy compaction effort traditionally using a Marshall Hammer (2 × 75 blows each end). More recently, other compaction equipment have become popular, e.g. Gyratory and Slab Compactors.

e. Curing Conditions, Testing and Optimum Binder Content (OBC) Determination

Curing for strength development of foamed-bituminous mixtures in the field can occur over several months as a result of the evaporation of water content. For laboratory testing, *oven curing at 60°C for 3 days* has been most widely used. Higher curing temperatures can cause excessive binder oxidation. This procedure results in moisture contents in the range of 0 to 4%, which represents the driest state achievable on site. Maximum strength can also be evaluated in the laboratory by curing the samples at 40°C, until a constant mass is achieved. At this condition the samples should have achieved full curing, [Lewis, 1998].

The oven cured samples are then *soaked* by immersing them in water and applying vacuum (50 mm mercury) for one hour. If a vacuum desiccator is not available, the samples can be fully immersed in water for 24 hours at 25°C [Wirtgen, 1998].

The cured samples are normally tested for Indirect Tensile Strength (ITS). It is recommended that the ITS of the dry and soaked samples should be at least 200 kPa and 100 kPa respectively [Maccarrone, 1997]. The Optimum Binder Content (OBC) should be selected as the binder content at which the soaked ITS value is maximum.

In terms of Resilient Modulus, foamed bituminous mixtures are expected to have resilient modulus values between a minimum of 1500 MPa and 6000 MPa for the soaked and dry samples respectively [Lancaster, 1994].

f. Safety Aspects

The bitumen should be heated to a high temperature, typically up to 170°C for the water reaction to produce acceptable foam. Extra safety measures should be applied to work on site at this high temperature.

Foamed bitumen mixtures can be reworked when a moisture content suitable for compaction can be maintained. The mechanical properties of the mixtures are comparable to emulsion mixtures and can be produced at a relatively lower cost, as foamed mixtures use a standard grade of bitumen. Foamed bitumen-aggregate mixtures have become competitors to Bituminous Emulsion Mixtures. Both mixture types appear to have low early life strengths.

3.2.4 Cold Bituminous Emulsion Mixtures (CBEMs)

CBEMs utilize bituminous emulsions to bind the mineral aggregates. In general, CBEMs are simple to manufacture, however they have an 'inherent problem', i.e. their insufficient early strength, hence their use is still limited to road pavements that accommodate low to medium traffic [Leech, 1994]. Under low to medium traffic conditions, CBEMs can be used in all pavement layers. CBEMs designed for use in heavily trafficked areas need to be overlaid with at least a 40mm hot bituminous mix layer [Ibrahim & Thom, 1997].

Previously, CBEMs were only used with open or semi-dense graded mixtures because of the 'intrinsic properties' of the CBEMs, in particular the long curing time requirement. Open or semi-dense gradations allow better air flow due to the higher degree of air voids. Currently cold mixtures can be produced even with dense gradations as a result of improvements in emulsion technology and mixing techniques.

Cold Bituminous Emulsion Mixtures, may consist of continuously graded or gap graded aggregate gradations and bituminous emulsions [Ibrahim & Thom, 1997]. These mixtures can be mixed at 'ambient' temperatures. The design of cold bituminous emulsion mixtures requires a suitable aggregate gradation, bitumen emulsion and added water. The breaking mechanism of the emulsion and the compact-ability of the mixture determine the behaviour of the mixture in service. The breaking of the emulsion during curing covers the total evaporation of water followed by an effective distribution of the mixture's components and coating of the aggregate by the bitumen emulsions [Zizi et. al, 1997].

Developments and utilisation of CBEMs vary significantly in different countries. France and the USA appear to have long term experiences in bituminous emulsions for road works. In France CBEMs are known as *Grave Emulsion* as had been described in Section 3.2.2. In the USA Cold Bituminous Emulsion Mixtures are called Emulsified *Asphalt Materials (EAMs).* The design of EAMs is described in the "Basic Emulsion Manual", Manual Series (MS-19) 3rd Edition 1997, published by the Asphalt Institute. The Asphalt Institute Thickness Design Manual (MS-1, 1991), categorises EAMs into three types as listed below:

EAMs Type 1: EAMs made with processed, dense-graded aggregates.

EAMs Type 2: EAMs made with semi-processed crusher-run, pit-run, or bank-run aggregates.

EAMs Type 3: EAMs made with sands or silts.

The grade of base bitumen is selected based on satisfactory coating. The percentage coating required for surfacing materials is 75% and for base materials the value is 50%. EAM's Type 1 do not require complete coating of aggregates but the aggregate particles should have 65% crushed faces for good interlock. Slow setting emulsions are often used with 0.5% to 2% ordinary portland cement added to improve early life strength.

Type 1 EAMs are considered to have strength equal to asphalt concrete. The layer thickness is recommended not to exceed 50 to 75 mm to facilitate accelerated curing. *However curing of EAMs may take from 2 months to 2 years*. EAMs also require a good level of compaction and any additional upper layers of EAM, if required, should be laid after the initial curing of the lower EAM layer has been completed which depends on the weather conditions.

3.2.4.1 The Prospective of CBEMs

It is clear that the demand for CBEMs increases in line with pressure on the environment, energy saving, health and safety issues. CBEMs have gained wide acceptance in the USA, France, Spain and some other European countries, whilst in the

UK they have not been widely used, but CBEMs use is being encouraged. *Slow acceptance and reluctance to accept cold mixture materials* may be attributed to the *natural resistance to change* in the related industry [Taylor, 1998].

The need for cold mixture materials is more apparent in *reinstatement and maintenance of road networks* that are *geographically wide spread* and *for small scale works*. Cold mixtures that can remain workable for sufficient periods have become an attractive option. Taking this into account the use of hot lay materials is not a viable choice. A general comparison between cold and hot mixtures is presented in Table 3.2 [Taylor, 1997].

Table 3.2 General Comparison between cold mixture and hot mixture.

Cold Mixture	Hot Mixture
Binder storage at refinery (cold).	Binder storage at refinery (hot).
No special transport.	Special tankers and equipment.
Simple storage plant.	Heated and lagged storage and pipe work at plant.
Mix cold with damp aggregates.	Mix hot with pre-dried aggregates.
Easy to store mixed materials.	Storage needs care.
No special transport for mixed materials.	Lagged and sheeted lorries required.
Wide time and temperature window for laying and compaction.	Narrow time and temperature window for laying and compaction.
Not affected much by temperature variation.	Hot weather - danger of wheel tracking and over embedment of chippings. Cold weather - poor compaction.

In terms of energy savings, cold mixtures appear to be much more efficient than hot mixtures. In the UK around 30 million tonnes of hot mixture coated road stone are used every year. The energy required is about 6 to 7 million MWh which equates to the production of 1 million tonnes of CO_2 [Kennedy, 1998]. Detailed case studies reported by Kennedy (1998) concluded that the total energy consumption using cold mixture technology is in the order of 40% to 60% of conventional hot mixtures. Stricter controls on emissions from hot mix plants provide further incentive towards the development of more advanced cold-mix technology [Leech, 1994].

Cold bituminous mixtures incorporating waste materials can additionally reduce the demand on virgin aggregates that are mostly explored from natural resources. Long term exploration of such materials can affect the equilibrium of the surrounding ecosystem. In a study reported by Dinnen (1998) on trials carried out at Lincolnshire in 1985, a road section was recycled to a depth of 100 mm using 5% bitumen emulsion. The total energy saving was between 20% and 30%.

With respect to Health and Safety issues, cold mixtures have much lower risks compared with hot mixtures especially the risk of skin burns.

Traditionally cold lay macadams have been used in trench reinstatements for footpaths and as 'temporary fill materials' for carriageways. These mixtures use 'flux oil', which is added to the bitumen for reducing the viscosity (cutback). The mixtures can remain workable at ambient temperatures for several days. However, studies have shown that cored samples of these mixtures revealed 'low stiffness values', well below the target values [Biczysco,1997]. Therefore these materials are only used as 'temporary or interim materials' that must be dug out and reinstated with hot-mix materials. This matter is targeted for reform as part of the Permanent Cold Lay Surfacing Materials (PCSMs), by the Highway Authority and Utility Committee (HAUC) Specification. The two main types of alternative cold mixtures that are expected to meet specifications are cold bituminous emulsion mixtures and cold foamed bitumen mixtures.

Despite the advantages with cold bituminous emulsion mixtures, there are a number of problems that still have to be overcome [Robinson, 1997]:

a. Poor coating due to the incompatibility between the emulsion and the aggregates.

b. Low binder film thickness on the coarse aggregates. More emulsion is attracted to the fines.

c. Binder drainage in storage due to the emulsion's low viscosity.

d. Poor early life adhesion/cohesion due to the presence of water.

e. Binder stripping off the aggregate caused by water sensitivity and poor adhesion.

f. High residual voids as a consequence of the materials inherent resistance to compaction.

g. Weak early life strength.

h. Long curing periods.

Most critically, the relatively wet and cold UK climatic conditions do not provide good support for a higher curing rate [Leech, 1994]. Additionally, there are sufficient Hot Mix Asphalt Plants available, and less remote areas. These matters increase the challenge for the emulsion mixture technology.

Emulsified bituminous mixtures will pass a '*cheesy state*' during or soon after laying. Within this period the mixture is in an intermediate stage of breaking where no cohesive strength exists [Leech, 1994]. At this stage cohesion can be accelerated by mechanical action such as rolling or compaction. Trials have been carried out in the UK involving collaboration between the government and the private sector. Test results have indicated that further studies need to be carried out to overcome the problems mentioned above whilst considering the following aspects:

a. The liquid bitumen should be free from flux oil.

b. The emulsion should break during compaction, and require short curing time.

c. Compaction should preferably be carried out during the summer with a low ground water table level.

d. The mixing technique / procedure and the affinity between the emulsion and the aggregates should be assessed. Most emulsions quickly absorb the fines leaving the coarse aggregates unevenly coated.

e. The selection of a suitable aggregate gradation.

f. Selection of the right proportion of added water to pre-wet aggregates, as water acts as viscosity reducing agent to facilitate better coating, replacing the need for heat.

g. The right water content that provides optimum lubrication during compaction for achieving maximum density.

h. Adoption of a higher compaction effort where possible, for reducing the air voids content.

i. Selecting suitable emulsion formulations.

3.2.4.2 Stages of the CBEMs Process

Three are three stages in the CBEMs process during which the emulsion is expected to perform different functions [Taylor, 1997], namely:

a. The First Stage is mixing the emulsion and the aggregates. The emulsion must remain sufficiently stable and coat the coarse and the fine aggregates evenly.

b. The Second Stage is storage and laying of the mixed materials. At this stage, the emulsion is required to remain workable and be partially broken or set, so that the emulsion can resist moisture and rain. The mixture must also remain un-drained (i.e. the emulsion does not drain off due to its low viscosity) after mixing with the aggregates.

c. The Third Stage is the compaction process during which the emulsion should break quickly and revert back to its original base bitumen.

Most emulsions require relatively long curing times, i.e. time for evaporation of the volatiles that results in a complete break of the emulsion by which time the mixture will achieve its maximum strength.

3.2.4.3 Recent Studies on Cold Bituminous Emulsion Mixtures in the UK.

In the UK there is limited laboratory and field data available on the performance of cold emulsion mixtures. A number of studies have nonetheless been carried out by some researchers in order to gain clearer understanding and improvement of the cold mixture characteristics.

Khalid and Eta (1997) investigated the effect of Polymer Modified Emulsions (PMEs) on the characteristics of Emulsified Bitumen Macadams (EBMs). A close graded wearing course and a dense graded base course were chosen. The emulsion used was a cationic type with 65% bitumen content. The base bitumen was 100 pen grade with Ethylene Vinyl Acetate (EVA) and Styrene-Butadiene-Styrene (SBS) admixtures.

Samples of 100 mm diameter for ITSM tests were prepared, each compacted by 85 kN static load applied for 5 minutes. This procedure was found to give the highest density and lowest void content. The samples were then cured in their compaction moulds for 48 hours before extrusion followed by oven curing at 40°C for 14 days to obtain full curing.

Three point flexural fatigue tests were carried out using beam specimens in the controlled stress mode. Initially roller compacted slabs of 305×305×50mm were prepared. The slabs were cured at 20°C for 21 days then cut into 240×40×30mm beams. Further curing for 15 days at 20°C was applied before testing. *Failure was defined as the point when the material's elastic stiffness was reduced to 10% of it's initial value.*

It was concluded that polymer modified bitumen emulsions improved stiffness and permanent deformation of the mixtures when compared with mixtures using unmodified bitumen emulsions. Wearing course mixtures containing 4% SBS and 6% EVA modified emulsions improved the mixture properties significantly. The fatigue lives of the 4% SBS and 6% EVA modified EBMs were about 45 to 35 times higher respectively compared to the fatigue life of the unmodified EBMs (which was approximately 8,119 load cycles).

Meanwhile Ibrahim and Thom (1997) studied and evaluated the behaviour of emulsion-aggregate mixtures with emphasis on the effect of curing procedure and compaction method. The aggregate gradation chosen was the mid point of BS 4987: Part 2 limits. Two cationic type emulsions were used; Redicote and EN998. In each case, the emulsion bitumen contents were 62% and the base bitumen was a 100pen. The aggregates were pre-wetted with 3.5% water. After 10 minutes of aeration, the aggregates were mixed with the emulsion. Optimum water content for compaction was approximately 4.5% by weight of dry aggregates. Marshall size samples were compacted using a vibrating hammer. The specimens were cured for 40, 55 and 70 days at 20°C, and 5 days in an oven at 48°C.

ITSM tests were carried out under controlled stress conditions according to BS DD213, 1993. The tests showed that the stiffness values were highly dependent on the stress level applied. The higher the tensile stress value, the lower the stiffness. This reduction in stiffness caused by an increase in the applied stress level was explained to be the result of the increased strain values. The stiffness of the EN998 emulsion mixtures were found to be less than the Redicote emulsion mixtures under all tensile stress levels applied, as illustrated in Figure 3.5.

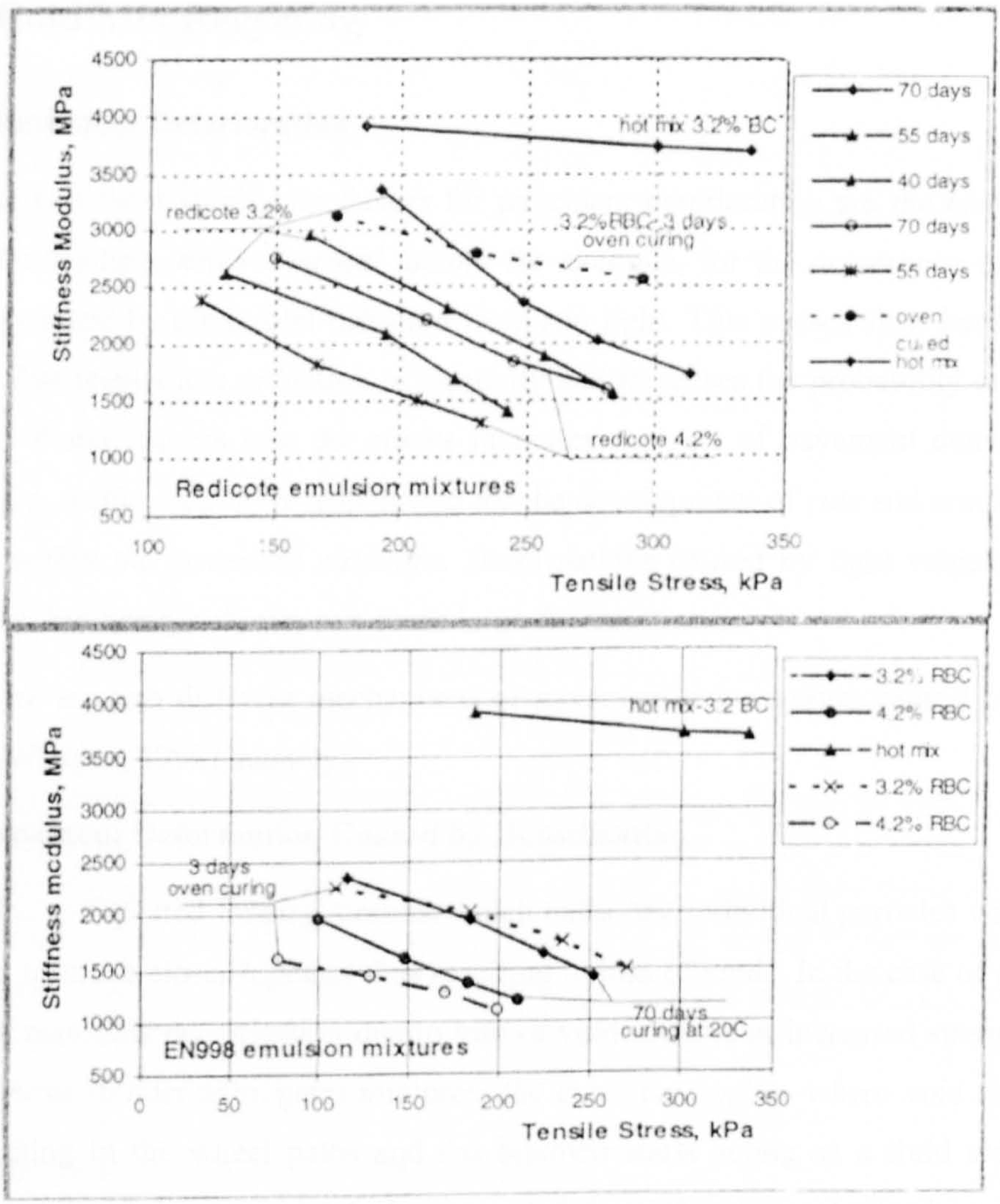


Fig. 3.5 Effect of horizontal tensile stress on stiffness modulus of specimens prepared using EN998 and Redicote emulsions, [Ibrahim & Thom, 1997].

Referring to Figure 3.5, it is clear that a longer curing time results in a stronger mixture. However, the ultimate strength values (Indirect Tensile Stiffness Modulus - ITSM) of the mixtures at full curing were not available.

Carefully analysing the above research work it became clear that different researchers had used different ways in preparing, compacting and curing the samples. This is due to the absence of uniformity in the design procedures and specifications of CBEMs. In general, the main problems with CBEMs remain with their long curing times, high porosity and low early life strength.

3.3 Cold Mix Recycling

3.3.1 Pavement Deterioration

The two most significant factors for pavement deterioration are the environment and traffic loading. *Environmental factors* for example, for the occurrence of *surface cracking* caused by ultraviolet radiation from sun light. This causes slow hardening of bitumen that results in a reduction of elasticity and increases the probability of thermal cracking. Water ingress into the cracks increases the rate of pavement deterioration. Meanwhile, *traffic loading* is responsible for the development of *ruts* and *cracking that initiates within the pavement structure.* Deformations caused by light vehicles is not significant, but heavy vehicles can cause large deformations.

There are two different mechanisms of pavement deterioration caused by traffic loading [Wirtgen, 1998], namely:

a. Permanent Deformation Caused by Densification

This is due to repeated loading stresses which cause the individual particles within the pavement to move closer together thus resulting in loss of voids. In the case of granular pavement materials densification due to loss of voids results in increased strength. But in bituminous (binder-aggregate) mixtures, the converse applies where void reduction causes rutting in the wheel paths and the bitumen starts acting as a fluid creating a medium for hydraulic pressures to be generated from the imposed wheel loads. This ends up in lateral displacement or shoving along the edges of ruts.

b. Fatigue Cracking of Bound Materials

Theoretically this initially occurs at the under-side of the bituminous bound layer as the tensile strains imposed by the wheel loads exceed the tensile strength of the bituminous layer. The cracks then propagate upwards to the surface layer of the pavement and become susceptible to water attack which can cause a softening effect. This results in strength reduction and increases the rate of deterioration under repeated loading.

When the materials are saturated with water, the water transmits loads into pressures which rapidly erode the structure of the granular material and cause stripping of bitumen from the aggregates. Under such conditions, the fine fractions may be expelled out through the cracks, otherwise known as "pumping action". This results in the development of large voids followed by potholes.

Free and trapped water within the cracks expand at temperatures below 4°C, creating hydraulic pressures even in the absence of loads. Frost heave caused by freeze-thaw cycles quickly deteriorate cracked pavements.

Under dry desert conditions a different problem may occur. When the temperature drops at night (often below freezing point) the surface contracts and this can cause surface cracks, which may then be filled by wind-blown sand. When the temperature rises during day time, the pavement surface expands, but is restricted from doing so by the trapped sand within the cracks. This results in large horizontal forces that lead to localised failure at the edges of the cracks and worsen the pavement condition.

3.3.2 Pavement Maintenance and Rehabilitation

Maintenance of pavements mostly involve preserving the pavement in a water proof state and ensuring effective drainage. Cracks on pavement surfaces should therefore be sealed off as soon as possible. This can be performed by the application of a surface dressing or a thin overlay. However, pavement deformation and fatigue cracking due to traffic loading should be treated more appropriately, as this requires a more suitable form of rehabilitation to improve the structural strength of the pavement.

3.3.3 Rehabilitation Options

In order to enable the selection of cost effective rehabilitation options, a survey of the existing pavement condition should include visual surveys and deflection measurements to evaluate the structural condition. Additionally, the maintenance strategy must cater for the requirements of the road owner, e.g. maintenance design life expectancy. Another aspect is the practicality of the various maintenance methods considering the conditions of traffic, weather and resource availability. The rehabilitation options may cover:

a. Surface Rehabilitation

This type of rehabilitation is commonly carried out on pavement surfacings as a result of surface cracking caused by bitumen aging. Some common methodologies of conducting this type of rehabilitation are:

- Paving a thin (about 40mm) hot bituminous mix overlay over the existing surface. Modified binders are often used to improve performance. However, repeated overlays increase road surface elevations, which may cause drainage and access problems.
- Mill and replace, which is also often known as "shave-n-pave". Here the cracked surface layer is removed using a "milling machine" and replaced by a hot bituminous mixture, often with a modified binder. This method can maintain the pavement level.
- Thin Recycling or *retread* the existing pavement either in place or in plant either as a hot or a cold process. The main aim is usually to rejuvenate the aged binder in the existing pavement. The utilization of modified binders and the addition of new materials to correct the aggregate grading is common practise. Retread involves scarification and reshaping. Imported aggregate materials are often added into the scarified existing pavement and sprayed with a bitumen emulsion, which is normally applied with a spray tanker and then thoroughly mixed. Thin recycling is generally carried out to a depth of about 75 mm.

b. Structural Strengthening

This type of treatment is usually the option for long term solutions. During service life, the granular base layer of the pavement may undergo densification, which in fact is a form of improvement. However densification of the bituminous layer can cause deformation problems. Structural rehabilitation should consider the salvage value of the existing pavement. There are several possible alternatives for pavement structural rehabilitation, namely:

- Total reconstruction combined with upgrading that may demand changes in road alignment. It may be possible to construct a new road section to avoid traffic congestion problems.
- Construction of additional layers on top of the existing pavement. This can be a granular and/or thick bituminous layer. This can be the easiest solution, but increases surface elevation.
- Deep recycling. This is also known as *structural recycling*. This is carried down to the depth of the pavement up to which the problems exist using a milling machine.

This technique will create a new thick recycled homogeneous layer with significant improvements in strength characteristics with the incorporation of stabilising agents such as lime, cement, or bituminous materials. Additional layers may be added on top of the recycled pavement. Note: structural recycling is deeper than retread, typically 150 to 250mm depth. It requires more substantial pulverising plant, which is a faster operation, but it can be more costly and noisier.

3.3.4 Cold Recycling Operations

a. The Process of Cold Recycling

Machines for cold milling operations had been developed over many years. Suitable sizes of milling machines are used depending on the depth of the milling operation. Currently, special milling machines are available which have the capability of recycling a thick pavement layer in a single pass and tend to be of large size. The core part of these machines is a milling/mixing drum equipped with a large number of special cutting tools. The drum rotates, milling the materials in the existing pavement (Figure 3.6), [Wirtgen, 1998].

The recycler is complete with a water sprayer, which sprays water into the mixing chamber during the milling operation. The amount of water sprayed should be metered to achieve the required amount. Water spray also reduces dust during the milling process. Stabilizing agents in liquid form, such as cement/water slurries or bitumen emulsions can also be introduced individually or in combination into the mixing chamber. When using foamed bitumen, the foamed bitumen is injected into the mixing chamber through a specially designed sprayer.

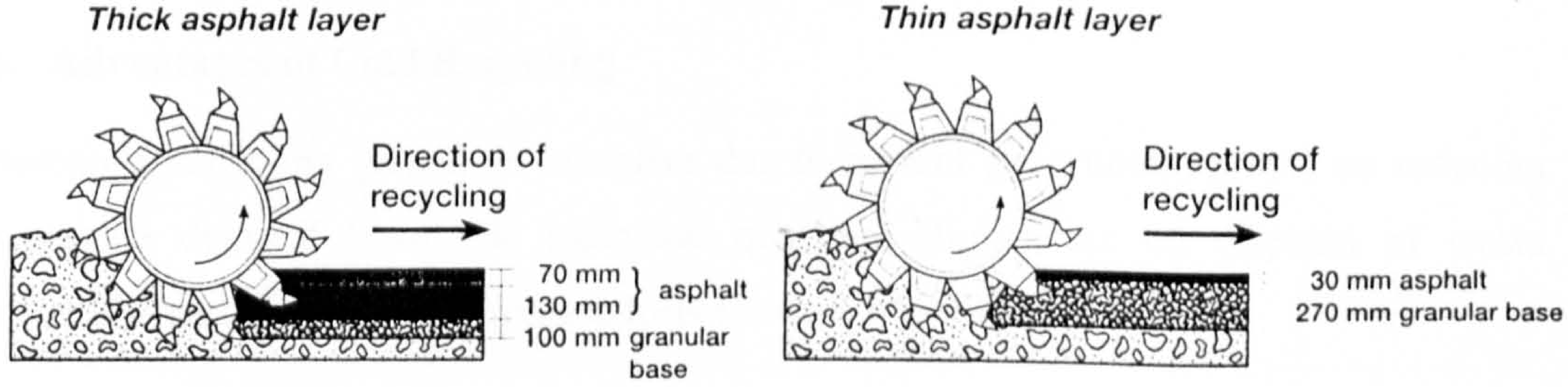


Fig. 3.6 Schematic of milling/mixing drum.

Powdered stabilizing agents such as cement or lime are usually spread onto the existing road surface ahead of the recycler, which passes over mixing it together with water. Recycling trains maybe of different configurations depending on the type of machine and the stabilizing materials used. The recycler can be either a *tyre or track mounted machine*. The milling machine acts as the locomotive which may be completed with equipment that can level the milled materials by either pushing or pulling. After recycling, the materials may require an initial pass from a roller to consolidate the material which is then profiled with a motor grader. The profiled surface is then compacted using a vibratory roller. Examples of recycling trains are given in Figures 3.7 and 3.8 with bitumen emulsion as the stabilizing agent.

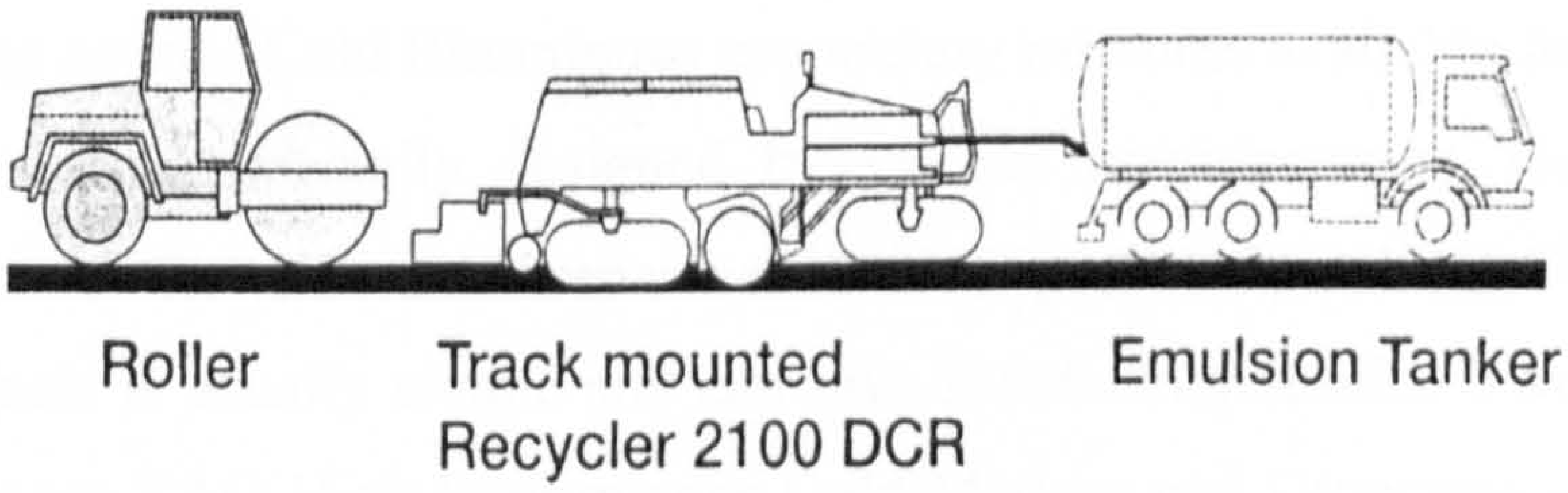


Fig. 3.7 Recycling Train with Track Mounted Recycler.

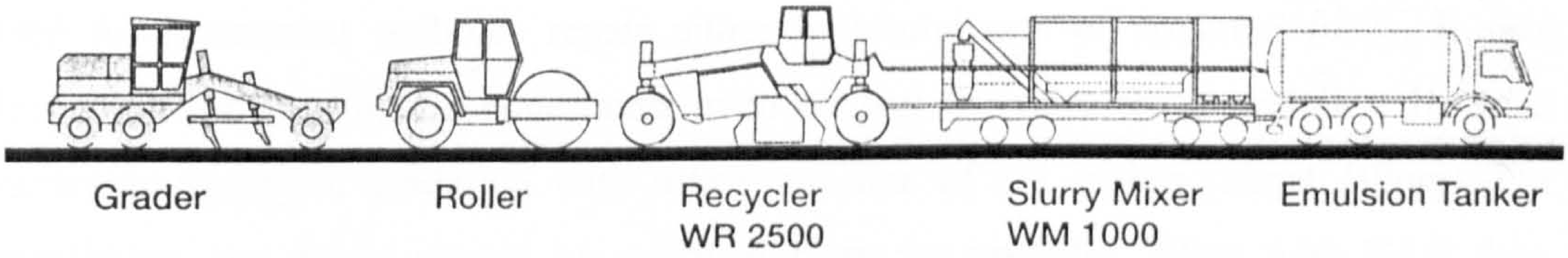


Fig. 3.8 Recycling Train with Tyre Mounted Recycler.

b. Advantages of Cold Recycling

Pavement recycling gained an incentive due to current government policy on reducing aggregate demand from UK mainland quarries. Higher tax on disposal of waste materials has also generated more efforts towards recycling

Reconstruction and maintenance of a road network often requires removal by planning or milling of a considerable amount of the existing road pavement that has been deteriorated due to deformation, cracks, wear etc. The reuse of these old bituminous materials, i.e. reclaimed asphalt pavement (RAP) is common practice as well as the incorporation of other industrial by-products.

The most obvious advantage that can be gained from cold recycling of pavements is *energy savings and reduced pollution* from gas emissions to the environment. The total energy consumption using cold mixture recycling is only 20 to 30%, of the consumption during hot recycling [Dinnen, 1998]. In other aspects cold recycling is basically comparable to hot recycling. This is in line with current developments in recycling technology.

3.4 Products of Cold Bituminous Mixtures for Road Repair

There are several Cold Bituminous proprietary mixtures available for repair work. These products are especially designed by various manufacturers. Generally these products are aimed for minor road repairs such as for potholes patching. The application of these products is usually simple and can give satisfactory results. Two examples of these products are TAG High Performance Cold Mixture and Ultracrete – Instant Road Repair (IRR).

The TAG (Throw And Go) is produced by Infratech Polymers Inc., USA. It is used for permanent potholes repair either in bituminous or concrete roads. It was developed for all climatic conditions, even on wet or water filled potholes. It has a particular aggregate gradation with maximum size of 3/8 inches (about 10mm). On installation, the debris should be removed from the potholes, filled with TAG then tamped with shovel, vibratory compactor, or wheel roller. It can be immediately opened to traffic. The physical appearance of the TAG is a granular solid [Infratech Polymer Inc., 2002, URL-12].

Ultracrete-IRR is manufactured by Instarmac Group plc, Kingsbury, UK. It utilises a formulated polymer modified bitumen. It is available in 10mm, 6mm, and 3mm maximum aggregate sizes. Detailed application guidelines are available from the manufacturer that comply with specifications approved by road authorities in the UK [Instarmac Group plc, 2002, URL-9].

3.5 Cold Mixture Practices World Wide

Cold bituminous mixtures are used in a number of countries including European countries, Scandinavia, the USA, Australia, New Zealand, Southern Africa and some developing countries [Needam, 1996]. Below is a brief review of cold mixture practices in some countries.

3.5.1 European Cold Mixture Practices

a. Grave Emulsion

Grave Emulsion is mainly used in France extensively as basecourse and for re-profiling. (see section 3.2.2).

b. Double Mixing Process

This mixture type is also used in France. This procedure is aimed at avoiding the preference of coating the fine fraction. The fine aggregates fraction is first mixed with part of the bitumen emulsion. Alternatively hot bitumen can also be used at this stage. This mixture is then stock piled for later use. The second step is mixing the coarse aggregates fraction with the second portion of emulsion, and then adding the coated fines, followed by remixing.

c. Two Stage Mixing

This mixture is used on lightly trafficked roads in remotes areas particularly in Sweden. The production of the mixtures is carried out in a '*two stage mixing*' process. Both virgin and recycled materials can be used. The principle of the procedure is that coating of the coarse and fine aggregates is carried out in two stages to prevent preferential break of the emulsion onto the fines. The base binder of the bitumen emulsion is a very soft binder to avoid low temperature cracking during extreme cold periods. The binder types are expressed in terms of viscosity at 60°C, with grades ranging from 2000 to 10,000 centistokes. For comparison, a 100 pen bitumen has a viscosity of 100,000 centistokes. The soft binder is obtained by adding heavy flux oil to a normal penetration grade bitumen and they are often doped with fatty amine based adhesion agents to improve durability. This soft bitumen is then emulsified with different levels of emulsifiers to give different setting rates.

Principally, in the two stage mixing process, the coarse aggregates are first charged into the mixer and then pre-wetted with water. A rapid setting bitumen emulsion is then added via a spray bar during the mixing process. After 20 seconds of mixing, the pre-wetted fine aggregate fraction is added, shortly followed by a medium setting bitumen emulsion. Final mixing is continued for about 20 more seconds.

d. Double Treatment Process

This process uses two types of binders, namely bitumen and cement. These binders can be added in various orders to both virgin and recycled materials. Due to the nature of the binders, the cured mixture is both hard and flexible, hence can give high resistance to cracking and also rutting at high temperatures. The binder is usually a slow setting bitumen emulsion and a hydraulic binder (cement). The ratio of the two binders affects the nature and the performance of the mixture. By careful mix design, best mixture performance can be obtained. This type of mixture is mostly used in France, Germany, and in the Benelux countries for base course layers.

e. Practices in other European countries.

So far, there does not exist any very special practices in cold mixture technology in other European countries.

3.5.2 North American Cold Mixture Practices

Cold Mixture Practices in North America are mostly developed in the USA. The binders used are cutback asphalts or bitumen emulsions. Two main design procedures that had been developed are the Hveem Method and the Marshall Method, with their recent modifications [Asphalt Institute MS-14, *Asphalt Cold Mix Manual* (1989); Asphalt Institute MS-19, *A Basic Asphalt emulsion Manual* (1997)]. Emulsified Asphalt Emulsions Mixtures (EAMs) have been described earlier in Section 3.2.4.

CHAPTER IV

INCORPORATION OF SELECTED MATERIALS INTO CBEMs

4.1 Fillers

The filler as a constituent of bituminous mixtures appears to have a very complex role. A filler can act as an *inert or an active material* depending on its fineness and surface characteristics. The amount of filler used is generally within a range of filler to bitumen ratio of 1.00 to 1.50 [Zoorob, 2000]. The filler content in bituminous mixtures depends on the gradation and type of mixture. Typically in a dense graded mixture, the filler content can be in the range between 4 to 5%, and may be from 10 to 15% in gap graded mixtures. The utilization of fillers in both hot and cold mixtures is briefly described in this chapter in order to explain its role in influencing the properties of the mixtures.

4.1.1 Inert Fillers and Reactive Fillers

Fillers may take different roles in hot and cold mixtures. They can be classified as active or non-active fillers.

Active and Non-Active Fillers in Hot Mixtures

Depending on their influence on bitumen consistency, fillers can be categorized as active or non-active fillers. Active fillers can *highly increase bitumen consistency*, meanwhile a *moderate increase* is expected from the non-active filler. Hydrated lime, kaolinite and montmorillonite are active fillers, whilst pulverized fuel ash, Portland cement, limestone dust and silt are non-active fillers.

The activity of fillers is closely related to their packing properties such as particle size, particle size distribution, and geometric irregularity. Compared with non-active fillers, active fillers normally have higher dry compacted voids, higher degree of geometric angularity, and rougher surface texture.

Reactive and Non-Reactive Fillers in Cold Mixtures

In cold bituminous mixtures, the fillers can also be distinguished as Reactive Fillers and Non Reactive Fillers [Zizi et al, 1997]. A reactive filler is a type of filler that reacts when in contact with, say, bitumen emulsion. For example, in the case of *Portland cement, which is classed as* a *reactive filler, a* hydration reaction occurs when the emulsion water (and or any added water) is mixed with the filler. Meanwhile, a *non-reactive filler* will not react with the emulsion. *Silica filler* (micro-silica) belongs to this category.

4.1.2 Size of Filler Particles

Mineral fillers have a wide range of particle sizes that influence the properties of bituminous mixtures. The part of the filler with particle sizes greater than the bitumen film thickness (from about 10 to 100 microns) improves the interlocking characteristics of the compacted aggregates. Meanwhile particles finer than the bitumen film thickness will be *suspended* in the bitumen and constitute part of the binder in the mixture.

With respect to particle size, limestone dust particles appear to have relatively larger sizes (3500 nanometers = 3500×10^{-9} m). This is indicated by the lower surface area compared to other fillers as can be seen in Table 4.1 [Puzinauskas, 1983]. In contrast, *carbon black* with a diameter between 100 to 150 nanometers (nm) can be completely *dispersed and imbedded* in the bitumen film. Due to this reason carbon blacks are referred to as '*micro fillers*' [Zoorob, 2000]. Additionally, carbon black is hydrophobic (dislikes water) and thereby is easily wetted by bitumen, hence carbon black can give good dispersion in bitumen and requires a short mixing cycle.

Table 4.1 S.G. and Surface Area of Mineral Fillers.

Filler type	S.G.	Specific surface area (cm^2/g) *
Limestone Filler	2.746	3.1
Kaolin Clay	2.670	60.0
Fuller's Earth	2.760	24.9
Asbestos	2.550	4.8

* using Glycerol Retention Method [Puzinauskas, 1983].

Fillers suspended in bitumen can impart two effects:

i. A small stiffening effect, as the fillers fill a volume of bitumen.

ii. A relatively larger stiffening effect due to the 'physico-chemical interaction' between the surface of the fillers and the bitumen, that results in a stiffer mixture.

The effect of fillers on bitumen consistency has been found to be influenced by the surface area, size of filler particles, and relative affinity between the fillers and the bitumen. This condition relates to the geometric irregularity and surface activity (physico-chemical properties) of the particles.

4.1.3 Effect of Mineral fillers on binders

Studies carried out by Puzinauskas (1983) on the effect of mineral fillers and binder (bitumen + filler) in bituminous mixtures concluded that the binder viscosity generally increases as a function of surface area and relative affinity of filler to bitumen. Fillers that increase binder viscosity tend to decrease ductility. Binders with high filler contents have less sensitivity to temperature. Temperature was observed to have a more significant role on influencing viscosity than type and amount of fillers.

Puzinauskas also concluded that a small amount of filler can convert bitumen to become a binder with properties similar to a higher penetration grade bitumen. In relation to compaction, mixtures with fillers that are spherical, smooth, and slippery require less compaction effort when compared to fillers with rough and angular surface texture which generate friction and resist compaction. Kaolin and asbestos fall in the earlier category, whilst limestone dust and fuller's earth are of the latter.

It was suggested to run immersion compression tests as a supplement to the regular procedures when determining optimum bitumen content in order to assess water sensitivity. Fuller's earth was observed to be sensitive to moisture damage although it gave a high binder viscosity. Limestone dust on the other hand performed in an opposite manner, i.e. was less sensitive to moisture damage, and gave lower binder viscosity.

Suhaibani et.al (1992) investigated effect of filler type on the rutting potential of asphalt concrete mixtures. Three filler types were studied, namely; limestone dust, hydrated lime, and Portland cement. The study results indicated that partial replacement of limestone dust by hydrated lime or Portland cement improved resistance to rutting.

Similar results were described by Anderson et. al (1992) in which fillers (calcite and quartz) improved resistance to rutting due to an increase in the viscous component. They also found that fillers significantly increased failure stress at all combinations of temperatures and loading times investigated.

In general, fillers improve properties of bituminous mixtures in different manners. *A particular filler type may perform better than others in a particular property but may under-perform in other aspects. Therefore, a specific study is required on each filler type to obtain the best results.*

4.2 Pozzolanic and Cementitious Materials

4.2.1 Definitions

A **pozzolan** is a siliceous or siliceous and aluminous material, which in itself possesses little or no cementitious property, but which will, in finely divided (ground) form and in the presence of *moisture*, chemically react with *calcium hydroxide* at ordinary temperature to form compounds possessing cementing properties. *Volcanic ashes, calcined clays, and pulverized coal ash* from thermal power plants are among the most commonly used pozzolanic materials [Malhotra & Mehta, 1996].

A pozzolan has to be mixed either with lime or with portland cement to develop cementing action. In the case of using cement, calcium hydroxide is one of the products of portland cement-water interaction.

A **cementitious** material is a finely divided (ground) and non crystalline or poorly crystalline material (similar to pozzolan), but containing sufficient calcium to form compounds which posses cementing properties after interaction with water.

Examples of cementitious materials are: *high calcium/lime fly ash and granulated blast-furnace slag.* High calcium fly ash is produced by the combustion of '*lignite coals and sub-bituminous coals*' from coal power generation plants. According to their carbon content, coals are classified as *peat, lignite, sub-bituminous, bituminous, and anthracite.* Peat has less than 60% carbon content, Lignite has about 60% carbon content, sub-bitumonous and bituminous have between 60 to 90% carbon content, and Anthracite which is the best coal quality contains more than 90% carbon.

The combustion of lignite or the sub-bituminous coal provides a high calcium or lime content that gives the cementitious properties. Meanwhile, granulated blast furnace slag is a non metallic product consisting essentially of silicates and aluminosilicates of calcium and magnesium. During iron/steel production the molten slag is quenched rapidly with a large quantity of water. It then forms a granular material which is essentially glassy or non crystalline. Granulated blast furnace slag was first developed in Germany in 1853. The *ground granulated blast furnace slag* (GGBS) had been widely used as an ingredient of blended Portland-blast furnace slag cements, with slag contents ranging from 25 to 70% by mass.

4.2.2 Development of Pozzolanic Materials

Lime-pozzolan composites have been used as cementing materials for buildings throughout the Roman empire. The pozzolan was an ash produced by the volcanic eruption of Mt. Vesuvius in 79 A.D. The name of 'pozzolan' was first used to describe the volcanic ash at 'pozzuoli', a village near Naples. Meanwhile in Greece, similar pozzolanic materials had been used as a result of volcanic eruption in Santorini Island in around 1500 B.C. [Malhotra & Mehta, 1996].

'Rheinisch trass' which is a volcanic tuff, is another pozzolanic material that was used widely in Germany during the Roman period. Besides the Greeks and the Romans, the Indian and the Egyptian civilizations were also familiar with water resisting mortars. The source of pozzolan was 'calcined clay' from crushed bricks, tiles, and pottery.

The discovery of 'hydraulic limes' (impure limes containing substantial amounts of calcined clay) during the 18th century, lead to the invention of Portland cement in 1824. Due to the faster setting and quick hardening characteristics of Portland cement, it quickly became the favorite in preference to lime-pozzolan cements. It is necessary to mention that industrial by-products such as *fly ash and silica fumes* from metallurgical operations have been used to substitute natural pozzolans and calcined clays.

Silica fume is a relatively new material which has been increasingly used in the concrete industry since the 1980s. It is a highly reactive pozzolan due to its extreme fine particle size and amorphous (various form or shape of particle) nature. Two other products which are highly reactive pozzolans are '*rice-husk ash and metakaolin*'. The combustion of rice-husk results in a very fine and amorphous silica ash. Metakaolin is made by low temperature calcinations of high-purity kaolin clay. The calcined product contains silica and alumina in amorphous form, which is finely ground. Metakaolin has been used as cement substitute in concrete production.

4.2.3 Benefits of utilizing pozzolanic materials

Three main benefits can be obtained when utilizing the materials described in the previous section on CBEMs. First, with respect to engineering properties, in general there will be an enhancement of ultimate strength due to the cementitious effect. Secondly, there is the economic benefit as the pozzolanic and cementitious materials used are mostly industrial by-products. Lastly, there is the ecological benefit. Many of those by-products contain toxic elements, which can be hazardous to human health if not disposed in a safe manner. Dumping of such materials in lakes, streams or landfills, in addition to the use of such materials in unbound road layers is not safe practice, because the toxic elements can find their way into the ground water [Malhotra & Mehta, 1996].

4.2.4 Production, Sources, and Utilization of Pozzolanic Materials

4.2.4.1 Natural Sources

All natural pozzolanic materials are derived from **volcanic rocks and ashes.** Molten lava from volcanic eruptions are composed mainly of aluminosilicates. Quick cooling lava results in the formation of vitreous phases (glass) with disordered structure or poorly crystalline minerals. The escaping gases and water vapor impart to the volcanic material a porous texture. Volcanic ashes are fine with high surface areas. The finely ground volcanic rocks and the volcanic ashes have a combination of glassy or poorly crystalline structure and high surface area. This causes the reactivity between the aluminosilicates when mixed with calcium hydroxide at normal room temperature.

Examples of this type of volcanic pozzolan ('un-altered' volcanic glass) are Santorin Earth of Greece, Bacoli Pozzolan of Italy, and Shirazu pozzolan of Japan.

The volcanic glass *can be further altered* by hydrothermal processes to become **'zeolitic trass or turf'**, which has a chemical composition of the type, $(Na_2Ca)O.Al_2O_3.4SiO_2.xH_2O$. This material if finely ground, has pozzolanic properties and when mixed with lime can form cementitious products. It is found in some countries, namely Germany, Italy, Russia, and China.

Clay minerals are not pozzolanic. They are formed by progressive alteration of volcanic glass. The crystalline structure of aluminosilicate minerals present in a clay or shale can be destroyed by heating to 700 to 800°C, resulting in pozzolanic materials, i.e. ***calcined clays or shale.***

Metakaolin, is produced from a kaolinitic clay that had been washed to remove un-reactive impurities resulting in high-purity kaolin. The high-purity kaolin is then calcined at low temperature of 700 to 800°C, then finely ground to an average particle size of 1 to 2 μm. Metakaolin is white in colour.

Diatomaceous Earth consists of amorphous hydrated silica derived from the skeleton of diatoms, which are tiny water plants with cell walls composed of silica shell. It is usually contaminated with clays and needs to be calcined to obtain the pozzolanic properties. Diatomaceous Earth can be found in some parts of the world, e.g.: Algeria, California, Denmark, and France.

4.2.4.2 Industrial by products

Selected industrial by-products which generate pozzolanic and cementitious materials include the following:

- **Rice-Husk Ash**

An abundant amount of rice-husk is available for disposal in many parts of the world. The rice-husk can be used as a substitute for fossil fuels for power generation. The combustion of rice-husk produces a lot of 'high-silica ash', which has a high surface area with very high pozzolanic property. The calorific value of risk-husk is about 50% that of coal.

- **Silica Fume**

Silica fume is a by-product of the ferro-silicon alloys and silicon metal industries. Silicon, ferro-silicon, and other silicon alloys are produced in electric arc furnaces where quartz is reduced by carbon at very high temperatures. In the process, SiO vapors are produced which oxidize and condense in the form of very tiny spheres of non-crystalline silica (0.1 μm average diameter). The product which is highly pozzolanic is recovered by passing the outgoing flue gas through a bag house filter. Silica fume contains more than 78% SiO_2 in amorphous form. Due to the extreme fineness of the particles and the low bulk density, handling and transporting silica fume is usually done in the form of slurry or as a pelletized product. The USA and Norway are major producers of silica fume. However, due to handling problems and the relatively high cost, silica fume utilization is still limited.

- **Coal Ashes and Metalurgical Slags**

These materials are described in more detail in sections 4.3.1 and 4.3.7.

4.2.5 Chemical Composition

Typical chemical composition of some pozzolanic/cementitious materials are presented in the Tables 4.2 to 4.5 [Malhotra & Mehta, 1996].

It can be seen in Tables 4.2 to 4.5 that the acidic constituent (silica and alumina) as well as the oxides of calcium and magnesium in the materials vary. In terms of natural pozzolans, the diatomaceous earth has the highest silica content (Table 4.2). Fly ash generally contains less silica and more alumina. Meanwhile blast furnace slag is generally rich in calcium and magnesium oxide. The highly active pozzolanic materials shown in Table 4.5 are characterized by a very high silica content (more than 80%).

Table 4.2 Typical Oxide Analysis of Natural Pozzolans.

Source	Mass Percent						
	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	Alkalies	Ignition Loss
Roman Tuff, Italy	44.7	18.9	10.1	10.3	4.4	6.7	4.4
Rheinish Trass, Gremany	53.0	16.0	6.0	7.0	3.0	6.0	9.0
Santorin Earth, Greece	65.1	14.5	5.5	3.0	1.1	6.5	3.5
Jalisco Pumish, Mexico	68.7	14.8	2.3	-	0.5	9.3	5.6
Diatomaceous Earth, California	86.0	2.3	1.8	-	0.6	0.4	5.2

Table 4.3 Typical of Oxide Analysis of North American Fly Ashes.

	Mass Percent							
Source	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	Alkalies	SO_3	Ignition Loss
Bituminous, U.S	55.10	21.10	5.20	6.70	1.60	2.97	0.50	0.60
Bituminous, U.S.	50.90	25.30	8.40	2.40	1.00	3.11	0.30	2.10
Bituminous, Canada	48.00	21.50	10.60	6.70	0.96	1.42	0.52	6.89
Bituminous, Canada	47.10	23.00	20.40	1.21	1.17	3.70	0.67	2.88
Sub Bituminous, U.S.	38.40	13.00	20.60	14.60	1.40	2.44	3.30	1.60
Sub Bituminous, Canada	55.7	20.4	4.61	10.7	1.53	5.65	0.38	0.44
Lignite, U.S.	26.9	9.1	3.6	19.2	5.8	8.6	16.60	-
Lignite, Canada	44.5	21.1	3.38	12.9	3.1	7.05	7.81	0.82

Table 4.4 Typical of Oxide Analysis of North American Blast Furnace Slag.

	Mass Percent						
Source	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	Alkalies	Ignition Loss
Atlantic Cement, Sparrow Pt., Maryland, U.S.	33.17	10.8	0.63	41.60	12.5	0.85	0.54
Halilton Steel Plant, Ontario, Canada	37.08	8.76	1.93	40.04	11.52	0.80	1.99
Algoma Slag, Ontario, Canada	38.35	8.76	0.61	32.34	18.64	0.93	0.95

Table 4.5 Typical of Oxide Analysis Silica Fume, Rice-Husk Ash, and Metakaolin.

	Mass Percent						
Source	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	Alkalies	Ignition Loss
Silica Fume from Silicon metal industry	94.00	0.06	0.03	0.50	1.10	0.10	2.50
Silica Fume from (75% Si) Ferrosilicon alloy ind.	90.00	1.00	2.90	0.10	0.20	2.20	2.70
Silica Fume from (50% Si) Ferrosilicon alloy ind.	83.00	2.50	2.50	0.80	3.00	2.30	3.60
Rice-Husk Ash	92.15	0.41	0.21	0.41	0.45	2.39	2.77
Metakaolin	51.52	40.18	1.23	2.00	0.12	0.53	2.01

Utilization of pozzolanic and cementitious materials, for example in concrete, had indicated that the chemical composition of the materials do not play a major role, but their mineralogical and granulometric characteristics do influence the engineering properties of concrete [Malhotra & Mehta, 1996].

4.2.6 Mineralogical Composition

Natural Pozzolans

These mainly consist of unaltered volcanic ashes with pozzolanic properties due to the presence of anywhere from 80 to 95% by mass aluminosilicate glass. In terms of mineralogical content, natural pozzolans contain significant amounts of quartz, feldspar, and clays in a glassy matrix that had undergone alteration to zeolitic minerals, such as analcite, chabazite, herschellite, phillipsite, and clinoptilolite [Malhotra & Mehta, 1996].

Fly Ashes

Mineralogical analysis of fly ashes typically contain 50 to 90% glass. The chemical composition and reactivity depends on the calcium content. Fly ashes with high calcium content are more reactive then those with low calcium content. Typical crystalline minerals found in fly ashes include; quartz, tricalcium aluminate ($3CaO.Al_2O_3$), calcium aluminosulfate ($4CaO.3Al_2O_3.SO_3$), anhydrite ($CaSO_4$), free CaO, periclase (free MgO), and alkali sulfates. These minerals react with water at ordinary room temperature.

Granulated Blast-furnace Slag

The mineralogical content of granulated blast-furnace slag can consist of 80 to 100% glass with chemical composition similar to melitite which is a reactive component. It is a solid solution phase between the gehlenite composition ($2CaO.Al_2O_3.SiO_2$) and the akermanite composition ($2CaO.MgO.2SiO_2$). The non-reactive constituent are generally the crystalline minerals gehlenite, akermanite, diopside ($CaO.Mg.O.2SiO_2$) and merwinite ($3CaO.MgO.2SiO_2$).

Silica Fume, Rice-Husk, and Metakaolin

These are highly reactive pozzolanic materials. Their lime-reactivity comes from the combination of two factors namely a *totally non crystalline structure and high surface area.* The non crystalline phase in silica fume consists of a disordered Si-O structure which is the product of solidification or condensation from a fused material. Rice-husk also contains a disordered Si-O, as it is burnt at temperatures less than or about 700°C. The Si-O here is obtained by the decomposition and sintering of opaline or hydrous silica without melting. Metakaolin consists of a non-crystalline aluminosilicate (Si-Al-O) phase that is obtained from a pure kaolin clay by a low temperature burning process similar to rice-husk ash production.

4.2.7 Particle Characteristics

Mineral admixtures such as pozzolanic and cementitious materials have been used widely in concrete production. Pozzolanic and cementitious properties are controlled by the mineralogical as well as the particle characteristics, i.e. particle size, shape, texture and surface area. Overall, particle characteristics have more influence compared to their chemical composition.

Usually there is nothing that can be done to alter the mineralogical characteristics of admixture materials. Therefore, the control of particle size distribution is the only practical method to enhance pozolanic or cementitious properties. ASTM C 618 Standard Specification for pozzolanic admixtures specifies, that the amount of particle size retained on sieve No. 325 (45μm) should be a maximum of 34%. It has been found that particles larger than 45μm show little or no reactivity under normal hydration conditions [Malhotra & Mehta, 1996].

4.3 Waste Materials

Road construction requires a considerable amount of materials, mainly aggregates that are of coarse and fine gradations. These materials are mostly obtained from natural deposits and the exploration of these resources will to some extent generate an environmental impact. To deal with this issue, efforts have been exerted on substituting natural virgin aggregates with waste materials. In the following paragraphs, a number of waste materials have been considered.

4.3.1 Coal Combustion by-products (Fly Ash, Bottom Ash)

These materials are by-products of coke (carbonated coal) combustion from power stations in which the coke that had been pulverized to a fine powder is burnt in a furnace of the power station. This process produces a very fine ash which is carried out of the furnace with the flue gases. The ash is removed from the flue gases by mechanical and electrostatic precipitation and is known as *Pulversied Fuel Ash (PFA)* or *Fly Ash* and accounts for about 75-85% of the total ash produced from the burning of coal. The remaining coarser fraction of the ash falls to the bottom of the furnace where it sinters to form *Furnace Bottom Ash (FBA)* [Sherwood, 1995].

Coke which is a *carbonized coal,* is made by baking coal in a heated oven at about 1100°C without contact with air *(free of oxygen).* This process allows the impurities in the coal to be burned off, while not allowing the *carbon content* of the coal to burn, the coal is turned into coke. The impurities burned off are the volatile matter contained in the coal, such as tars, oils and gases. When burned, coke generates an intense amount of heat but produces *very little smoke,* qualities that make it an ideal fuel for use in producing iron and steel [Texas Dot, 2002, URL-15].

Fly Ash, Hydrated Fly Ash

Fly ash is the fine gas-born particles of non-combustible materials. These materials have been widely used in the concrete industry, for soil stabilization and as fillers in bituminous mixtures. Hydrated fly ash are aggregate sized particles that are produced by crushing hard fly ash, which are a result of hydration when fly ash is mixed with water. This material can be used as sub-base or base course material.

In the USA, according to ASTM Standard C 618, fly ashes are classified into two classifications, namely Fly Ash Type C and Type F. *Fly Ash Type C* is the by product of coal combustion using *Lignite Coal or Sub-bituminous Coal.* It has a high lime content (CaO) of about 20 to 30%. It has pozzolanic properties that give cementitious effects in the presence of water. Therefore it is suitable for use in cold bituminous mixtures. **Fly Ash Type F**, is the result of coal combustion using **Anthracite Coal or Bituminous Coal** with **low lime (CaO) content**, which can be less than 10%. It does not harden in the presence of moisture, unless it is combined with cement or lime.

In the UK, Fly Ash is more popularly called Pulverized Fuel Ash (PFA). As an example, the PFA described in the following section was obtained from Eggborough Power Station. PFA is used in many engineering works such as for fill works, grouting, partial replacement of cement in concrete mixtures, road-base and sub-base stabilization, light weight aggregates, land reclamation, bituminous mixtures, etc. [National Ash, 2001]. There are three types of PFA:

1. Conditioned Ash: PFA taken directly from the silos to which water had been added to assist with delivery and subsequent compaction on site. The particle sizes are in the range of 1 and 100 microns.

2. Stockpiled Ash: previously conditioned ash which has been placed temporarily on a stockpile before delivery to site.

3. Lagoon Ash: PFA which has been taken from the ash silos, pumped to storage lagoons and allowed to settle and drain prior to re-excavation and delivery to site.

This material contains coarser particle sizes, i.e. 6 to 5000 microns (5.0 mm) [National Ash, 2001]. A typical particle size distribution of PFA is shown in Fig. 4.1, and the typical physical properties of PFA are shown in Table 4.6.

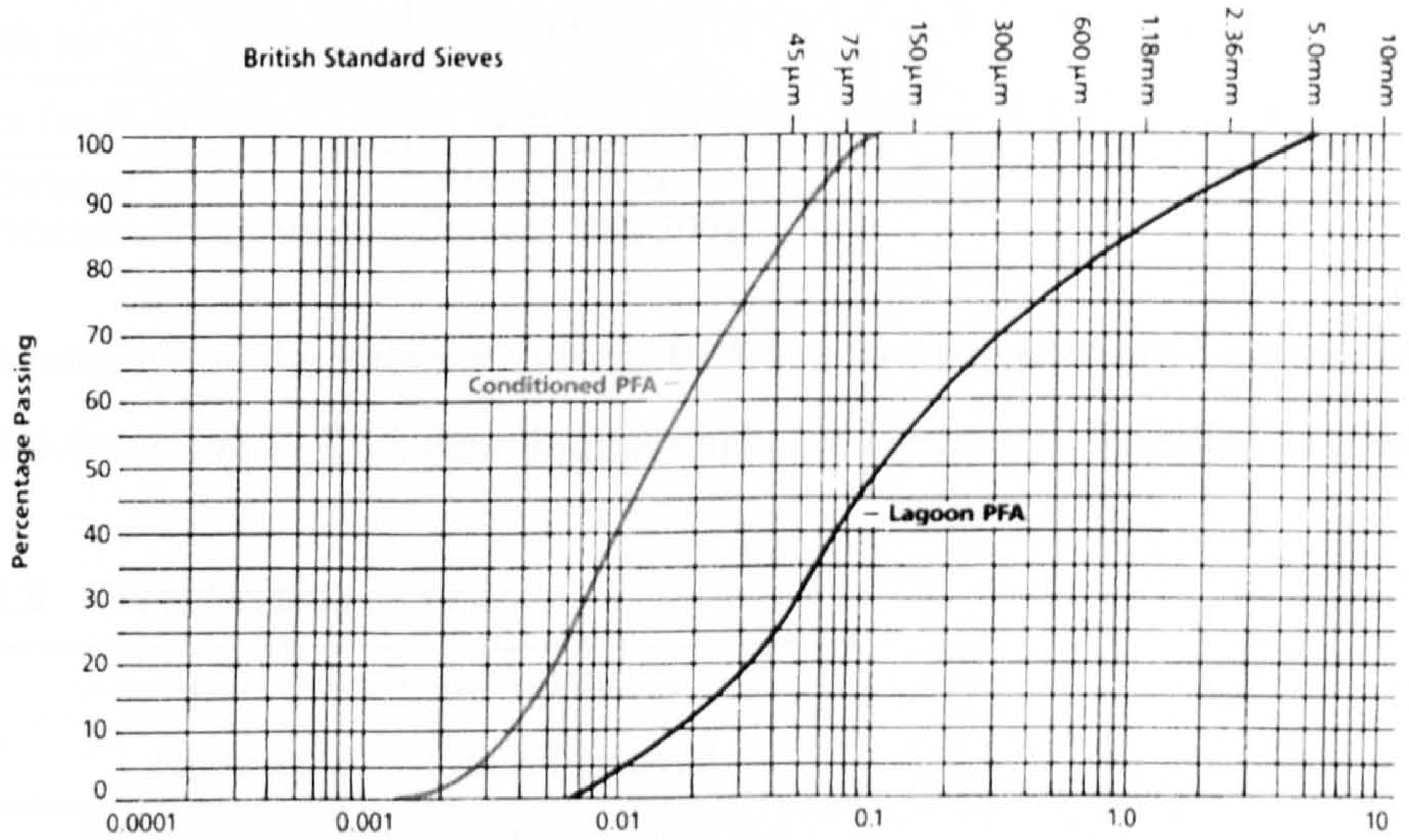


Fig. 4.1 Typical PFA Particle Size Distribution.

Table 4.6 Typical physical properties of U.K. PFA.

Description	Physical Properties
Appearance	a fine grey powder
Odour	virtually none
pH	Moderately alkaline when damp
Bulk Density	1.2 to 1.7 g/cm^3
SG	1.8 to 2.4 g/cm^3
Solubility	< 2 % in water

Source: National Ash, National Power, U.K.

The PFA filler used in this investigation was conditioned PFA and specific gravity tests using a *helium pycnometer* gave a dry S.G. value of 2.26. Chemically there are three main elements in PFA namely: silicon, aluminium, and iron. Typical chemical composition of PFA is presented in Table 4.7

Table 4.7 Typical chemical composition of PFA.

Description	% by mass
Silicon (as SiO_2)	50
Aluminium (as Al_2O_3)	28
Iron (as Fe_2O_3)	10.5
Calcium (as CaO)	2.3
Magnesium (as MgO)	1.6
Potassium (as K_2O)	3.6
Sodium (as Na_2O)	1.2
Titanium (asTiO_2)	1.0
Sulphur (as SO_3)*	0.7
Chloride (as Cl)	0.08
Sulphur (as SO_3)**	2.0 g/l

* Total Sulphur ** Water Soluble
Source: National Ash, National Power, UK, 2001.

Depending on the Sulphur content, PFA can be classified into 5 classes as shown in Table 4.8. Generally PFA from Eggborough, for instance is classified as Class 3.

Table 4.8 Classification of PFA.

Class	Concentration of Sulphates expressed as SO_3 in fill material	
	Total SO_3 (g/l)	SO_3 in 2:1 Water : material extract (g/l)
1	< 2	< 1
2	0.2 to 0.5	1.0 to 1.9
3	0.5 to 1.0	1.9 to 3.1
4	1.0 to 2.0	3.1 to 5.6
5	Over 2	Over 5.6

Source: National Ash, National Power, UK, 2001.

Bottom Ash

Bottom Ash is the heavier non-combustible sand-like particles and fused clinkers that fall to the bottom of the utility boiler. This material can be used as aggregates for road base and structural fill.

Flue Gas Desulfurization (FGD) Materials

This material is fine powder that is principally of calcium sulfate and sulfite (gypsum-like material). FDG is produced by a power plant that is equipped with equipment to reduce amount sulfur oxide emission to the air. This oxide can cause acid rain. This material can be used as constituent of sub-base and base materials [Texas DoT, 2000, URL- 15].

4.3.2 Stone Crushing by-products

Stone dust is a by-product of stone crushing processes. This material is mostly fine, therefore it can be used as fine aggregate and/or filler. An example of this material is *Red Porphyry Sand.* The Red Porphyry Sand (RPS) is a by-product of stone crushing during the production of decorative red coloured coarse aggregates that are commonly used for chipping applications. There are limited amount of this waste materials currently available in the UK (e.g about 5000 tones at Harden Quarry, North Rothbury, Tarmac Northern Ltd.). The availability fluctuates depending on the need for the coloured coarse aggregates. Currently this material is mainly used as fill material and for subgrade formation. There is also limited use in hot bituminous mixtures.

4.3.3 Synthetic Aggregates

The synthetic aggregates described here were obtained from an ongoing project on "The production of synthetic aggregates from a quarry waste using an innovative style rotary kiln" being carried out at the School of Civil Engineering, the University of Leeds [Wainwright & Cresswell, 2000]. The material used for this project was taken from the washings of crushed stone aggregates and sand from the Glensanda super quarry off the West Coast of Scotland. The quarry produces up to 10 million tonnes of aggregates and 1 million tonnes (dry weight) of the washings cake per year. At present this material is either stored in a lagoon on site or stockpiled at an off site processing plant in the south east of England. The major oxide composition of the washings is shown in Table 4.9 and the grain size distribution in Table 4.10

Table 4.9 Major oxide composition of Glensanda granite washings.

Oxide	%
Silica SiO_2	66
Aluminium Al_2O_3	17
Iron Fe_2O_3	3
Magnesium MgO	1
Calcium CaO	2
Sodium Na_2O	3
Potassium K_2O	5
Phosphorous P_2O_5	0.2
Titanium TiO_2	0.5
Manganese MnO	0.1

Table 4.10 Particle size distribution of Glensanda granite washings.

Sieve size (microns)	Cumulative % passing
300	99.5
150	96.0
75	82.8

Extrusion and 'Balling'

The first stage in the aggregate production process involved blending the quarry waste with a binder to enable the mixture to be extruded and balled (pelletised). In the Leeds study the binders used were clay, river dredgings and paper sludge in a range of proportions. In this investigation, the aggregates used in the cold emulsion mixtures were composed of 90% Glensanda washings and 10% paper sludge as binder. Once blended, the mixtures were extruded and pelletised by means of a rotary drum pelletiser that also incorporated a burner/drier. The combined action of the drier and the rolling motion of the drum produced a skin on the pellets that aided green pellet strength and created a thin denser outer skin on the fired pellet.

Aggregate firing

Firing of the synthetic aggregate consisted of two stages: the first is known as the 'burnout' stage where the carbon content of the ash/binder mixture is allowed to combust (burnout) at a temperature below that where the inert material in the pellet begins to sinter. From previous experience it was known that this stage can take between 5 and 40 minutes and occurs between 700°C and 800°C, depending on the amount and type of carbon (organic material) present and the internal structure of the pellet. The second stage is the sintering of the green pellets at temperatures between 1200°C and 1230°C, to fuse the particles of the granitic filter cake into a single mass.

During the second sintering stage, when either the temperature or the firing time are not carefully calibrated, the pellets can adhere and form big lumps of agglomerate pellets. In this investigation, it was decided to utilise these lumps by crushing them and incorporating them into the CBEMs as coarse aggregate replacement. The properties of the crushed synthetic aggregates are shown in Table 4.11.

Table 4.11 Properties of the Synthetic Aggregates.

Property *	Value
SG, Oven Dried (Bulk)	1.88
SG, S.S.D	1.94
SG, Apparent	1.99
Water Absorption	3.05

* in accordance with BS 812, Part 2, 1995.

4.3.4 Crushed Glass or Glass Cullet

Glass cullet is produced by crushing waste glass into relatively fine uniform sizes. The current prime use is in recycling as new glass containers. On average new glass bottles/jars can contain over 25% recycled glass [Ollie Recycles, 2000, URL-16].

Since the introduction of public bottle banks in the UK in the late 1970s, container glass recovery has steadily grown to about 0.5 Mt of glass collected per year (Fig. 4.2). This represents approximately 25% of UK glass production which is significantly less than the target of 50-65% set for June 2001 (Table 4.13).

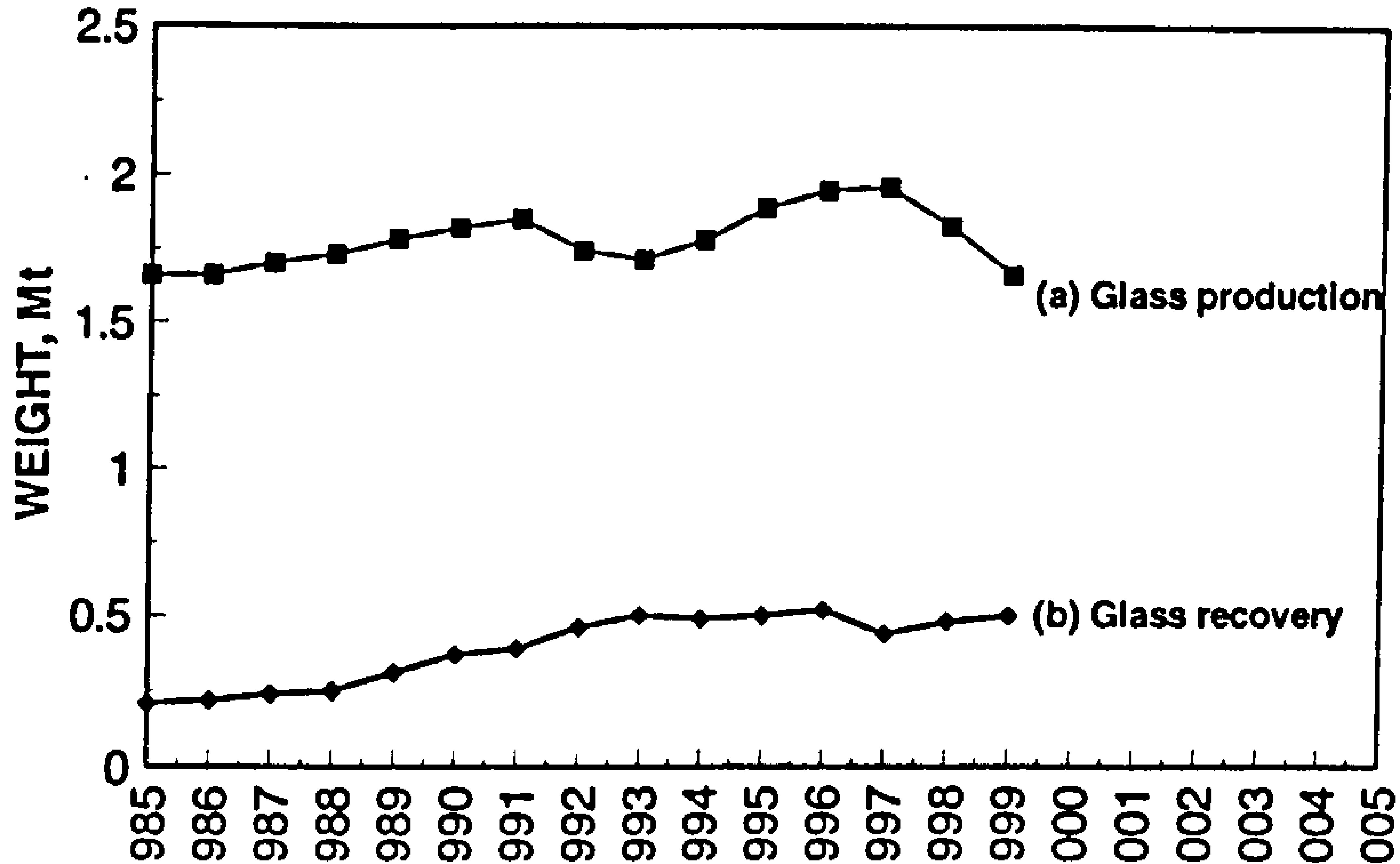


Fig. 4.2 Glass production and recovery in the UK since 1985, [Dhir, et al, 2001]

Glass recycling requires colour sorting and the demand on clear glass is normally greater than other colours. Glass waste and production in the UK is presented in Table 4.12 from which it can be seen that there is a significant amount of excess green glass wastes.

The UK government demands that glass recycling should grow from about 500,000 tones in the year 2000 to more than 1,000,000 tones in 2001 [Nick Nuttall, 2000].

Table 4.12 Amount of Glass Waste and Production in the UK.

Colour	Amount of Glass Waste (%)	Glass Production (%)
Green	56	15
Brown	13	17
Clear	31	68

Source: The Times Newspaper, Edition Monday, July 17, 2000.

The most recent information on waste packaging recovery and recycling targets was obtained from "Packaging and Packaging waste: Revised recovery and recycling targets", Select committee on the European Union, Session 2001-02, 33rd Report, House of Lords Paper 166. The report contained the European Parliament and the Council targets for recovering packaging and packaging waste from the waste stream.

The 1994 Directive required the U.K. to recover, overall, between 50 and 65% by weight of all packaging and packaging waste placed on the market and, within this figure, to recycle overall between 25 and 45% by weight by June 2001. In addition, the Directive specified four materials – paper, glass, metals and plastics – and required a minimum recovery or recycling of these materials by weight of 15% each by July 2001.

The new draft Directive was designed to raise the targets so that by June 2006 overall recovery of packaging and packaging waste by weight will be between 60 and 75%, and overall recycling between 55 and 70%. The data are shown in Table 4.13, in which it is revealed that the UK had achieved target of 33% glass recycling in year 2000. This value exceeds the target for 2001 set by the directive, i.e. 15%.

Table 4.13 Targets set by the Directives.

	Achieved by UK in 2000	Targets for 2001 set by the 1994 Directive	Targets proposed for 2006 by the new Directive
Overall Recovery*	42%	50-65%	60-75%
Overall Recycling	36%	25-45%	55-70%
Recycling:			
Paper	49%	min. 15%	55%
Glass	33%	min. 15%	60%
Metal	30%	min. 15%	50%
Plastic	12%	min. 15%	20%

* Recovery includes recycling and recovering energy from waste. For example, waste incineration may produce steam or electricity.

Table 4.14 Estimated total tonnage of packaging flowing into the UK waste stream 1998-2001 (thousands of tones).

Packaging material	1998	1999	2000	2001	Estimated in 2006
Glass	4000	2155	2155	**2200**	2190
Plastic	1700	1600	1600	1678	1958

Table 4.15 Packaging Recycled 1998-2001 (thousands of tones).

Packaging material	1998	1999	2000	2001
Glass	504.0	582.6	715.0	**735.6**
Plastic	125.5	198.5	204.4	270.0

From Tables 4.14 and 4.15 it can be seen that in 2001, the UK recycled 735,600 tonnes of glass, which is about 33% (as shown in Table 4.13) of the 2,200,000 tones of glass flowing into the UK waste stream.

The accumulation of coloured glass in the UK is particularly due to the high consumption of wine. Wines are traditionally contained in coloured (mostly green) bottles to protect them from deterioration caused by the effects of light. A similar case applies to other parts of the world. Using crushed glass in glass making saves energy and cost.

In Minessota-USA [Minesota Office of Environmental Assistance, 2000, URL-17], glass beads have been used as abrasive cleaners and as reflective paint additive to improve visibility of highway lane delineator. Limited amounts of recycled glass have been implemented in Minessota, Texas and other parts of the USA as road base and land fill. Texas Department of Transportation (TxDOT) recommends a range of applications of glass cullet as presented in Table 4.16.

Table 4.16 Percentage of Glass Cullet Permitted in Various Applications.

TxDOT Item No.	**Application**	**Percentage of Glass Cullet Permitted**
132	Embankments	Shall not exceed 20% by weight of total mixture
247	Flexible Base (Grad Type D)	Shall not exceed 20% by weight of total mixture
301	Asphalt Anti-Stripping Agents	When cullet is used as an aggregate in an asphalt stabilized bases, lime and some liquid anti-stripping agents may not perform adequately. Silane based agent may be used effectively.
345	**Asphalt-stabilized base**	**Shall not exceed 5% of the total weight of the aggregate.**
400	Excavation and backfill for structures	a) Utility bedding material may comprise up to 100%. b) Backfill that will support any portion of roadbed or embankment shall include less than 20% c) Backfill that does not support any portion of the roadbed or embankment may include up to 100%.
423	Retaining wall	Structural backfill limited to maximum of 20%. Non structural backfill up to 100%
556	Pipe underdrains	Up to 100%
Other	**Open-graded base courses**	**The use of cullet in this application shall be governed by item 345, "Asphalt-stabilized base". Not to exceed 5%.**

Source: Texas Department of Transportation [Texas Dot, 2002,URL-15]

With regards to health issues, dust created from glass crushing contains 'silica' rather than 'crystalline silica', which is the case in natural sand. It is believed that glass dust is not a 'hazardous' dust, unlike stone or sand dust that can contribute to lung disease.

Recently, Glass cullet has been incorporated into concrete as partial coarse aggregates substitute [Dhir et al, 2001]. However there remains the potential problem of 'alkaline silica reactivity' (ASR). ASR is a physicochemical process that can result in mechanical damage / deterioration of the concrete. Reactive silica in the glass aggregates an dissolve in the alkaline cement solution, producing a gel which expands, hence causing deterioration and reduction in concrete strength. Amorphous silica in glass can be highly reactive and therefore could potentially increase the ASR. However, recent findings have indicated that if finely ground glass cullet is incorporated, despite its high amorphous silica content, it can reduce the expansion caused by the ASR. Additionally, glass colour is important in terms of ASR expansion, with green glass giving much lower expansion than amber or clear glass [Pascoe et al, 2001].

Glass is rich in *silica* as shown in Table 4.17. Due to its amorphous nature and high silica content, when *finely ground,* glass possess *pozzolanic properties* [Pascoe et al, 2001].

Table 4.17 Typical properties of glass cullet, [WRAP Fact Sheet, 2002, URL- 20].

Property	Value
Bulk Density	1.3 t/m^3
SG	2.5
Chemical Composition	
SiO_2	73 %
Al_2O_3	14 %
Na_2O	14 %
CaO	10 %

Bituminous mixtures containing glass cullet have not yet become an attractive choice because of the potential for bitumen stripping in moist conditions. Anti stripping agents such as lime may not perform adequately and the use of 'silane based' anti stripping agents has been recommended.

Efforts have been made towards alternative recycling options which basically do not require colour sorting. Waste glasses had been made as *glass beads* for abrasive surface cleaners, *reflective paint* additives, and as road *construction materials.* In the U.K., **'Glasphalt'** a Hot Bituminous Mixture (containing 30% crushed glass) is commercially available and has already been tested full scale [Nicholls & Lay, 2002]. Finely ground glass, which in fact has pozzolanic properties, is attractive for incorporation into cement concrete or CBEMs.

4.3.5 Crumb Rubber from Scrap Tires

The stocks of scrap tires are increasing in line with the increasing demand for tires especially in the transportation sector all over the world. New tire production by re-using (recycling) scrap tires remains minimal in the tire manufacturing industry, due to safety measures. Damaged tires are seldom repaired and reused. The re-treading and re-grooving industry absorbs only small amounts of scrap tires, although it is slowly gaining popularity as concerns for environmental issues grow, as the cost of new tires rises, and as improvements in re-treading technology are introduced to the market [Ahmed & Klunderf, 1994]. Therefore, there are large amounts of scrap tires available ready for utilization [Texas Dot, 2002, URL-15]. An example of the amounts of annual scrap tires in selected countries is presented in Table 4.18 [Ahmed & Klunderf, 1994]

Table 4.18 Estimated amount of annual scrap tires in selected countries in 1994, [Ahmed & Klunderf, 1994].

Country	**Amount of tires (Million)**
USA	242
Canada	10
UK	0.74
Germany	0.60
France	0.40
Italy	0.37

The use of scrap tires can be divided into four main categories:

- Tire Derived Fuel (TDF)
- Civil Engineering applications

- Crumb and Chunk Rubber
- Miscellaneous

The extent of utilization of scrap tires varies from one country to another. For example, the end use of scrap tires in the UK in 1997 [Ahmed & Klunderf, 1994] was only 74% out of a total of 399,212 tones. This is compared to 100% utilization in Texas in 1998 as shown in Tables 4.19 and 4.20 respectively [Texas Dot, 2002, URL-15]. More recent data on used tyres in Europe is given in Table 4.21 [Halleett, 2001].

Table 4.19 End uses of scrap tires in the UK in 1997, [Ahmed & Klunderf, 1994].

Scrap Tire Uses	%
Burnt for energy	29
Retreaded	26
Recycled / reused	14
Engineering / landfill	5
Total	74

Table 4.20 End uses of scrap tires in Texas in 1998, [Texas Dot, 2002, URL-15].

Scrap Tire Uses	%
Tire Derived Fuel (TDF)	62
Civil Engineering	30
Crumb Rubber	4
Other	4
Total	100

Table 4.21 Tyre arising and recovery in EU countries in 1998, [Halleett, 2001].

Country	Tyre arising (tones)	Overall recovery rate (%) **
Belgium	45,000	94
Finland	30,000	80
France	370,000	39
Germany	596,000	92
Italy	330,000	60
Netherlands *	45,000	100
Spain	241,000	19
Sweden	58,000	98
UK	468,000	70

* car only ** for various recovery options: reuse, retread, recycling, energy recovery, and export.

Tire Derived Fuel (TDF)

The use of scrap tires (as whole or shredded) as TDF has been mostly utilized as fuel supplement to boilers and cement kilns. TDF can reduce the demand on fossil fuels such as coal. Tires are of low moisture content, compact, consistent composition, and give high heat levels. With proper emission control devices, tires can generate less pollution compared to coal.

Civil Engineering

Prior to utilization in civil engineering areas, tires should be shredded. Shredded tires can be used in landfill, land reclamation, and highway embankments. For highway embankments, the shredded tires should be completely covered by geotextile fabric and covered with at least 0.9m natural soil. The size of the shredded tires for this application is between 3 to 12 inches (about 7.5 to 30cm).

Crumb and Chunk Rubber

Rubber that has been reduced in size by mechanical shredding or cryogenic crushing (process of crushing at low temperatures) with sizes less than 3/8 inch (about 10mm) is called 'crumb rubber'. When the rubber size is in the order between 10mm and a maximum of 125mm, it is referred to as 'chunk rubber'. Crumb rubber can be used in the production of asphalt-rubber (bituminous) mixtures via two processes, i.e. the 'wet' and the 'dry' process [Hossain et al, 2000, URL-18].

The wet process incorporates 10 to 30% rubber by weight of the asphalt binder (bitumen). The rubber is mixed with the asphalt binder at high temperature causing a reaction to take place. The resulting product is called 'asphalt-rubber binder'. This product can be used in hot mix asphalt mixtures and in spray applications such as in stress absorbing membrane (SAM) or stress absorbing membrane inter-layers (SAMI). The use of asphalt-rubber binder is more suitable for conditions of heavy traffic, extreme temperatures, and low rainfall [Texas Dot, 2002, URL-15]. Asphalt-rubber binders have not yet been widely used due to the fact that their performance and cost effectiveness has not yet been universally proved, although they are attractive from an environmental point of view.

The substitution of aggregates with crumb or chunk rubber in asphalt concrete (bituminous mixtures) can result in weaker mixtures, and hence they are more suitable for low volume roads. However, the utilization of scrap tires in this area clearly provides some contribution towards reducing the waste tire problem. In the 'dry' process, about 2 to 3% crumb rubber is used as aggregate substitute. Using this process, chunk rubber has been used as aggregate in both hot and cold bituminous mixtures.

Miscellaneous uses of scrap tires

For miscellaneous applications, scrap tires can be used for rubber-die-cut products such as for traffic control devices: delineator posts, and weighting bases on traffic cones. They can also be used in playgrounds, silage cover, mulch, etc.

4.3.6 Plastics

Plastics can be categorized into two broad categories, i.e. 'thermoplastics and thermosets' [Texas Dot, 2002, URL-15]. For recycling purposes, thermoplastics are more suitable since they can be heated and reformed over and over again. Meanwhile thermosets cannot be re-melted. Thermoplastics are of different properties and different melting points. For recycling purposes plastics should be selected prior to processing.

Among thermoplastics, Polyethylene Terephthalate (PET), High Density Polyethylene (HDPE), and Low Density Polyethylene (LDPE) are most commonly recycled. In the transportation field, recycled plastics have been used as traffic cones, barricades, channelizers, delineators, safety fencing, etc.

The incorporation of plastics into road pavement materials has not been widely implemented and there is very limited information available. Suparma & Zoorob (2000) at Leeds University investigated the properties of hot bituminous mixtures incorporating crushed waste plastics (CWP) as aggregate replacement. The amount of CWP, which was of single size (5 to 2.36mm), was varied. The materials were pre-heated at different temperatures before compaction. In general, compared to the control mixture (without plastics), the mixtures incorporating plastics (plastiphalts) had lower densities (lighter), higher stabilities (approximately 2.5 times) and were much more elastic (very high elastic recovery).

4.3.7 Slag

Slag is a non metallic by-product or impurities from metal manufacturing. Slag consists mainly of combinations of calcium, magnesium, and aluminium silicates. Slag floats on the metal when it is molten and has to be separated and then left to cool down and solidifies. For construction application, the slag is crushed and screened into different sizes.

The two most popular types of slag are **'Blast Furnace Slag'** from *iron making* and **'Steel Slag'** from *steel refining process.*

Blast Furnace Slag

Blast Furnace Slag consists essentially of silicates and alumino-silicates of lime and other bases which is produced simultaneously with iron in a *blast furnace* (Fig. 4.3). An iron blast furnace is a piece of equipment for converting iron ore into iron, at that stage called 'pig iron'. The furnace is a tall, circular, steel shell lined with firebricks. The bottom part in which the molten products are collected is called the 'hearth'. The name of blast furnace is derived from the fact that the *pre-heated air* to support combustion must be forced into the furnace under pressure (blast), because of the resistance offered by the column of raw materials to the passage of the combustion gases [Lee, 1974].

In the iron production process, the iron ore (which is a mixture of oxides of iron, alumina and silica), fluxing stone (limestone and dolomite), and the pulverized coke are charged into the furnace. Pre-heated air is then blown (blasted) into the furnace. *The fluxing stones function to remove the unwanted impurities which then formed as slag.* The oxygen from the pre-heated air combines with the carbon of the pulverized coke producing heat and carbon monoxide. The heat required to allow a chemical reaction to occur is between 1300°C to 1600°C. At this temperature the iron ore that contains oxides of iron, silica and alumina react causing the iron oxides to become iron; the silica and alumina oxide combine with the fluxing stones to form the iron slag or Blast Furnace Slag which collect and float in a molten stratum above the molten iron in the furnace. The slag is then allowed to flow into a slag pit and to cool down. There are three common methods used to cool down slag namely: *air cooling* which produces *rock-like materials* with a crystalline structure; *water cooling* which results in a rapid solidification producing a *granulated or pelletized* glassy materials; *steaming* using jets of water arranged around the bottom and the sides of the pit/pond where the molten slag is poured, produces *foamed slag* [Sherwood, 1995].

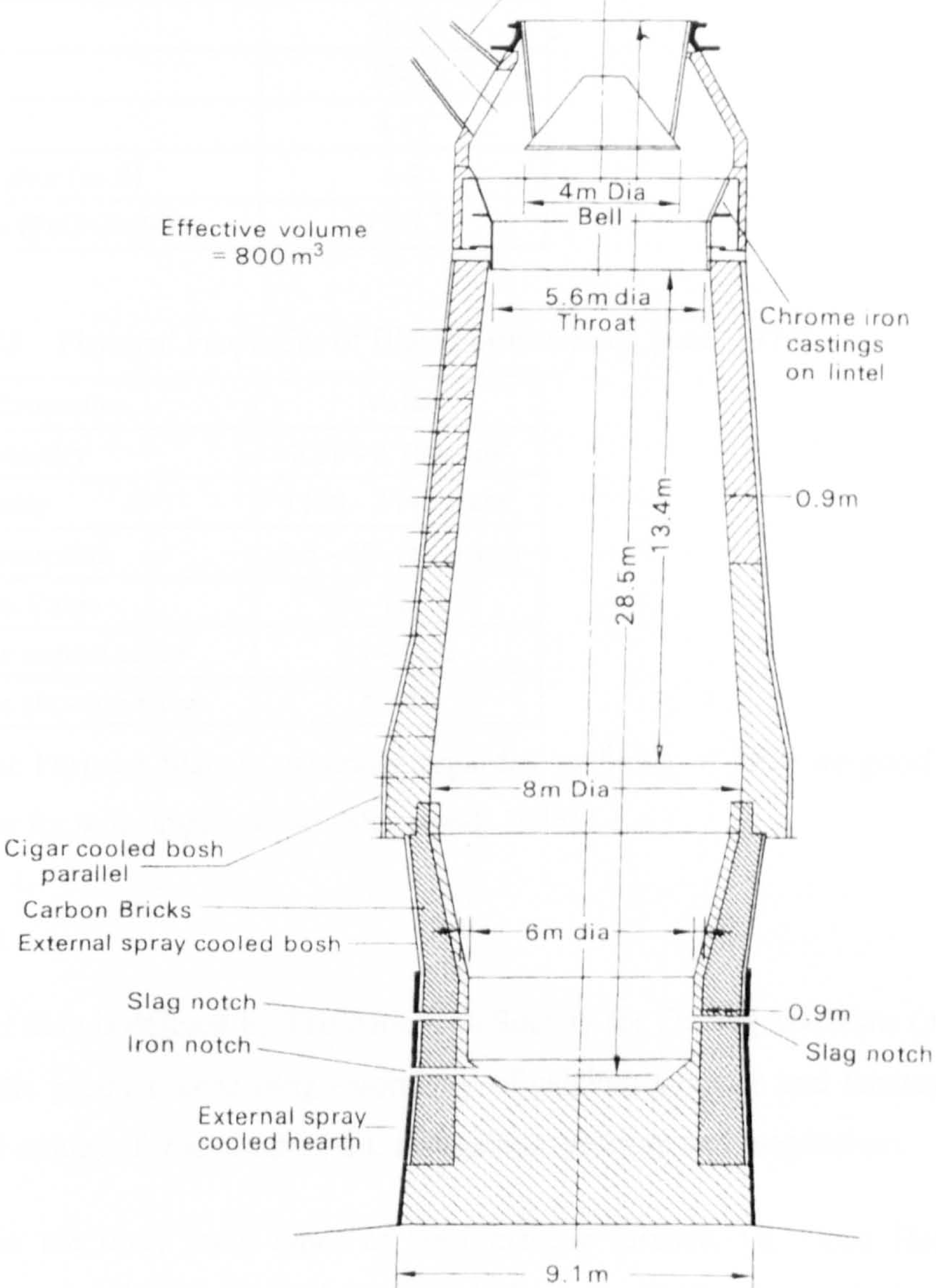


Fig. 4.3 Sectional diagram of a modern blast furnace, [Lee, 1974].

The chemical composition of blast furnace slag is shown in Table 4.22, and the physical properties of an air cooled blast furnace slag is presented in Table 4.23 [Lee, 1974].

Table 4.22 Chemical Composition of Blast Furnace Slag, [Lee, 1974].

Component	Range (% by mass)
CaO	36-43
SiO_2	28-36
Al_2O_3	12-22
MgO	4-11
Total Sulphur (as S)	1-2
Total iron (FeO+Fe2O3)	0.3-2.7

Table 4.23 Physical Properties of Blast Furnace Slag, [Lee, 1974].

Properties	Value
Particle Density	2.38 - 2.76 g/cm^3
Bulk Density	1150 - 1440 g/cm^3
Water Absorption	1.5 - 5% (by mass)
10% Fines Value	70 - 160 kN
Aggregate impact value	21 - 42%
Aggregate abrasion value	5 - 31

Blast Furnace Slag is generally regarded as being *at least as good as natural aggregates* for *most applications* [Sherwood, 1995].

Steel Slag

Steel Slag is defined by The American Society for Testing Materials (ASTM) as a non-metallic product, consisting essentially of calcium silicates and ferrites combined with fused oxides of iron, aluminium, manganese, calcium and magnesium.

There are three main types of steel making furnace, i.e. Open Hearth, Basic Oxygen Furnace (BOF), and Electric Arch Furnace (EAF). In the UK the BOF is mainly used. The EAF is used less, and no furnaces are of the Open Hearth type [Sherwood, 1995]. Presently in the UK almost 75% of steel is produced by the BOF process [Steel-UK, 2001, URL-19].

Steel may be produced almost entirely from *iron* or *steel-scrap,* but normally it is from the combination of the two. The proportion is varied according to the manufacturing process. In an integrated iron and steel production, the iron is taken molten from the blast furnace and then charged into the steel furnace. A lime and dolomite flux (*to fuse the metal)* is combined with the oxidized constituents to form steel slag. The manufacturing process involves the removal from the iron any excess quantities of carbon and silicon by oxidation. There are variations in the types and chemical compositions of steel slag and a few are suitable for use in the construction industry [Lee, 1974].

Expansive Nature of steel slag

The expansive nature of steel slag can be traced back to the steel making process in which the conversion of pig iron to steel involves the controlled adjustment of various impurities, and the addition of small quantities of constituents that give special properties to the steel. While the steel slag constituents are similar to those of blast furnace slag, the proportions are quite different, with the higher iron content being reflected in the high specific gravity of 3.2 to 3.5 for steel slag compared to 2.2 to 2.5 for air cooled blast furnace slag [Emery, 1982].

The mineral composition of steel slags are fundamentally different from *blast furnace slags* where the calcium oxide (CaO) and magnesium oxides (MgO) are *always combined* in silicate and aluminosilicate minerals. Meanwhile the CaO and MgO in steel slag not only in higher amount (Tables 4.22 and 4.24) but also *not completely combined* (free) in silicate and aluminosilicate minerals which causes volume expansion [Emery, 1982].

The steel slags have particle density of about 1.25 times that of the blast furnace (iron) slags and are mechanically stronger. However, they may contain residues of iron, free lime (CaO), and free magnesia (MgO). In the presence of moisture, the hydration of lime and magnesia makes steel slags *unstable and liable to expand.* The hydration of calcium oxide is rapid but is locked up within the slag particles, hence the rate of reaction is much reduced. Meanwhile the hydration of magnesium oxide is slow even under favourable conditions. The hydration process that is accompanied by *volume* *expansion* can cause disintegration of the slag particles over a considerable period of time [Sherwood, 1995].

At the moment, there are no specifications available in the UK relating to the use of steel slag in the road construction industry. For comparison, in Belgium the utilization of steel slag is allowed provided the following requirements are met: [Sherwood, 1995]

a. Steel slag should not have a free lime (CaO) content of more than 4.5% at the time of production.

b. Before use, the steel slag should be weathered for one year.

c. Maximum particle size should not exceed 20-25 mm.

d. Before use, the volumetric stability should be checked by a volumetric swelling test.

EAF slags have less free oxides content (iron oxides, MgO, and CaO) compared to the BOF slags, therefore the EAF slags has less risk of volume expansion [Sherwood, 1995]. Additionally the EAF process is more economical, and mainly used when *cold steel* such as *scrap steel* is included in the steel making.

In this investigation, the steel slag was obtained from Teesside East Coast Slag Products, Middlesbrough, Cleveland, UK. This material had been stockpiled outdoors and had therefore been naturally weathered at least from October 1996 until the time it was tested in October 1998. The properties of the coarse steel slag aggregates are presented in Table 4.24. Outdoor exposure includes weekly spraying with water in order to reduce the potential for future slag expansion caused by the hydration process in the presence of moisture.

Steel slag phase compositions are similar to that of a weak portland clinker and, while the dicalcium silicate present is in the potentially metastable β form, it appears to be inactive [Emery, 1982]. *The calcium and magnesium oxides are not completely combined in steel slags*, and there is general agreement in the literature that the hydration of the unslaked lime (free CaO) and magnesium oxide (MgO) in contact with moisture is largely responsible for the expansive nature of most steel slags. *The unslaked lime hydrates rapidly* and can cause *large volume changes in a few weeks.* Magnesium oxide hydrates more slowly and contributes to the *long term expansion* that may take several years to develop in the field, even when old steel slag dumps are being used.

Table 4.24 Properties of Coarse Steel Slag Aggregates

Properties *	Values
Aggregate Abrasion Value	2 %
Aggregate Crushing Value	13 %
Aggregate Impact Values	12 %
10% Fines Value (Wet)	300 kN
Specific Gravity, Oven Dried (Bulk)	3.34
Specific Gravity, S.S.D.	3.40
Specific Gravity, Apparent	3.54
Water Absorption	1.70 %
Micro Deval Coefficient	9 %
Chemical Analysis (main constituents):	
- Fe (total)	19.37 %
- CaO	**45.47 %**
- SiO_2	10.90 %
- MgO	**5.77 %**
- Free CaO	7.92 %
- CaO/SiO_2	4.21 %
- $CaCO_3$	1.25 %
- Mn	2.26 %

* in accordance with BS 812, Part 2, 1995.
Source: Tarmac Northern Ltd. Durham DH3 2ST, Product Data, 1998.

The mineralogical information on steel slag can also be used to examine some of the possible reactions that may occur when these materials are exposed to accelerated weathering conditions [Coomarasamy & Walzak, 1995]. Some of the possible reactions involve the calcium silicates hydrating to form C-S-H gels as in Portland cement or the calcium oxides forming portlandite ($Ca(OH)_2$) and goethite ($FeO(OH)$). Instability of steel slags have been attributed to the existence of bicalcium silicate in β-metastable form and the presence of free lime. The periclase may also be expected to hydrate and form brucite ($Mg(OH)_2$). With the presence of CO_2 in the moisture on the slag surface, there would be further formation of calcium carbonate ($CaCO_3$) from the calcium hydroxide species present.

Results obtained from both scanning electron microscopy coupled with energy dispersive x-ray analysis (SEM/EDX) of the deposits on the slag surfaces after weathering and from the slag asphalt interface, both in-service and after weathering, show that the formation of calcium carbonate is the main reaction. From the phases present, the most likely source of the calcium would be the various calcium silicate phases detected in the slag cross sections. Another source of calcium was the calcium rich layer found in regions at the outer surface of many of the slag samples examined [Coomarasamy & Walzak, 1995].

Coomarasamy conducted two accelerated weathering experiments: the first set involved only slag particles exposed to 100% *relative humidity* (RH) at 30°C, the second set involved both asphalt specimens and slag particles exposed to 100% RH at 50°C. In each experiment, each slag or asphalt sample was placed in an open beaker, and the beaker was placed in a humidity controlled chamber. In the humidity chamber the samples were maintained continuously wet with a thin film of water but were never completely immersed [Coomarasamy & Walzak, 1995].

It was observed that high humidity exposure at 30°C resulted in a change in the surface morphology. The surface of the slag became covered in a thick crystal like deposit. This change in surface morphology was accompanied by an increase in the calcium levels of the surface compared with initial surface composition. These calcium rich crystals (calcium carbonate-type compound) were easily removed from the surfaces and easily fractured. Such weathering products would obviously be undesirable as an interface layer for wetting with bitumen because of their fragility. These surface deposits were first observed after 10 days at 30°C. Longer exposures did not seem to change the composition of the surface deposits. This surface deposition was observed for all slags examined. Similar results were obtained at 100% RH at 50°C, the main difference being the rate at which the deposits were formed.

By exposing asphalt samples made with steel slag aggregates to periods of only 10 days at 50°C there were signs of deposits on all the exposed slag faces. These deposits were uniformly distributed across the slag surfaces and were also calcium rich. At longer exposure times, it was observed that internal voids and cracks were also becoming filled with this white crystalline deposit. It seems the calcium rich crystals were able to form wherever a moisture film was present. This concurs with the assumption that the thin film of moisture in contact with exposed slag faces would become saturated with dissolution products from the slag. These dissolution products would then be free to move in the water phase to other locations; hence deposition would also occur within these cracks and pores.

Utilization of steel slag in road construction

Due to possibility of long term expansion, steel slags are only used in situations where expansion is unlikely to occur, or if volume expansion does occur, that it is not likely to cause a serious problem. Their use is therefore ideal in the *upper (hot) bituminous layers* of the road structure where the layers are relatively impermeable and the aggregate particles are coated with bitumen, thus sealing the particles from the effects of water and preventing any hydration. If expansion did occur, it would be limited to the upper layers which would cause less serious disruption than expansion in the lower layers [Sherwood, 1995].

CHAPTER V

COMMON TESTS FOR COLD BITUMINOUS EMULSION MIXTURES (CBEMs)

5.1 General Properties of Bituminous Mixtures

Bituminous mixtures are known to have complex mechanical performance, which is more appropriately described as elasto-viscoplastic response. For a small number of repeated loadings (N) and at *very small strains* (ϵ) the behavior is highly nonlinear. At a higher number of repeated loading cycles, which induce a certain range of *small strains* ($< 10^{-4}$), the behavior is approximated as 'linear viscoelastic'. Viscoelastic behavior of bituminous mixtures is briefly described in Section 5.3.2. At an even greater number of load cycles, the materials display fatigue damage response. An illustration of this complex behavior is shown in Fig. 5.1. [di Beneto & de La Roche, 1998].

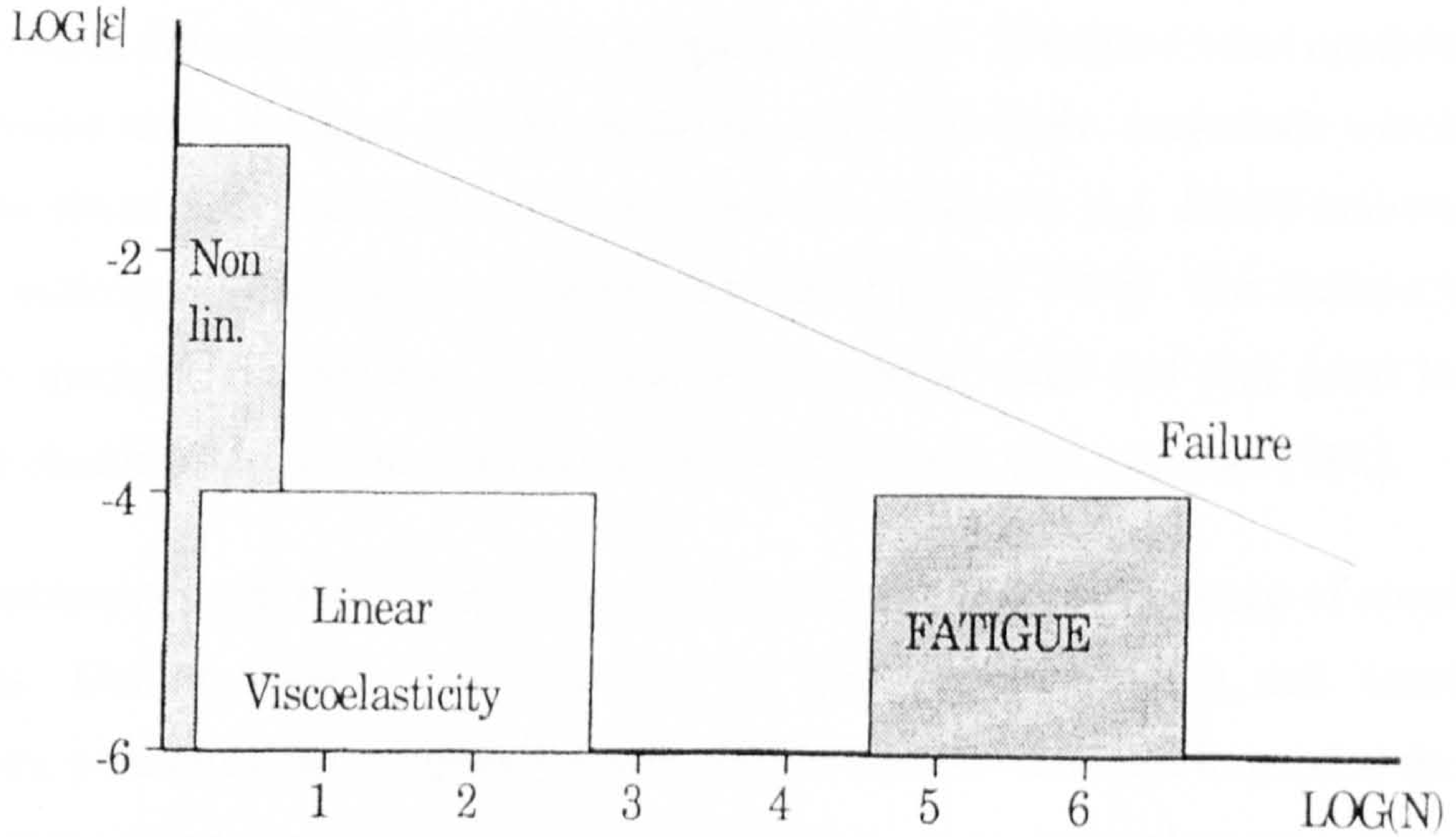


Fig. 5.1 Typical behavior of bituminous mixtures: strains (ϵ) vs. number of loading cycles (N).

The properties of bituminous mixtures must be evaluated using appropriate testing methods and there exists a wide range of different test types and configurations for assessing these properties. Some testing modes do not take into consideration the elastic properties of the bituminous mixtures, such as the Marshall stability test, and the indirect tensile strength test (ITS), whilst in other testing modes, the elastic properties of the bituminous mixtures play a significant role, in particular in the indirect tensile stiffness modulus (ITSM) Test.

Bituminous material tests that take into consideration the material's elastic properties can be grouped into two categories, i.e. homogeneous and non-homogeneous tests [di Beneto & de La Roche, 1998].

"Homogeneous tests give *direct access to the stresses and strains*, and therefore to constitutive laws (whether viscoelastic or not)". The most common homogeneous test is the creep test (static or dynamic), which is a tension/compression type test (with or without confining pressure). This test can provide parameters for determining the stiffness of the mixture (S_{mix}). Other test types that can be included in this category are: shearing tests, constant height shearing tests, and co-axial shear tests.

Non-homogeneous tests call for postulating a constitutive law (e.g. linear viscoelastic). By taking into account the geometry of the specimens visco-elastic parameters can be calculated. If the behavior of the materials is not 'linear viscoelastic' as postulated, the test results can be in a significant error. *Therefore when applying non-homogenous tests, the load applied should be within the right magnitude which gives sufficient strain which is neither too small nor too excessive, e.g. 50±20 micro-strains for the indirect tensile stiffness modulus test [BS DD213: 1993].* The indirect tensile stiffness modulus (ITSM) test, two point bending tests, three and four point bending tests are classified as non-homogeneous tests [di Beneto & de La Roche, 1998].

Bituminous mixtures can be composed of a relatively wide range of constituent materials. Due to such a wide diversity in mixture composition and constituent properties, predicting the composite properties can become difficult. In general there are three main properties that must be considered, namely; volumetric properties, mechanical properties, and durability.

Currently, there is no single mixture design procedure, testing protocol or end Result specification for CBEMs that are universally accepted. *In principle, most testing methods routinely used for characterizing hot bituminous mixtures can be adopted for testing CBEMs. However, some modifications are required bearing in mind the nature of CBEMs, such as the influence of curing time and test temperature on mixture performance.*

5.2 Tests for Volumetric Properties

For ease of description, the shape of all CBEM compacted samples are assumed to be 'cylindrical', following the Marshall standard.

5.2.1 Dry Bulk Density (D)

This volumetric property requires the mass and the volume of the samples to be measured. The weight can be readily determined using any balance. Meanwhile, the volumes of compacted samples are commonly measured using one of two simple methods: 1- directly by using water displacement or 2- indirectly by physically measuring the dimensions of cylindrical specimens and hence calculating the volume.

In the first method, the samples are weighed dry in air and when fully immersed in water. The volume (V_l) is taken as the difference between the sample's weight in air and in water.

$$V_l = (\text{weight in air} - \text{weight in water}) \qquad (5.1a)$$

This principle is based on Archimedes Law, i.e. the sample in water is subjected to uplift pressure of magnitude equal to the weight of water displaced by the sample. The density of water can be assumed to be 1 g/cm^3 at room temperature and therefore the weight of the water displaced (in grams) equals to the volume of the sample (in cm^3). Due to the nature of bituminous samples, which *may not be totally impermeable* (i.e. the presence of some surface accessible air voids), some amount of water may be able to penetrate the samples or may be absorbed. Therefore, the volume can also be calculated using the equation shown below [Asphalt Institute, 1989]:

$$V_l = (\text{weight of the sample in SSD condition} - \text{weight in water}) \qquad (5.1b)$$

SSD is saturated surface dry condition, which is obtained by towel drying the samples that have been weighed in water.

With samples that contain high void contents (such as CBEMs samples), using the first method as explained above can be less accurate due to the likelihood of water rapidly penetrating the surface accessible voids. This problem can be overcome by sealing the samples (e.g. using commercially available adhesive foil). Sealing the samples must be carried out with some care, as any air bubbles trapped within or underneath uneven or excess foil layers will appear as voids in any subsequent air void calculations. Additionally, this method may not be very easy to apply on CBEMs at early life as the samples are generally weak and can deform during handling. *However, when the samples had achieved sufficient strength, this method has been commonly used.*

In the second method, the average height (h_{avg}) of the cylindrical bituminous samples are calculated, preferably from averaging height measurements at four equally spaced locations. The volume (V_2) can thus be calculated knowing the diameter (d) is constant, as follows:

$$V_2 = \tfrac{1}{4}.\pi.d^2.\ h_{avg},\ \text{in cm}^3 \tag{5.1c}$$

It was experienced by the author that this second method gives around 1 % higher volume than the first method. Using this method, the density will be slightly smaller, hence slightly higher porosity calculated (more conservative). As CBEMs porosity targeted is of wide range, e.g. 5-10 % [MPW-RI, 1990], the overall effect is therefore relatively minor. *This method is suitable when the samples are still very weak during handling or contain high porosity.*

The bulk density (D) is finally calculated by dividing the weight of the sample in air (W) by its volume, either V_1 or V_2. Choosing the higher V value will be more conservative.

$$D = \frac{W}{V} \quad \text{g/cm}^3 \tag{5.2}$$

In the early step of CBEMs design procedure, i.e. in determining *suitable level of compaction effort (see Section 8.5.5)*, it is necessary to determine the dry density of the samples soon after compaction, i.e. whenever it is possible to extrude the compacted samples from their compaction moulds, e.g. after 1 day curing. This is to assist in determination of the (wet) bulk compacted density of each specimen and hence the porosity at the selected compaction level.

After mechanically testing a compacted specimen, the specimen should be broken down (pulverized), and about 500 grams from each test specimen was used for water content determination (w % at time of testing). The value 'w' was subsequently used to calculate the dry density (D_d):

$$D_d = \frac{(100+\mathrm{RBC})}{(100+\mathrm{RBC}+w)} \times D \tag{5.3}$$

where: D_d = dry compacted bulk density, RBC = residual bitumen content, w = water content at time of testing, D = bulk density at testing [Asphalt Institute MS-14, 1989; MPW-Indonesia, 1990].

5.2.2 Porosity (P)

Porosity is determined using the formulae shown below:

$$\text{Porosity (P) \%} = \left(1 - \frac{D_d}{SG_{mix}}\right) \times 100\% , \tag{5.4}$$

where the SGmix is determined from Eq. (5.5)

$$SG_{mix} = \frac{100}{\frac{\%CA}{SG_{CA}} + \frac{\%FA}{SG_{FA}} + \frac{\%F}{SG_F} + \frac{\%Binder}{SG_{Binder}}} \quad \text{by weight of total mixture} \tag{5.5}$$

The voids in mineral aggregates (VMA) and voids filled with bitumen (VFB), are calculated using the following formulae [Asphalt Institute, MS-2, 1995]:

$$\text{Voids in mineral aggregates (VMA)} = 100 - \left[\frac{\%W_{agg}}{SG_{agg}}\right] \times D_b , \tag{5.6}$$

where the: % W_{agg} = % of aggregates by weight of total mixture.

$$\text{Voids Filled with Bitumen (VFB)} = \frac{(\text{VMA} - \text{P})}{\text{VMA}} \times 100 \qquad (5.7)$$

where the $SG_{agg.}$ is given by Eq. (5.8)

$$SG_{agg} = \frac{100}{\frac{\%CA}{SG_{CA}} + \frac{\%FA}{SG_{FA}} + \frac{\%F}{SG_{F}}} \quad \text{by weight of total aggregates} \qquad (5.8)$$

In Eq. (5.8), SG = specific gravity, CA = Coarse Aggregate, FA = Fine Aggregate, F = Filler.

5.3 Tests for Mechanical Properties

Common Tests for mechanical properties of bituminous mixtures include stability, stiffness, resistance to deformation, fatigue and tensile strength. Mixtures should have sufficient mechanical strength in terms of stability or stiffness to provide load spreading ability, ability to resist excessive deformation, and adequate fatigue cracking resistance.

5.3.1 Tests for Marshall Stability and Flow

The test for stability and flow on CBEMs utilizes the same Marshall stability apparatus used for testing hot bituminous mixtures.

The procedure, however, had been modified so that the samples are tested at room temperature [MPW-RI, 1990], instead of at 60°C as is common for hot mixtures and has been renamed the Modified Marshall test.

The objective of this test is to determine, using the Marshall apparatus shown in Figure 5.2, the stability (maximum shear strength) in N or kN, and flow (deformation) in mm at peak strength. The loading procedure is identical to the conventional Marshall stability test, where cylindrical specimens, about 101mm diameter by approximately 62.5mm high, are loaded along a diameter at a constant rate of compression of 51mm/ minute.

The recommended thickness of a standard Marshall bituminous test specimen is 63.5mm. Compacted specimens with heights that deviate from the standard height have their stability values corrected using the correction factors shown in table 5.1.

Table 5.1 Stability Correction Factors, [Asphalt Institute, MS-2, 1998].

Volume of specimen (cm^3)	Approximate Thickness of Specimen (cm)	Correction Coefficient
200 - 213	2.54	5.56
214 - 225	2.70	5.00
226 - 237	2.86	4.55
238 - 250	3.02	4.17
251 - 264	3.18	3.85
265 - 276	3.34	3.57
277 - 289	3.49	3.33
290 - 301	3.65	3.03
302 - 316	3.81	2.78
317 - 328	3.97	2.50
329 - 340	4.13	2.27
341 - 353	4.29	2.08
354 - 367	4.45	1.92
368 - 379	4.60	1.79
380 - 392	4.76	1.67
393 - 405	4.92	1.56
406 - 420	5.08	1.47
421 - 431	5.24	1.39
432 - 443	5.40	1.32
444 - 456	5.56	1.25
457 - 470	5.72	1.19
471 - 482	5.88	1.14
483 - 495	6.03	1.09
496 - 508	6.19	1.04
509 - 522	**6.35**	**1.00**
523 - 535	6.51	0.96
536 - 546	6.67	0.93
547 - 559	6.83	0.89
560 - 573	6.99	0.86
574 - 585	7.14	0.83
586 - 598	7.30	0.81
599 - 610	7.46	0.78
611 - 625	7.62	0.76

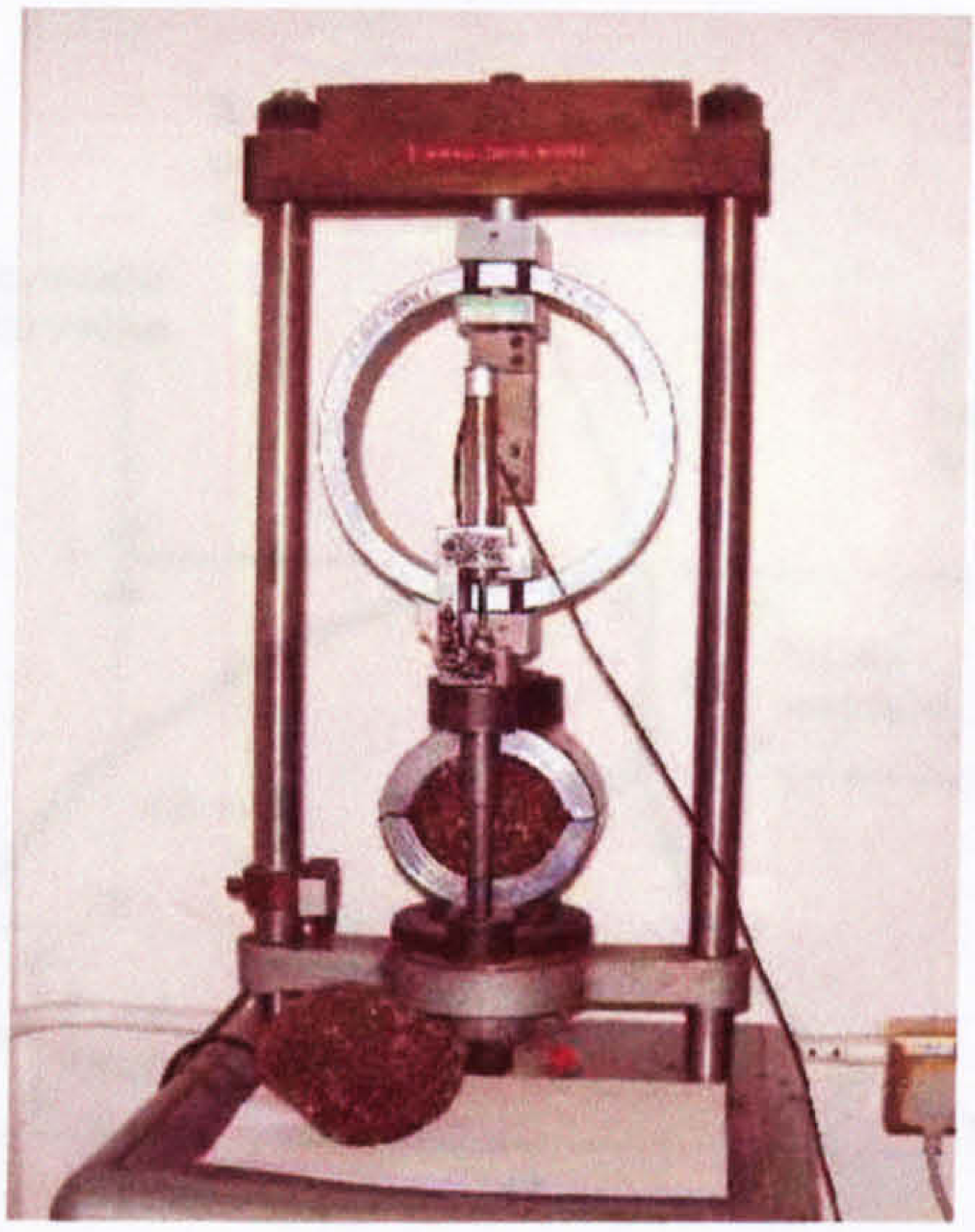


Fig. 5.2 Marshall Stability Apparatus.

5.3.2 Test for Stiffness

This property indicates the load spreading ability of a compacted mixture and is commonly expressed as the elastic modulus, i.e. the ratio of maximum stress to maximum strain of a test specimen normally under a uniaxial loading mode.

Stiffness can be subdivided into an *elastic stiffness* and a *viscous stiffness* component. Elastic stiffness is used in analytical design for calculating critical strains. This property is obtained under low temperatures and short loading times such as in the indirect tensile stiffness modulus (ITSM) test. Meanwhile viscous stiffness is tested at higher temperatures and at longer loading times, and is used for assessing resistance to permanent deformations, e.g. using the creep test [Whiteoak, 1991].

Bituminous mixtures are visco-elastic, i.e. their mechanical properties depend on both the loading time and test temperature. In visco-elastic materials, the strain resulting from a stress application is out of phase (i.e. lagging behind) the stress by a parameter referred to as the phase angle. Due to this nature, the elastic modulus of a visco-elastic material is referred to as the 'complex modulus' [Croney & Croney, 1998]. At the end of a loading pulse, there remains an '*irrecoverable strain*' (Newtonian Flow). A typical visco-elastic response to a load pulse in bituminous materials is illustrated in Fig. 5.3.

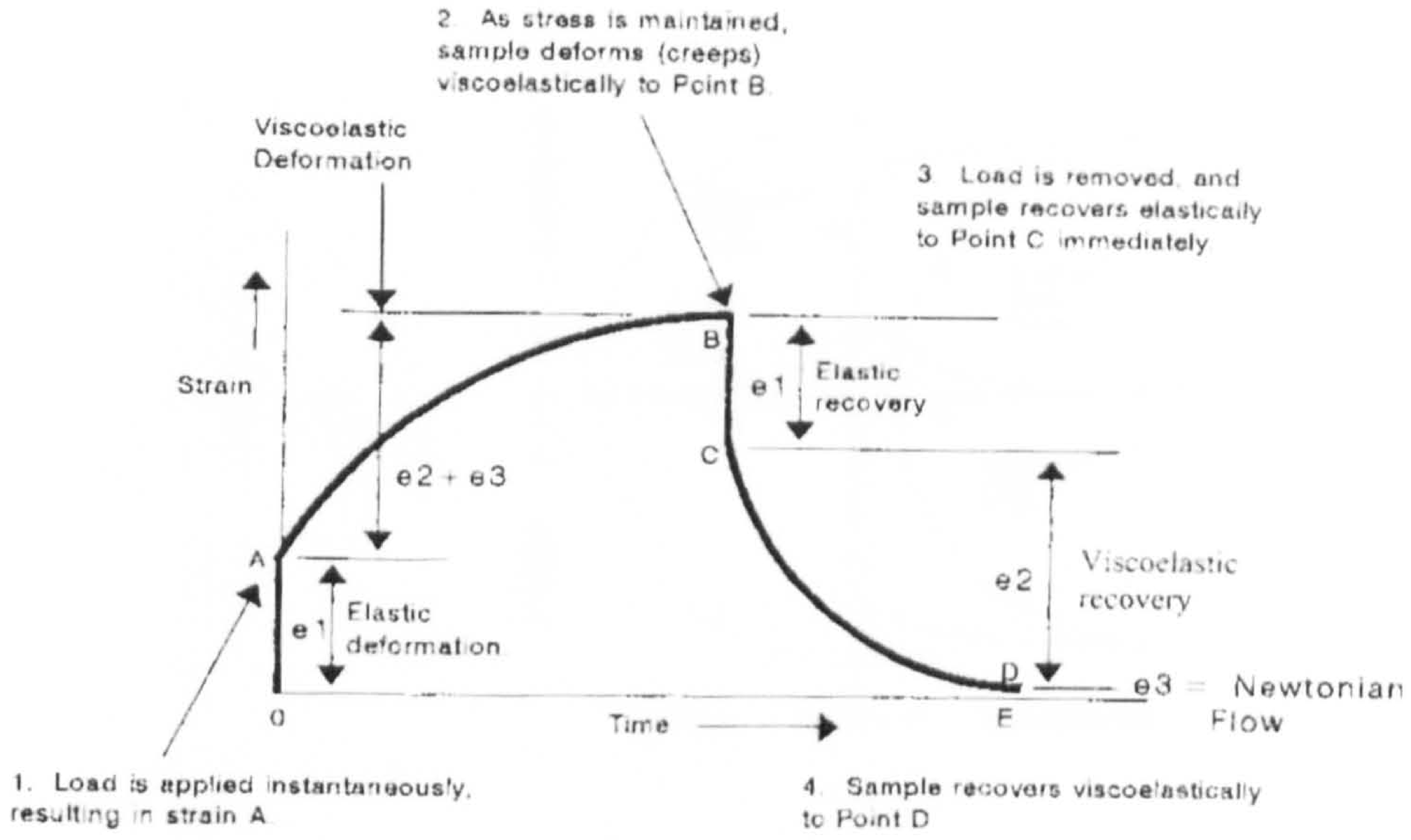


Fig. 5.3 Visco-elastic response to loading.

5.3.2.1 The Indirect Tensile Stiffness Modulus (ITSM) Test

Bituminous mixtures are more sensitive to tensile stresses than to compressive stresses and therefore the tensile stiffness of bituminous mixtures are routinely measured. Testing bituminous samples in an indirect tensile mode is more practicable than direct tension tests. Direct tension tests require the test assembly to allow perfect alignment of the test specimens with the loading plates so as not to cause eccentric stresses during loading and failures are commonly experienced at the points of specimen attachment (glued or clamped ends) to loading plates instead of failure around the center.

The indirect tensile stiffness modulus test is used for measuring small recoverable strains on bituminous specimens and consists of a set of pulse loadings applied on two diametrically opposed generating lines of a cylindrical specimen. The stress distribution in a cylindrical specimen is as shown in Figure 5.4 with the central part of the sample under a uniform tensile stress [Read & Collop, 1997; di Benedito & de La Roche, 1998].

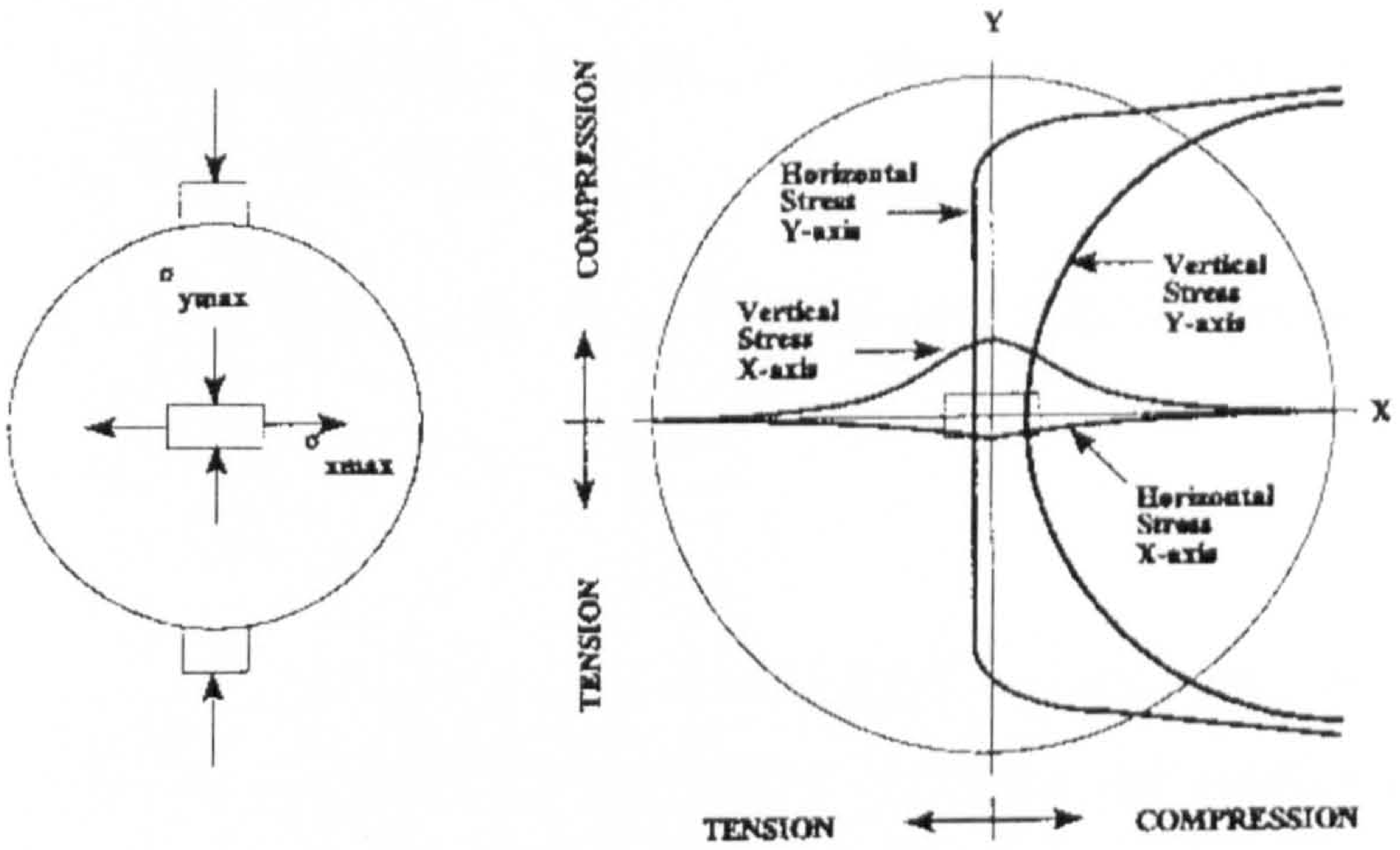


Fig. 5.4 Stress distribution in a cylindrical specimen with applied line loading.

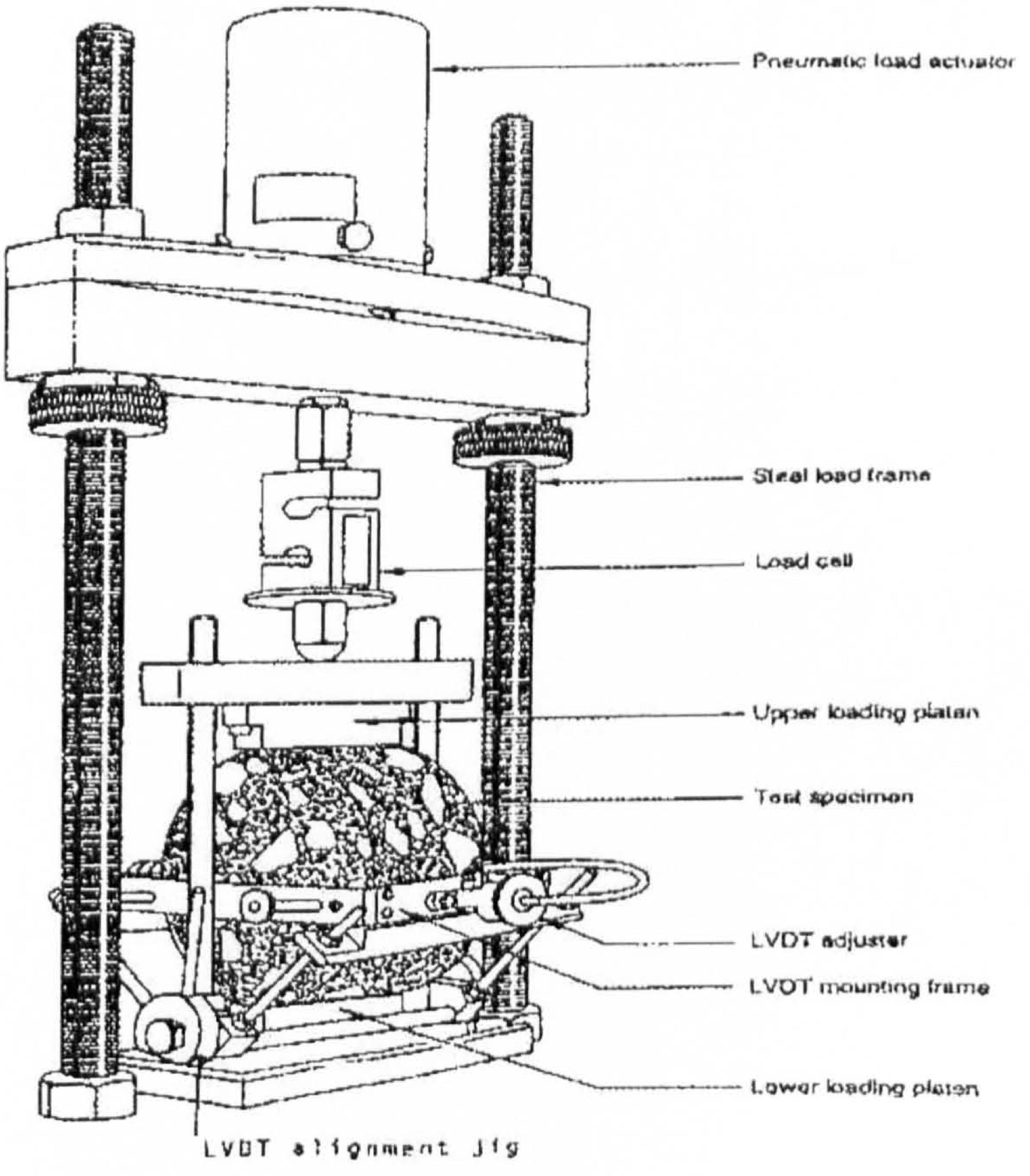


Fig. 5.5 The ITSM test configuration.

The current method of specifying the indirect tensile stiffness modulus is based on British Standard Draft for Development BS DD213:1993, *Method for determination of the indirect tensile stiffness modulus of bituminous mixtures.* A typical test configuration is illustrated in Figure 5.5.

The indirect tensile stiffness modulus (S_m) is calculated using equation 5.9 below:

$$S_m = \frac{L(\upsilon + 0.27)}{D \times t} \qquad (5.9)$$

where: S_m = indirect tensile stiffness modulus (MPa), L = the peak value of the applied vertical load (N), D = the mean amplitude of the horizontal deformation obtained from 2 or more applications of the load pulse (mm), T = the mean thickness of the test specimen (mm), and υ = Poisson's ratio, a value of 0.35 shall be used (for standard materials) at the standard testing temperature of 20°C.

Table 5.2, from BS DD213: 1993, shows Poisson ratios that can be used at other test temperatures. Prior to testing, all samples shall be conditioned for at least two hours at the correct test temperature.

Table 5.2 Poisson ratios for the calculation of stiffness modulus.

Test Temp. (°C)	Poisson's Ratio
0	0.25
10	0.25
20	0.35
30	0.45

The standard rise time used in the BS DD213 is 124 ± 4 ms. The rise time is defined as the time taken for the load to increase from zero to maximum value. The pulse period is the period from the start of the application of load until the start of the next load application which in BS DD213 is taken to be (3000±50) ms. *The peak load should be adjusted, so it gives a transient horizontal deformation value of 7μm ± 2μm for 150mm nominal diameter specimens, and 5μm ± 2μm for 100mm nominal diameter specimens. These horizontal deformations will give rise to horizontal (tensile) strains or total recovery strains of 50 ± 20 micro strains.*

A range of machines is available for testing the stiffness of bituminous materials in terms of indirect tensile stiffness modulus (ITSM). Two of the most popular are the

UK Nottingham asphalt tester (NAT) and the Australian universal material testing apparatus (UMATTA). A direct comparison of the test protocol for these two machines is shown below in Table 5.3.

Table 5.3 Characteristics of the Indirect Tensile Stiffness Modulus (ITSM) Test.

Feature	NAT method (UK)	MATTA method (Australia)
Rise time (milliseconds)	125 ± 10	50 ± 5
Deflection requirements	50 ± 20 micro strains	30-70 micro strains
Load (N)	As required	Above 1200
Pulse duration	3 s	2 s and 3 s
No. of conditioning pulses	5	5
Number of test pulses	5	5
Test temperature (°C)	20 ± 0.5	25 ± 1
Poisson's ratio	0.35	0.4
Rotation of sample	90 + 10°	Not required
Time to reach equilibrium	> 4 hours	Not specified
Specimen height (mm)	30 to 80	35-70
Test result	The mean of two measurements on the one specimen 90° apart and which do not differ by more than 10% from the mean	The mean of three replicate specimens which do not differ by more than 15% from the mean

Source: Leech & Sexton [1996].

5.3.2.2 Using Stiffness Data for Fatigue Life Prediction

The stress state around the centre of an indirect tension specimen as shown in Figure 5.4 is very similar to the stress state that occurs in a bituminous road layer. Figure 5.6 presents typical stress states in a bituminous layer underneath an applied wheel load.

The critical location for load-induced cracking or *fatigue cracking* is generally considered to be at the bottom of the bituminous layer and immediately underneath the load, where the stress state is composed of longitudinal and transverse tension combined with vertical compression [Rogue & Buttlar, 1992].

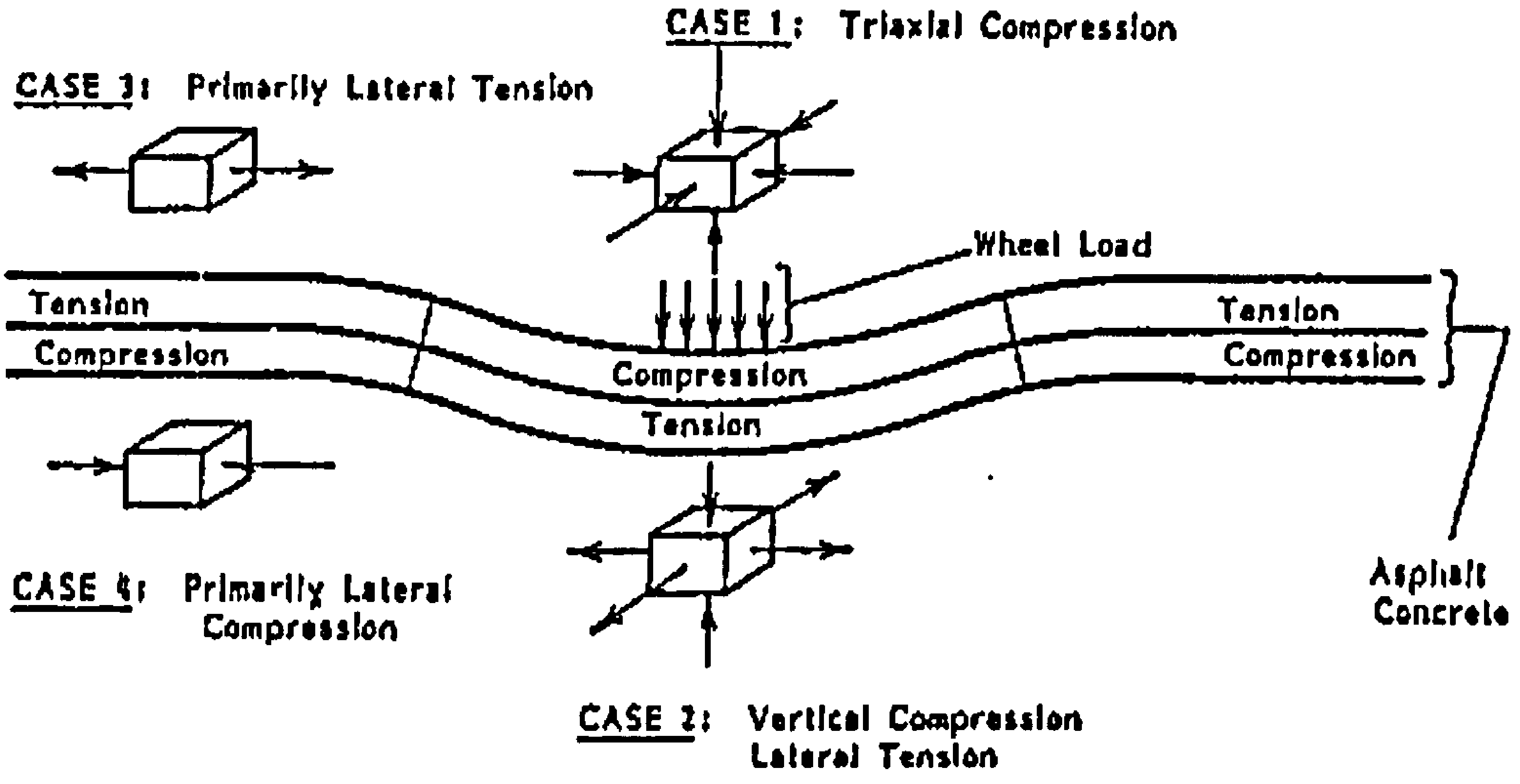


Fig. 5.6 Typical stress states in a bituminous layer with wheel load applied, [Rogue & Buttlar, 1992]

The Highways Agency in the UK has recently introduced the concept of 'long life pavements', which is based on good observational evidence. The mode of deterioration in pavements which consist of thick asphalt layers are surface dominated, either by cracking or rutting. Maintenance of pavements by regular renewal of the wearing courses has been commonly carried out, provided the base layers were still in good condition. Long term experience with pavement monitoring of major highways in the UK, has very rarely found evidence for 'bottom-up' cracking, whereas a lot of 'top-down' cracking cases have been observed. It was also noted that bituminous materials have continued to gain in stiffness as the materials aged. Increases in stiffness in bituminous materials can cause a tendency for the materials to become more brittle. The effects of hardening and embrittlement, and the prediction of top-down cracking have not yet been incorporated within traditional analytical pavement design methods which normally calculate the tensile strain at the base of the asphaltic layer thus assuming bottom-up cracking. This is not an accurate representation of the deterioration mechanism exhibited by thick pavements [Thom et al, 2002].

Asphalt mixtures are susceptible to tensile stresses which can cause micro-cracks. Stresses cause strains to develop within a bitumen film at a particle contact. Micro cracking at particle contact is illustrated in Fig. 5.7.

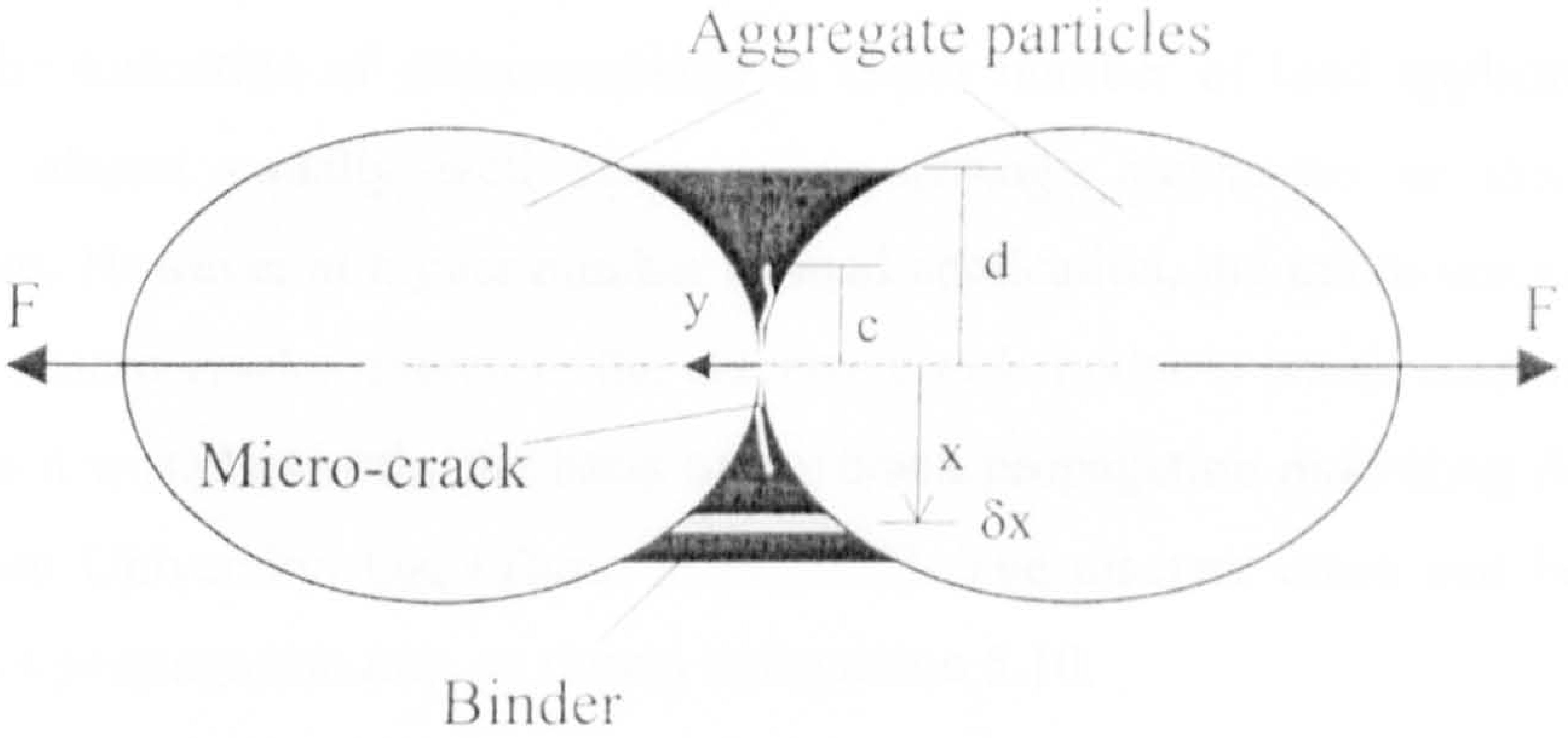
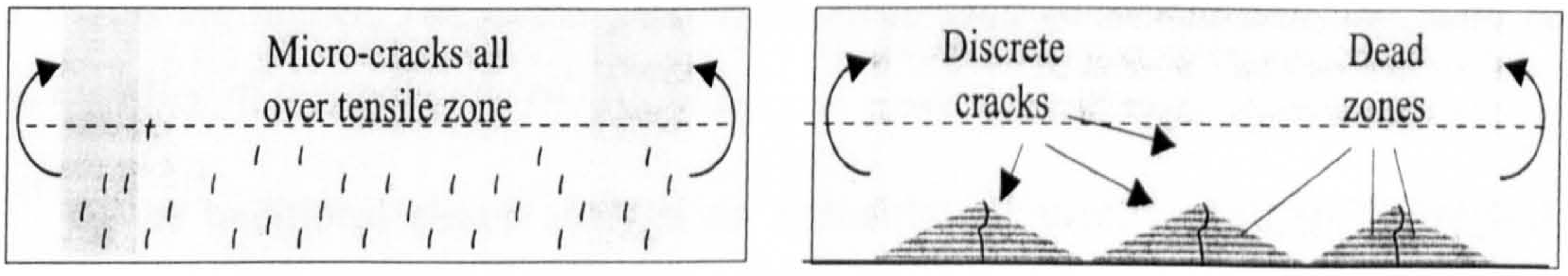


Fig. 5.7 Micro-crack development at a particle contact under tension.

As stresses continue, the micro-cracks propagate to form larger cracks. Several numbers of micro-cracks can be present with differing orientations. Some micro-cracks may join to form macro-cracks, and in the end cause failure of the asphaltic material. [Thom et al, 2002].

There are two main, significantly different, approaches for describing the formation of cracks, namely 'damage mechanics' and 'discrete crack' models [Thom et. al., 2002]. This is illustrated in an example case of an asphaltic mixture in the form of beam in bending as illustrated in Fig. 5.8.



(a) Damage Mechanics Model (b) Discrete Crack Model

Fig. 5.8 Diagramatic Comparison of Model.

In the concept of 'damage mechanics', the build up of cracks are assumed to occur everywhere within the body of the beam where a tensile stress is experienced. As the stress is higher at the edges (in beam bending) where damage is greatest, macro cracks may start to form. Whereas in the 'discrete crack' concept, a set of individual cracks are assumed to start at the tensile face of the beam, while the whole of the remainder of the material remains intact. Crack spacing has to be sufficient such that no crack is significantly affected by its neighbour. As the cracks develop some cracks will dominate and others will stop growing.

The formation of micro-cracking at lower number of load application can be modelled almost equally well using either *damage mechanics* or *discrete crack* mechanism. However at higher number of load application, the micro cracks propagate and form macro cracks, therefore the *discrete crack* model is considered more logical and hence it was chosen for the basis of the crack propagation modelling developed at Nottingham University, UK [Thom et al 2002]. The discrete crack can be modelled using *crack propagation law*, as shown in Equation 5.10.

$$dc/dN = A\,\epsilon^{n} \qquad (5.10)$$

where c is the crack length, N is the number of load applications, ϵ is the strain in the region of a crack tip, A and n are constants. The constants A and n can be deduced from the intercept and slope of a traditional 'strain against life to failure' from indirect tensile fatigue test results.

Thom et al (2002) developed a crack propagation model where propagation of cracks are assumed to take place both from the surface and from the bottom of the bituminous layer. *Calculation of strain near the surface allows the prediction of top-down cracking* using Equation 5.10. The cracks will naturally dominate at a place where the strains are higher. *The model was restricted to load generated cracking, with no consideration of thermal induced cracking.*

In the traditional elastic analysis for asphaltic mixtures, asphalt stiffening with time and embrittlement are not taken into consideration. Long term site evaluations in the UK have shown that asphalt pavements do become harder and more brittle with time. Therefore, for modelling purposes, asphalt stiffening with time and embrittlement must be taken into account. Based on a conservative interpretation of published data, a logarithmic law was used to predict that the stiffness of bituminous materials doubles every 30 years [Leech & Nunn, 1997]. There is less data available on embrittlement, which is also likely to be affected by temperature. *The complex nature of the matter was* *simplified by assuming 'a logarithmic deterioration' fatigue behaviour, such that each doubling of stiffness every 30 years, is accompanied by a halving or a quartering of the laboratory fatigue life.*

Thom et. al, 2002, carried out a design exercise using 'model for prediction of crack growth including stiffening and embrittlement', on a base material with a 35 pen. grade binder, a design life of 200×10^6 standard (80kN) axles over a 40 year life . The thickness of the bituminous layer to satisfy the design life was calculated at 325mm. With input of typical stiffness and fatigue data, *the crack growth* was predicted for each of the following cases: 1- the case *without adjustment of stiffness and fatigue* (*unadjusted*), 2- with *adjustment to stiffness* only due to asphalt ageing, 3- and with *adjustment to both stiffness* and *fatigue* (either by a halving or quartering of lab fatigue life) due to embrittlement. The predicted crack growth is illustrated in Fig. 5.9. [Thom et al, 2002].

Referring to the example shown in Fig. 5.9, it was predicted that *initial damage* will take the form of *top-down cracking* which then slows down, allowing bottom-up cracks to form and, eventually to cause failure. At early stages of the prediction, no actual top-down cracks should be visible before about 30×10^6 standard axles in this example, whilst no bottom-up crack would be observable until well over 100×10^6 standard axles.

The model can also describe the significance of life reduction due to 'embrittlement' that can be caused by for example ageing or ingress of water. As can be seen from Fig. 5.9, with no allowance for embrittlement, the life of the asphalt layer becomes almost indefinite due to stiffness gain. Whilst the inclusion of an embrittlement factor to the stiffness gain can bring the fatigue line back closer to the original design life line, or maybe even result in a slightly reduced life [Thom et al, 2002].

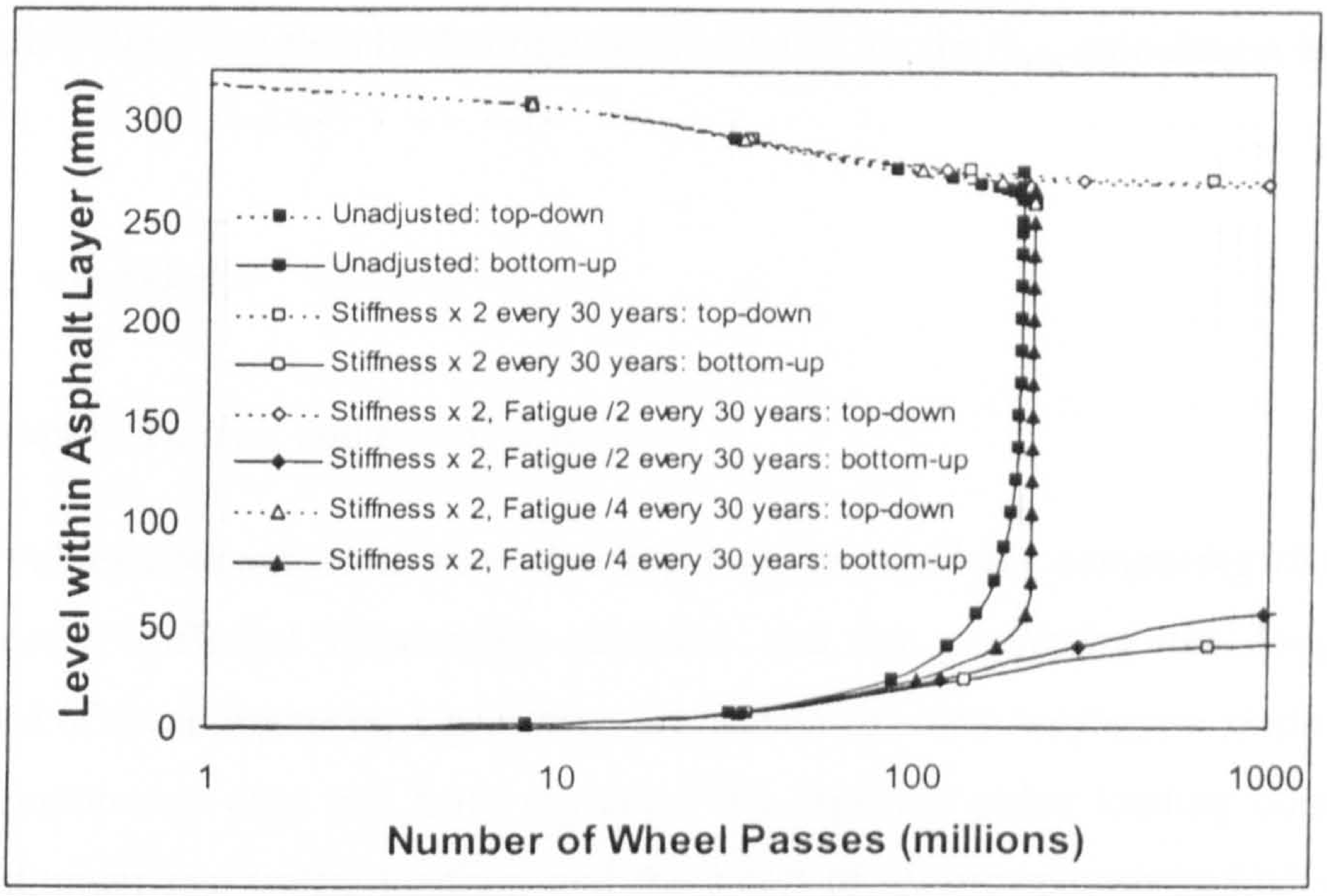


Fig. 5.9 Prediction of Crack Growth – including asphalt stiffening and embrittlement.

5.3.3 Test for Deformation Resistance

Resistance to deformation of bituminous mixture relates to mixture stiffness, which in turn is largely controlled by temperature and time of loading. Besides these factors, there are a range of other variables involved including grade of binder, grading of aggregates, aggregate shape and texture, degree of interlock, and compaction level [Whitoeak, 1991].

The unconfined uniaxial creep test is commonly used to assess resistance to deformation of bituminous mixtures, a schematic illustration of the apparatus is shown in Fig. 5.10. The standard test temperature is 40°C, which is occasionally raised to 60°C for very hot climates. Samples are conditioned for at least two hours at the test temperature and the test can be conducted in the static or dynamic mode of loading.

5.3.3.1 Static Creep Test

In the standard test protocol, a cylindrical sample is subjected to a static axial stress (σ) of magnitude 100 kPa for a loading duration of 1 hour. During the test, axial deformation is measured as a function of loading time. At the end of the test, sufficient recovery time is allowed (at least 15 minutes) during which the sample recovery is monitored. The axial strain and mixture stiffness (S_{mix}) at any loading time (e.g. S_{mix} at 1 hour loading time) can thus be determined. The basis for the S_{mix} calculation is shown below:

$$S_{mix} = \frac{\sigma}{\varepsilon} \text{ (kPa)}, \tag{5.11}$$

where: σ = applied stress; and ϵ = axial strain.

S_{mix} values obtained from static loading tests are used for *comparing the creep performance of* different bituminous mixtures and for *predicting the long term performance* of these mixtures, e.g. *rutting evaluation of slow moving or static loads*. The static creep test does not truly represent the repeated pulse loading conditions experienced under real traffic loadings and the effect of *elastic recovery* which occurs during the rest periods in between load pulses is not adequately simulated in the static creep test. Traffic conditions are more appropriately simulated when using the dynamic creep test.

5.3.3.2 Dynamic Creep Test

The dynamic creep test uses the same test configuration as the Static Creep Test, except that the load is applied in pulse form using a pulse width of 1 second and a pulse period of 2 seconds.

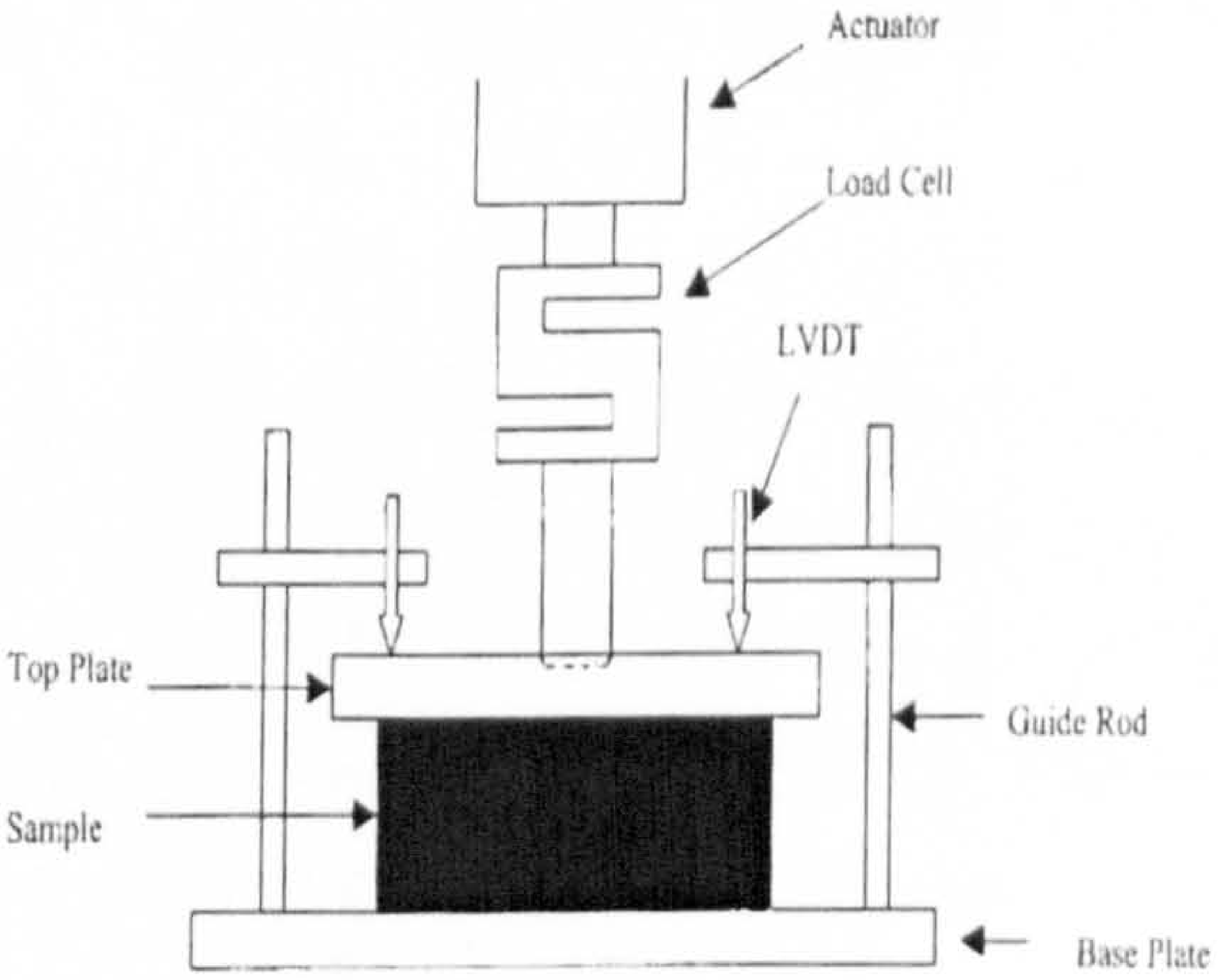


Fig. 5.10 Schematic of creep test configuration.

The Dynamic Creep Test is normally terminated at 3600 pulses (1 hour cumulative loading time), which requires a total testing time of 2 hours, i.e. 2 seconds/pulse × 3600 pulses.

a. Creep stiffness modulus (S_{mix})

At the conclusion of a static or dynamic creep test, the **creep stiffness modulus (S_{mix})** of the test specimen, can be obtained. The S_{mix} value is indicative of the resistance to permanent axial deformation and for bituminous specimens is basically obtained from the ratio of applied stress to the cumulative compressive strain at a defined temperature and time of loading [BS DD-185:1990]. During the test, axial deformations are continuously monitored as a function of time. Knowing the initial height of the specimen and the magnitude of axial strain ε, the creep stiffness modulus S_{mix} at any loading time can thus be determined:

$$S_{mix\,(t,T)} = \frac{\sigma}{\varepsilon_t}$$

where: $S_{mix\,(t,T)}$ = creep stiffness modulus (MPa) at test temperature T and loading time t; σ = applied stress, e.g. 100 kPa; $\varepsilon_{(t)}$ = cumulative irrecoverable axial strain at time t.

The creep stiffness value at the end of the test or after 1 hour cumulative loading time is normally reported. This value is obtained using the above equation by inputting the measured cumulative irrecoverable axial strain after 1 hour loading time.

b. Slope of creep curve

By plotting the dynamic creep test results against time, the '*minimum slope*' of the creep line can be identified. *The smaller the minimum slope value, the better the mixture's resistance to deformation.* The materials testing apparatus (MATTA) software will automatically locate the minimum slope in units of microstrains per loading cycle by analyzing the creep results with respect to time. Fig. 5.11 shows a typical creep dynamic creep test curve and the derivative of the creep line (both plotted on *a normal scale*). Typical slope values obtained from testing asphalt concrete mixtures in repeated load axial creep tests are given in Table 5.4 [Alderson, 1995].

It can be seen in Fig. 5.11, that at the beginning of the test there is a rapid increase in permanent strain, this phase is called the 'primary stage'. This is followed by a secondary creep stage in which *an approximately linear relationship* between the number of load pulses and permanent deformation is observed. The final 'tertiary stage' is indicated by an increasing rate of permanent deformation and is related to failure of the test specimen.

The *point of inflection* or '*minimum slope*' is always located in *the secondary stage*, and is *usually in the first third of this stage.* The coordinates of the point with minimum slope are required, which allow the intercept with the y-axis to be determined, using the following equation [Alderson, 1995]:

$$\text{Intercept} = Y - (\text{Minimum Slope}) \times X \qquad (5.13)$$

where: Y = strain at minimum slope (micro strain), and X = number of load cycles to minimum slope.

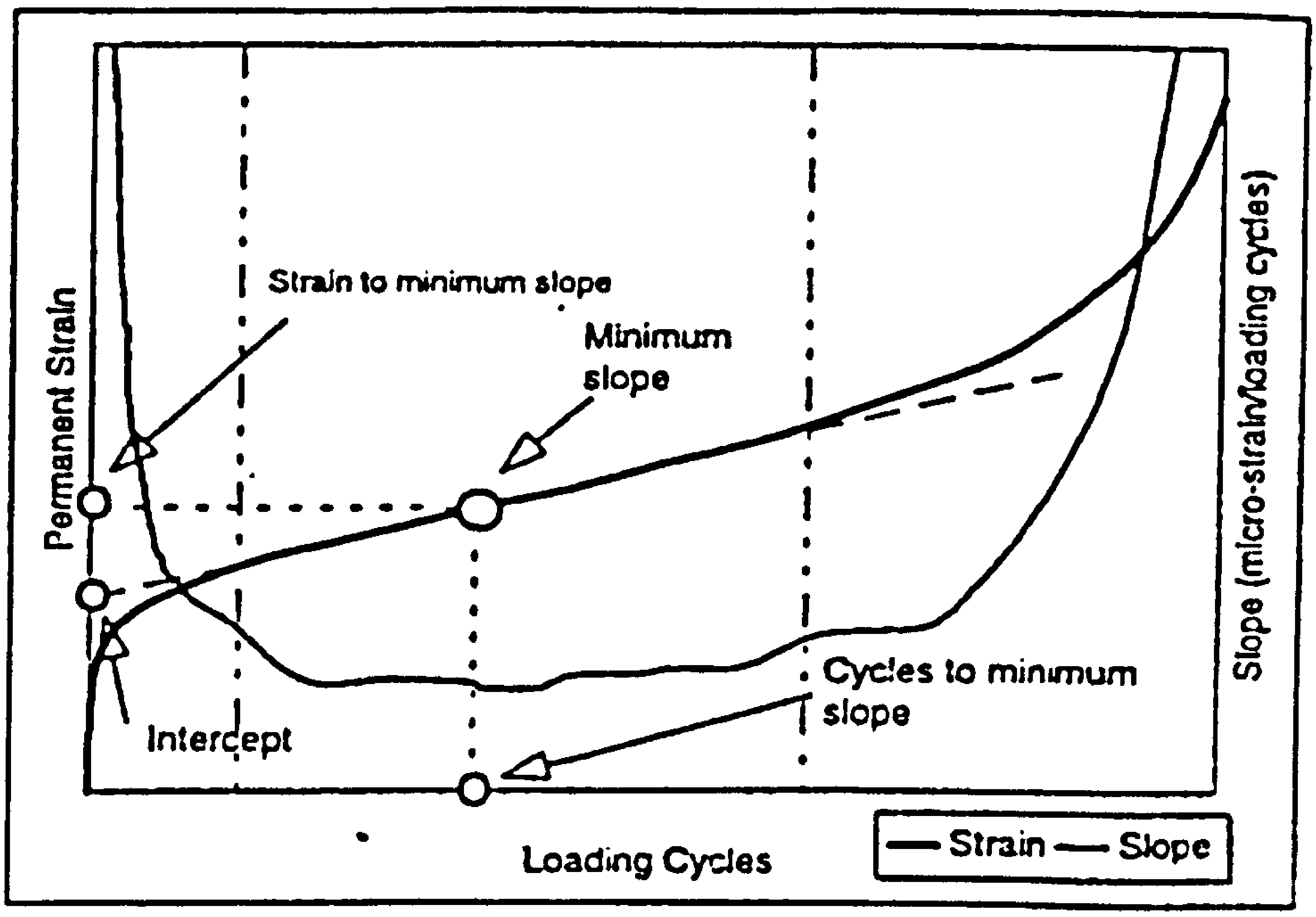


Fig. 5.11 Typical dynamic creep curve including slope curve, [Alderson, 1995].

Alternatively, an equation can be derived for the secondary stage using a 'best fit' linear trend line of the data in the secondary stage. Care must be exercised when determining the starting and end points of the secondary loading stage.

Table 5.4 Typical Laboratory Determined Minimum Dynamic Creep Slope values, [Alderson, 1995].

Average Annual Pavement Temperature (°C)	Heavy Traffic $> 10^6$ ESA	Medium Traffic 5×10^5 to 10^6 ESA	Light Traffic $< 5\times10^5$ ESA
> 30	< 0.5	0.5 - 3	> 3 - 6
20 - 30	< 1	1 - 6	> 6 - 10
10 - 20	< 2	2 - 10	Not Applicable

Resistance to deformation can be influenced by several factors including; temperature, aggregate gradation, mixture porosity, filler content, and aggregate angularity. In the case of new bituminous mixtures (e.g. new aggregate or binder types), comparison is normally carried out in terms of deformation resistance to a standard (control) mixture whose performance is well known.

c. Creep recovery

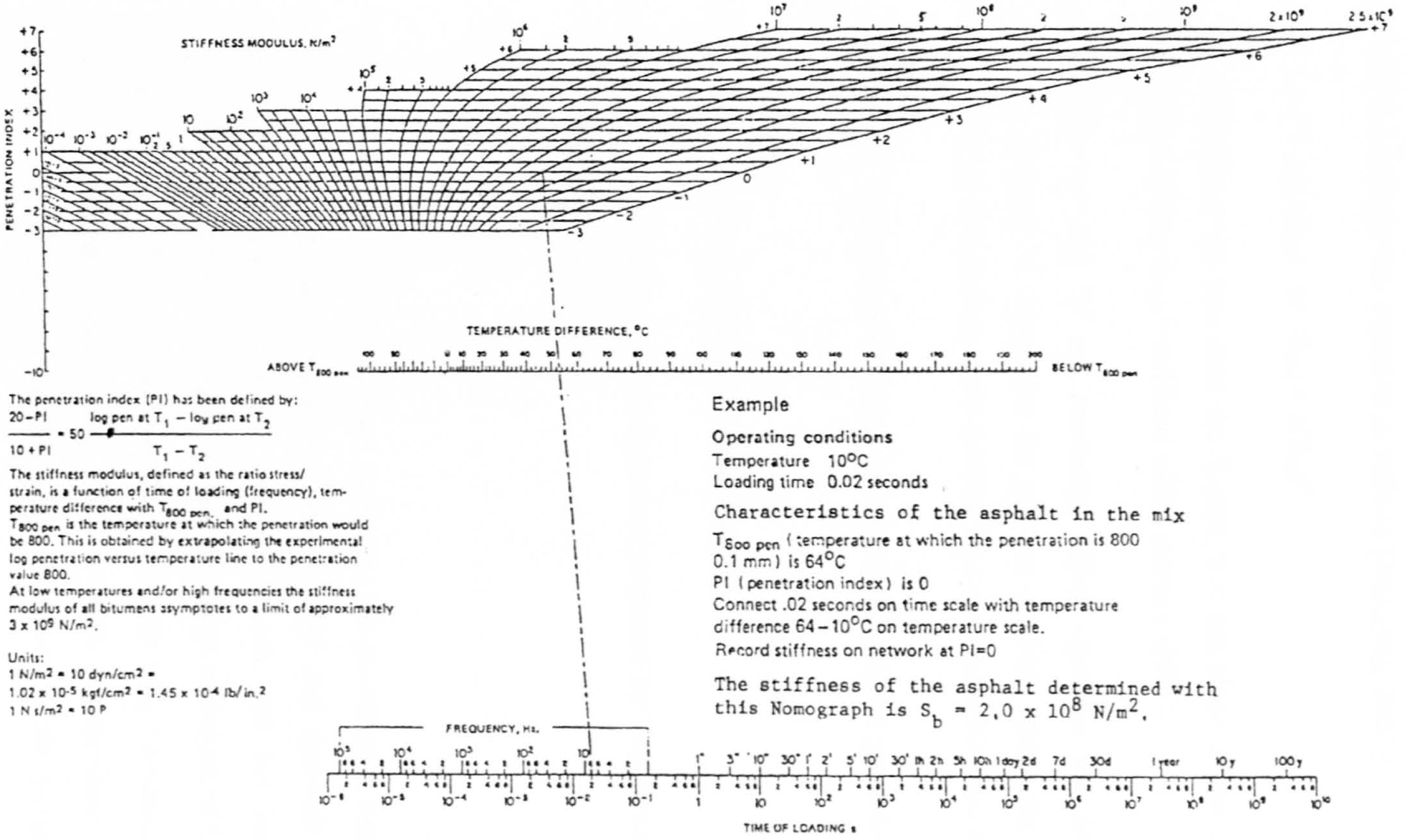
Creep recovery values are presented based on the recovery strains recorded after a predetermined unloading time (e.g 1 hour). The values are expressed as percentage of the recoverable axial strain value divided by the cumulative strain at the end of the creep test. Creep recovery values reflect the elastic behaviour of the bituminous mixtures subjected to compressive loading and are important in polymer and rubber modified mixtures.

$$C_{Rec} = \frac{\varepsilon_R}{\varepsilon_L} \qquad (5.14)$$

where: C_{Rec} = creep recovery (%); ε_R = recoverable axial strain (loading strain-recovery strain after unloading) in microstrains ; ε_L = cumulative axial strain or loading strain (microstrain) at the end of a static or dynamic creep test (see Appendix 4.4).

d. Relationship between stiffness of mixture (S_{mix}) and stiffness of bitumen (S_{bit})

The relationship between creep stiffness of mixtures S_{mix}, and stiffness of bitumen S_{bit}, (at the same loading time) can be used to characterize the mixture's behavior in terms of resistance to permanent deformation. Stiffness of bitumen values (S_{bit}) can be determined using the Van der Poel nomograph (Fig. 5.12) or obtained using the BANDS software (developed by Shell).



The penetration index (PI) has been defined by:

$$\frac{20-PI}{10+PI} = 50\,\frac{\log \text{pen at } T_1 - \log \text{pen at } T_2}{T_1 - T_2}$$

The stiffness modulus, defined as the ratio stress/strain, is a function of time of loading (frequency), temperature difference with $T_{800\ pen.}$ and PI.

$T_{800\ pen}$ is the temperature at which the penetration would be 800. This is obtained by extrapolating the experimental log penetration versus temperature line to the penetration value 800.

At low temperatures and/or high frequencies the stiffness modulus of all bitumens asymptotes to a limit of approximately 3×10^9 N/m².

Units:
1 N/m² = 10 dyn/cm² =
1.02×10^{-5} kgf/cm² = 1.45×10^{-4} lb/in.²
1 N s/m² = 10 P

Example

Operating conditions
Temperature 10°C
Loading time 0.02 seconds

Characteristics of the asphalt in the mix

$T_{800\ pen}$ (temperature at which the penetration is 800 0.1 mm) is 64°C
PI (penetration index) is 0
Connect .02 seconds on time scale with temperature difference 64−10°C on temperature scale.
Record stiffness on network at PI=0

The stiffness of the asphalt determined with this Nomograph is $S_b = 2.0 \times 10^8$ N/m².

Fig. 5.12 Van der Poel Nomograph for determining the Stiffness Modulus of Bitumen (S_{bit}).

Graphs relating S_{mix} and S_{bit} at pre-selected test temperatures (e.g. 40 or 60°C) are plotted on a double logarithmic scale. The relationship between these two parameters can be expressed by a straight line equation as follows [Suparma, 2002]:

$$\text{Log}\,(S_{mix}) = a + b\,\log\,(S_{bit}) \quad \text{or} \quad S_{mix} = a(S_{bit})^{b} \qquad (5.15)$$

where: a = the intercept of the line with the Y-axis; b = the slope of the line; S_{bit} = stiffness of the bitumen (MPa); S_{mix} = stiffness of the mixture (MPa).

The coefficients 'a' and 'b' are constants, which represent the deformation performance of the mixture. 'a' represents the mixture stiffness whilst the slope 'b' indicates the sensitivity of the mixture to the loading time and hence bitumen stiffness.

A mixture with high 'a' value and low 'b' can be expected to exhibit good deformation performance.

e. Prediction of Permanent Deformation (Rut Depth)

A procedure for the prediction of rut depth was proposed by Van de Loo [1978] with the aid of the following relationship:

$$R_d = \frac{C_m \times h \times \sigma_{avg}}{S_{mix}} \qquad (5.16)$$

where: R_d = rut depth (mm); C_m = correction factor for the-so-called dynamic effect which takes account of differences between static (creep) and dynamic (rutting) behaviour. This factor is dependent on the type of mixture and has been found empirically to be in the range 1.0-2.0. Table 5.5 lists the range of values of C_m factors for different mixture types; h = pavement layer thickness (m); σ_{avg} = average stress in the pavement under a moving wheel (MPa), which in turn is dependent upon the bituminous layer type and pavement temperature; S_{mix} = stiffness of the mixture (MPa) derived from creep testing.

Table 5.5 Suggested C_m values, [Van de Loo, 1978].

Mixture type	C_m
Sand sheet and lean sand mixtures, lean open asphaltic concrete	1.6 – 2.0
Lean bitumen macadam	1.5 – 1.8
Asphaltic concrete, Gravel and asphalt dense bitumen macadam	1.2 – 1.6
Mastic asphalt, HRA	1.0 – 1.3

5.3.4 Fatigue Testing (Indirect Tensile Fatigue Test, ITFT)

Fatigue can be defined as the phenomenon of fracture under repeated stress, whose magnitude is usually less than the tensile strength of the material. Tensile strains of the order of 30 to 200 microstrains can cause fatigue cracking of bituminous mixtures.

Fatigue life largely depends on stress magnitude, loading frequency, and duration of rest period between load applications, or in other words it depends on the test procedure used [Croney & Croney, 1998]. Additionally, fatigue life is longer at low temperatures in which the mixtures are stiffer [Whitoeak, 1991].

Raithby and Sterling (1972) found that the introduction of 'rest periods' between stress applications (to simulate traffic loading) has a significant influence on fatigue life. Sawn beams of rolled asphalt surfacings were subjected to direct tensile and compressive fatigue loading. It was found when a rest period of 0.3 seconds was introduced between each load pulse, fatigue life was increased by as much as 25 times in the temperature range of 10 to 25°C.

In their investigation, Cooper & Pell (1974) concluded that the main mixture factors affecting fatigue performance are the binder content and binder type. Increasing the mixture binder content increases fatigue life. Meanwhile, increasing stiffness by reducing the porosity (for example by compaction), and modifying the aggregate gradation have relatively little direct effect on fatigue life.

Sousa et. al. (1998) also studied the effect of aggregate gradation on fatigue life of asphalt concrete and they found that:

a) The fatigue performance of mixtures made with finer gradations were better than mixtures containing coarser gradations.

b) Harder binders have better performance in thicker pavements.

Fatigue life can be determined in the laboratory using the following relationship [Whiteoak, 1991]:

$$N_f = c \times \left(\frac{1}{\varepsilon_t}\right)^m \qquad (5.17)$$

where: N_f = number of load applications to initiate a fatigue crack ; ε_t = maximum value of applied tensile strain; c and m = factors depending on the composition and the properties of the mixture; m is the slope of the strain-fatigue line.

Fatigue tests can be carried out in a '*controlled stress*' or in a '*controlled strain condition*'. In controlled stress testing, which is more applicable to thick bituminous constructions (> 150mm), the peak value of the cyclic load applied on the specimen is kept constant whilst the resultant strains are monitored. In constant stress tests, the measured lives do not usually contain a large amount of crack propagation time and the end point of a test is very definite, i.e. complete fracture of the specimen.

Controlled strain tests are more applicable to thin bituminous surfacings (≤ 50mm). As the damage accumulates during testing, the stress required to maintain the initial strain level gradually decreases following crack initiation, as the stiffness of the mixture is effectively decreased. When a crack initiates, to maintain the same strain level, a reduction in stress occurs around the crack and therefore crack propagation is relatively slow compared with the controlled stress mode of failure. Hence failure does not occur with a sudden or total specimen fracture/collapse and since the damage is continuous, failure is an arbitrarily defined point. The test specimen is usually deemed to have failed when the load required to maintain the initial level of strain has fallen to 50% of its initial value. Therefore in general, controlled strain tests give greater fatigue lives than controlled stress tests.

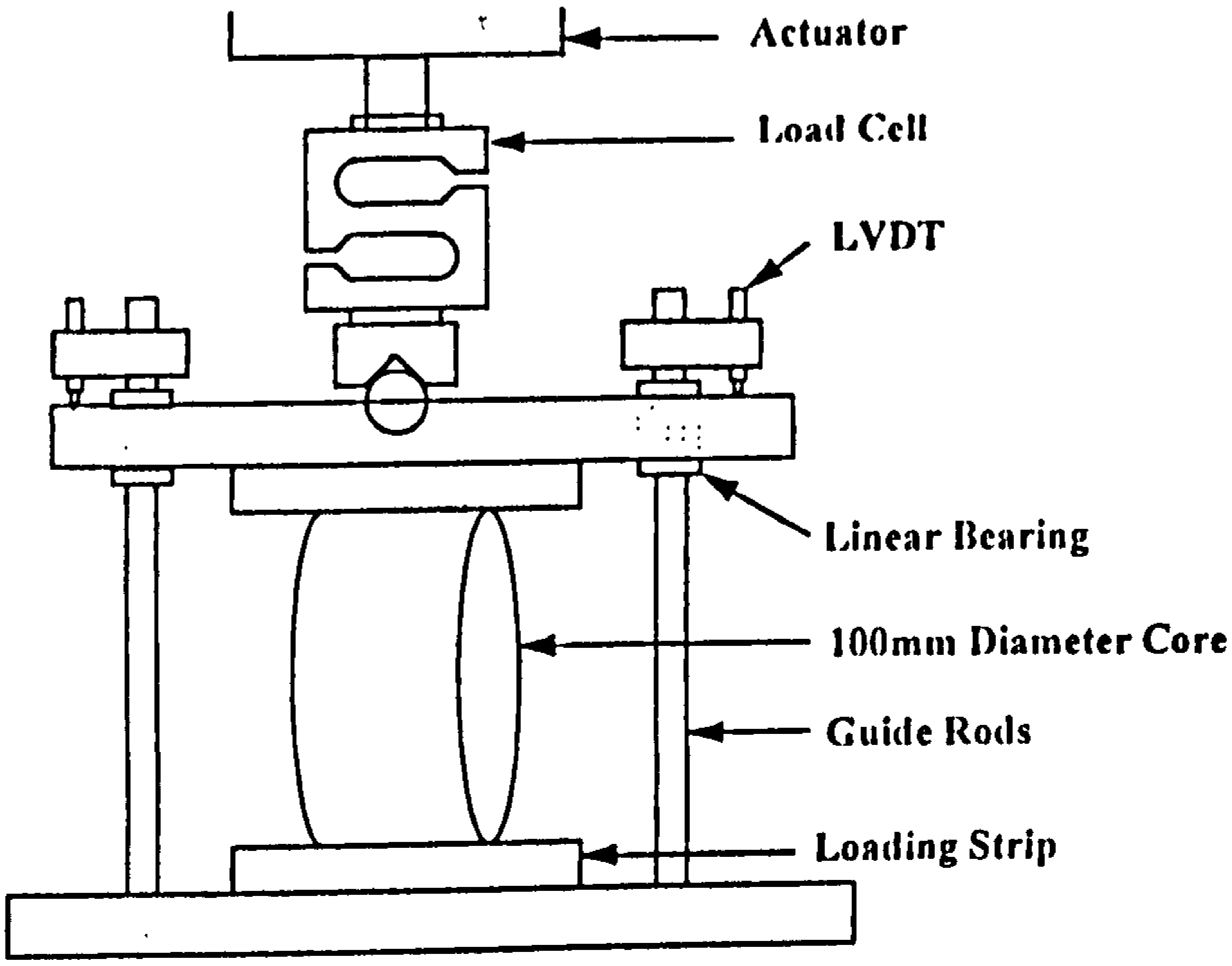


Fig. 5.13 Sample arrangement for an ITFT.

A typical example of a controlled stress test configuration is the indirect tensile fatigue test (ITFT). In the ITFT test, a cylindrical specimen is subjected to repeated diametrical line loading along the vertical diameter which produces an indirect tensile stress on the horizontal diameter (Fig. 5.13). The test is capable of characterising the fatigue life of a bituminous mixture by testing a small number of specimens (less than 10) at high temperatures (in excess of 25°C) and at high stress levels (> 450 kPa).

As shown in Figure 5.13, the vertical applied load produces both a vertical compressive stress and a horizontal tensile stress along the diameters of the tested specimen. The magnitudes of the stresses vary along the diameters but a maximum value occurs at the centre of the specimen. Based on the following assumptions, the stress and strain conditions can be calculated using elastic theory [Read *et al.*, 1997]:

- The specimen is subjected to plane stress conditions ($\sigma_z = 0$).
- The material is homogeneous and behaves in an isotropic and linear elastic manner.
- Poisson's ratio (υ) for the material is known.

- The force (P) is applied as a line loading.

By accepting the above assumptions, the following equations can be used for calculating the maximum horizontal tensile stress (σ_{xmax}) and strain (ε_{xmax}) at the centre of the specimen:

$$\sigma_{x\,max} = \frac{2P}{\pi \times d \times t} \tag{5.18}$$

$$\varepsilon_{x\,max} = \frac{\sigma_{x\,max}}{S_{mix}}(1 + 3\upsilon) \times 1000 \tag{5.19}$$

where: σ_{xmax} = maximum horizontal tensile stress at the centre of the specimen (kPa); ε_{xmax} = maximum initial horizontal tensile strain at the centre of the specimen (microstrain); S_{mix} = the indirect tensile stiffness modulus (MPa); d = diameter of the test specimen (mm); t = thickness of the test specimen (mm); P = vertically applied vertical force (kN); υ = Poisson's ratio (assumed to be 0.35).

In the latest ITFT Test protocol as covered within BS - DD ABF (2002), the recommended dimensions for a test specimen are *a diameter (d) of 100mm and a thickness (t) of 30-75mm.* During the test, the cumulative permanent vertical deformation is measured using linear variable differential transducers (LVDTs) and *the failure point of the test is defined as a permanent vertical deformation value of 9mm*, or specimen failure.

Table 5.6 Target fatigue test stress levels for the second, third and fourth specimens based on the number of cycles to failure of first specimen tested at a stress level of 500 kPa (N_{500})

	Test stress level (kPa)			
	$N_{500} \leq 200$	$200 < N_{500} \leq 500$	$500 < N_{500} \leq 1000$	$N_{500} > 1000$
2nd specimen	400	425	450	475
3rd specimen	300	350	400	450
4th specimen	200	250	300	400

According to the standard, the target stress level for the first specimen to be tested shall be 600 kPa. If this stress level cannot be reliably obtained, the target stress level shall be 500 kPa. The target stress levels suggested for subsequent test specimens are given in Table 5.6 and Table 5.7.

Table 5.7 Target fatigue test stress levels for the second, third and fourth specimens based on the number of cycles to failure of first specimen tested at stress level of 600 kPa (N_{600})

	Test stress level (kPa)			
	$N_{600} \leq 200$	$200 < N_{600} \leq 500$	$500 < N_{600} \leq 1000$	$N_{600} > 1000$
2nd specimen	500	525	550	575
3rd specimen	400	450	500	550
4th specimen	300	350	400	500

The normal test temperature for the ITFT test is 20°C ± 0.5°C and the target test temperature shall not exceed 40°C. For specimens that need to be stored prior to testing, if the storage period is less than 4 days, the storage temperature shall not be more than 25°C and for storage periods of more than 4 days the temperature shall not exceed 5°C.

The fatigue lines can be derived by plotting the number of cycles to failure (N_f) on the x-axis, and the initial tensile strain ($\varepsilon_{x\ max}$ in microstrains) on the y-axis, both on a log-log scale. The regression line that best fits the test results should have a correlation coefficient (R^2) of more than 0.90, otherwise more data or tests shall be added until $R^2 \geq 0.90$ is achieved.

If it is accepted that fatigue life can be primarily controlled by the magnitude of strain, then under controlled stress testing conditions, stiffness will largely determine the position of the stress-life line, higher stiffness values giving longer lives. Therefore, improved fatigue performance on the basis of stress-life may simply be a reflection of increased stiffness. Consequently, results that are analysed on the basis of strain and the resulting *strain-life relationships* are considered to be *a better basis for the comparison of different mixes*, life at a given strain (e.g. at 100 microstrains) being taken as a measure of fatigue performance.

5.3.5 The Indirect Tensile Strength (ITS) Test

In the indirect tensile strength test, a cylindrical sample is subjected to compressive loads along two generators which creates tensile stresses perpendicular to and along the diametral plane causing a splitting failure. Testing is carried out in accordance with any of the following standards: BS 99/108553, BS EN 12697-23, ASTM D4123 and PrEN 12697-23. The test is normally carried out at room temperature using a Marshall loading frame fitted with 12.5mm wide concave surface loading strips below and above the Marshall sized bituminous sample, as shown in Fig. 5.14. The rate of loading is the same as in the Marshall Stability test, i.e. 51mm per minute.

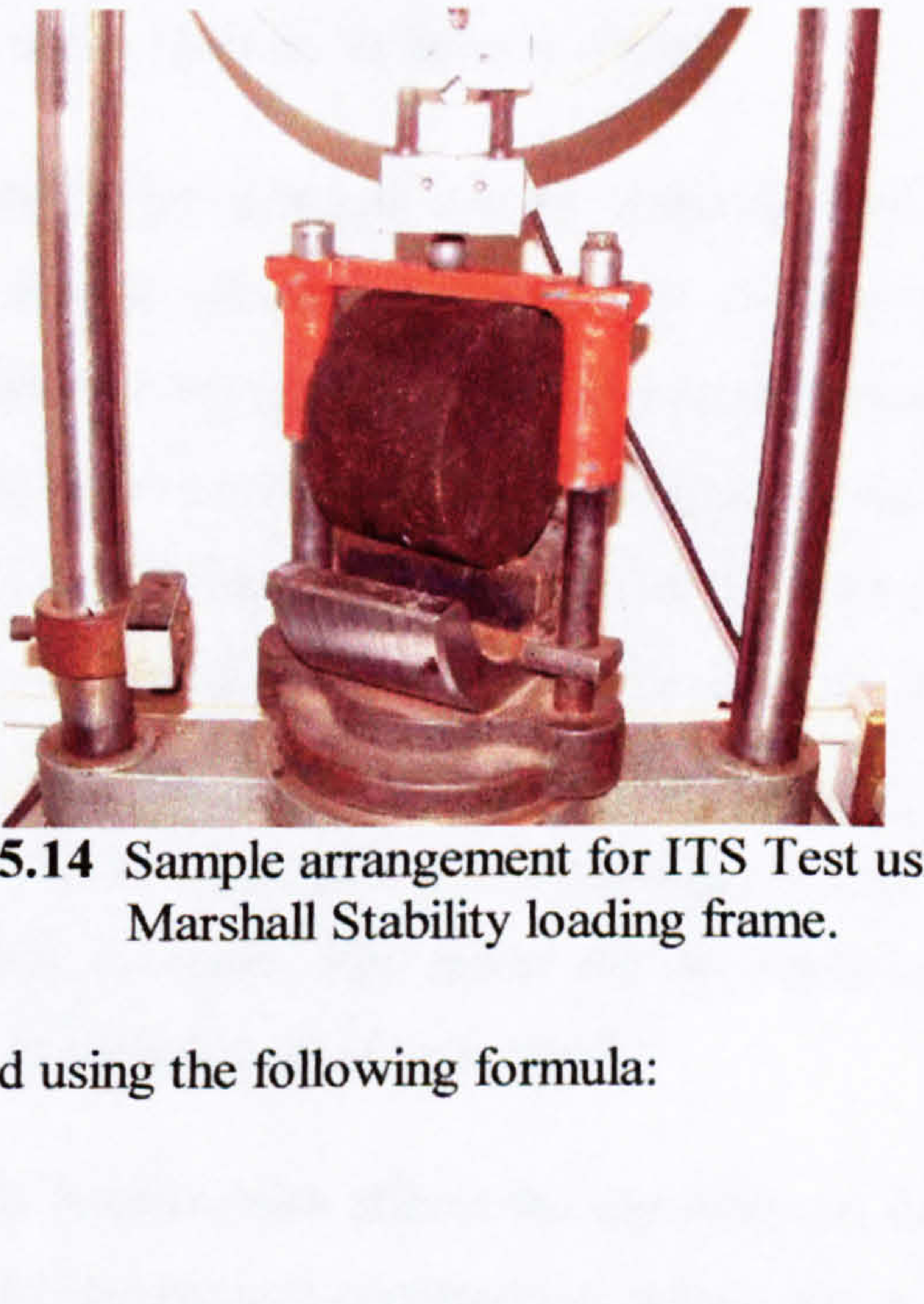


Fig. 5.14 Sample arrangement for ITS Test using Marshall Stability loading frame.

The ITS value is calculated using the following formula:

$$\mathrm{ITS} = \frac{2P\max}{\pi \times t \times d} \tag{5.20}$$

where: ITS = indirect tensile strength (kPa); $P_{max.}$ = maximum load (kN); t = average height of sample (m); d = diameter of sample (m).

The ITS test provides two mixture properties that are useful in characterising bituminous mixtures. The first property is *tensile strength*, which is often used in *evaluating water susceptibility* of mixtures. The tensile strength is also sometimes used to help evaluate cracking potential of a bituminous mixture. However, the second property determined from the indirect tensile test, which is *tensile strain at failure*, is more useful for predicting cracking potential. Mixtures that can tolerate high strains prior to failure are more likely to resist cracking [Suparma, 2001].

5.4 Durability Tests

5.4.1 General

Durability can be defined as the ability of the materials in the asphalt pavement structure to withstand the effects of environmental conditions, such as water, ageing and temperature variations without any significant deterioration for an extended period for a given amount of traffic loading [Scholtz & Brown, 1996].

It is generally agreed that damage due to water (moisture damage) and age hardening are the major factors affecting the durability of bituminous mixtures. There are two mechanisms by which water can damage the structural integrity of the bitumen-aggregate interface, firstly water can cause loss of cohesion (strength) and stiffness of the bitumen and secondly, water attacks the adhesive bond between the bitumen and the aggregate in the mixture (stripping). These two water damage mechanisms result in decreasing the strength of the pavement layer [Scholtz & Brown, 1996]. The detachment of bitumen off the aggregate (or stripping) is more likely to occur in mixtures that are permeable to water. The lower the air voids content in compacted mixtures, the less the risk of stripping [Suparma, 2001].

Age hardening of the bitumen also affects the durability of the bitumen-aggregate matrix. Primarily during the mixing and construction stages, but additionally whilst the pavement is in service, the bitumen becomes gradually stiffer due to oxidation which chemically affects the bitumen chemical composition [Whiteoak, 1991]. Bitumen slowly oxidises when in contact with air, the oxygen in the air causes an increase in viscosity and makes the bitumen harder and less flexible. The degree of oxidation is highly dependent on the temperature, time and the bitumen thickness. The loss of volatile components is also dependent on temperature and exposure conditions. This stiffening is often referred to as ageing or age hardening and results in the stiffening of the bituminous layer. Age hardening during the construction process is categorised as short-term ageing and age hardening of the mixtures in service is categorised as long-term ageing [Suparma, 2001].

Other forms of bitumen hardening include: 1- Physical hardening, which occurs at ambient temperatures and results from reorientation of molecules and the slow crystallisation of waxes, 2- Exudative hardening which occurs due to the movement of an oily component from the bitumen into the mineral aggregates.

5.4.2 Water Damage Tests

Water Damage Tests for CBEMs are generally carried out following a *certain moisture conditioning procedure* and are expressed in terms of 'retained mechanical property', i.e. the ratio of the sample's strength or stiffness *after and before the conditioning procedure.* Mechanical characterisation is commonly performed using the Marshall stability test, the indirect tensile strength (ITS) test or the non destructive indirect tensile stiffness modulus (ITSM) test. With destructive tests such as the Marshall stability test, two identical sets of specimens are required for obtaining the retained strength. One set is tested in an unconditioned state and the other set is tested following moisture conditioning. Using a non-destructive test, e.g. the ITSM Test, the same samples can be tested before and after moisture conditioning.

A range of moisture conditioning procedures is available in the literature suitable for CBEM specimens as described below:

a. Capillary Soaking

In this procedure, samples are tested that had been manufactured and cured according to a pre-determined curing procedure. Half the thickness of each specimen is soaked in water at room temperature for 24 hours, the specimen is then inverted and the other half is soaked for a further 24 hours. During soaking, each sample rests on a bed of approximately 15 to 20mm coarse sand to allow full contact with water, as shown in (Fig. 5.15). The samples are subsequently towel dried and then tested for water absorption and Marshall stability at room temperature, and the results are referred to *as soaked stability* values [MPW-RI, 1990].

Fig. 5.15 Capillary soaking.

b. Vacuum Saturation

Moisture exposure using vacuum saturation simulates the effect of exposure of the cold bituminous mixture to subsurface water. The equipment consists of a glass jar (dessicator) connected to a vacuum pump capable of applying a *reduced pressure* of up to 100mm of Hg as shown in Fig. 5.16. The specimen is placed in the vacuum apparatus and completely covered with water. The dessicator is evacuated at 100mm of Hg and held under vacuum for one hour. The vacuum is slowly released and the specimen is allowed to soak in water for another one hour. The specimen is then removed from the water, towel dried, weighed in air and then tested for its strength [Asphalt Institute, 1989].

Fig. 5.16 Vacuum saturation apparatus.

5.4.3 Ageing Tests

Durability of bituminous mixtures with respect to ageing are more commonly evaluated with hot bituminous mixtures compared to cold mixtures. One of the main concerns of CBEMs is their low early life strength. This has prompted many researchers to explore ways of accelerating the rate of increase of CBEMs early life strength. In theory, once a CBEM achieves full curing, it should behave like a hot mixture.

When considering hot bituminous mixtures, there are two types of ageing, namely *short term oven ageing (STOA)* to simulate mixture ageing during the mixture manufacture stage, and *long term oven ageing (LTOA)* for simulating the ageing of the mixture on the road during service.

In short term oven ageing for hot mixtures, *the loose,* bitumen coated bituminous mixtures are aged at 135°C for 4 hours prior to compaction. This ageing *procedure is not suitable for CBEMs,* as the CBEMs production process, i.e. mixing and compaction is carried out at ambient temperatures with the mixtures contain substantial amounts of liquids (water of emulsion + pre-wetting water) to achieve adequate workability and binder coating. The excess water remains trapped in the compacted specimens for extended periods of time as the mixture cures and gradually loses its excess moisture.

On the other hand, with hot bituminous mixture long term ageing, the compacted samples are cured in an oven at 85°C for 2 or 5 days to represent 5 and 10 years ageing on site respectively, in accordance with SHRP A-003A recommendations [Kliewer, et al. 1995]. Long term ageing for CBEMs can only be logically carried out after the samples had achieved a full curing condition. It is not considered that researching the ageing of CBEMs is an urgent matter as it is overshadowed by concerns for their low early life strengths, in some cases unacceptably long curing times and high compacted mixture porosities.

CHAPTER VI

REVIEW OF CBEMs MIXTURE DESIGN PROCEDURES AND SPECIFICATIONS

6.1 Review of selected CBEMs Design Procedures

Currently, there exists a whole range of CBEMs mixture design procedures among the various road authorities and research organizations. The majority of design and test procedures (in particular those written in English) are based on American design procedures, e.g. The Asphalt Institute or AASHTO, with some modifications.

In general the design procedures cover the following aspects: determination of suitable aggregate gradations, binder coating tests, optimum water content at compaction, and the determination of optimum residual bitumen content. Few additional, more recent tests have been proposed, e.g. by Nynas UK, for testing loose cold bituminous mixtures during storage and laying stages [Taylor, 1997].

In the following sections, three design procedures for cold bituminous emulsion mixtures are briefly described, namely: the Asphalt Institute Design Procedure, Ministry of Public Work, Republic of Indonesia (MPW-Indonesia) Design Procedure, and a Design Procedure suggested by Nikolaides A.F. Additional tests suitable for loose bituminous emulsion mixtures as proposed by Nynas are also presented.

6.1.1 Design Procedures covered by the Asphalt Institute

The Asphalt Institute Manual Series No.14 (MS-14), Asphalt Cold Mix Manual, 1989, Third Edition covers two design emulsified asphalt aggregate cold mixture design methods, these are:

i. The Modified Hveem method for emulsified asphalt aggregate cold mixture design.

ii. The Marshall method for emulsified asphalt aggregate cold mixture design.

The modified Hveem method requires the use of specific equipment that are less widely available, such as the Hveem Stabilometer (a stability / bearing capacity type test), and the Cohesiometer (for measuring cohesive resistance or tensile strength of compacted mixtures).

Meanwhile, The Marshall method for cold mixtures uses the popular Marshall testing apparatus for stability and flow determinations, and ***is applicable for base courses of low traffic volume and dense graded mineral aggregates with maximum size of 25mm, at ambient temperature.*** The Marshall method for emulsified asphalt aggregate cold mixture design is based on research conducted at Illinois University. The detailed design procedure is described in manual series MS-14 and has been largely adopted by the author in his investigation as described in Chapter VII. In general the design procedure covers the following steps:

a. Determination of a suitable aggregates gradation based on empirical aggregate grading specifications (see section 6.2.2).

b. Determination of an initial approximate amount of emulsified asphalt (bitumen emulsion) required based on empirical formulae.

c. Coating test; this test is designed to assess the ability of the emulsion to sufficiently coat the aggregate particles.

d. Determination of optimum water content at compaction.

e. Variation of emulsion content and hence the Residual Bitumen Content for the chosen gradation.

f. Curing protocol:

    1- Dry specimens; the specimens are kept in the compaction moulds for 24 hours. Following extrusion, the specimens are subsequently cured in an oven at 38°C for a further 24 hours.

    2- Soaked specimens; after carrying out the procedure for producing the "dry specimens" as in stage 1, the same specimens are then allowed to cool to room temperature and are then subjected to vacuum saturation under water for one hour at the end of which, the negative pressure is released and the specimens brought back to atmospheric conditions. The specimens are subsequently kept fully immersed in water for a further one hour (see Chapter V, Section 5.4.2.b.).

g Modified stability test carried out on both dry and soaked specimens (after surface drying with a towel) at 22.2°C.

h. Determination of optimum residual bitumen content (ORBC) taking into consideration the following parameters: stability, % of stability change or percent stability loss, i.e. (dry stability - soaked stability)/dry stability, in percent (see Equation 6.4), dry bulk density, moisture absorption, and total voids at various residual bitumen contents, as illustrated in Fig. 6.1.

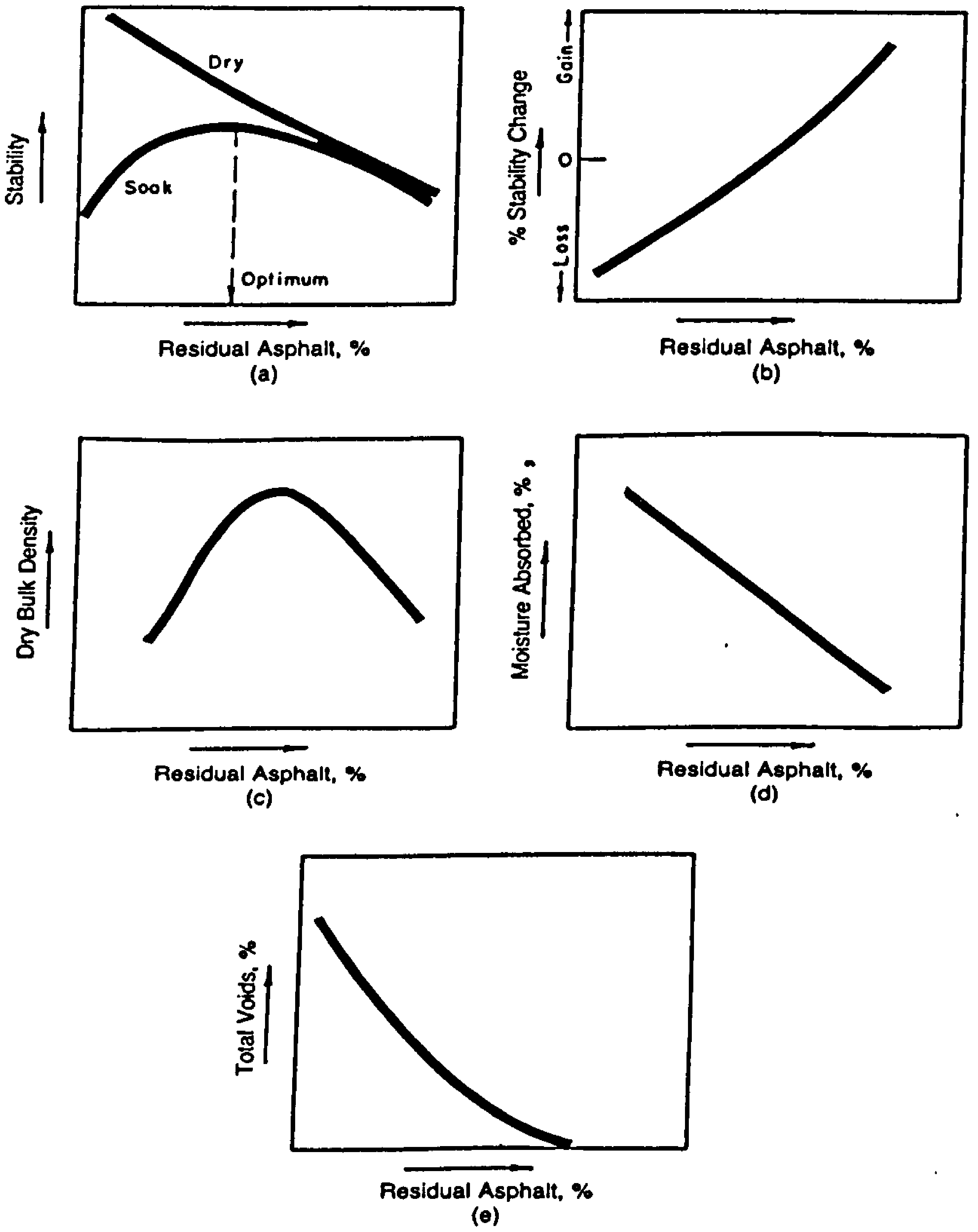


Fig. 6.1 Typical Emulsified Asphalt-Aggregate Mixture Design Plots.

The *latest* manual on asphalt emulsions published by **The Asphalt Institute** in association with the **Asphalt Emulsion Manufacturers Association (AEMA)** is **Manual Series (MS) No. 19, 'A Basic Asphalt Emulsion Manual', Third Edition, 1997.** When compared with MS-14, some changes have been introduced in the design procedures within this new manual (***which should be viewed as provisional***, as research on this matter is ongoing). The main points of CBEMs design procedure in **(MS-19)** are listed below:

A. Dense Graded Aggregate Cold Mixture Design

a. The Asphalt Institute proposes aggregate gradations (see section 6.2.3), with maximum aggregate size recommended is 25mm.

b The recommended emulsion types are either, a Medium Setting emulsion for use with gradations having no excessive amounts of particles passing 0.075mm (filler), or slow setting emulsions for use with gradations having excessive amounts of filler.

c. Determination of trial emulsion content:

- Base Mixtures: $$\%\,\text{Emulsion} = \frac{[(0.06\times B)+(0.01\times C)]\times 100}{A} \qquad (6.1)$$
- Surface Mixtures: $$\%\,\text{Emulsion} = \frac{[(0.07\times B)+(0.03\times C)]\times 100}{A} \qquad (6.2)$$

where: % Emulsion = estimated initial percent asphalt emulsion by dry weight of aggregates; A = % residue of emulsion or emulsion bitumen content; B = % of dry aggregates passing 4.75mm sieve; C = (100-B) = dry aggregate retained on 4.75 mm sieve.

d. Coating test, water resistance and adhesion tests; this procedure is described in more detail in Chapter VIII Section 8.5.3, and Chapter IX Section 9.5.1.

e. Specimen preparation including mixing, compaction, and curing procedures. The list below presents only the main aspects:

1- For simulating damp aggregate conditions on site, a water content equivalent to that in the stockpile or some other pre-selected value of moisture content must be added to the batched aggregates, mixed and covered to prevent moisture loss for about 24 hours prior to the addition and mixing of the emulsion.

2- Aeration or drying of the loose mixture is often required prior to compaction. This is to avoid the situation where the total liquid volume (emulsion + water in the aggregate) exceeds the voids in mineral aggregate (VMA) which prevents proper compaction from being carried out, particularly when using a Marshall hammer. This is characterised by the hammer bouncing during impact as a result of the high pore water pressures developed during such fast loading rates. *However, it was experienced by the author that if compaction is carried out using a Gyratory compactor, the loose mixtures can be compacted in a wetter condition, as the Gyratory compactor provides a kneading or squeezing effect. This matter is further described in Chapter IX, Section 9.4.*

3- MS-19 suggests that compaction be carried out at medium compaction effort, i.e. 50 blows Marshall hammer each end of the specimen.

4- For the curing stage, the specimens are kept in their compaction moulds. After removing the base, collar, and paper disc from each mould, the specimens are cured in an oven on a perforated shelf at 60°C for 48 hours. It is necessary to push each specimen in the compaction mould all the way to the bottom of the mould to avoid creep deformation during the curing stage.

5- After curing, each mould containing a compacted specimen is then removed from the oven and whilst still at 60°C, a static load of 178 kN is applied by a double plunger method, i.e. a free-fitting plunger is placed at the bottom and another on top of the specimen whilst in the mould. The load is applied at a rate of 1.3 mm/minute (0.05 inch/min.) of compression and once 178 kN is achieved, the load is maintained for one minute before being released.

6- Each compacted specimen is allowed to cool down in the compaction mould for at least one hour prior to extrusion from the moulds for further testing.

f. Determination of volumetric properties.

g. Stability testing:

- Marshall stability and flow tests; the specimen shall be placed in an air bath for at least 2 hours at the test temperature of 25±1°C. A stability value of 2224 N or greater has been found to be satisfactory for most pavements with low to medium traffic volumes.

B. Open-Graded Aggregate Mixture Design

The recommended aggregate gradations for open graded cold mixtures utilizing medium setting emulsions (see section 6.2.3). The main points are listed below:

a. A correctly graded representative aggregate sample is dried to a constant weight in a force draft oven at 60°C. After oven drying, the sample is cooled down to ambient temperature for a minimum of two hours.

b. A sufficient number of 2000 grams batched dried aggregates are pre-weighed.

c. Using a stainless steel bowl, each batch of aggregates is mixed with 40g of water (equivalent to 2%) until all of the aggregates are damp. Each bowl and its contents are covered with a clean cloth and left covered and undisturbed for 15 minutes.

e. The appropriate amount of asphalt emulsion pre-heated to 60°C is then added to the dampened aggregate and mixed by hand for 2 minutes using a metal spoon. Meanwhile, the workability of the mixture and the degree of binder coating is observed and recorded, (e.g. mixture has low workability / stiff with 80% aggregate coating). It is recommended that emulsion contents for the various batches are varied in 1% increment by weight of dry aggregates. As a starting point, an emulsion content of 4% is recommended for very coarse maximum sized aggregates and 6% for the finest maximum size aggregates.

f. Soon after mixing, each whole batch of loose emulsion coated mixture is transferred onto a 2.36mm (No.8) sieve that is placed on a balance. Prior to transferring the mixture, the 2.36mm sieve is lightly dampened with water.

g. The mixture is allowed to drain into the pan at ambient temperature for 30 minutes.

h. The sieve containing the drained mixture is lifted and the mass of the pan containing the drained emulsion/run-off (W_1) is recorded. This allows the amount of asphalt run-off in grams to be determined.

i. The drained mixture is removed from the 2.36mm sieve and spread onto a paper-lined tray. The mixture is surface dried with a fan and the percent of asphalt coating the aggregates is evaluated.

j. The pan containing the run-off is then placed in a force draft oven at 110±5°C and dried to a constant mass. The final mass (W_2) is recorded. Subtract the tared weight of the pan to determine the asphalt residue run off in grams. The tared weight of the pan is the weight of the pan containing no materials.

k. The % emulsion content by weight of aggregates versus the asphalt residue run off in grams is plotted on a graph and a smooth curve is drawn through the results as shown in Fig. 6.2.

l. A horizontal line at 10 grams of asphalt residue run off on the Y-axis is extrapolated to intersect with the curve. The corresponding % emulsion content on the X-axis is read to the nearest 0.1% and is classified as the **optimum emulsion content.**

m. At the optimum emulsion content, the mixture should have satisfactory workability, with a minimum coating of 85% for surface courses, and a minimum of 60% for base courses.

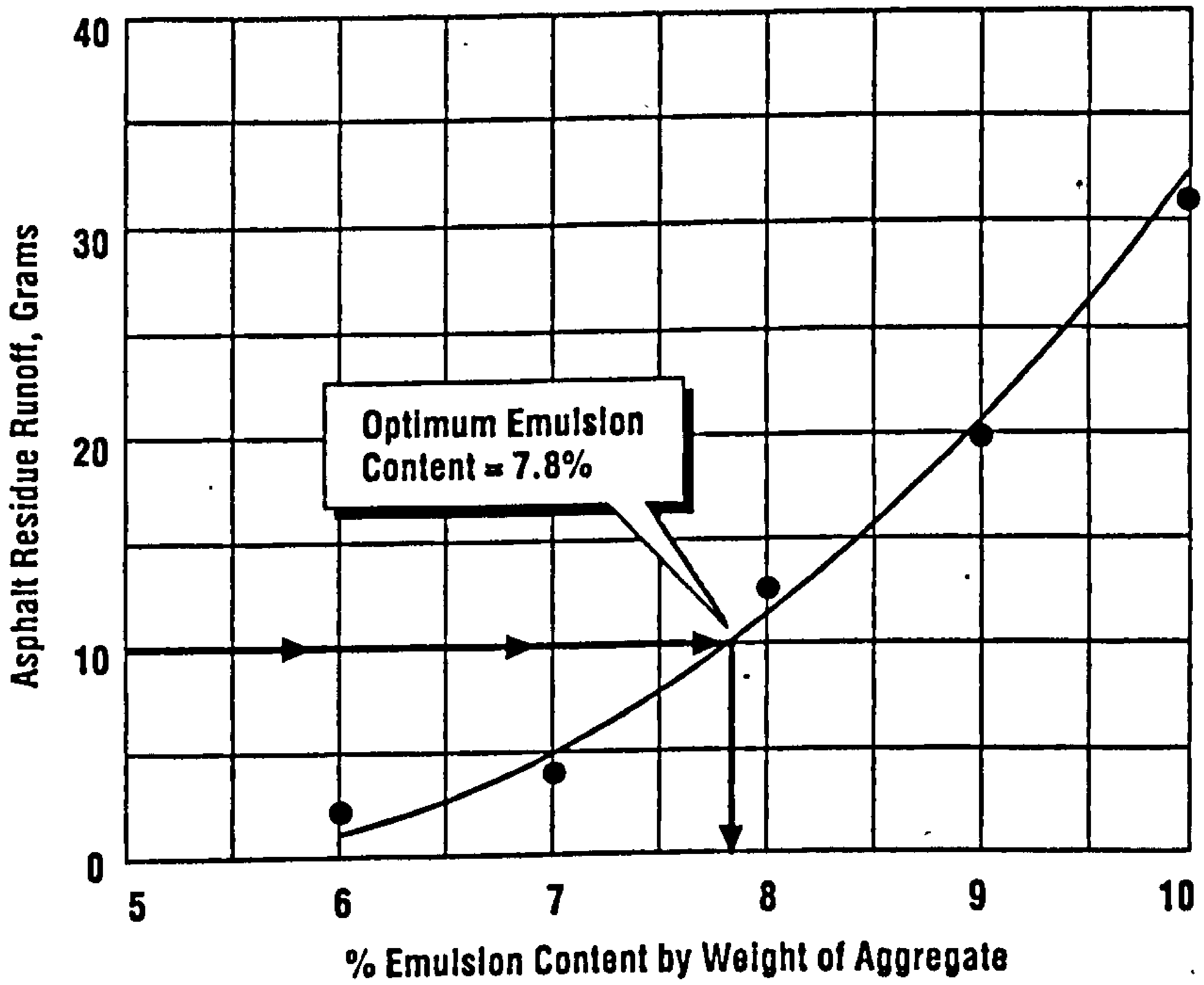


Fig. 6.2 Selection of the Optimum Emulsion Contentfor an Open Graded Asphalt Emulsion Mixture, [Asphalt Institute, MS-19, 1997]

6.1.2 Design Procedure of The Ministry of Public Work Republic of Indonesia, [MPW - Indonesia, 1990]

Specifications of the MPW-Indonesia (1990) covers two main areas, namely **open graded emulsion mixtures (OGEMs) and dense graded emulsion mixtures (DGEMs)**. The MPW-Indonesian Specification was set up for The Department of Public Work, Directorate General of Highways, Directorate of Planning, Republic of Indonesia by a team of Overseas Consultants who worked in Indonesia on World Bank funded Highway Projects in 1990.

In principle, the design procedure is mainly based on AASHTO and is similar to the Marshall design procedure covered in the Asphalt Institute MS-14 (1989), with modifications taking into consideration the regional/national conditions (see section 6.1.4). The *Test Procedure is referred to as the modified Marshall test procedure.* In this procedure the samples are tested using a Marshall Testing Frame **at ambient temperature** *(the standard testing procedure for the Marshall stability test for hot mixtures includes pre-conditioning the specimens in water at 60°C for 30 minutes. This is not recommended for cold mixtures).*

In this investigation, the author largely adopted the design procedure of the MPW-RI 1990, which are presented in detail in Chapter VII.

6.1.3 Design Procedure of Nikolaides A.F.

Nikolaides A.F. is currently a Professor of pavement engineering at Aristotles University of Thessaloniki, Greece, who worked in Indonesia as a consultant on road projects in 1990. His design procedure basically incorporates the American Standard and the specifications of the Ministry of Public Work Republic of Indonesia (MPW-Indonesia, 1990), as described in Chapter VII. The main differences are given in section 6.2.2.

In addition, Nikolaides introduced the method of characterising the **permanent deformation performance.** The permanent deformation performance is controlled by limiting **the maximum permissible value of bitumen content.** This value is derived from a chart showing the relationship between the residual bitumen content (RBC) and the creep stiffness coefficient (CSC) as shown in Fig. 6.3.

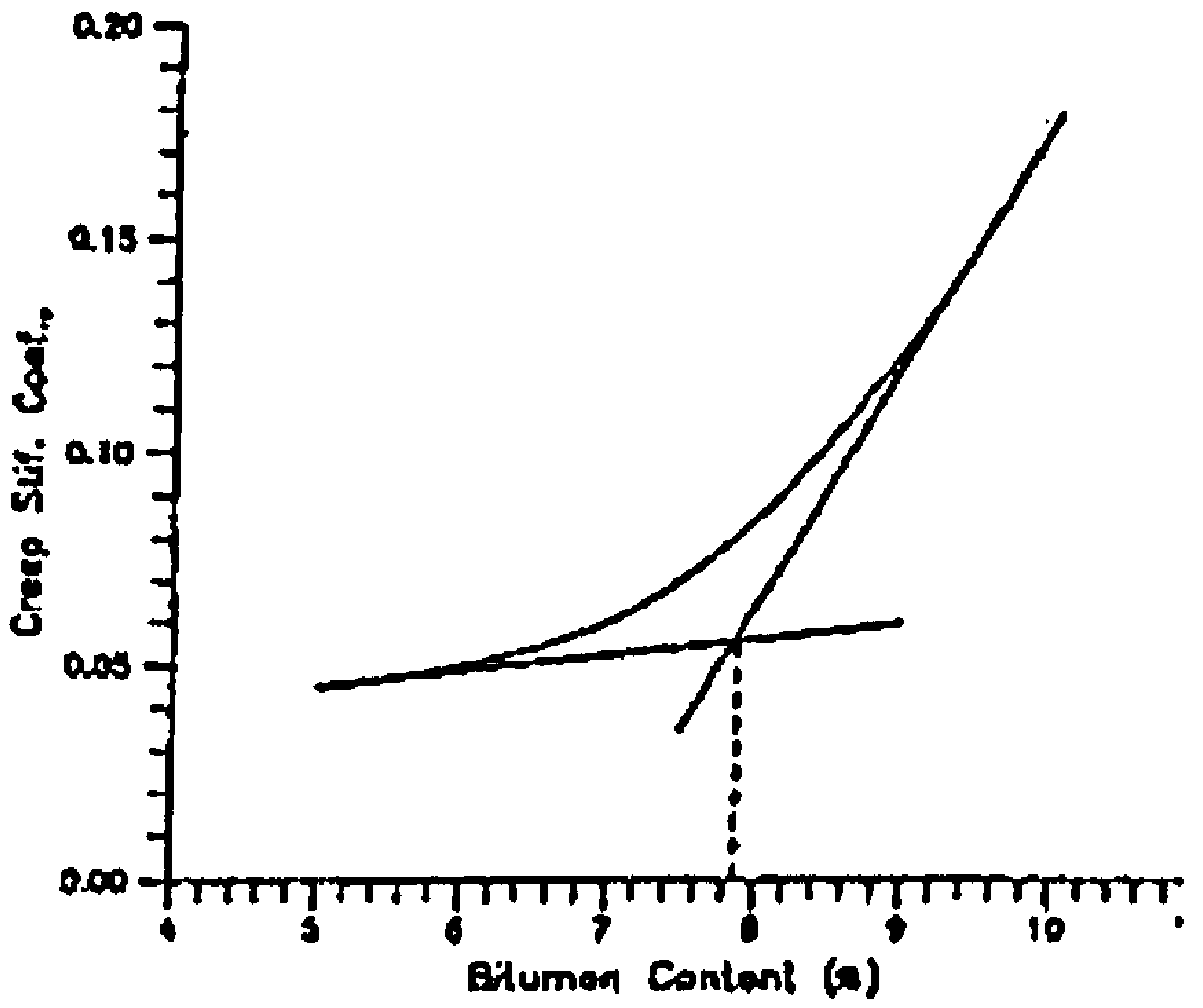


Fig. 6.3 Determination of maximum allowable bitumen content for optimum permanent deformation performance, [Nikolaides A.F., 1994].

The CSC is derived from creep strain results obtained from the 1 hour static creep test, typically conducted at a 40°C test temperature and a vertical applied constant static stress level of 100 kPa. For a particular test at one bitumen content, the stiffness of mixture (S_{mix}) can be calculated at any time of loading (ratio of static stress to resultant cumulative creep strain). The stiffness of bitumen (S_{bit}) can also be determined independently using the *van der Poel nomograph* at any loading time for a given grade of bitumen or *BANDS software* from Shell company, softening point temperature, and test temperature. Therefore, for any particular test specimen at one bitumen content, the results from the static creep test can be plotted in the form of S_{mix} v.s. S_{bit}. When the results are plotted in terms of Log S_{bit} (x-axis) v.s. Log S_{mix} (y-axis), a linear trend line can normally be obtained with the following equation describing the line:

$$\text{Log}\,Y = b\,\text{Log}\,X + \log A \qquad (6.3)$$

where: $Y = S_{mix}$, $X = S_{bit}$, A = constant, "b" is the slope of the linear line named CSC. However, this procedure is still being introduced and further evaluation may be required to ensure wider acceptance.

6.1.4 Summary of the selected CBEMs Design Procedures Reviewed

Table 6.1 below compares the specimen preparation and testing procedures for the *three main mixture designs* described earlier.

Table 6.1 Summary of Procedures for Preparing and Treating Samples.

Description	**The Asphalt Institute MS-14, 1989**	**Nikolaides A.F.**	**MPW Indonesia**
Degree of coating (%), for Base Courses Wearing Courses	 50% 75%	 85% 85%	 75% 75%
Compaction (Marshall Hammer)	2 × 50 blows	2 × 50 blows	2 × 50 blows
Curing in mould	24 hours at room temp.	24 hours at room temp.	24 hours at room temp (28 ± 1°C for tropical climate)
Curing in oven	24 hours at 38°C.	24 hours at 40°C.	24 hours at 40°C.
Samples Testing Temperature	22.2°C.	Room temp.	28 ± 1°C.
Soaking Procedure for the soaked sample.	The specimen is immersed in water and vacuum saturated at 100 mm of Hg for 1 hour, followed by immersion in water (no vacuum) for a further 1 hour at 22.2°C.	Capillary soaking: Half the depth of each compacted specimen is soaked in water at room temp for 24 h, the specimen is then inverted and the other half soaked for a further 24 h.	Capillary soaking: Half the depth of each compacted specimen is soaked in water at room temp (28 ± 1°C) for 24 h, the specimen is then inverted and the other half soaked for a further 24 h.

6.1.5 The Nynas Test Procedures

These tests are applied only on the loose mixture during the second stage of the cold mix process, i.e. during storage or before laying (Chapter III, Section 3.2.4.2.b). These tests are the runoff test, washoff test, and workability test.

The runoff test is carried out by pouring 500g of loose bitumen emulsion mixture into a funnel with a mesh (< 2 mm) at the bottom to prevent the mixture falling through. The quantity of material which runs-off in 30 minutes is collected and weighed, and the quantity of bitumen that has run off is calculated.

The washoff test is carried out immediately following the Runoff Test. In the washoff Test, 200ml of water is poured over the mixture in the funnel. The wash water and any washed-off bitumen is collected and measured. The test result is expected to give zero % bitumen that runs-off or washes-off. The equipment is shown schematically in Fig. 6.4

The Workability Test is performed using the Nynas workability tester presented in Fig. 6.5. The test is carried out by scraping the top few mm of an uncompacted cold bituminous mixture during storage or just before laying and the maximum force required to shear the top of the mixture apart is measured.

Note: it was not specified by the Nynas test protocol precisely how deep the scraping action should be carried out into the loose mixture. An example of test result on a 16mm maximum nominal aggregate size open graded mixture is given in Table 6.2.

Table 6.2 An example of Limits for Maximum Force when using the Nynas Workability Test on loose CBEMs, with a 16mm maximum nominal aggregate size open graded mixture.

Maximum Force	Description
30 to 90 N	Soft Mixture
90 to 130 N	Slightly Viscous Mixture
130 to 170 N	Viscous Mixture
> 170 N	Stiff Mixture

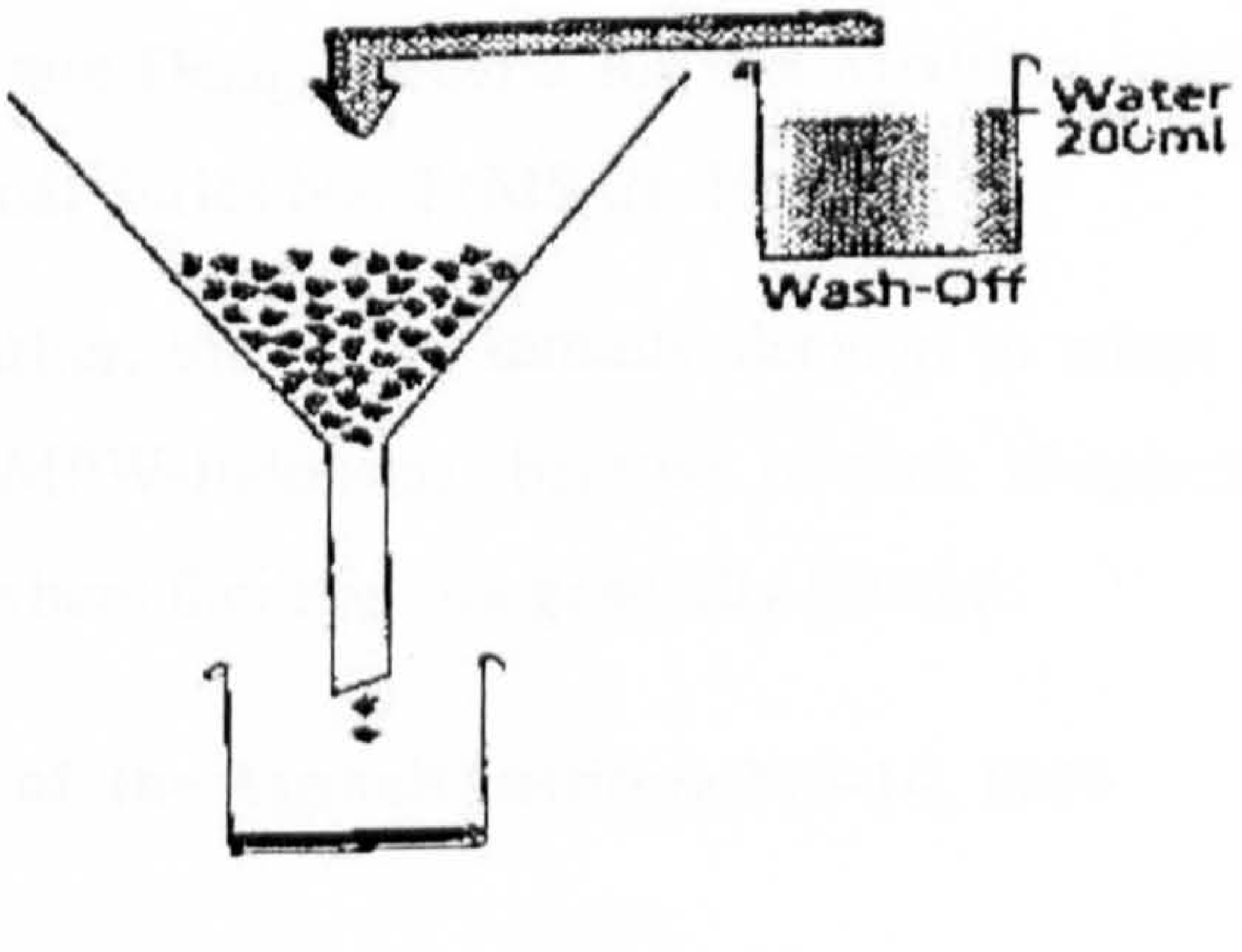


Fig. 6.4 The Runoff and Washoff Tests.

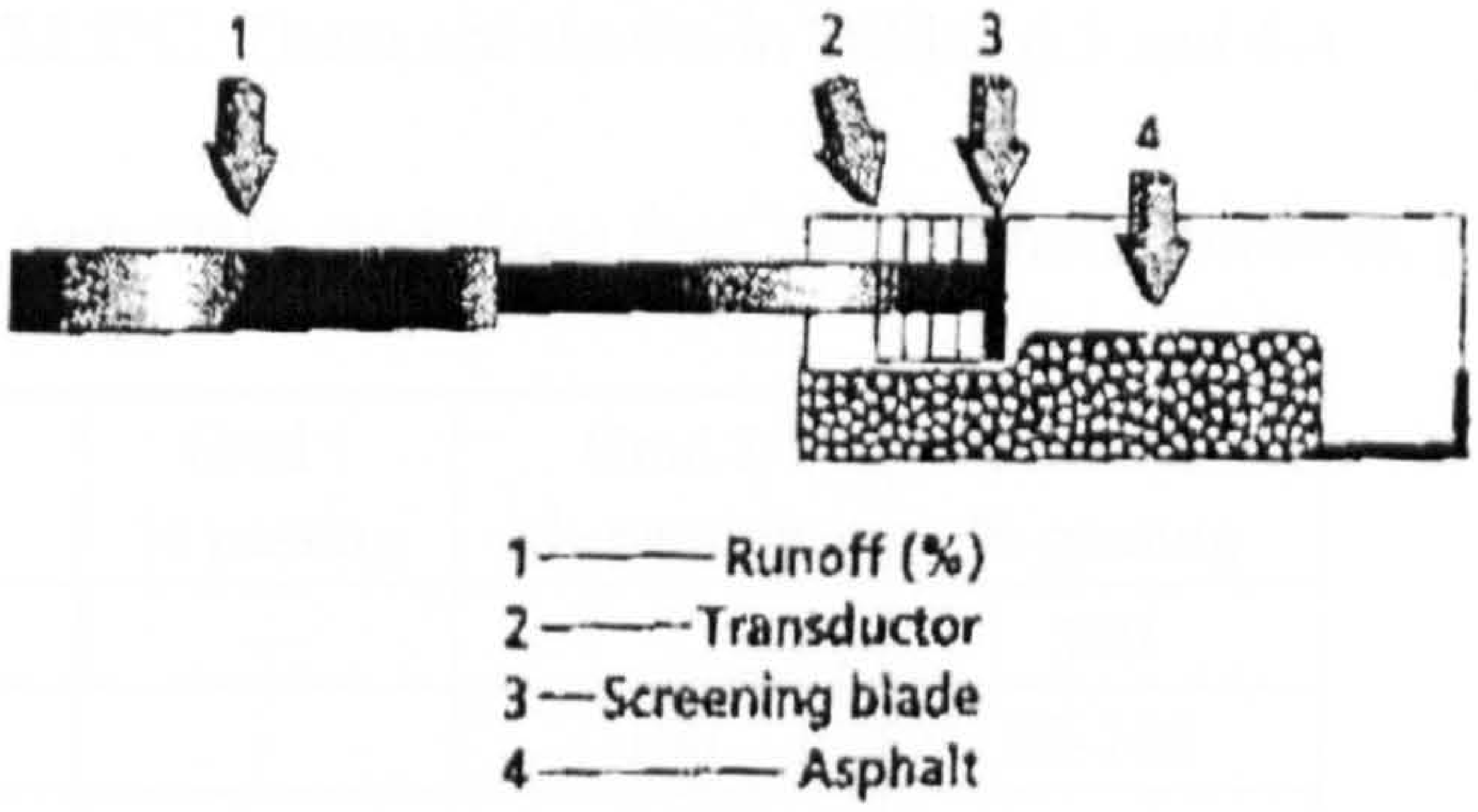


Fig. 6.5 Basic Principle of the Nynas Workability Tester.

6.2 Specifications for Cold Bituminous Emulsion Mixtures (CBEMs)

6.2.1 General

Currently there is no universally accepted specification for cold bituminous emulsion mixtures mix design or testing. Mixture design and testing procedures vary amongst the various road authorities, research institutions and asphalt researchers. Within this report five CBEMs specifications have been reviewed, namely; 1- Asphalt Institute Specifications [Asphalt Institute, 1989]; 2- [Asphalt Institute, 1997]; 3- Specification of Nikolaides [Nikolaides A.F., 1994]; 4- Ministry of Public Work Republic of Indonesia [MPW-Indonesia], 1990; 4- the HAUC Specifications U.K., 1992; and 5- Transport Road Laboratory TRL Report 386, U.K, 1999. The Asphalt Institute Marshall Mixture Design Criteria for Hot Mixtures, was used for comparison [Asphalt Institute Manual Series No. 2 (MS-2), 1998].

As mentioned earlier, the author initially decided to adopt the specification and testing procedures of MPW-Indonesia, because of their simplicity and suitability for developing countries, where facilities are generally limited.

6.2.2 Specifications of the Asphalt Institute MS-14, 1989

The MS-14 criteria is applicable for low volume traffic base course mixtures composed of dense graded mineral aggregates with maximum size of 25mm, at an ambient temperature 22.2°C. These are shown in Tables 6.3 and 6.4.

Table 6.3 Proposed aggregate gradations for Cold Asphalt Mixtures, [Asphalt institute MS-14, 1989].

Sieve Size (mm)	Grad 1 % passing	Grad 2 % passing	Grad 3 % passing
25.0 mm (1 in.)	-	-	100
19.0 mm (¾ in.)	-	100	90-100
12.5 mm (½ in.)	100	90-100	-
9.5 mm (3/8 in.)	90-100	-	60-80
4.75 mm (No.4)	60-80	45-70	35-65
2.36 mm (No.8)	35-65	25-55	20-50
0.03 mm (No.50)	6-25	5-20	3-20
0.075 mm (No.200)	2-10	2-9	2-8

Table 6.4 Emulsified Asphalt-Aggregate Mixture Design Criteria, [Asphalt institute MS-14, 1989].

Test Property	Minimum	Maximum
Soaked Stability at 22°C (72°F), kN	2.225	-
Percent Stability Loss, after vacuum saturation and immersion at 22°C (72°F)	-	50
Aggregate Coating (%)	50	-

Soaked stability is a measure of the specimen resistance to the effects of water in wet and cold climates. The percent of stability loss is calculated as shown below:

Percent of stability loss ={(dry stability-soaked stability)/dry stability}x100% (6.4)

Additionally, the Asphalt Institute *does not recommend the construction of cold bituminous emulsion mixtures when the temperature is below 10°C, or in wet weather.*

6.2.3 Specifications of the Asphalt Institute MS-19, 1997

The design criteria in MS-19, as shown in Tables 6.5 to 6.7, are applicable for either base courses or surface courses of low to medium traffic volumes with maximum aggregate size of 25mm, at ambient temperatures (25 ± 1°C or 77 ± 1.8°F).

Table 6.5 Aggregate gradations for Dense Graded Emulsion Mixtures, [Asphalt Institute MS-19, 1997].

Sieve Size	Semi-Processed Crusher, Pit or Bank Run	Processed Dense-Graded Asphalt Mixtures, Percent Passing by Mass					
50.0 mm (2 in.)	-	100	-	-	-	-
37.5 mm (1-1/2 in.)	100	90-100	100	-	-	-
25.0 mm (1 in.)	80-90	-	90-100	100	-	-
19.0 mm (¾ in.)	-	60-80	-	90-100	100	-
12.5 mm (½ in.)	-	-	60-80	-	90-100	100
9.5 mm (3/8 in.)	-	-	-	60-80	-	90-100
4.75 mm (No.4)	25-85	20-55	25-60	35-65	45-70	60-80
2.36 mm (No.8)	-	10-40	15-45	20-50	25-55	35-65
1.18 mm (No.16)	-	-	-	-	-	-
600 µm (No.30)	-	-	-	-	-	-
300 µm (No. 50)	-	2-16	3-18	3-20	5-20	6-25
150 µm (No. 100)	-	-	-	-	-	-
75 µm (No. 200)	3-15	0-5	1-7	2-8	2-9	2-10
Sand Equivalent (%)	30 min.	35 min.	35 min.	35 min.	35 min.	35 min.
Los Angeles Abrasion @ 500 revolutions (%)	-	40 max.	40 max.	40 max.	40 max.	40 max.
Percent Crushed Faces	-	65 min.	65 min.	65 min.	65 min.	65 min.

Table 6.6 Aggregates for Open Graded Emulsion Mixtures, [Asphalt Institute MS-19, 1997].

Sieve Size	Base			Open Graded Surface Course
	Coarse	Medium	Fine	
37.5 mm (1-1/2 in)	100	-	-	-
25.0 mm (1 in.)	95-100	100	-	-
19.0 mm (¾ in.)	-	90-100	-	-
12.5 mm (½ in.)	25-60	-	100	-
9.5 mm (3/8 in.)	-	20-55	85-100	100
4.75 mm (No.4)	0-10	0-10	-	30-50
2.36 mm (No.8)	0-5	0-5	0-10	5-15
1.18 mm (No.16)	-	-	0-5	-
75 µm (No.200)	0-2	0-2	0-2	0-2
Los Angeles Abrasion @ 500 revolutions (%)	40 max.	40 max.	40 max.	40 max.
Percent Crushed Faces	65 min.	65 min.	65 min.	65 min.

Table 6.7 Emulsified Asphalt-Aggregate Mixture Design Criteria, [Asphalt Institute, MS-19, 1997].

Test Property	Minimum	Maximum
Soaked Stability at 25 ±1°C (77±1.8°F), kN	2.224	-
Percent Stability Loss, after vacuum saturation and immersion at 25 ±1°C	Not available	Not available
Aggregate Coating for surface courses (%)	85	-
Aggregate Coating for surface courses (%)	60	-

6.2.4 Specification of Nikolaides, A.F., 1994.

Nikolaides (1994) introduced 9 types of gradations for cold dense graded bituminous emulsion mixtures (CDGBEM) as shown in Table 6.8 covering a wide range of aggregate sizes enabling better use of the available aggregates. The proposed mixture property requirements are shown in Table 6.9. The minimum **bitumen film thickness** (BFT) is aimed at safeguarding from premature ageing of the mixture especially for wearing courses. The formula for calculating the BFT is shown below:

$$\text{Bitumen film thickness (T)} = \frac{\%\text{Binder}}{100 - \%\text{Binder}} \times \frac{1}{\text{SGBinder}} \times \frac{1}{\text{ASA}} \qquad (6.5)$$

The aggregate surface area (ASA) is calculated using the surface area factor adopted from Appendix E, Asphalt Institute MS-14, 1989. An example calculation of ASA and BFT (T) is given in Appendix 1.8 and Appendix 1.9 respectively. For his investigation, the author closely followed Nikolaides gradation limit type V for wearing courses (WC).

Table 6.8 Gradation Limits for Cold Dense Graded Bitumen Emulsion Mixtures (CDGBEMs) - [Nikolaides, 1994].

Sieve Size (mm)	PERCENTAGE PASSING (% by weight)								
	Type I (*)	Type II	Type III	Type IV	Type V	Type VI	Type VII	Type VIII	Type IX
50.00	100	-	-	-	-	-	-	-	-
38.10	90 - 100	-	-	-	-	-	100	-	-
25.40	-	90 - 100	100	-	-	-	90 - 100	100	100
19.10	60 - 80	-	90 - 100	100	-	-	65 - 98	97 - 100	97 - 100
12.70	-	60 - 80	-	90 - 100	100	100	50 - 75	70 - 100	77 - 98
9.52	-	-	60 - 80	-	90 - 100	-	40 - 60	56 - 80	67 - 85
4.76	20 - 55	25 - 60	35 - 65	45 - 75	60 - 80	75 - 100	35 - 50	50 - 60	60 - 70
2.36	10 - 40	15 - 45	20 - 50	25 - 55	35 - 65	60 - 80	30 - 48	45 - 60	55 - 70
0.60	-	-	-	-	-	-	10 - 45	15 - 60	18 - 70
0.30	4 - 15	5 - 18	5 - 20	5 - 20	5 - 25	15 - 65	-	-	-
0.074	0 - 5	0 - 5	3 - 8	3 - 8	3 - 11	5 - 20	3 - 6	3 - 8	3 - 8
Sand Equivalent	> 45%	> 45%	> 45%	> 45%	> 45%	> 45%	> 45%	> 45%	> 45%
Plasticity Index	NP	NP	NP	NP	NP	NP	NP	NP	NP
Utilisation	BC	BC / WC	BC / WC	BC / WC	WC	WC	BC	BC / WC	WC

(*) If aggregate size > 25.4 mm , use nominal bitumen content as near optimum and adjust during construction using the criterion for proper coating

BC = Base Course
WC = Wearing Course

Table 6.9 Properties of Cold Dense Graded Bituminous Emulsion Mixtures (CDGBEMs) - [Nikolaides, 1994].

Properties of Mixturers	Limits	Cold Dense Graded Emulsion Mixtures								
		I	II	III	IV	V	VI	VII	VIII	IX
Nominal Bitumen Content (%) (by. W.t. of total mix)	-	4	4.5	5	5.5	6	6.5	6.5	6	5.5
Soaked Stability (N)	Min	1335	1335	1335	1335	1335	1110	1335	1335	1335
Retained Stability (%) (after 48 hours soaking)	Min	50	50	50	50	50	50	50	50	50
Total Void Content * (%)	Min	6	6	6	6	6	6	6	6	6
	Max	12	12	12	12	12	12	12	12	12
Water Absorption (after 48 hours soaking)	Max	4	4	4	4	4	4	4	4	4
Bitumen Film Thickness μm	Min	6	6	6	6	6	6	6	6	6
Degree of Aggregate Coating (%)	Min	85	85	85	85	85	85	85	85	85
Permanent Deform. Performance	-	(Optimum bitumen content less then max. allowable determined by Creep Test)								
Proposed layer thickness (mm)	Min	80	50	40	30	25	25	30	30	30
	Max	150 **	100 **	100 **	75 **	75 **	75 **	75 **	75 **	75 **

* Total void content = air voids + voids filled with water

** It can be changed after compaction test trials on the job site

6.2.5 Specifications of The Ministry of Public Work Republic of Indonesia, 1990, [MPW-Indonesia, 1990]

This specification uses the *modified marshall test procedure* for the design of open graded emulsion mixtures (OGEMs), and dense graded emulsion mixtures (DGEMs) in which the Marshall procedure for testing hot mixtures has been modified.

Mixture composition limits for OGEMs are given in Table 6.10. Meanwhile DGEMs (shown in Table 6.11) are divided into 6 types: Types I to V are gradations suitable for continuously graded mixtures, whilst Type VI is an emulsion sand mixture which is mainly used as a base course. Types I to V gradations are identified based on the maximum nominal aggregate size. For example, Type I/50 means the gradation is continuous with 50mm maximum nominal aggregate size. Type I is the coarsest mixture for use in sub-bases, Type II – IV are suitable for sub-base or base courses. Type V is the finest and can be used in either base or surface layers. The mixture property limits of the six types of DGEM's are given in Table 6.12.

Table 6.10 OGEMs Mixture Composition Limits, [MPW-Indonesia, 1990].

Property	Unit	Friction Course	Base Course
Grading			
25 mm	%	100	100
19 mm	passing	100	80 - 100
12.5 mm		100	-
9.5 mm		80 - 100	20 - 55
6.75 mm		10 - 40	5 - 30
2.36 mm		0 - 10	0 - 5
1.18 mm		0 - 5	-
75 microns		0 - 2	0 - 2
Nominal Layer Thickness	mm	25	-
Minimum Effective/ Residual bitumen content	% by weight of total mixture	3.9	3.3
Minimum Total Emulsion content	% by weight of total mixture	6.6	5.7

Table 6.11 DGEMs Mixture Design Fractions (guide only), [MPW-Indonesia, 1990].

Mixture Design Fractions	% Passing					
	Type I/50 Sub-Base	Type II/37.5 Sub-Base and Base	Type III/25 Base, Surface	Type IV/19 Base, Surface	Type V/12.5 Base, Surface	Type VI Base
CA Fractions (> 2.36)	60 - 85	55 - 80	50 - 75	45 - 70	40 - 60	25 - 35
FA Fractions (2.36-0.075) mm	10 - 30	13 - 35	17 - 40	20 - 40	30 - 50	60 - 65
FF (< 0.075 mm)	0 - 4.5	1 – 6.5	2 – 8	2 – 8.5	2 - 9	6 - 15

Note: CA = coarse aggregate; FA = fine aggregate; FF = filler fraction

Table 6.12 DGEMs Mixture Property Limits, at ambient temperature 28±1°C, [MPW-Indonesia, 1990].

Mixture's Property	Units	Value range	Type of DGEMs					
			I/50	II/37.5	III/25	IV/19	V/12.5	VI
Soaked Stability	kN	Min.	3	3	3	3	3	2.5
Retained Stability	%	Min.	50	50	50	50	50	50
Void Content	%	Min.-Max.	5-10	5-10	5-10	5-10	5-10	5-10
Moisture Absorption	%	Max.	4	4	4	4	4	4
Bitumen Film Thickness (BFT)	µm	Min.	8	8	8	8	8	8
Degree of Coating	%	Min.	75	75	75	75	75	75
Recommended layer thickness	mm	Min.-Max.	80-150	50-100	40-100	30-75	25-75	25-75

Note: The Retained Stability is the ratio of the Soaked Stability to the Dry Stability.

6.2.6 The Highways Authority and Utility Committee (HAUC) Specification – UK, 1992

In 1992 HAUC produced a code of practice: 'Specification for the Reinstatement of Openings in Highways', in response to government legislation 'The New Roads and Streets Works Act 1991 (NRSWA)'. Appendix 10 of the code as shown in Table 6.13, approves the use of permanent cold lay surfacing materials (PCSMs) based on elastic stiffness criteria at the end of a 2 years guarantee period. The specification does not cover a mix design procedure for the PCSMs.

In the most recent amendment, the specification requires that the **total voids** after compaction **for footpath materials is in the range 2 to 12%,** and **for carriageways 2 to 10%.** In addition the PCSMs should give a satisfactory surface profile within the 2 years guarantee period [Robinson, 1997].

Table 6.13 The Minimum Permanent Cold Lay Surfacing Materials Requirement, Appendix 10, [HAUC, 1992].

Permanent Cold-lay Surfacing Materials (PCSMs)	Minimum Property Requirement at 20°C for equivalence to:		
	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	4600	2400	900
10mm nominal size wearing-course	3800	1900	800
6mm nominal size wearing-course	2800	1400	600

6.2.7 Design Guide and Specification for Structural Maintenance by Cold in-situ Recycling, TRL Report 386, UK, 1999

This TRL Report addresses the issue of **structural maintenance by cold in-situ recycling.** The structural layers referred to in this report are the foundation or main structural layers of the road pavement **(base-courses or sub-base layers).**

Part 1 of TRL Report 386, 1999 mainly contains design guidance for cold in-situ recycling of pavements using either cement or foamed bitumen binders. Part 2 of the report sets out specification clauses and associated notes for guidance on the construction of pavement structural layers using cold in situ recycling with cement, and cold in situ recycling with foamed bitumen. The minimum amount of *cement* for cement bound layers, and the minimum amount of *residual bitumen* for bitumen bound layers *were set at 3% and 3.5%* respectively to ensure satisfactory performance in terms of coating and bonding.

The majority of cold recycling road works in the UK have traditionally used hydrated lime or Portland cement as binder. Recently, due in particular to *health and safety issues* when using *hydrated lime,* and the problem of *thermal cracking* associated with *cement bound layers*, alternative binders have become more popular, namely bitumen emulsions and foamed bitumen [TRL Report 386, Milton & Earland, 1999].

The materials in cold in situ recycling consist of the pulverized road layers, and any supplementary aggregates and/or filler. The particle size distribution of recycled materials have been divided into two zones as shown in Table 6.14.

Table 6.14 Particle size distribution of granular materials for cold in-situ recycling, [TRL Report 386 ,UK, 1999].

Sieve Size (mm)	% passing	
	Zone A	Zone B
50	100	-
37.5	94 - 100	-
20	66 - 100	100
10	48 - 75	75 - 100
5	35 - 57	57 - 95
2.36	25 - 42	42 - 77
0.6	13 - 28	28 - 52
0.3	10 - 24	24 - 45
0.075	5 - 20	20 - 35

Cement bound recycled materials are generally not highly sensitive to aggregate grading, therefore both Zones A and B can be used for cement bound recycled materials with a recommended maximum amount of material passing 0.075mm of 35%.

On the otherhand, bitumen bound recycled materials are often sensitive to aggregate grading particularly the fines component. The cold bitumen tends to mix and preferentially bond with the fines fraction and hence partially coat the coarse aggregates. For this reason, Zone A grading (Table 6.14) is recommended for bitumen bound recycled materials. Zone A grading limits the amount of particles passing 0.075mm (filler fraction) to ideally a minimum of 5% and a maximum of 20%, none of which should be composed of clay.

Finer materials may be tolerated, provided any clay fraction is modified by treatment with lime or cement to form a non cohesive component. This treatment is required when the fraction passing 0.075mm is more than 20% (Zone B material), and if the Plasticity Index of the materials passing 0.425mm is more than 6 (either Zone A or

B). TRL Report 386 proposes that cold in-situ recycled bitumen bound materials should have properties as shown in Table 6.15.

Table 6.15 Compliance criteria for process control tests on cold in-situ recycled bitumen bound material (base and sub-base courses), [TRL Report 386, UK, 1999].

Material Property or Characteristics	Individual results	Mean from test set
Relative in-situ density	93% minimum	95% minimum
Added bitumen content [BS 598:Part 102]	Target ± 0.6 %	N/A
Moisture content [BS 1377: Part 2]	Opt. ± 3%	Target ± 1%
Layer thickness [site measurement]	± 25 mm of specified	± 15 mm of specified
Cement content [site rate of spread measurement]	Target ± 2%	Target ± 1%
Indirect Tensile Modulus [ITSM] - dry specimens	2000 MPa	2500 MPa
ITSM - water saturated specimens	1500 MPa	2000 MPa
Percentage of air voids content	12% Maximum	9% maximum

Cold in-situ recycling is applicable to road classes that carry traffic loads up to 10 million 8 tonne standard axles (msa), and in circumstances where thicker layers of good quality aggregates are available, traffic loads up to 20 msa are possible. The Highway Authority and Utility Committee (HAUC) specification for reinstatement of opening in highways, 1992, categorized roads as shown in Table 6.16.

Table 6.16 Road Type Categories, [HAUC, UK, 1992].

Road Type category	Traffic Design Standard (msa)
1	More than 10 up to 30
2	More than 2.5 up to 10
3	More than 0.5 up to 2.5
4	Up to 0.5

6.3 Specifications for Hot Mixtures (for comparison)

The Asphalt Institute Marshall Mixture Design Criteria (MS-2, 1998) specifications are reproduced in Tables 6.17 and 6.18 below for the purpose of comparison between the CBEMs and Hot Bituminous Mixture specifications. The criteria for Stability and Flow of Laboratory Design Asphalt (BS 594: Part 1: 1992) is given in Table 6.19. Correction Factors for the Stability values were shown in Table 5.1, Chapter V.

A direct comparison between CBEMs and Hot Bituminous Mixture design criteria is only possible with respect to *porosity* requirements. The porosity requirements shown in Table 6.17 for hot mixes are far lower (almost half) those of CBEMs: 5- 10 % [MPW, Indonesia], or 6-12 % [Nikolaides, 1994]. Other properties are not comparable as the testing temperature, curing conditions, and design parameters between CBEMs and Hot Mixtures are somewhat different (see Tables 6.9, 6.12, and 6.17).

Table 6.17 Asphalt Institute Marshall Mixture Design Criteria for Hot Bituminous Mixtures, [Asphalt Institute, MS-2, 1998].

Description	Heavy Traffic $>10^6$ EAL*	Medium Traffic $10^4 - 10^6$ EAL*	Light Traffic $< 10^4$ EAL*
No of blows, compaction	2 × 75	2 × 50	2 × 35
Stability (minimum), kN at 60 °C	8.006	5.338	3.336
Flow (maximum), mm	2.0 - 3.5	2.0 - 4.0	2.0 - 4.5
% Air Voids, surface and base-course	3 - 5	3 - 5	3 - 5
% Void Filled with bitumen (VFB)	65 - 75	65 - 78	70 - 80

* Equivalent Axle Load (EAL)

Table 6.18 Minimum % proposed Voids in Mineral Aggregates (VMA) for Hot Bituminous Mixtures, [Asphalt Institute, MS-2, 1998].

Nominal Max. Particle Size (mm)	3% design air Voids	4% design air Voids	5% design air Voids
1.18	21.5	22.5	23.5
2.36	19.0	20.0	21.0
4.75	16.0	17.0	18.0
9.5	14.0	15.0	16.0
12.5	13.0	14.0	15.0
19.0	12.0	13.0	14.0
25.0	11.0	12.0	13.0
37.5	10.0	11.0	12.0
50.0	9.5	10.5	11.5
60.0	9.0	10.0	11.0

Table 6.19 Criteria for Stability and Flow of Laboratory Designed Asphalt (Hot Bituminous Mixtures), [BS 594: Part 1: 1992].

Traffic (Commercial Vehicles per lane per day)	Marshall stability of complete mixture (kN)	Flow (mm)
< 1500	3 to 8	≤ 5
1500 to 6000	4 to 8	≤ 5
> 6000	6 to 8 or above	≤ 7

Notes:

- Commercial vehicles are vehicles with unladen (empty) weight over 15 kN (including PSV, OGV1 and OGV2)
- PSV: Public Service Vehicle (Buses and Coaches). OGV: Other Goods Vehicle.
- OGV1 covers: 2 axle rigid, 3 axle rigid, 3 axle articulated.
- OGV2 covers: 4 axle rigid, 4 axle articulated, 5 axle or more.

CHAPTER VII

INITIAL CBEMs DESIGN PROCEDURE DEVELOPED DURING THE INVESTIGATION

7.1 General

The design procedure described in this chapter is basically a combination of three design procedures, namely the Marshall design procedure as covered by the Asphalt Institute MS-14, 1989; the Design Procedure of The Ministry of Public Work Republic of Indonesia (MPW-Indonesia), 1990; and the Design Procedure of Nikolaides A.F., 1994. The prime objective was to incorporate the best features from each of the aforementioned design methods whilst maintaining simplicity and practicality of the proposed mixture design procedure.

7.2 Types of materials used at this stage

The coarse aggregate chosen was a carboniferous limestone from Swinden Quarry, Skipton North Yorkshire – UK, with angular shape and rough texture. The fine aggregate (sand) chosen was a rough, angular *red porphyry* which is a by-product of stone crushing during the production of decorative red coloured coarse aggregates commonly used for road chipping applications. During this investigation there was about 5000 tones of this by-product material stockpiled, mainly for use as fill material and for subgrade formation. The red porphyry sand was obtained from Harden Quarry, Northumberland – UK. The filler used was *limestone filler* with a specific gravity value of 2.765 and a minimum of 65% passing 75μm. All the aforementioned aggregates were supplied by Tarmac Northern Ltd. The characteristics of the aggregates are presented in Tables 7.1.

Two types of bitumen emulsion were initially trialled, namely K3-60 (obtained from a supplier named 'Joplin Purser', Newcastle-UK), and a bitumen emulsion produced by TotalFinaElf-UK. Both emulsions were produced by processing a

Venezuelan 'heavy crude oil'. At a later stage of the investigation, another emulsion type was introduced, obtained from Nynas. The emulsion properties are given in Table 7.2.

Table 7.1 Characteristics of the Coarse Aggregates (Carboniferous Limestone) and the Fine Aggregates (Red Porphyry Sand).

Properties	Values for Coarse Aggregate	Values for Fine Aggregates
Aggregates Abrasion Value (%)	15.5	1.7
Aggregates Crushing Value (%)	25	14
Aggregates Impact Value (%)	21	14
10% Fines Value (Dry) (kN)	150	300
10% Fines Value (Wet) (kN)	140	290
Magnesium Sulfate Soundness Value (%)	94	96
Polished Stone Value (%)	34	55
Bulk Density Uncompacted (kg/m3)	1357	1450
Bulk Density Compacted (kg/m3)	1518	1760
Specific Gravity, Oven Dried (Bulk)	2.63	2.54
Specific Gravity, S.S.D.	2.65	2.59
Specific Gravity, Apparent	2.70	2.67
Water Absorption (%)	1	0.90

Source: Tarmac Northern Fellbank Birtley Co. Durham DH3 2ST, Product Data

Table 7.2 Properties of Bitumen Emulsions used.

Description	K3-60	TotalFinaElf Emulsion	Nynas Emulsion *
Type	Cationic $^{(+)}$	Cationic $^{(+)}$	Cationic $^{(+)}$
Base Bitumen	300 pen	100 pen	100 pen
Bitumen Content	60 %	62 %	60 %
SG of Base Bitumen	1.01	1.02	1.02
Reactivity	Slow setting	Slow setting	Slow setting

Sources: Joplin Purser, TotalfinaElf Bitumen, and Nynas Product Data, 2000.
* used at the later stage of the investigation (in section 7.13).

7.3 Determination of Aggregate Gradation

Materials for use as coarse aggregates were graded into the following fractions: (12.7-10mm), (10-5mm), (5-2.36mm). The fine aggregates were sieved to obtain the following fractions; (2.36–1.18mm), (1.18-0.60mm), (0.60-0.30mm), (0.30-0.075mm) in order to obtain consistent gradation control.

To maintain a degree of uniformity and to allow for direct comparison of mechanical properties, the aggregate gradations of all the bituminous mixtures studied in this investigation were restricted to wearing course (WC) gradations. The gradation was determined using the following modified Fuller's gradation curve proposed by Cooper (Cooper et. al., 1985):

$$P = \frac{(100 - F)(d^n - 0.075^n)}{D^n - 0.075^n} + F \tag{7.1}$$

where: P = % material passing sieve size d (mm), D = maximum aggregate size (mm), F = % filler, n = an exponential value that dictates the concavity of the gradation line.

The gradations designed in this investigation were also compared with the gradation recommended by Nikolaides for the same top aggregate size, i.e. gradation Type V [Nikolaides, A.F., 1994] as shown in Table 7.3 and Fig. 7.1 which also includes lines with several n values for comparison. The choice of 4% filler content, i.e. F = 4 and n = 0.45 for all mixtures was intended to keep all the gradation curves within the Nikolaides recommended gradation limits. The design gradations adopted for this investigation were found to be very close to the lower recommended Nikolaides limit. Furthermore, n = 0.45 is the exponent used in Superpave Level 1 Mixture Design recommendations for best aggregate packing.

Table 7.3 Selected Gradation with F = 4% and n = 0.45, compared to the Nikolaides gradation limits.

Sieve Size (mm)	Grad with F = 4% n = 0.45	Nikolaides Gradation Limits Mixture Type V for wearing course		
		LL	Mid	UL
12.70	100.00	100.0	100.0	100.0
10.00	89.13	90.0	95.0	100.0
5.00	63.48	60.0	70.0	80.0
2.36	43.39	35.0	50.0	65.0
1.18	30.00	28.0	41.5	55.0
0.075	4.00	3.0	7.0	11.0

Note: LL = lower limit; Mid = Mid Point; UL = upper limit

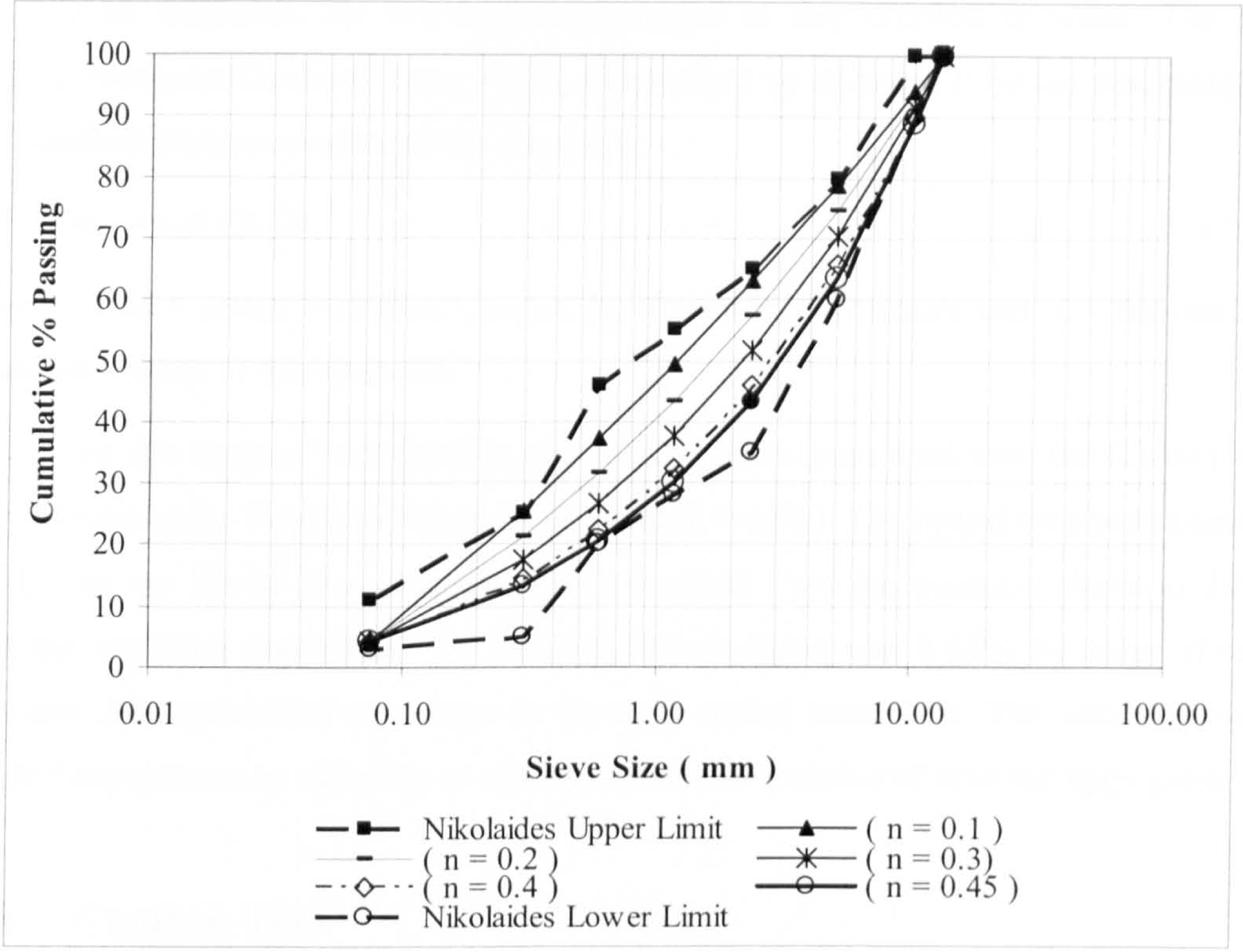


Fig. 7.1 Gradation of the Wearing Course used in this investigation (F = 4% and n = 0.45) compared to other gradations.

7.4 Estimation of Initial Residual Bitumen Content (IRBC) and Initial Emulsion Content (IEC)

The first step in the mixture design procedure was to calculate the estimated Initial Residual Bitumen Content (IRBC), designated as P, utilizing the Asphalt Institute empirical formula as shown below [Asphalt Institute, MS-14, 1989]:

$$P = (0.05\,A + 0.1\,B + 0.5\,C) \times (0.7) \tag{7.2}$$

where: P = % Initial Residual Bitumen Content by mass of total mixture, A = % of aggregate retained on sieve 2.36mm, B = % of aggregate passing sieve 2.36mm and retained on 0.075mm, C = % of aggregate passing 0.075mm.

The values A, B and C were determined based on the selected aggregate gradation as shown in Table 7.3. The values of A, B and C were found to be 56.61%, 39.39%, and 4% respectively. This results in a P value of 6.14%. Therefore the initial bitumen content requirement of the mixture was determined at approximately 6%.

In an emulsion, the bitumen is suspended as tiny droplets in water. The IEC (Initial Emulsion Content) value can be determined by dividing P by the percentage of the residual bitumen content in the emulsion:

$$\text{IEC} = (P / X)\,\% \tag{7.3}$$

where: IEC = Initial Emulsion Content by mass of total mixture and X = the residual bitumen content of the emulsion.

At this stage of investigation, the two emulsion types used were the K3-60 (with X = 60%) and the TotalFinaElf emulsion (with X = 62%). The **initial emulsion content (IEC)** for the **K3-60** type emulsion was calculated from the equation above as **10%**, and the emulsion content for the TotalFinaElf emulsion was **9.68%** by mass of total mixture. The emulsions were kept in air tight sealed containers. The emulsion were made homogenous by agitating or stirring before it is introduced onto the aggregates.

7.5 Coating Test

This test is also known as the '**binder compatibility test**'. The test indicates the degree of affinity and compatibility between the emulsion and the mineral aggregates and provides a means for selecting the most suitable emulsion type according to the aggregate type being used. The test was carried out at room temperature 24°C, with the following steps:

a. Aggregate Pre-wetting at Various Water Contents for Optimum Coating

Several proportioned dry graded aggregate batches 500 grams each were prepared in metallic containers /bowls for this stage of the experiment. The amount of added water to pre-wet the mineral aggregates in each batch was varied. A typical starting point would be 3% water by weight of dry aggregates, which was subsequently varied in 1% increments. A fixed amount of bitumen emulsion, i.e. *the initial emulsion content* (IEC) was then added and mixed with each of the pre-wetted aggregate samples. The pre-wetting water acts as a viscosity reducing agent that promotes workability and aids in the even distribution of any added emulsion onto the aggregate surfaces.

The amount of pre-wetting added water was subsequently adjusted if the mixture was observed to be either too stiff or too sloppy.

b. The Mixing Procedure

Several trials were carried out in order to determine the most suitable mixing procedure that would provide the best coating. All mixing trials were carried out manually in a metal bowl with the aid of a large spatula. Manual mixing was observed to be more convenient for the small quantities of materials used to manufacture the specimens than using a mechanical mixer, and satisfactory bitumen coating was achieved in all cases. In the case of the mechanical mixer, mixture segregation was noted on some occasions (finer aggregate particles remain at the bottom of the mixing bowl unevenly coated with bitumen). The total recommended mixing time was ***about 2 – 3 minutes***. Excessive mixing may also result in segregation, and possible stripping. Four mixing procedures were tested, referred to in this investigation as procedures A, B, C and D:

- **Procedure A: (CA+FA+F) + W + E**

  In this procedure, the coarse aggregate (CA), the fine aggregate (FA) and the filler (F) fractions were first dry mixed together, the pre-wetting water (W) was then added and re-mixed. After that the required amount of emulsion (E) was added and mixing continued until the mixture was evenly coated with the emulsion.

- **Procedure B: CA + W + E + (FA+F)**

  In this procedure, the coarse aggregate was first evenly wetted with water, the entire emulsion was then added and mixed. The fine aggregate and filler was introduced and mixed in at the end.

- **Procedure C: CA + ½ W + E + ½ W + (FA+F)**

  In procedure C, the coarse aggregate was first wetted with half the required amount of pre-wetting water, after that the entire emulsion content (IEC) was added and mixed in. The remainder amount of water was added next and mixed, finally followed by the fine aggregates and filler.

- **Procedure D: (CA+½ W) + E + ((FA+F)) + ½ W)**

  In procedure D the coarse aggregate and the fine aggregate fractions and filler are pre-wetted with half the amount of pre-wetting water in separate containers. The entire amount of emulsion (IEC) was then added to the damp coarse aggregate and mixed, followed by the pre-wetted fine aggregate.

d. Selection of Mixing Procedure by observation of Degree of coating

Upon completion of the mixing stage, *the emulsified mixtures were placed on an impermeable tray* and the degree of bitumen coating was observed (for various pre-wetting contents at the same IEC). A better visual estimate was normally achieved after approximately one hour as any mixture colour changes became much clearer (darker). Depending upon the emulsion type (rate of break) longer times may be required for visual observation.

In general, ***mixing procedure A was the simplest***, but may not necessarily give the best binder coating. For different kinds of aggregate and different types of emulsions other procedures may give better workability and coating. In this investigation it was found *that sufficient workability and degree of coating* were achievable with mixing procedure A and since it was the easiest procedure, it was adopted for this part of the investigation.

e. Determination of Optimum Pre-Wetting Water Content for Best Coating (OPWwc)

The degree of coating was ascertained visually to determine the optimum pre-wetting moisture content that would result in all the surfaces of the mineral aggregate particles being fully and evenly coated with (black/dark colour) bitumen. If for the same added emulsion content, the required degree of coating was achieved by more than one variation of pre-wetting water content, the mixture ***with the lowest pre-wetting water content was selected.*** This pre-wetting water content was referred to as the **optimum pre-wetting water content for best coating (OPWwc)** in which *the mixture was neither too sloppy nor too stiff.*

Whilst keeping the IEC content constant, the pre-wetting water was varied in increments/decrements of 1%. The pre-wetting water and hence the **(OPWwc)** was proportioned by mass of dry aggregate. The **Total Liquid Content (TLC) was thus defined** as the percentage of **(pre-wetting water + emulsion)** by mass of total mixture during the mixing stage.

When the required degree of coating could not be achieved by varying the pre-wetting water content, the amount of emulsion was increased by 10% and the mixing procedure repeated. It is possible in some cases, that the required degree of coating is not achieved even after repeatedly increasing the pre-wetting water and emulsion contents. In such a case, the emulsion type may not be suitable or compatible with the aggregate [MPW-Indonesia, 1990- Appendix 6.10.B, Coating Test].

Figure 7.2 illustrates the influence of pre-wetting water content on the bitumen coating ability of the emulsion. In this example, aggregate batch 4 contains the minimum required pre-wetting water content for adequate binder coating.

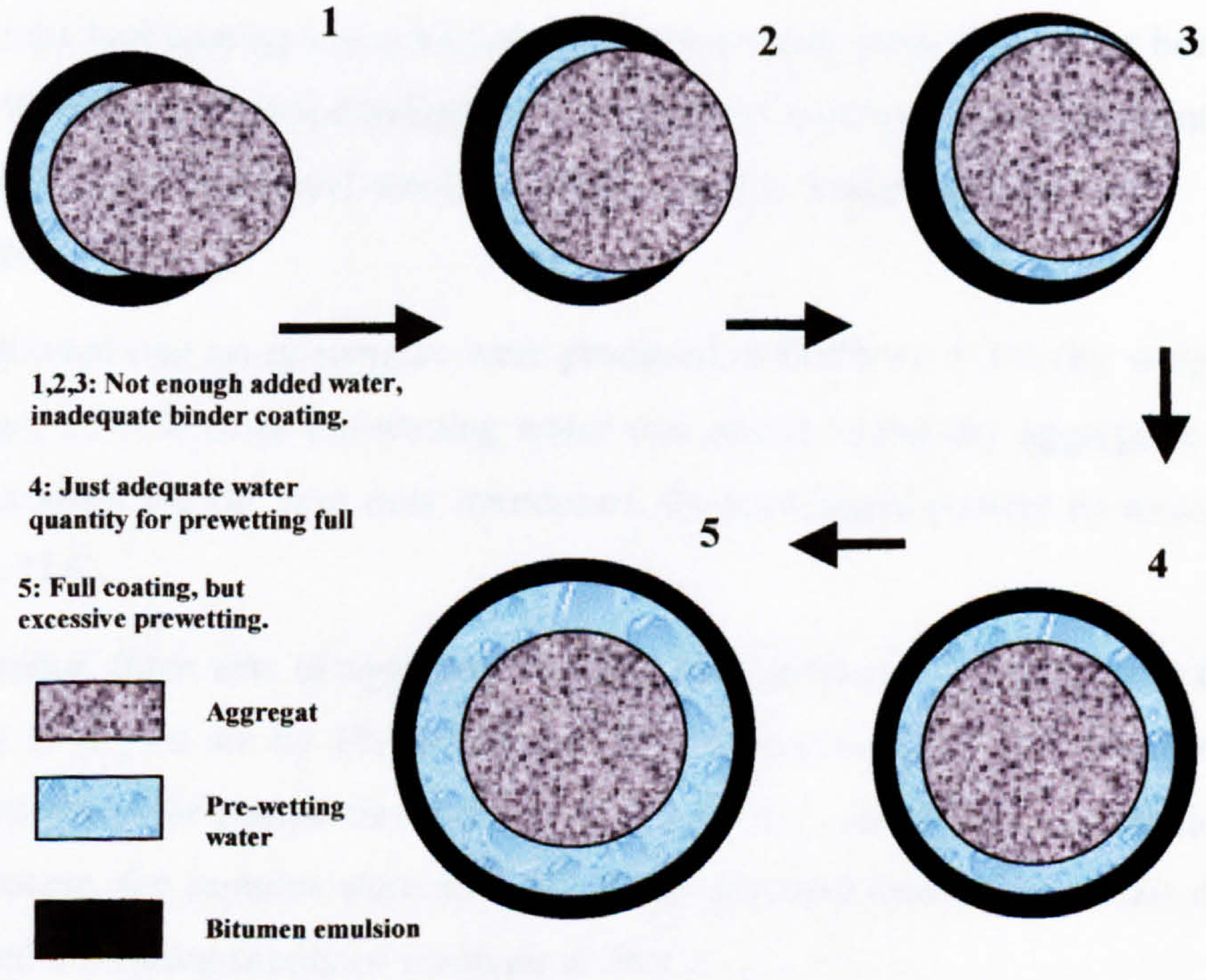


Fig. 7.2 Illustration of the influence of the pre-wetting water content on the bitumen coating ability of the emulsions.

7.6 Determination of Optimum Total Liquid Content (OTLC) at Compaction

The OTLC at compaction was determined using the following procedure:

a. Preparation

Approximately 1200 grams of proportioned mixed dry aggregates were prepared for each sample.

b. Production of loose Mixture

The aggregates were pre-wetted with water **at optimum pre-wetting water content for best coating (OPWwc),** and mixed according to the mixing procedure for best coating, i.e. by adopting Mixing Procedure A.

c. Compaction at a range of Total Liquid Contents (TLC)

The **total liquid content** (TLC = pre-wetting water content + emulsion content) influences the workability, packing and volumetric properties of the compacted bituminous specimens. The **optimum total liquid content** for compaction **(OTLC)** was thus determined by mixing and compacting the bituminous mixtures at a range of *total liquid contents* **(TLC).**

Since the best coating was achieved at the pre-wetting water content for best coating **(OPWwc)** as determined in Section 7.5e, this was used as the starting point together with the estimated initial emulsion content **(IEC). It shall be noted that: (OPWwc + IEC = TLC_0).**

An additional *one set of samples* were produced at **OPWwc** + 1% (by weight of dry aggregate), i.e. 1% more pre-wetting water was added to the dry aggregates than optimum, and when the IEC was next introduced, the total liquid content by mass was referred to as TLC_1.

Furthermore, three sets of aggregate batches were prepared at **TLC_0** , but these were allowed to dry in air by 1% increments prior to compaction. Thus these three mixtures when ready for compaction were at **TLC_{-1}, TLC_{-2},** and **TLC_{-3}.** To accelerate the drying process, the samples were air dried to the required mass using a hair dryer which provided a constant supply of warm air at 38°C.

The compaction procedure for all mixtures, i.e. mixtures at **TLC_{-3}, TLC_{-2}, TLC_{-1}, TLC_0,** and **TLC_1** was carried out in accordance with the protocol detailed below:

d. Compaction

Preparation

The compaction moulds all had 100mm internal diameter by 150mm height. Each mould included a base plate and two wearing plates. A sufficient amount of mixture (about 1200 grams) was placed into each compaction mould to give a compacted sample with height close to the standard Marshall sample height (i.e. 63.5mm). Prior to gyratory compaction, the mixture was evenly distributed in the mould and was tamped using a steel rod (dia. 12mm) 15 times around the perimeter and 10 times around the centre.

Compaction Process

Compaction was carried out using a 'Gyropac' kneading compactor (Fig.7.3a) with the following compaction effort settings; Vertical pressure = 240 kPa for 100mm diameter samples, or 540 kPa for 150mm diameter samples, angle of gyration = 2° for 100mm diameter samples, or 3° for 150mm diameter samples (Fig. 7.3b).

In this study the compaction was carried out at two levels, i.e.:

Medium compaction effort: 80 revolutions, equivalent to 50 Marshall blows each end.

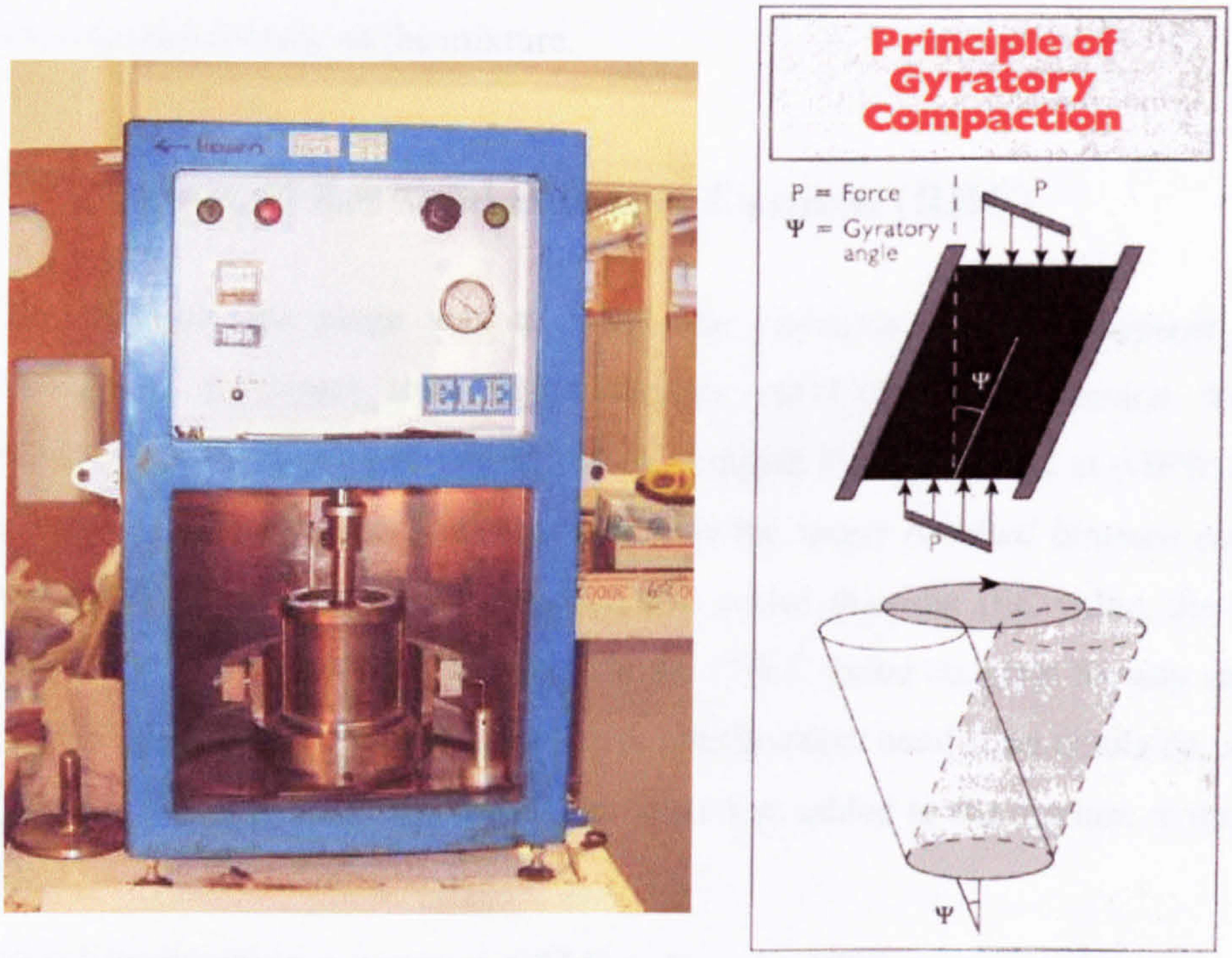
Heavy compaction effort: 120 revolutions, equivalent to 75 Marshall blows each end.

Excess water generated during and following compaction was removed with the aid of paper towels.

Curing Compacted Specimens

Unlike hot bituminous mixtures, freshly compacted CBEM mixtures lack adequate cohesion and must remain confined. The specimens hence cannot be removed from their compaction moulds without some form of curing. The moulds containing the compacted CBEMs were positioned vertically on a flat surface in which case, care was taken to ascertain that each specimen was fully supported in the compaction mould by a bottom steel base plate and was hence not suspended (hanging) by friction with the inner sides of the moulds. The upper steel wearing plate was not removed during this stage. Alternatively the compacted samples in their moulds could be laid on their sides. Both curing positions give equal opportunity for evaporation of the excess water from the top or bottom ends of the samples.

Samples were left in their compaction moulds to cure at room temperature (approximately 24°C) for 24 hours (prior to any additional oven curing). Following 24 hours of room temperature curing, the specimens were extruded from their compaction moulds and in general were found to be stiff enough for careful manual handling and volumetric measurements.



a. Gyropac Compactor. **b.** Angle of Gyration.

Fig. 7.3 Gyropac Compactor and Angle of Gyration.

e. The Optimum Total Liquid Content (OTLC) at Compaction

Following 24 hour room temperature curing, the specimens were extruded from the moulds, and the mass in air and submerged in water were measured. The saturated surface dry mass (towel dried after weighing in water) was also measured. The volumetric parameters were used for determination of the *wet density* (D) of the specimens. The *water content* of each specimen was then determined by breaking up and oven drying about 500 grams of the specimen, thus enabling the calculation of *dry density* using the following formula (see also Appendix 1.3).

$$D_d = \frac{(100 + \mathrm{RBC})}{(100 + \mathrm{RBC} + w)} \times D \tag{7.4}$$

where: D_d = dry compacted bulk density, RBC = residual bitumen content, w = water content at time of testing, D = bulk density (at time of testing).

Major assumption:

The main assumption in this part of the work was that at the optimum total liquid content (OTLC), varying the % of emulsion in the TLC did not affect the workability and hence compactability of the mixture.

7.7 Variation of Residual Bitumen Content (RBC)

The idea at this stage was to **vary the residual bitumen content** whilst maintaining the Optimum total liquid content (**OTLC**) at compaction for best workability. In order to achieve this objective, samples were produced at (OPWwc) but with a range of emulsion contents (according to the target *residual bitumen content*). The problem was that when more emulsion was added than the IEC value, the OTLC value was automatically altered. To maintain the OTLC value constant for any value of added emulsion, the mixtures had to be dried. On the other hand if an emulsion content lower than the IEC was required, then more water was added to the mixture to maintain the OTLC.

Whilst maintaining a constant OTLC value, the RBC was varied at two points above and two points below the 6 % initial RBC in steps of 0.50 %. Specimens were manufactured and tested at each of these RBC values.

7.8 Design Curing

The curing procedure adopted at this stage of investigation was referred to as ***Design Curing***. The procedure was used to assist in determination of the mechanical and volumetric properties during the mixture design procedure to assess the performance of the specimens.

When determining the optimum residual bitumen content, design curing was carried out in two stages:

- **Design Curing Stage A;** Oven Curing Compacted Samples for Dry Stability Test:

This conditioning procedure consisted of keeping the newly compacted samples for *1 day in their compaction moulds.* The samples were then extruded and kept for *1 more day in an oven at 40°C.* They were then removed from the oven and stored for 1 day at room temperature (24°C). Some of the samples were subsequently tested for Marshall Stability at room temperature and the results obtained were referred to as (dry stability values).

- **Design Curing Stage B;** Water Conditioning Samples for Soaked Stability Test (capillary soaking):

After having been subjected to *oven curing* as explained earlier in Design Curing Stage A, the remainder of the un-tested dry samples were water conditioned. In this procedure (also referred to as *capillary soaking*) half the thickness of each compacted specimen was soaked in water at room temperature for 24 h, the specimen was then inverted and the other half was soaked for a further 24 h. During soaking, the samples would rest on a bed of approximately 15 to 20mm coarse sand (see Fig.5.6, Chapter V). The samples were subsequently towel dried, their mass in air (before and after capillary soaking) were measured to determine the amount of water absorption and they were then tested for Marshall stability at room temperature. The Marshall stability test results obtained were thus referred to as (soaked stability values).

7.9 Calculation of Design Parameters

The parameters required for the design procedures are listed below. The procedure for calculating the volumetric properties was given earlier in Chapter V, Section 5.2.

a. Measurement and calculation (after drying the samples) of *dry bulk density* (values obtained from *design curing stage* A, taking into account the remaining water content in the samples at the time of testing),

b. Calculation of *retained stability* (ratio of Stage B soaked / Stage A cured stabilities), (a minimum of 50% is required in most specifications),

c. Calculation of *specific gravity of mixture* (S.G. Mix),

d. Calculation of *porosity* (P) or total voids (values obtained following design curing stage A),

e. Calculation of water absorption (values obtained following design curing stage B), (typically a maximum allowable value of 4% is recommended),

f. Calculation of *aggregate surface area* (ASA) and *bitumen film thickness* (BFT), typically a minimum of 8μm is desirable, (See Chapter VI, Section 6.2.4). Further detailed examples of the calculations are given in Appendices 1.8, 1.9 and 1.10 for the CBEMs Wearing Course I (WC I).

7.10 Determination of Optimum Residual Bitumen Content (ORBC)

This was achieved by optimizing the parameters listed in Section 7.9, namely the *dry bulk density* and Stage B *soaked stability* values of samples containing a range of residual bitumen contents (RBCs). The objective was to target the residual bitumen content (RBC) which produces the maximum dry density value whilst ensuring that the minimum soaked stability requirement (Table 7.6) was achieved. In addition to the soaked stability values, the specimens at ORBC were expected to meet minimum requirements for retained stability, porosity, water absorption and finally bitumen film thickness.

The parameters for the determination of *optimum residual bitumen content* (ORBC) at medium and heavy compaction are presented in Section 7.12: Figs. 7.6a to 7.6f. Each data point in the figures is based on an average of three samples. Fig. 7.7 shows the variation in the Stage A cured and Stage B cured (soaked) stability values as a function of the residual bitumen content at one level of compaction (heavy compaction level).

7.11 Full Curing

The author found that the indirect tensile stiffness modulus (ITSM) results of the samples tested were far below the ITSM Target, i.e. 2000-2500 MPa (see Chapter I). It was therefore considered essential to cure the samples with ORBC to *full curing* in order to evaluate *the ultimate strength* of the samples, in addition to the tests carried out following *design curing procedures A and B discussed earlier.* The ITSM test results of the CBEMs are given in Table 7.7.

In the *full curing procedure*, the specimens at ORBC were left in their compaction moulds for 1 day at room temperature before being extruded. The specimens were then placed in an oven at 40°C (until a constant mass was achieved). Finally the samples were left to cool down to room temperature (24°C) for 1 day. At the end of this curing procedure the specimens were tested for ITSM at 20°C. A typical curing duration required to achieve full curing at 40°C for a 100mm diameter sample was approximately 14 days for the samples compacted with *heavy compaction* effort having *porosity* values around 9%.

7.12 Test Results and Analysis of Results up to this stage of investigation

7.12.1 Optimum Pre-wetting Water Content (OPWwc) and Optimum Total Liquid Content (OTLC) at Compaction

Results of coating tests, OPWwc, and OTLC are presented in Table 7.4. Determination of OPWwc and the OTLC at compaction are illustrated in Figs. 7.4, and 7.5.

Table 7.4 Results of Coating Test, OPWwc, and OTLC.

Mixture	RBC (%)	IEC (%)	Variation Of Added Water (%) by wda*	Added Water for Best Coating - OPWwc (%)	Estimated Degree Of Coating (%)	Mixing Procedure	OTLC (%)
CBEMs using K3-60 emulsion	6	10	4, 5, 6, 7	6% by wda * (which equates to 14.3% TLC **)	± 85 ***	B	Med. Comp.: 12% Heavy Comp.: 11.2 %
CBEMs using TotalFinaElf emulsion	6	9.68	4, 5, 6 , 7	5% by wda * (which equates to 13.3% TLC **)	± 90 ***	A	Med./ Heavy Comp : 13%

* wda: weight of dry aggregates ** %TLC: % total liquid content (water +emulsion) by weight of mixture during the mixing stage. *** best coating achievable

As shown in Table 7.4, the coating tests were carried out at added water contents of 4, 5, 6 and 7%. The OPWwc for best coating was found to be 6% and 5 % for each mixture, which was used to produce the loose mixtures. Figs. 7.4 and 7.5 illustrates (see the x-axis) the TLC at compaction after the loose mixture with OPWwc were air dried and compacted at a range of TLC values.

As explained earlier in section 7.6, the optimum total liquid content (OTLC) at compaction may differ slightly from (OPWwc+IEC), and hence specimens were compacted at a range of total liquid contents (TLC).

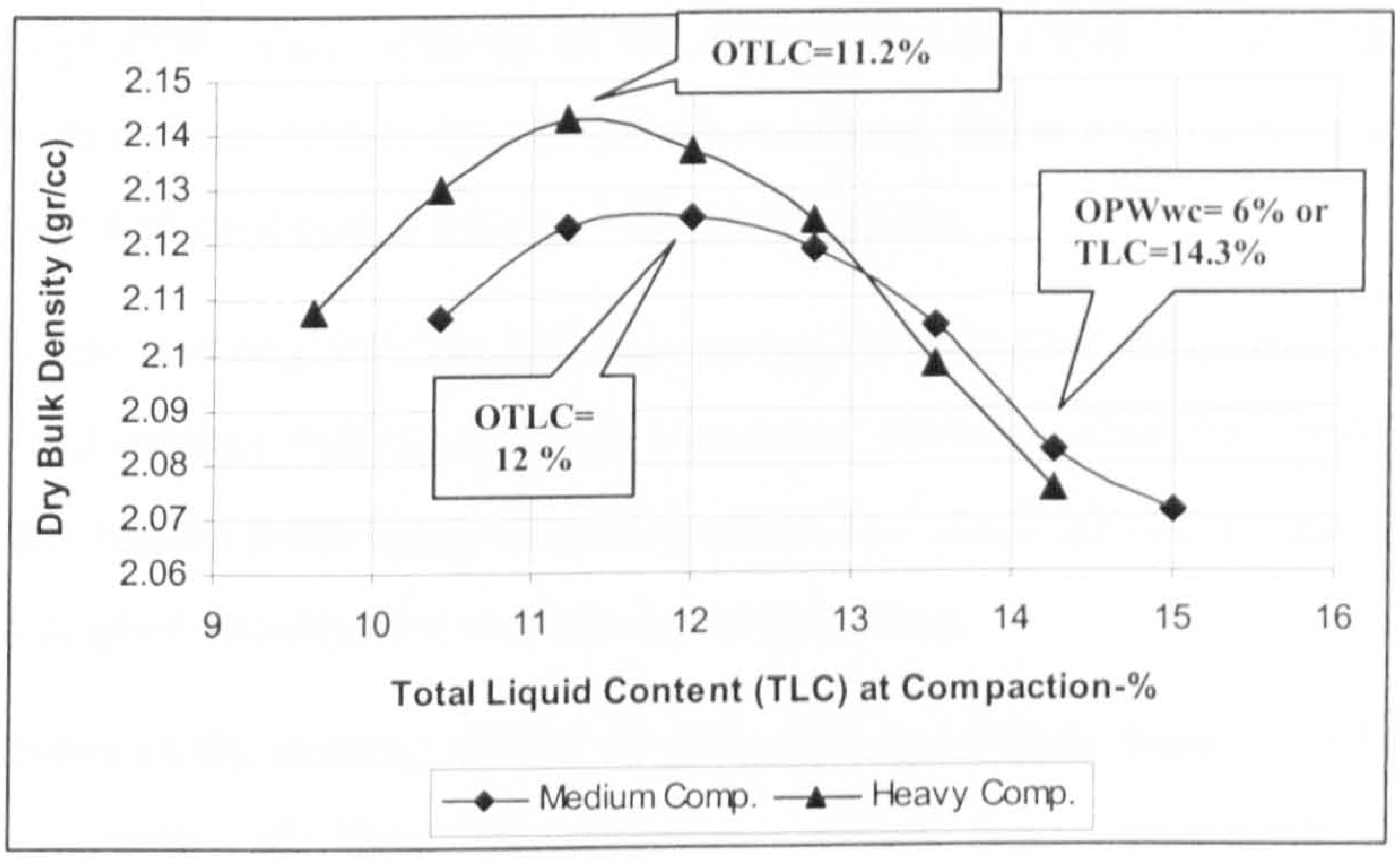


Fig. 7.4 Determination of OTLC at Compaction on CBEMs using K3-60 emulsion.

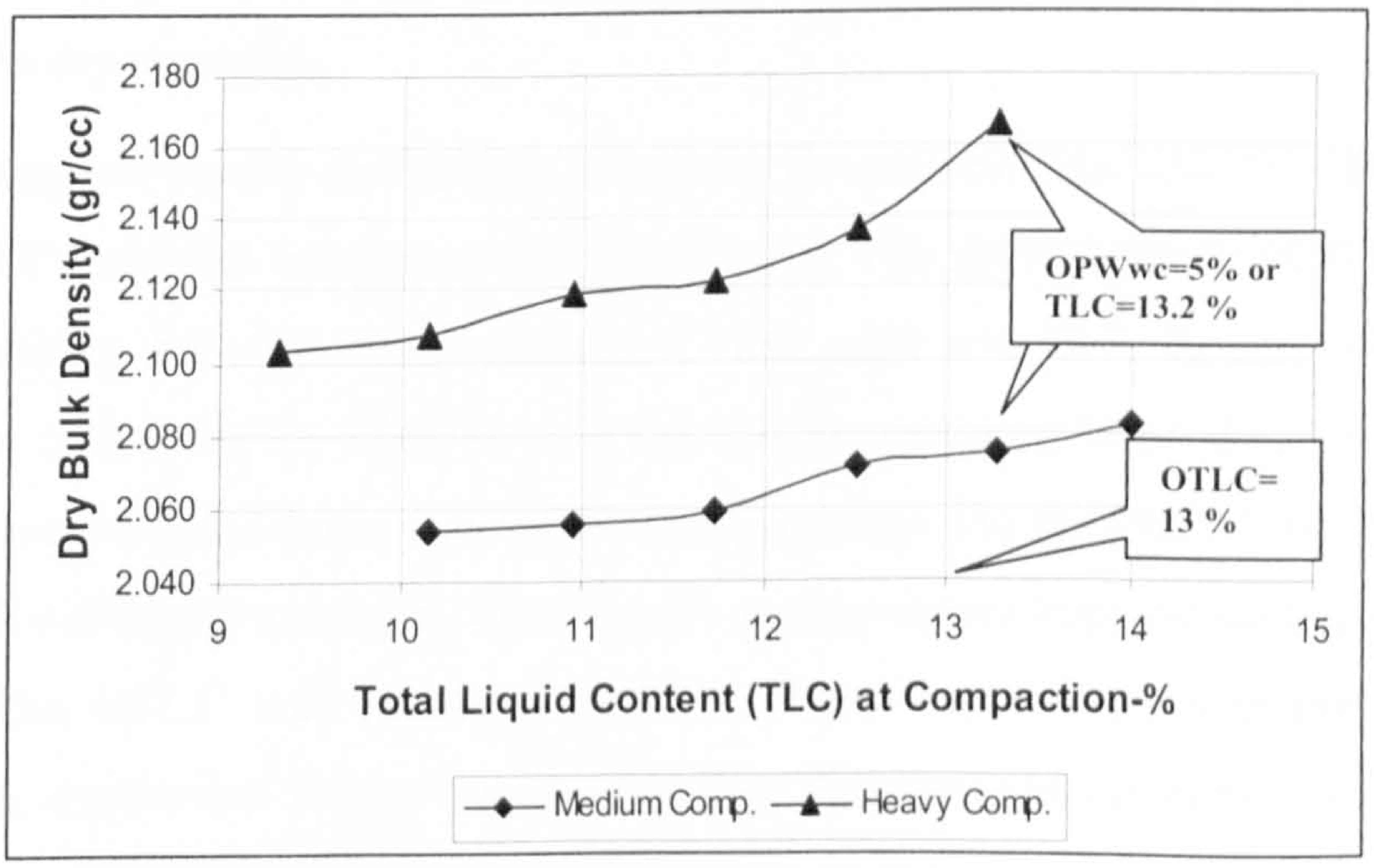


Fig. 7.5 Determination of OTLC at Compaction on CBEMs using TotalFinaElf emulsion.

An estimate of the degree of coating during the *coating test* was carried by visual observation. The 'best coating' among the mixtures with a range of added water contents was estimated and is shown in Table 7.4. It was felt that this procedure was rather subjective.

The best coating for CBEMs when using the K3-60 emulsion was obtained using *mixing procedure* B. This procedure gave better coating of the coarse aggregates as well as of the whole mixture when compared to other procedures. The simplest procedure was procedure A, however with this emulsion it did not give good coating especially on the coarser aggregates. During mixing it was observed that with all mixing procedures, the emulsion preferentially adhered to the fine particles which resulted in 'balling' of the emulsion and fines which thus required crushing. Such a mixture would therefore require a mixer that produces high crushing/shear action.

Meanwhile, coating tests on CBEMs using TotalFinaElf emulsion indicated good workability and coating with all mixing procedures. Better workability and coating were observed at the higher percentage of added water. For these mixtures, *mixing procedure A* was found to give satisfactory workability and coating.

Differences in the coating ability **of the same emulsions** were observed with time during the course of this investigation which was attributed to potential settlement/segregation of the emulsion during storage. In such cases, to maintain adequate mixture workability and coating ability, extra pre-wetting water had to be added to the dry aggregate.

Coating test results on CBEMs using the K3-60 emulsion and the CBEMs using TotalFinaElf emulsion are presented in Table 7.4. The determination of OTLC values are presented in Fig. 7.4 which showed a clear peak *dry bulk density* value at both compaction efforts for the CBEMs with the K3-60 emulsion. The OTLC was 12% and 11.2% for medium and heavy compaction respectively. On the other hand, for CBEMs using the TotalFinaElf emulsion, the *dry bulk density* values kept increasing as shown in Fig. 7.5. The OTLC was estimated at 13% where the mixture was practicable for compaction, neither too sloppy nor too dry. The difference in the density trends shown in Figs.7.4 and 7.5 may be due to the difference in formulation of the emulsions. The TotalFinaElf emulsion displayed better lubrication with the increase of water content. However, when the water content was too high, the mixture became too sloppy which was not practicable for compaction. Therefore engineering *judgment* was exercised with selecting the correct pre-wetting water content. In a separate set of experiments, OTLC test results obtained from CBEMs mixtures incorporating other types of materials are presented in Fig. 7.11, section 7.13.

7.12.2 The Optimum Residual Bitumen Content (ORBC)

Using TotalFinaElf emulsion Figs. 7.6a to 7.6f provide an illustration of the relationships between the parameters required for ORBC determination and the residual bitumen content (RBC). Similar trends were obtained for the CBEMs using the K3-60 emulsion but these have not been included in this report.

Determination of the ORBC was based on the *maximum soaked stability* and the *maximum dry bulk density (following stage A curing)*, whilst other characteristics namely *porosity, water absorption*, and *bitumen film thickness* (BFT) were expected to meet pre-set specifications. The optimum RBCs for the two emulsion types are given in Table 7.5. The properties of the mixtures at ORBC are presented in Table 7.6.

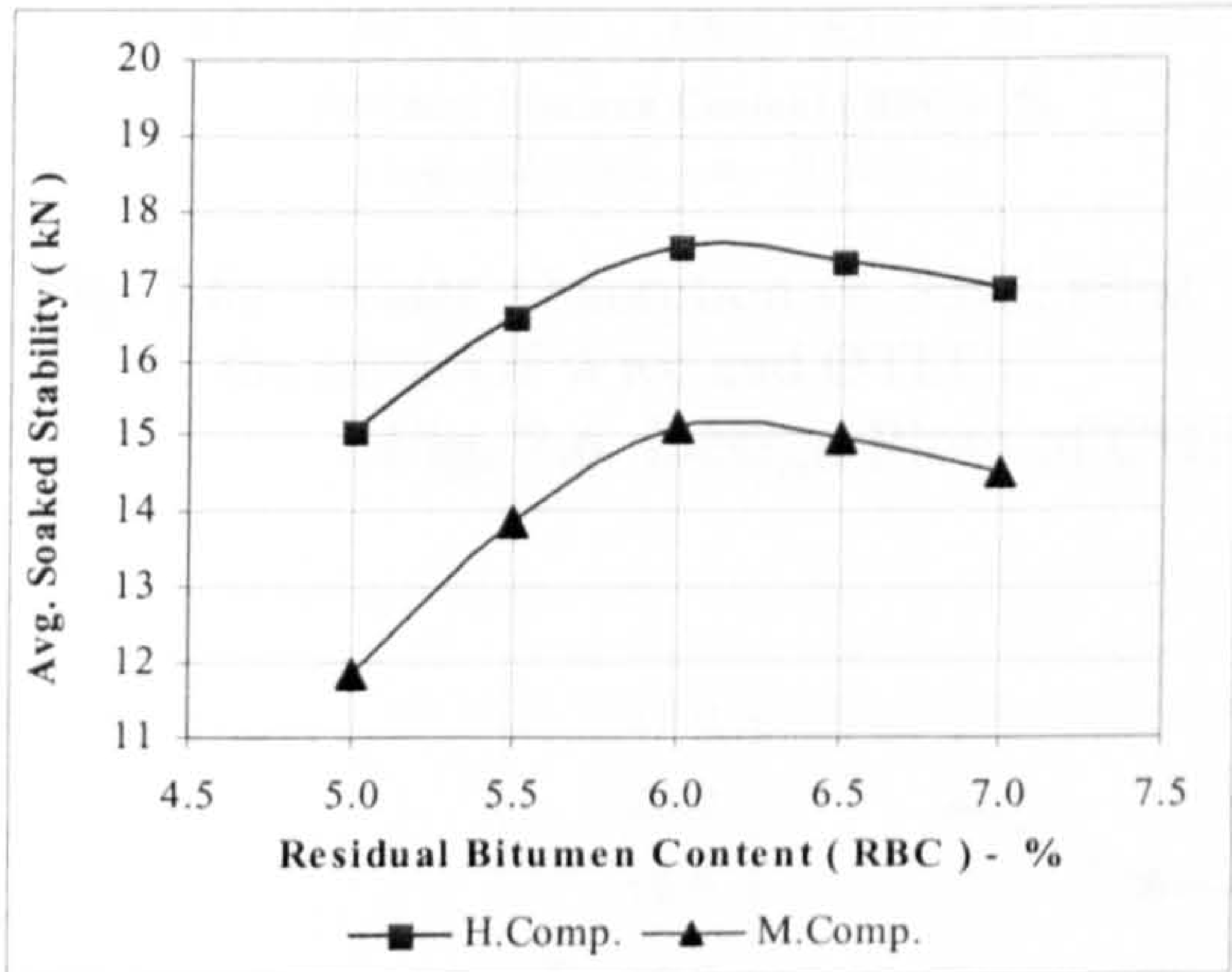


Fig. 7.6a Stage B cured (soaked) Stability vs. RBC, all at the same OPWwc and OTLC.

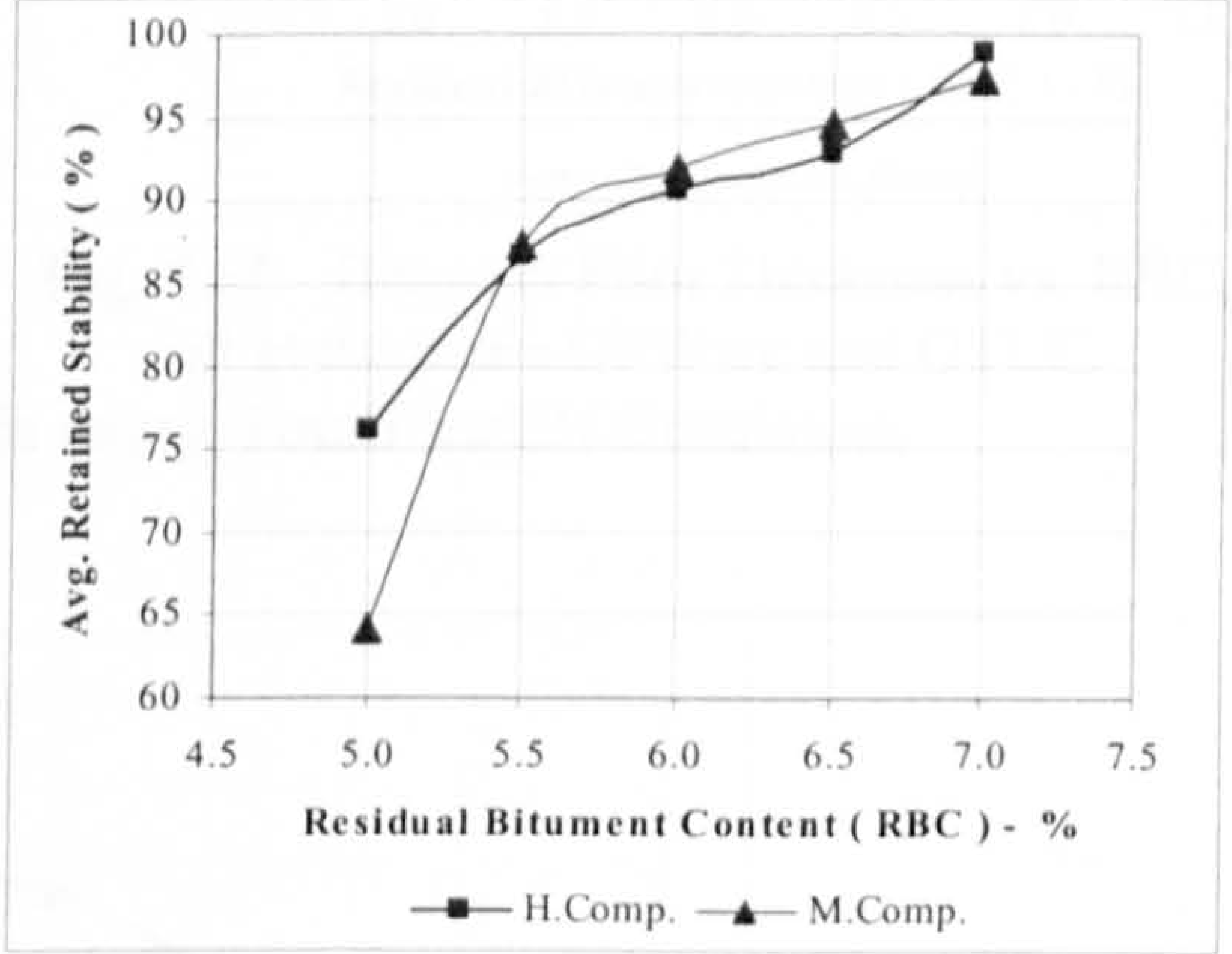


Fig. 7.6b Retained Stability vs. RBC, all at the same OPWwc and OTLC.

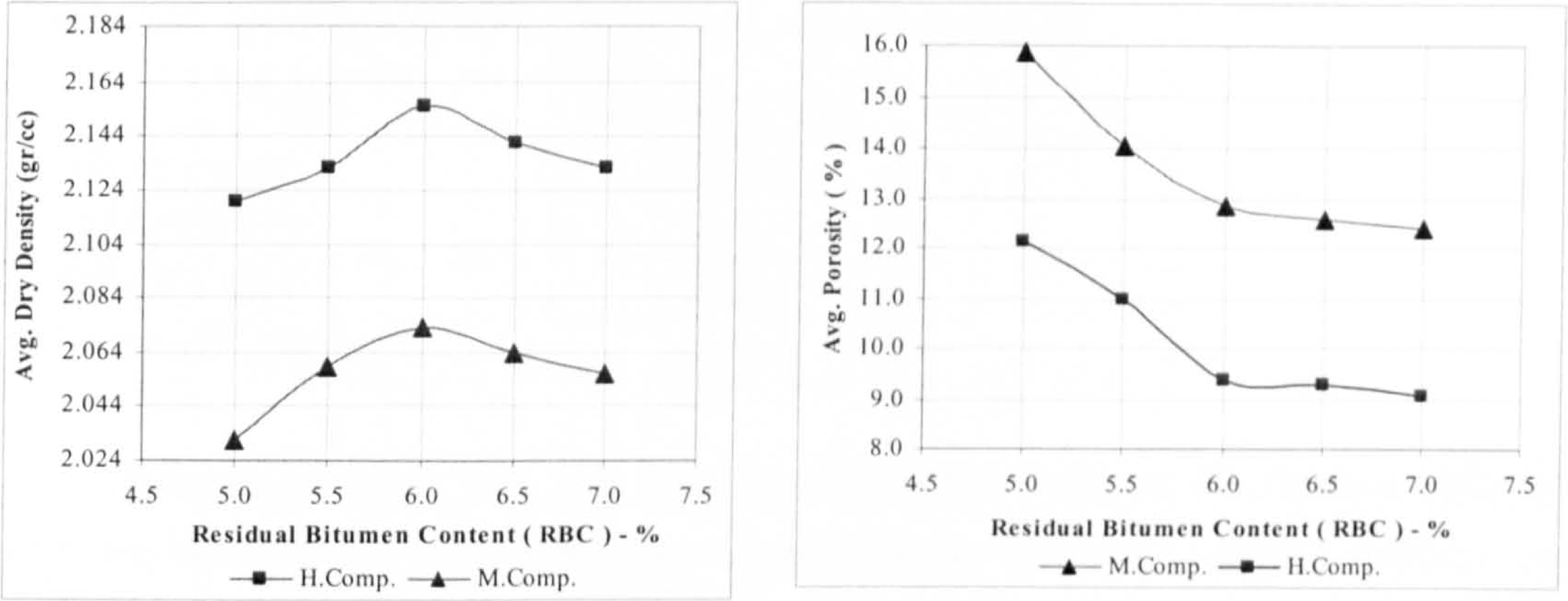


Fig. 7.6c Dry Density vs. RBC, all at the same OPWwc and OTLC.

Fig. 7.6d Porosity vs. RBC, all at the same OPWwc and OTLC.

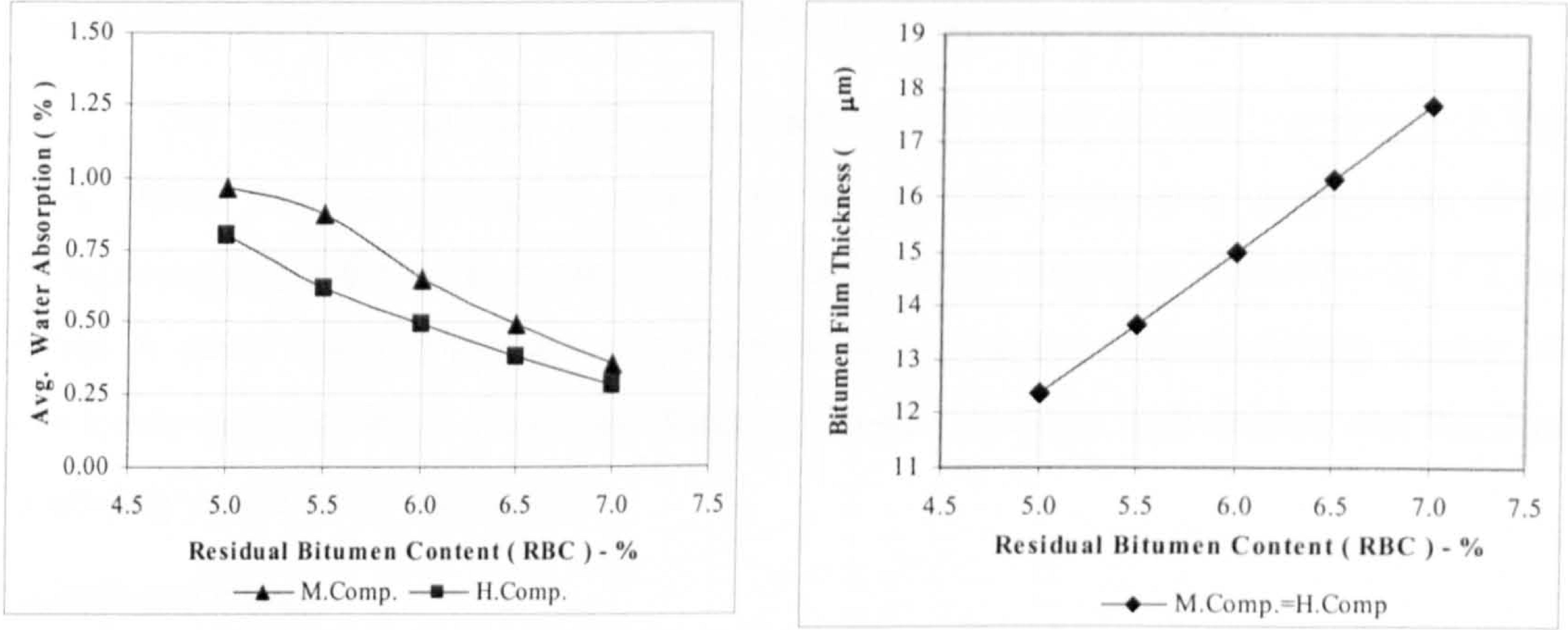


Fig. 7.6e Water Absorption vs. RBC, all at the same OPWwc and OTLC.

Fig. 7.6f Bitumen Film Thickness vs. RBC, all at the same OPWwc and OTLC.

Fig. 7.6 Design Plots of CBEMs using TotalFinaElf Emulsion.

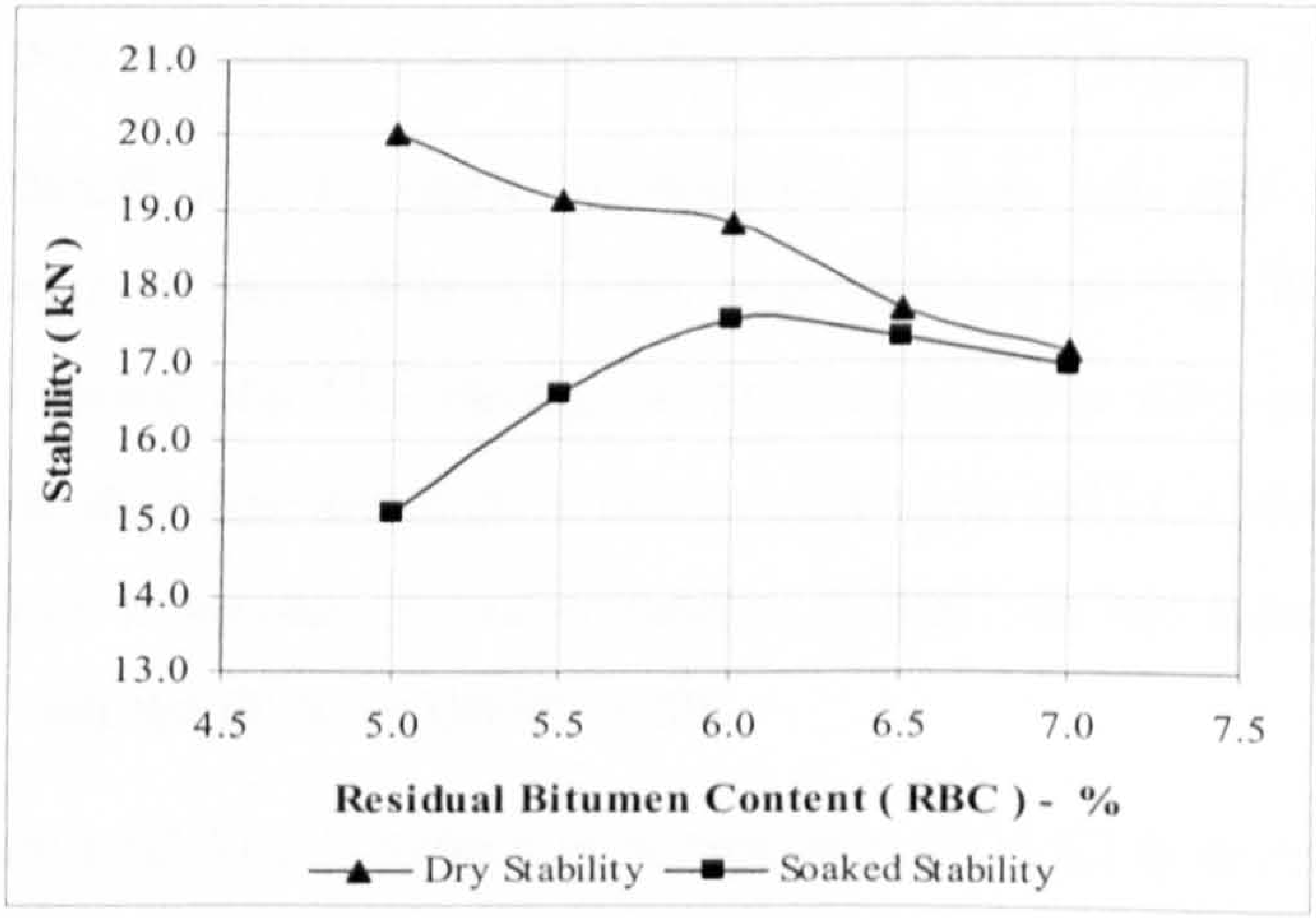


Fig. 7.7 Comparison of Stage A cured (dry) and Stage B cured (soaked) Stability values (at 24°C-room temp.), all at the same OPWwc and OTLC produced with heavy compaction.

Table 7.5 The Optimum Residual Bitumen Content (ORBC).

Mixture	Compaction level	ORBC (%)
CBEM I using K3-60 emulsion	Medium	6.5
	Heavy	6
CBEM II, using TotalFinaElf emulsion	Medium	6
	Heavy	6

Referring to Figs. 7.6a, and 7.6c, the relationship between the Soaked Stability, and dry bulk density v.s. RBC gave similar trends where the soaked stability and densities increased with increasing RBC up to an optimum RBC value beyond which stability and density decreased. It was also clear that heavy compaction gives higher *soaked stability* and *dry bulk density* results than medium compaction.

The *retained stability* values increased at all values of RBC as shown in Fig. 7.6b. There was no distinctive pattern of results with respect to compaction effort. When comparing the Stage A cured *v.s. soaked stability* values as shown in Fig. 7.7, the Stage A cured *stability* results were found to lie above the *soaked stability* results. At the lower range of RBC values the Stage A cured *stabilities* were higher and declined gradually as the RBC increased.

Additional Notes:

> Reference can be made to Appendices 1.1, 1.2, and 1.3 which describe the early stages of the design procedure, where the amount of residual bitumen was kept constant at the Initial RBC (IRBC) = 6%. Based on the IRBC, the OTLC was then determined, as mentioned at the end of Section 7.6e.
>
> Reference should also be made to Appendices 1.4, 1.5, and 1.6. The Pre-wetting water contents (OPWwc) were kept constant at 8%. When the RBC values were varied, the *TLC during mixing would hence also vary.* The loose mixtures were therefore air dried to achieve the same OTLC=14.8% by weight of total mixture in mixing, for all variations of RBC. In this case, it was just a coincidence that the IRBC = ORBC = 6%.

Since the optimum total liquid content at compaction (OTLC) in each of the compacted specimens was kept constant, and the only variable within the OTLC was the residual bitumen content (RBC) (which forms part of the TLC), it was initially assumed that the workability of all the mixtures would be similar.

Hence, there was no clear explanation to justify the shape of the dry density curves. In theory, since we have the same OTLC for all specimens, they should all compact to the same volume at the same compaction effort, and therefore should give the same dry densities when the specimens are oven dried.

One possible way to justify the dry density trends is to argue that workability is influenced by the RBC and beyond a certain value (in this case approximately 6% RBC), the increased RBC begins to manifest itself in the form of reduced mixture workability and less aggregate contact as the BFT increases (RBC has higher viscosity than the surrounding water).

Another possible explanation, is that during the 24 hour 40°C oven curing stage, at the same OTLC, more water is available for evaporation at the lower RBC's and therefore the reduced compacted specimen mass would result in reduced dry densities. This would become less of a factor as the RBC values increase, as more of the TLC would be composed of the (non-volatile) bitumen.

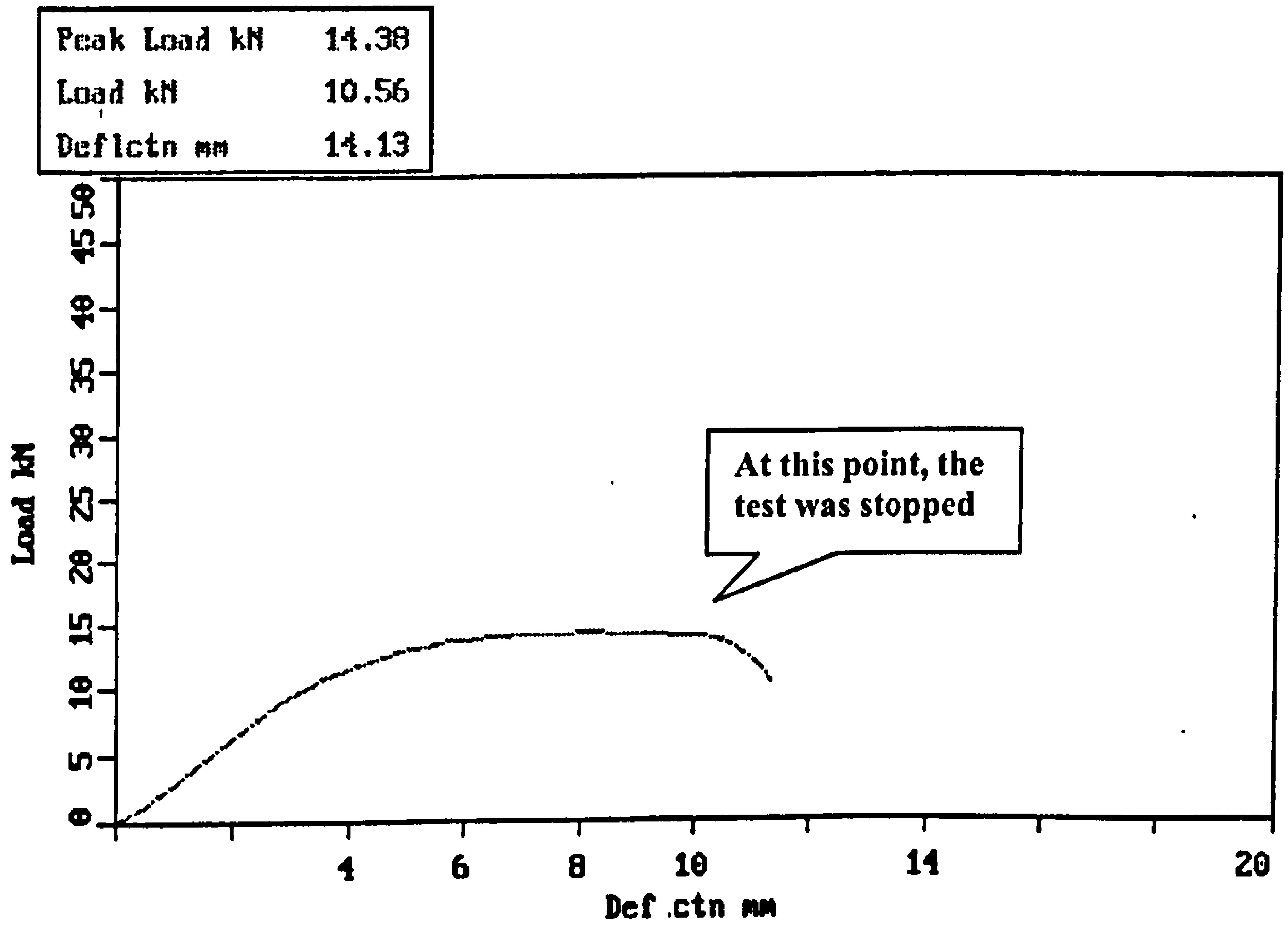
Therefore, at the optimum dry density value, there is a balance between optimum mixture workability and the resistance offered by the RBC.

Explaining the trends in Figure 7.7 proved to be even more challenging. Starting with the Stage B cured (soaked) stability trend-line, it can be expected that a specimen with less RBC (accompanied by reduced dry density) to absorb more water during the capillary soaking stage (and hence cause more damage / softening) to the remaining RBC, which should result in reduced shear strength (and hence reduced stability).

Increasing the RBC will cause a reduction in the absorbed water (increased binder film thickness) during the soaking stage and hence less damage to the binder cohesion. Nonetheless, this increase in soaked stability values can only be maintained as long as the dry density of the specimen does not reduce, in which case a less dense specimen will naturally offer lower shear resistance up to failure (i.e. reduced stability).

With respect to the Stage A cured (dry) stability trend-line, water damage is no longer an issue. In this case, due to the nature of the semi-confined Marshall stability shear test, which tends to favor mixtures with greater aggregate interlock / internal friction, a reduced RBC content (especially in this case where the bitumen is still in a relatively soft mobile state and had not had enough time to fully cure) will result in lower binder film thickness and hence greater aggregate interlock / friction. This would not be the case if the specimens were soaked and become fully saturated, whereby the trapped water would act as a lubricant during the stability test.

Deformation or flow of the CBEMs following *design curing procedure A were* carefully observed during Marshall stability tests at room temperature (24°C). It was noted that the flow of CBEMs did not give a clear peak at *maximum stability*. The flow steadily increases and then flattens off as peak strength is approached. The tests were continued up to a maximum practicable flow value dictated by the *Marshall test frame* dimensions (Fig. 7.8 and Fig. 7.9). For safety reasons the tests were terminated when the upper and the lower curved sample holding frames came to within 2-3mm. It was very clear that CBEMs possess a highly plastic mode of failure when tested in shear as demonstrated in the Marshall stability test.



At this point, the test was stopped

Fig. 7.8 Typical Trend of Stability vs. Flow of CBEMs.

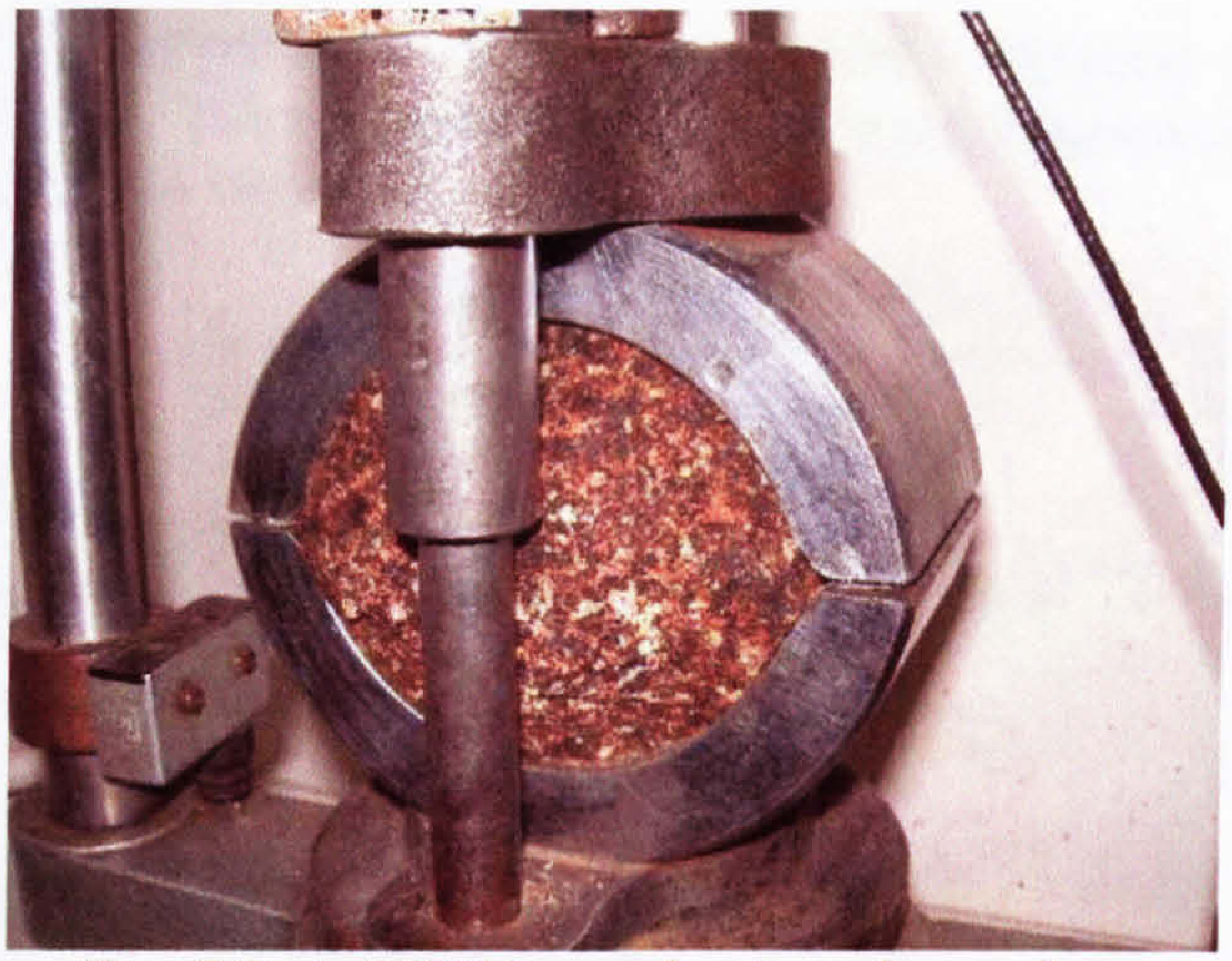


Fig. 7.9 A CBEM sample at maximum flow, showing plastic mode of failure.

The author had also observed *deformation or flow* of CBEMs which were tested *at the full curing condition* at room temperature using the Marshall testing apparatus. Some fully cured samples were also tested in accordance with the standard Marshall test protocol, i.e. after soaking in a water bath at 60°C for 30 minutes. The trend test results on stability vs. flow of the samples subjected to both treatments mentioned before were similar, i.e. showing plastic mode of failure as shown in Figs 7.8 and 7.9.

The porosity (or total voids) and consequently, water absorption values decreased as the RBC increased, in particular when specimens were manufactured using heavy compaction. As the RBC increases (at the same OTLC) as more bitumen fills the available VMA, porosity reduces and the bitumen film thickness coating the mineral aggregates increases causing a reduction in permeability and improved protection against water damage.

The porosity of the CBEMs using K3-60 meets specifications (Table 7.6). Meanwhile the porosity of the CBEMs using TotalFinaElf emulsion was higher in value at both medium and heavy compaction efforts compared to the K3-60. This is likely because the TotalFinaElf emulsion was more viscous (100 pen base bitumen) compared to the K3-60 (300 pen base bitumen), with the consequence of more resistance to compaction. With heavy compaction the CBEMs using TotalFinaElf emulsion gave total void values of less than 10% which met the porosity specifications.

Table 7.6 CBEMs Properties at Optimum Residual Bitumen Content (ORBC) subjected to *Design Curing* Condition at room temperature 24°C, compared with Specifications.

Description	Characteristics of CBEMs (Average values)					
	Soaked Stability (kN)	Retained Stability (%)	Dry Bulk Density (gr/cc)	Porosity (%)	Water Abs. (%)	BFT (μm)
CBEMs using K3-60 Emulsion (300pen)*:						
Medium Compaction Opt. RBC = 6.5 %	6.6	88.6	2.128	**9.9**	4.19	16.38
Heavy Compaction Opt. RBC = 6 %	7.1	84.3	2.141	**7.9**	3.99	15.04
CBEMs using TotalFinaElf Emulsion (100pen)*:						
Medium Compaction ORBC = 6 % (soaked sample)	15.125	92.137	2.073	**12.575**	0.647	14.98
Heavy Compaction ORBC = 6 % (soaked sample)	17.556	90.676	2.155	**9.155**	0.494	14.98
Ratio of H Comp./Medium Comp.	1.16	0.98	1.04	0.73	0.760	-
Dense Emulsified Bitumen Macadams (100pen):**						
Medium Compaction 2 × 50 blows Marshall Hammer	-	-	-	18.73	-	-
Heavy Compaction 2 × 75 blows Marshall Hammer	-	-	-	16.31	-	-
Summary of Specifications:						
The Asphalt Institute, at 22°C	2.225	50 (min)	-	-	-	-
Nikolaides, at room temp.	1.335	50 (min)	-	**6 – 12**	4 (max)	6 (min)
The MPW-RI, 1990 at room temp.	3.0	50 (min)	-	**5 – 10**	4 (max)	8 (min)
Compactor: Marshall Hammer	2 × 50 blows Marshall Blows (Medium Compaction)					

* by Thanaya A., First Year Report (2000); ** by Khalid and Eta (1999); BFT = Bitumen Film Thickness

Finally, the bitumen film thickness that coat the aggregate particles which was determined from a theoretical calculation, naturally increases in a linear manner as the RBC increases.

In general, it can be clearly seen that heavy compaction effort can improve the properties of the mixtures, especially with respect to a reduction in the Total Voids content.

Evaluation of the sample's mechanical properties were also carried out using the indirect tensile stiffness (ITSM) test and the test data are given in Table 7.7. The ITSM tests could only be performed on the CBEMs made with TotalFinaElf emulsion. Even at the lowest applied force with the MATTA, the samples with the K3-60 gave *total recoverable horizontal strains* that exceeded the range detectable by the linear voltage differential transformers (LVDTs), which resulted in *error readings,* which meant that the samples were still too weak. At full curing, ITSM tests carried out on CBEMs using K3-60 Emulsion compacted with heavy compaction produced a stiffness value of 554 MPa. This is much lower than the ITSM values of the CBEM samples using TotalFinaElf Emulsion, which was caused by the lower penetration grade of the base bitumen.

Table 7.7 Summary of Indirect Tensile Stiffness Modulus (ITSM) test results of the CBEMs compacted with Heavy Compaction Effort.

Mixture Type	**ITSM (MPa)**	
	Medium Comp.	Heavy Comp.
CBEMs using K3-60 emulsion:		
After design curing stage A	*	*
After full curing	*	554
CBEMs using TotalFinaElf emulsion:		
After design curing stage A	475	543
After full curing	1045	1142

Note; Target ITSM values: 2000 – 2500 MPa.
* the samples gave error results due to large horizontal deformations (the samples were too weak).

The strength of the mixtures subjected to *design curing procedure B* in terms of *Marshall soaked stability* were well above the requirements set in Table 7.6. Therefore, of greater concern was the porosity or total voids of the compacted mixtures. Similar findings were obtained from results of an earlier study carried out at Liverpool University – UK [Khalid H.A. & Eta K.E., 1996], where *dense emulsified bitumen macadams* with 100 pen base bitumen were compacted using a *Marshall hammer* with 50 blows (medium compaction) and 75 blows (heavy compaction). The porosity values obtained were 18.7% and 16.3% respectively and it was hence concluded that CBEMs require heavier compaction effort to meet the porosity targets. This is due to the inherent resistance to compaction of the loose mixture. The bitumen emulsion appears to quickly absorb the fine particles causing partial set during the mixing stage.

Furthermore, air drying the loose mixture results in additional stiffening or hardening, hence the requirement for heavier compaction. In this investigation, increasing the compaction effort from medium to heavy compaction reduced the porosity of the compacted mixture from 12.5% to 9.1%, which fell within the specifications set in Table 7.6.

Although, the soaked stability minimum requirement was satisfactorily met, the ITSM value however was still far below the target, even at full curing condition as shown in Table 7.7.

7.13 Additional Experiments

Further experimentation was considered necessary as some new materials were introduced at this stage of the investigation. The performance of the mixtures were found to be markedly different at most stages of the design procedure when using different materials, in particular when conducting the 'coating test'.

7.13.1 Materials used

Further trials were carried out using a new emulsion (Nynas Emulsion), produced by Nynas-UK. Two waste materials were incorporated at this stage of the investigation, namely *pulverised fuel ash (PFA)* or fly ash, and *steel slag*. The PFA was obtained from Eggborough Power Station, courtesy of National Ash, National Power, UK. The steel slag was obtained from Teeside, Middlesbrough, Cleveland, UK courtesy of Tarmac Northern Ltd. The properties of the Nynas emulsion was given earlier in Table 7.2. The properties of the PFA were presented in Chapter IV: Fig. 4.1 and Tables 4.6, 4.7, and 4.8. The steel slag properties were shown in Chapter IV, Table 4.17.

The steel slag filler was obtained by sieving the steel slag sandy component (referred to as dust) which contained about 5.4% particles passing 75μm. This material was used as a filler substitute (Table 7.8) and was tested for specific gravity using a helium pycnometer which gave a specific gravity value of 3.2.

The gradation was based on a modified Fuller's maximum packing curve (F = 4%, n = 0.45%). For these trials the CBEMs were designated as shown in Table 7.8.

Table 7.8 Mixture Designations.

Mixture Designation.	Coarse aggregate type	Fine aggregate type	Filler	Maximum nominal aggregate size (mm)
WC I	limestone	red porphyry sand	PFA	12.70
WC II	limestone	steel slag dust	PFA	12.70
WC III	steel slag	steel slag dust	50% PFA + 50% steel slag	14.00

7.13.2 Design Procedure Adopted

In the following set of experiments, the initial design procedure that had been described earlier in sections 7.3 to 7.10, was adopted whilst using *heavy compaction effort* (Gyropac, 120 revolutions, 240 kPa axial pressure) instead of *medium compaction effort*. This was found to give satisfactory porosity values. All the *samples at ORBC were brought to full curing.*

7.13.3 Results

An example of a coating test result of WCI is shown in Fig. 7.10 for comparison, where the optimum pre-wetting water contents (OPWwc) was found to be 8%. The OPWwc and the optimum total liquid contents at compaction (OTLC) of all the mixtures listed in Table 7.8 are presented in Fig. 7.11. The parameters for determining the optimum residual bitumen content are given in Figs. 7.12a to 7.12f, and Figs. 7.13a to 7.13c. The main properties of the mixtures at ORBC based on *design curing* are shown in Table 7.9.

Table 7.9 Properties of the CBEMs WC I, WC II, and WC III at ORBC.

Property	WC I	WC II	WC III
ORBC (%)	6.0	5.5	5.5
Soaked stability (kN)*	22.4	24.9	22.5
Retained stability (%)*	91.3	91.4	86.8
Porosity (%) *	11.3	12.6	15.8
Water absorption (%) *	0.96	0.97	0.97
BFT (μm) *	14.8	13.5	13.8
ITSM at full curing (MPa)	1095.4	**	**

* based on *design curing stages A and B*; ** samples developed cracks after 7 days during the full curing process.

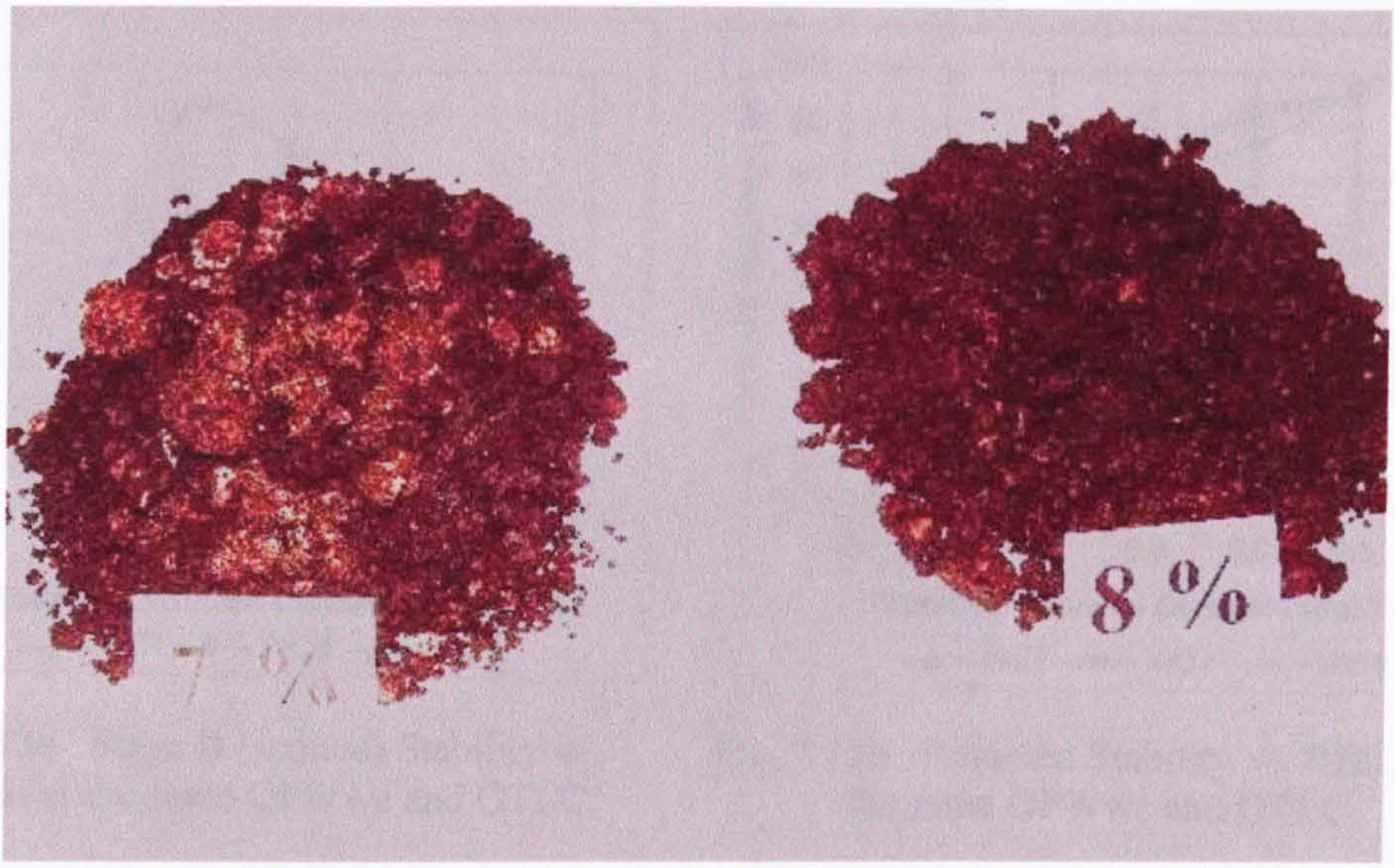


Fig. 7.10 Degree of Coating Test Result of WC I using a previously supplied (1st supply) emulsion with 7% and 8% pre-wetting water content.

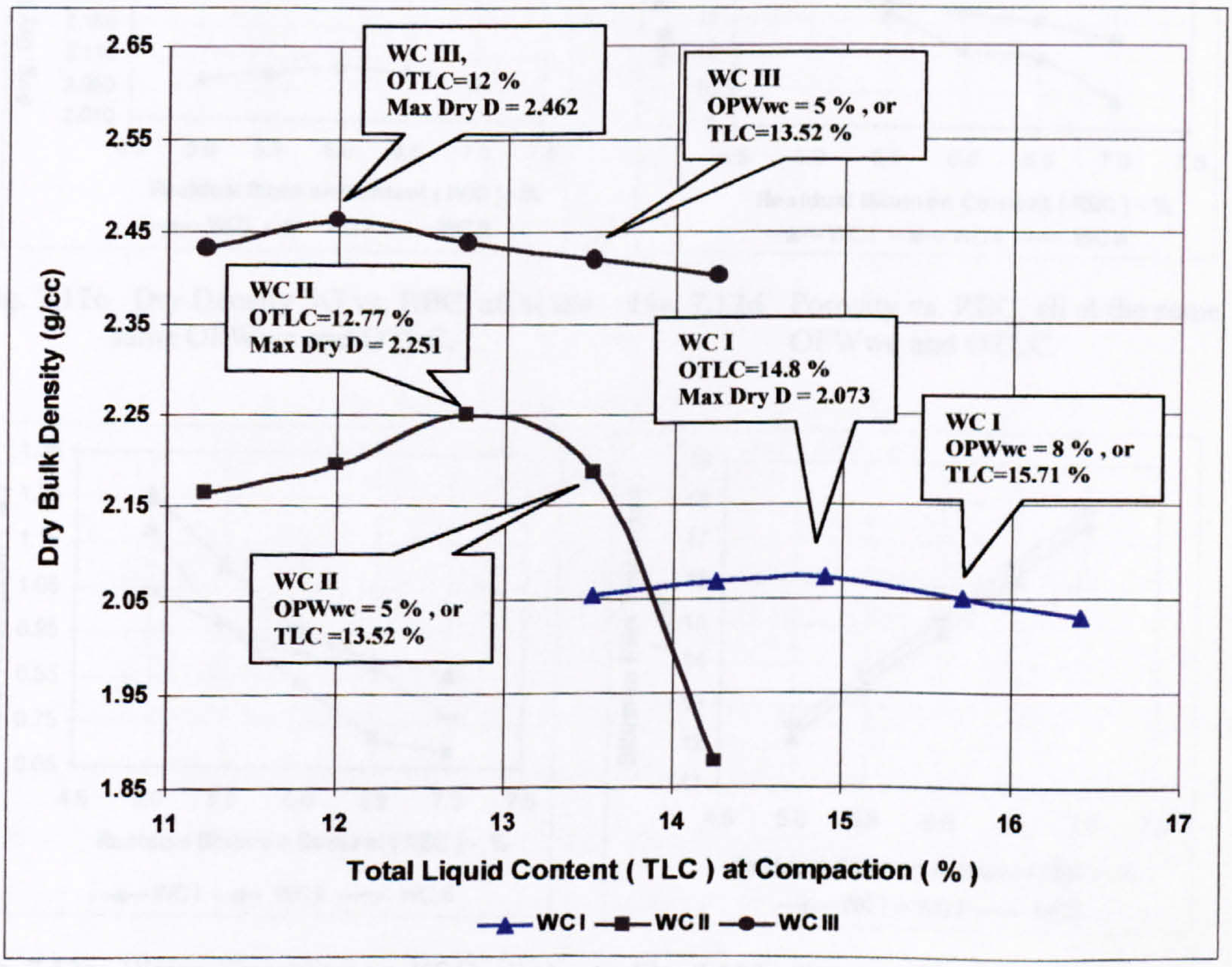


Fig. 7.11 The OPWwc and the OTLC for mixtures WCI, WC II, and WC III.

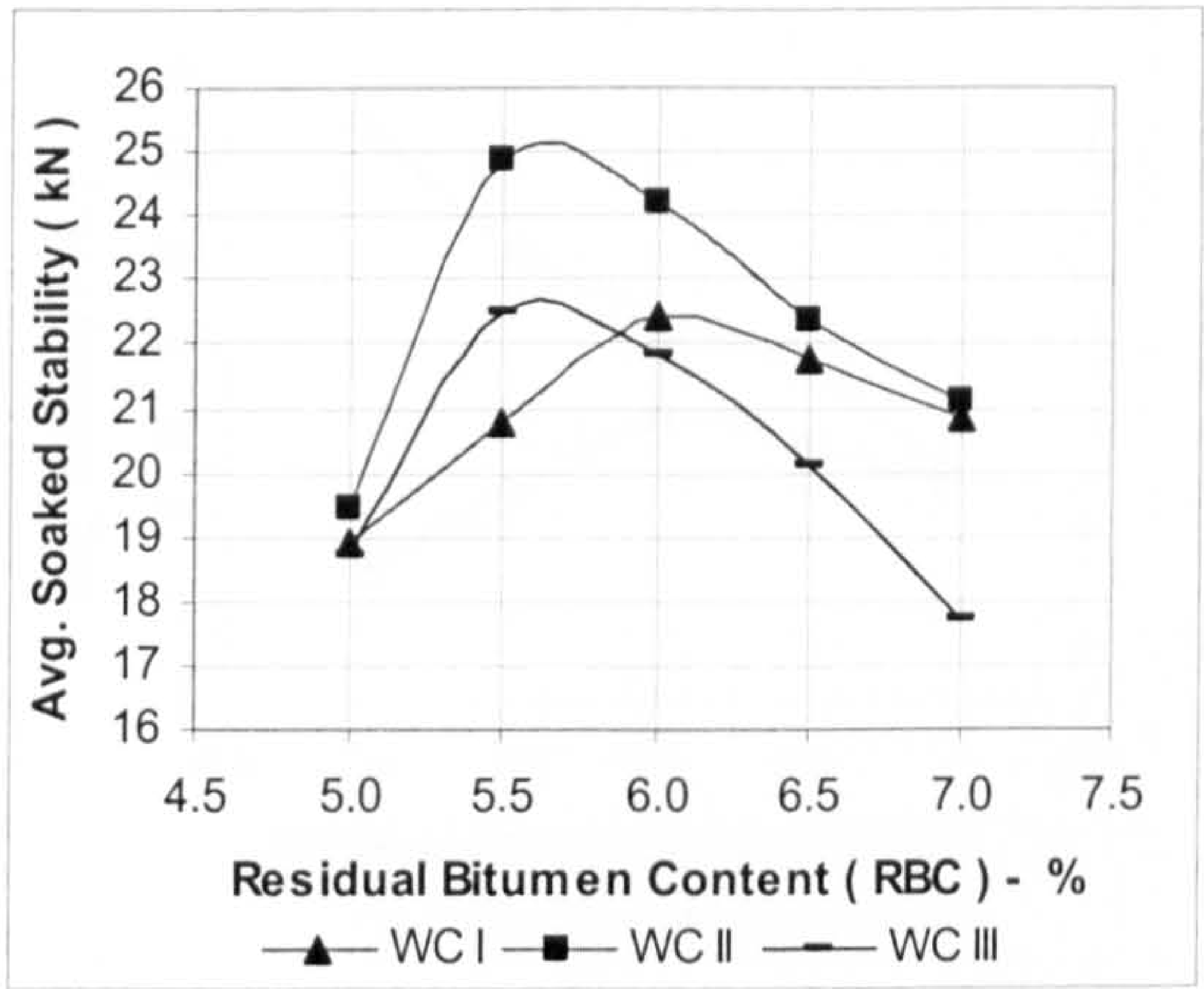


Fig. 7.12a Stage B (soaked) Stability vs. RBC, all at the same OPWwc and OTLC.

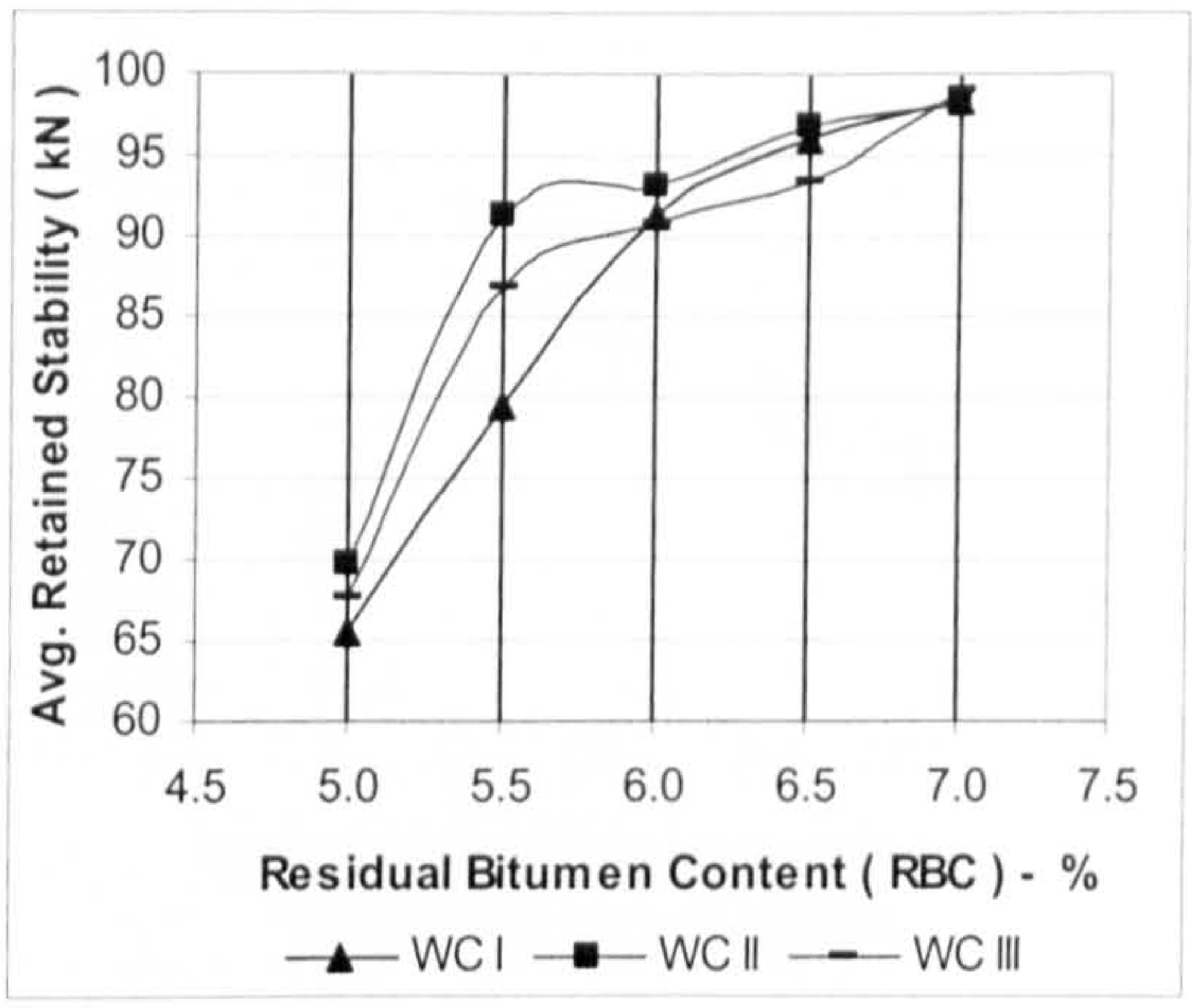


Fig. 7.12b Retained Stability vs. RBC, all at the same OPWwc and OTLC.

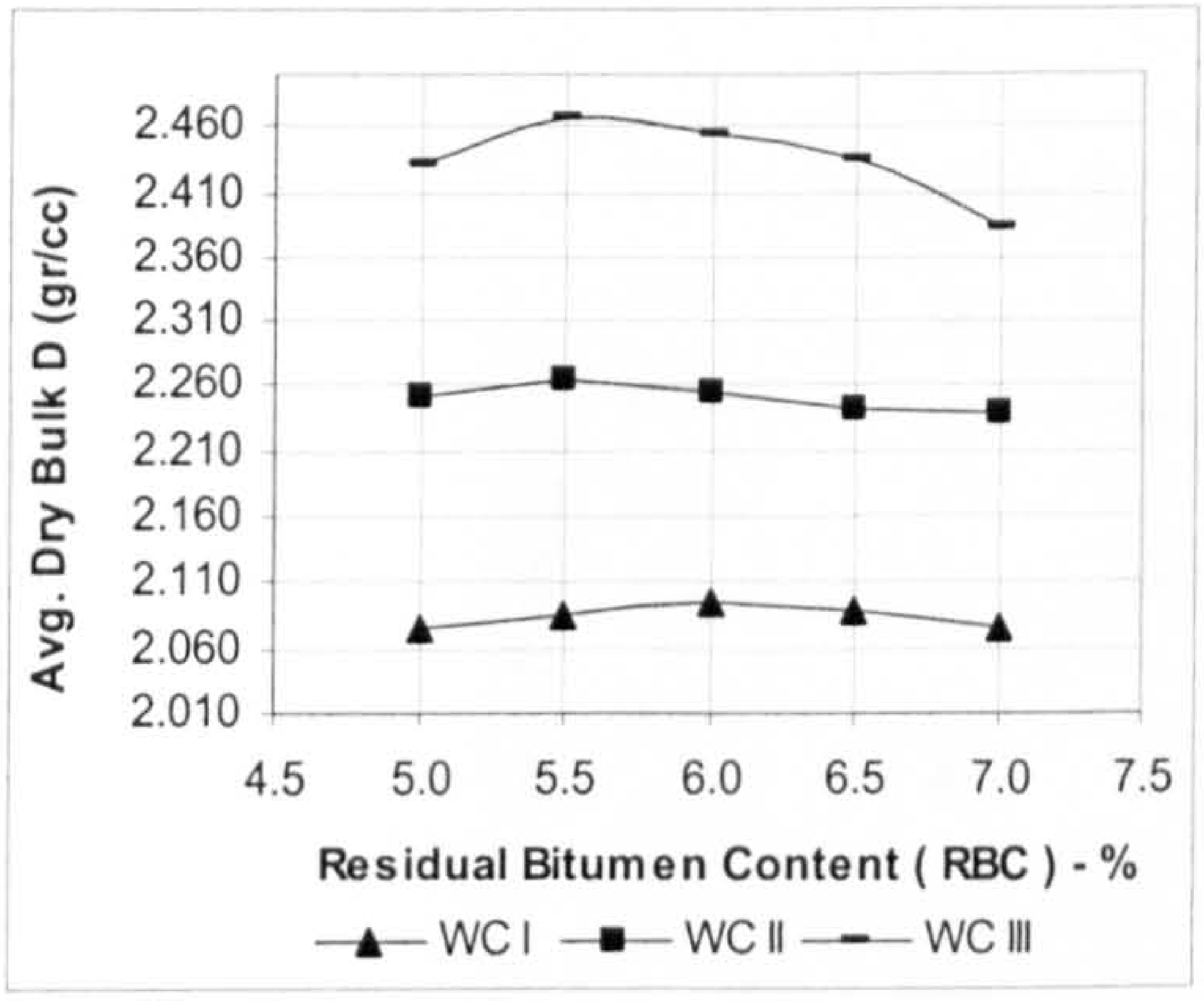


Fig. 7.12c Dry Density SG vs. RBC, all at the same OPWwc and OTLC.

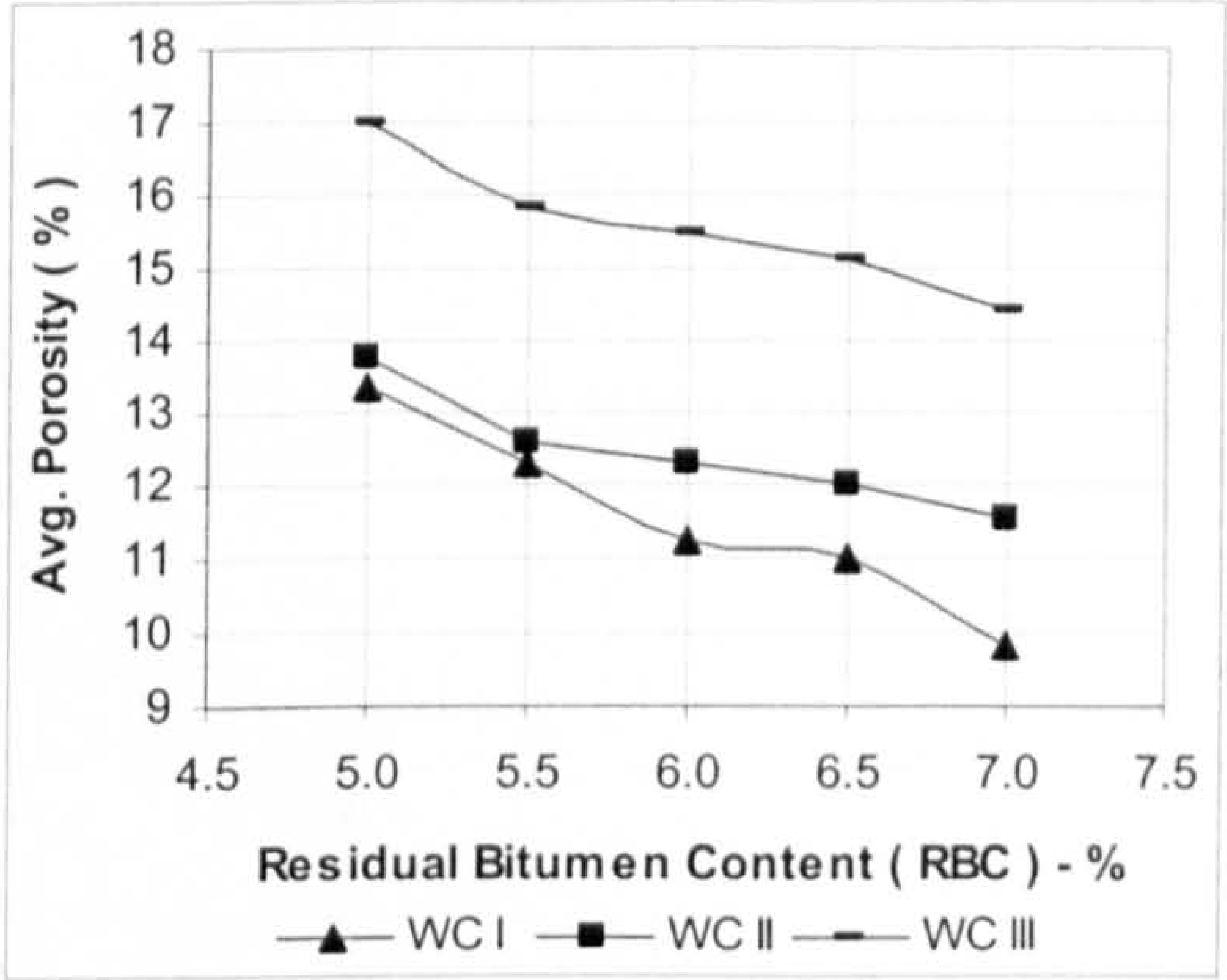


Fig. 7.12d Porosity vs. RBC, all at the same OPWwc and OTLC.

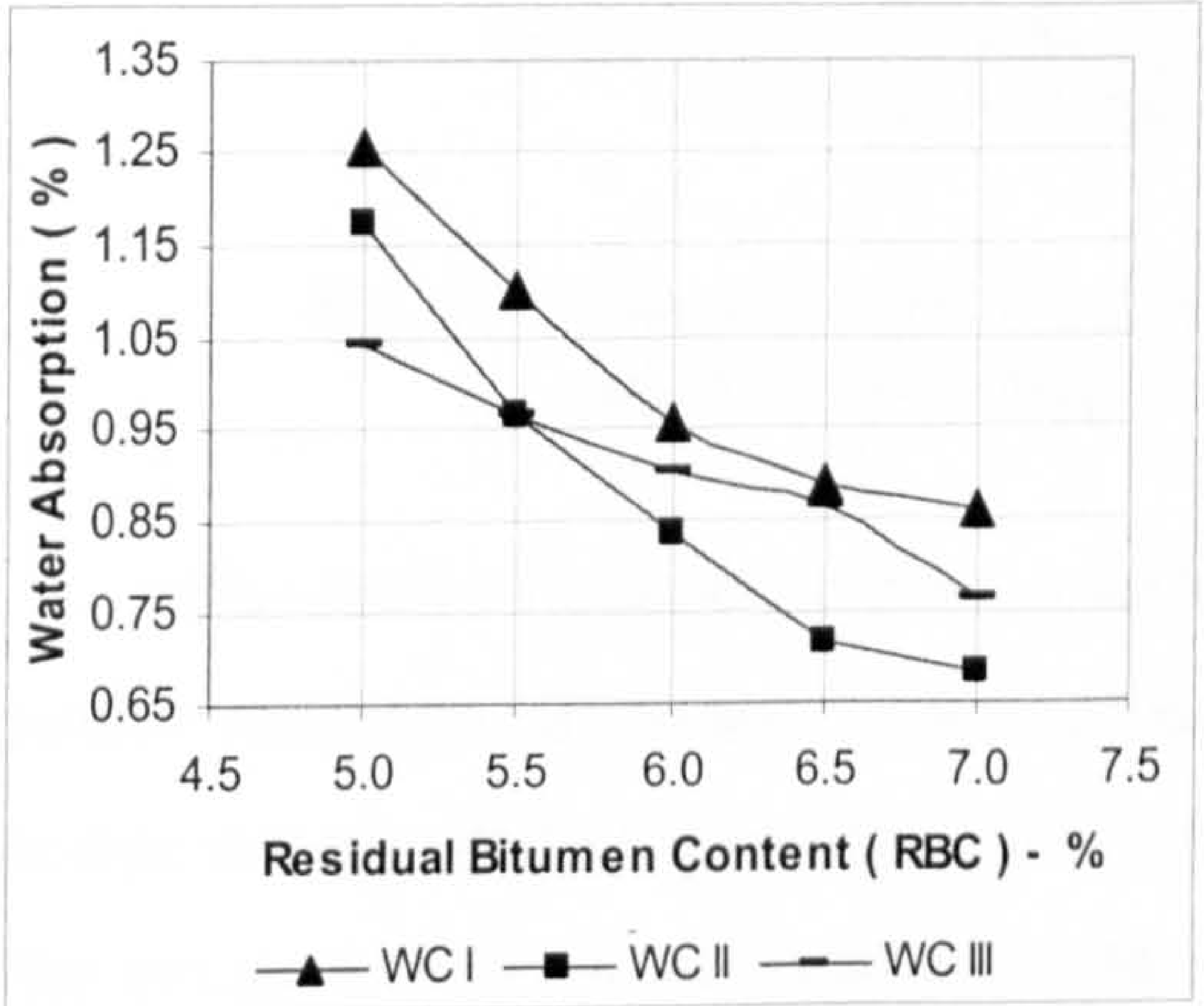


Fig. 7.12e Water Absorption vs. RBC, all at the same OPWwc and OTLC.

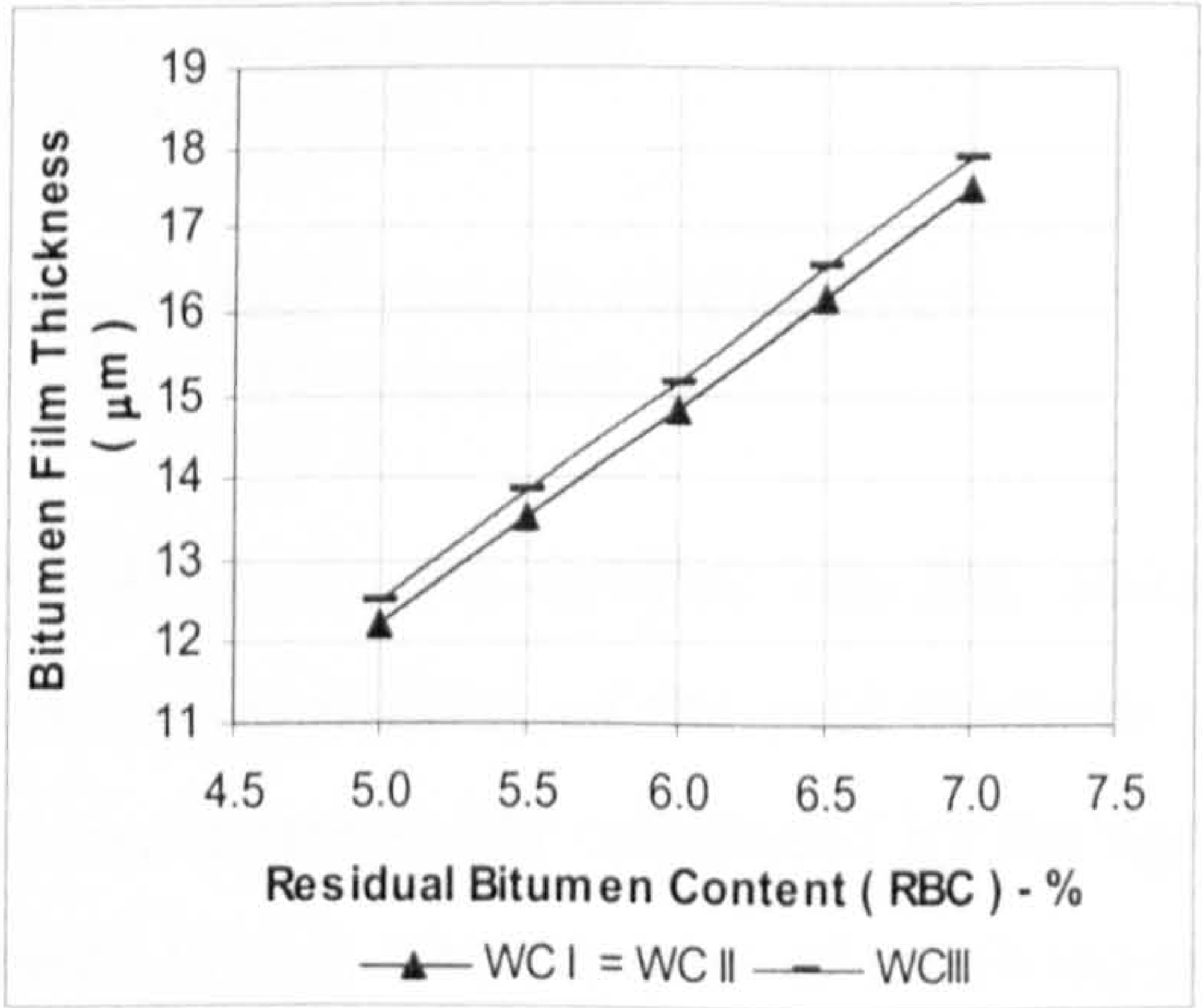


Fig. 7.12f Bitumen Film Thickness vs. RBC, all at the same OPWwc and OTLC.

Fig. 7.12 Design Plots for the WCI, WCII, and WC III.

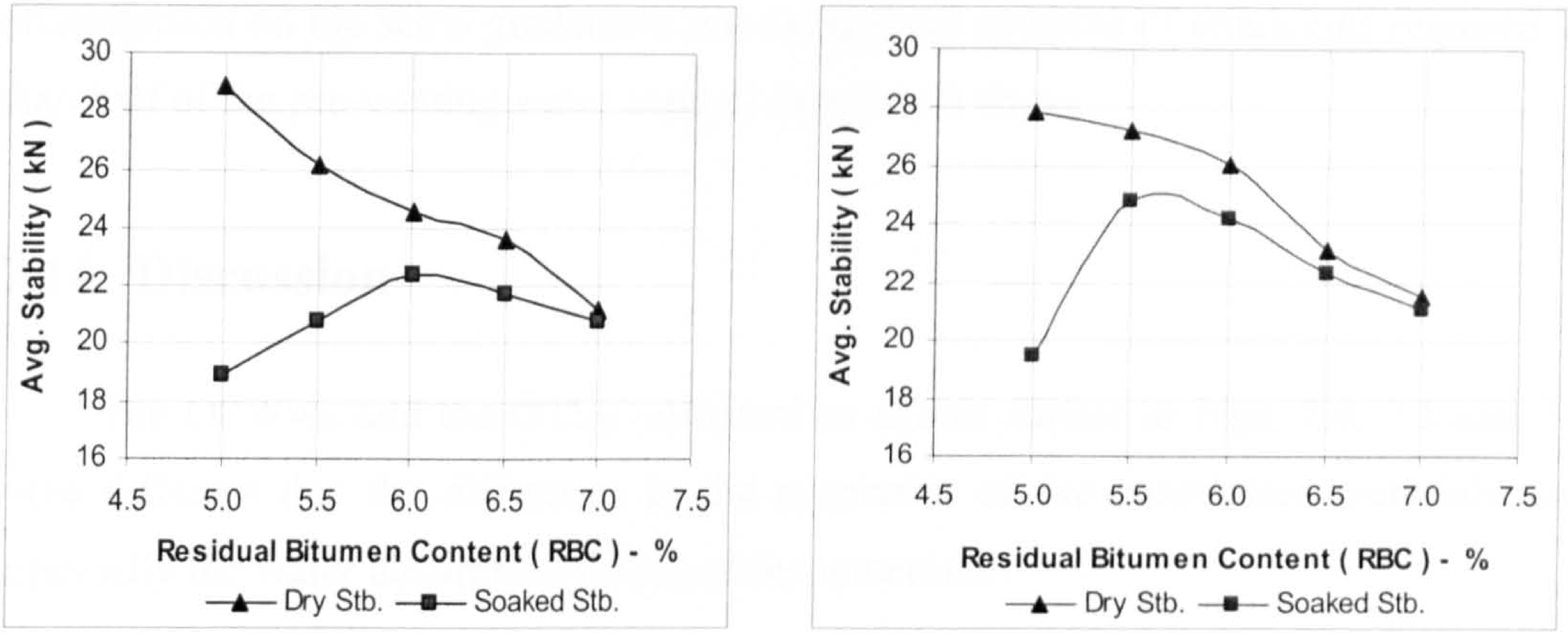


Fig. 7.13a Stage A cured (dry) and Stage B cured (soaked) stability values (at 24°C-room temp.) of WC I, all at the same OPWwc and OTLC.

Fig. 7.13b Stage A cured (dry) and Stage B cured (soaked) stability values (at 24°C-room temp.) of WC II, all at the same OPWwc and OTLC.

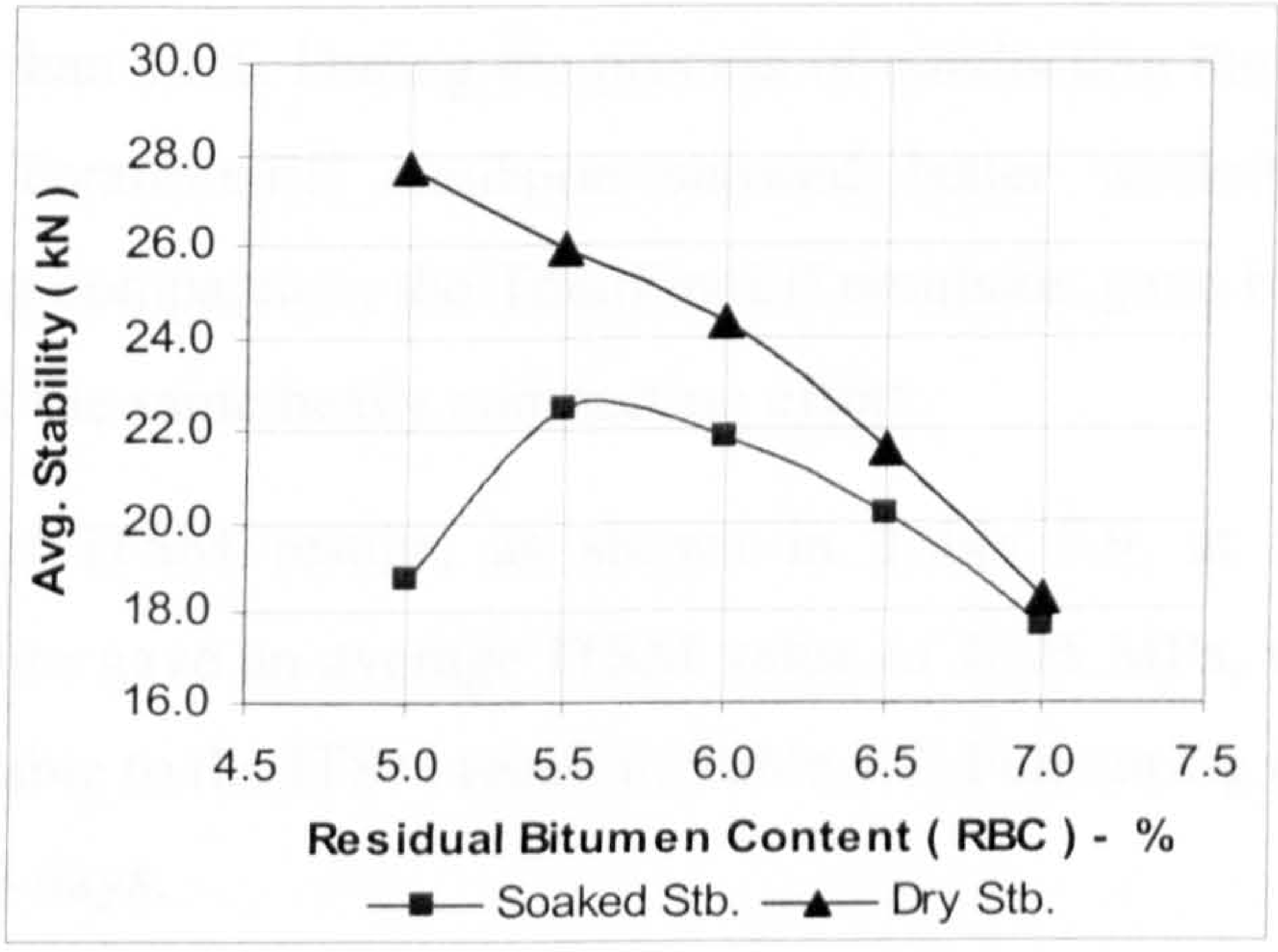


Fig. 7.13c Stage A cured (dry) and Stage B cured (soaked) stability values (at 24°C-room temp.) for WC III, all at the same OPWwc and OTLC.

Fig. 7.13 Stage A cured (dry) and Stage B cured (soaked) stability values (at 24°C-room temp.) for the WCI, WCII, and WC III.

The author recognizes (for example with WC I) that the OPWwc was 8%, which is rather excessive. In general it was found that the workability of the cold mixtures and hence the required pre-wetting water values were dramatically influenced by the age of the emulsion (i.e. storage stability and exposure times). The samples of emulsion used in this investigation had been proportioned into 2-litre metallic containers and stored for approximately 2 months prior to their use. Trials carried out at the end of the investigation on the same gradations but using fresh samples of emulsions required less than half of the pre-wetting water content mentioned above.

7.14 Discussion

The OPWwc and the OTLC obtained as shown earlier in Figs. 7.4, 7.5 and 7.11 were different due the difference in the properties of the constituent materials used, especially the water absorption values of the materials.

The porosity values obtained with heavy compaction in the last experiment in Section 7.13 (using the Nynas emulsion) as shown in Table 7.9 indicated higher porosity results (above 10%) than the test results using TotalFinaElf emulsion (porosity below 10%) as presented in Table 7.6. The target porosity value in this investigation was a lot lower than 10%. During the process of conducting the coating test, it became clear that the TotalFinaElf emulsion showed better workability than the Nynas emulsion. During compaction, the TotalFinaElf emulsion gave better lubrication, hence lower porosity at the same heavy compaction effort.

In terms of ITSM results, as shown in Table 7.9, at full curing the WC I compacted mixture gave an average ITSM value of 1095 MPa, which is slightly lower but very comparable to the ITSM result in Table 7.7. Full curing in an oven at 40°C was achieved after 14 days.

Meanwhile, the WC II and WC III samples (Table 7.9) *whilst performing normally during the design curing process (i.e. stages* **A and B)**, the *samples developed cracks* after about 7 days curing in an oven at 40°C (i.e. during the process of bringing the samples to a full curing state). The cracks (of very irregular pattern) were clearly visible on the specimen's surfaces and varied in widths from hair-line cracks up to approximately 0.5mm wide and at least 2mm deep. The cracks were very likely due to the *volume expansion of the steel slag* in the presence of moisture which causes the hydration process of the *free lime* (CaO) and *free magnesium* (MgO). This may have caused the ITSM values to scatter in the range of 1400 to 2000 MPa. The cracks did not heal after the curing process even though the specimens were left at room temperature for about one month.

7.15 Conclusions and Recommendations

It can be logically concluded that by increasing the compaction effort from medium to heavy compaction, the porosity can be reduced. However, in order to safely meet porosity targets an *even heavier compaction effort* may be required.

It was found that carrying out the 'coating test' was very essential. Coating test results were significantly affected by the type of materials used. Any material changes within the mixture composition can significantly change the mixture's workability and degree of coating.

It was felt that the design procedure was rather complicated, particularly in the determination of optimum total liquid content (OTLC) for compaction which occasionally requires air drying of the loose mixture to a certain level. This may not be practicable or difficult to apply on site, in particular during large mixture production, or when involving low skilled labour and limited equipment availability. It was therefore decided that the proposed design procedure must be improved as described in Chapter VIII Section 8.5.

Further efforts are still needed in order to accelerate the low early life strength of conventional CBEMs in general and to shorten the long curing periods required. Furthermore, alternative waste materials must be assessed in support of environmental conservation. The addition of *cementitious materials* was felt essential to accelerate the rate of strength gain and to improve the ultimate strength of CBEMs to meet ITSM targets.

CHAPTER VIII

LIMITATIONS OF THE CBEMs DESIGN PROCEDURE INITIALLY DEVELOPED AND A PROPOSED NEW DESIGN PROCEDURE

After carrying out a large number of laboratory tests, it became evident that there were some inherent limitations with the initially adopted cold mixture design procedure. The main limitations encountered are described in Sections 8.1 to 8.4, and a new CBEMs design procedure is therefore proposed in Section 8.5.

8.1 Determination of Optimum Total Liquid Content at Compaction (OTLC)

Although the procedure was based on logical principles, it was however not very practical and was envisaged to be difficult to implement on site particularly when involving low skilled labour and conventional asphalt plant for mass production. This could be best described by referring to the example in Figure 7.11 in Chapter VII for the wearing course I (WC I) mixture, in which the data were based on a 6 % IRBC by mass of total dry mixture. All the samples were produced at the same **optimum pre-wetting water content (OPWwc)**, which was 8% *by mass of dry aggregates*. This had been calculated to result in 15.71% total liquid content (TLC = water + emulsion) *by mass of total mixture during the mixing stage*. The three result points shown on the left of the (15.71% TLC point in WC I), were mixed **at the same (OPWwc), and hence had the same *total liquid content* (TLC), but were subsequently air dried** to various levels prior to compaction.

In this example, the **OTLC** that gave the *maximum compacted mixture density* value was found to be **14.8%.** In the laboratory, this procedure was practical but required extra care by regularly weighing the air dried mixtures until they had achieved the required OTLC. *This procedure is very unlikely to be practical on site.*

8.2 Compaction Effort

The medium compaction effort (80 revolutions, 240kPa, 2° angle of gyration) that was adopted in the early parts of this investigation was found to be inadequate for achieving sufficient compacted density, and hence unacceptable porosity values were produced which far exceeded the porosity targets (Table 7.6, Chapter VII). On the other hand, at medium compaction effort, the samples did **meet the soaked stability specifications.**

When heavy compaction was subsequently introduced (120 revolutions, 240kPa, 2° angle of gyration), the CBEMs were just capable of meeting the porosity target, but the values were nonetheless very close to the upper limit. Earlier investigations, e.g. a study by Khalid & Eta (1996) on *Dense Emulsified Bitumen Macadams* demonstrated that even at heavy compaction effort the porosity values calculated were still higher than the targets (see Table 7.6, Chapter VII). This is generally influenced by the type and the proportion of materials used, but the main reason is likely to be the excessive total liquid content in the system, which causes pore water pressure to develop and which in turn resists compaction and maintains trapped water in the matrix.

After carrying out extensive laboratory compaction trials, it was found that the mixtures required '*extra heavy compaction*' (i.e. up to two times the conventional heavy compaction effort) to safely meet the porosity targets.

Unfortunately, there does not seem to be a universally accepted specification currently available regarding porosity requirements for CBEMs. The Asphalt Institute for instance does not recommend any porosity limits.

8.3 Retained Stability: (Soaked Stability / Dry Stability)

This parameter was obtained from samples at all variations of *residual bitumen content* based on testing specimens following *design curing stages A and B.* The Marshall stability test is a destructive test, and therefore the test protocol suggested in this investigation requires the production of a large number of samples, i.e. stage A cured (dry) samples and stage B cured (soaked) samples. This was felt to be *unnecessary* and that it would be more efficient if this parameter was *only determined at*

Optimum Residual Bitumen Content (ORBC). Some *scatter in the results was also observed,* which may be attributed to the mixtures not achieving full curing or containing some trapped water.

8.4 Ultimate Strength of CBEMs

Current design procedures do not refer to *ultimate strength* requirements. The *soaked stability values* recommended in typical specifications are based on the *design curing* (stage A followed by stage B) as described in Chapter VII (Section 7.8). In this investigation it was found that following this curing regime, the samples were very likely to have *retained moisture* and hence *not have yet achieved full curing.* It was therefore decided to introduce a ***full curing procedure.*** In this procedure, the specimens were left in their compaction moulds for 1 day at room temperature before being extruded, this was followed by several days in an oven at 40°C (until a constant mass was achieved). At that condition, all the water content trapped within the samples would have evaporated (full curing condition). Finally the samples were left to cool down to room temperature (24°C) for 1 day. At the end of this curing procedure the specimens were tested (same as for conventional hot mixtures) either for Marshall stability at 60°C or for Indirect Tensile Stiffness Modulus (ITSM) at 20°C. Again it is recommended that this full curing regime shall *only be applied to* mixtures at optimum residual bitumen contents (**ORBC**) to evaluate the mixture's ultimate performance.

8.5 A Proposed New CBEMs Design Procedure

As a result of the mixture design limitations mentioned earlier in this chapter and based on a wide range of trials carried out by the author, the following CBEMs optimized design procedure has been proposed [Zoorob and Thanaya, 2002], and [Thanaya and Zoorob, 2002].

8.5.1 Determination of Aggregate Gradation

In order to obtain better gradation control, it is recommended that the coarse aggregate (retained on 2.36mm sieve) and the fine aggregate (passing 2.36mm and retained on 0.075mm sieve) fractions are sub-divided into as many *smaller fractions* as practicable. The filler component is defined as material with not less than 65% passing 75µm.

The *aggregate gradations normally follow national specifications. In the cases where it is possible to design the gradation from basic principles, it is recommended that a Modified Fuller's Maximum Packing Curve be adopted.* An example of a Modified Fuller's Maximum Packing Curve was proposed by Cooper [Cooper et. al., 1985], the formula used is as follows:

$$P = \frac{(100 - F)(d^n - 0.075^n)}{D^n - 0.075^n} + F \tag{8.1}$$

where: P = % material passing sieve size d (mm), D = maximum aggregate size (mm), F = % filler, n = an exponential value that dictates the concavity of the gradation line.

The filler fraction F can be chosen by referring to conventional specifications or based on the experience of the asphalt engineer. An exponential n value close to 0.45 is recommended for all mixtures (this is applicable for any nominal maximum aggregate size). The value n = 0.45 is the exponent adopted by Superpave Level 1 Mixture Design for best aggregate packing.

8.5.2 Estimation of Initial Residual Bitumen Content (IRBC), and Initial Emulsion Content (IEC)

An estimate *initial residual bitumen content* (IRBC), designated as P, can be determined by utilizing the Asphalt Institute empirical formula as shown below [Asphalt Institute, MS-14, 1989]:

$$P = (0.05\,A + 0.1\,B + 0.5\,C) \times (0.7) \tag{8.2}$$

where: P = % Initial Residual Bitumen Content by mass of total mixture, A = % of aggregate retained on sieve 2.36mm, B = % of aggregate passing sieve 2.36mm and retained on 0.075mm, C = % of aggregate passing 0.075mm (filler fraction).

The IEC (initial emulsion content) value can be determined by dividing P by the percentage of the residual bitumen content in the emulsion:

$$\text{IEC} = (P / X)\% \tag{8.3}$$

where: IEC = initial emulsion content by mass of total mixture and X = the residual bitumen content of the emulsion.

8.5.3 Coating Test

This test is also known as '**binder compatibility test**' and it provides the initial stage for selecting suitable emulsions according to aggregate type being used. The coating test indicates the degree of affinity and compatibility between the emulsion and the mineral aggregates used in a particular investigation. The test shall be carried out at room ambient temperature with the following steps:

8.5.3.1 Material Preparation

About 500 grams of proportioned aggregate materials shall be prepared and placed in a metal container of suitable size and mixed dry with a spatula, or metal spoon. The emulsion that had been kept in an airtight sealed container is gently agitated / stirred and an adequate quantity is poured into a beaker.

8.5.3.2 Pre-wetted or Added Water at Various Water Content for Optimum Coating Requirement

Several proportioned dry graded aggregate batches, 500 grams each are prepared at this stage of the experiment. The amount of added water to pre-wet the mineral aggregates in each batch is varied. A typical starting point would be 3% water *by weight of dry aggregates*, this is subsequently varied in steps of 1%. A fixed amount of bitumen emulsion, i.e. the initial emulsion content (IEC) is then added and mixed with each of the pre-wetted aggregate samples. The added water acts as a viscosity reducing agent that promotes workability and aids in the even distribution of any added emulsion onto the aggregate surfaces. ***The amount of added water is subsequently adjusted if the mixture becomes either too stiff or too sloppy.***

8.5.3.3 Mixing Procedure

One method of introducing the pre-wetting water is obviously to add the correct mass of water to the dry aggregate batches at several percentages prior to introducing the emulsion. Alternatively, to simulate wet aggregates in stockpiles on site, the dry aggregates can be *dampened* with an estimated amount of water and kept in a sealed container for 24 hours.

The experience gained from this investigation indicated no significant difference in terms of workability and degree of coating between the two pre-wetting techniques. Therefore, the first technique, i.e. adding all the pre-wetting water to the *dry aggregates* in one go was recommended as it is simpler. A suitable mixing procedure is suggested as below:

Mixing Procedure: (CA+FA+F) + W + E

In this procedure, the coarse aggregate (CA), the fine aggregate (FA) and the filler (F) fractions are first combined and mixed dry. The pre-wetting water (W) is then added and the aggregates re-mixed. After that the required initial emulsion content (IEC) is added and the mixing continued for an additional 2 minutes until the mixture becomes evenly coated with the emulsion. Excessive mixing shall be avoided to prevent binder stripping off the aggregate surfaces.

Additional batches of mixed aggregates are pre-wetted in increments/decrements or steps of 1% by weight of dry aggregate, whilst keeping the IEC content constant in all cases. At this stage, consideration should be given to the *degree of workability depending on the type of mixing equipment used.* This may require further adjustments to the pre-wetting water content to guarantee a satisfactory degree of coating.

In mixtures where the target 85% degree of coating can not be achieved, this does not necessarily mean that the emulsion should be rejected. It is recommended that the mixture design procedure is continued by targeting **'the best degree of coating'** possible. The Asphalt Institute for example, only requires a degree of coating of 50% for base course mixtures. However, the overall performance of the specimens, namely; the rate of increase in strength, the ultimate strength at full curing, and performance following water conditioning must remain satisfactory. Incorporation of a small amount of cementitious materials (1-2%) had been found to give a significant increase in the rate of strength gain and to improve ultimate strength. It was observed in this investigation that the addition of cementitious materials can affect the amount of pre-wetting water content required (more pre-wetting water was normally required) for satisfactory workability and coating.

8.5.3.4 Determination of Optimum Pre-Wetting Water Content for Best Coating (OPWwc)

At this stage, the degree of coating is ascertained by visual observation to determine the optimum pre-wetting moisture content that would result in all the surfaces of the mineral aggregate particles being fully and evenly coated with (black/dark colour) bitumen. If for the same added emulsion content, the required degree of coating can be achieved by more than one pre-wetting water content, the mixture ***with the lowest pre-wetting water content is selected.*** This pre-wetting water content is referred to as the **optimum pre-wetting water content for best coating (OPWwc).** At the OPWwc, the mixtures are normally neither too sloppy nor too stiff.

8.5.4 Loose Mixture Preparation: assessment of storage and air drying

After determining the OPWwc, all loose CBEMs shall subsequently be prepared using the OPWwc. At this stage, some practical site related aspects need to be considered, including: expected mixture storage time and storage conditions (i.e. sealed or unsealed), and air drying that may be required for practicable optimum compaction (ideally, the mixtures shall be just loose enough with no indication of balling of the fines with the emulsion). Depending on the type of compaction method/effort used, air drying will normally be required, as typically, slightly more pre-wetting water is needed at the mixing stage for convenient workability (depending on the type of mixing equipment used) than the optimum total liquid content for compaction.

For laboratory scale work, drier loose mixtures may be required when using the Marshall hammer impact type compaction (as opposed to Gyratory compaction) as a consequence of the high pore water pressures developed in the mixture during the compaction process resulting in high porosities of the fully cured specimens. Gyratory compaction on the other hand with its shear/kneading type effect enables the compaction of wetter loose mixtures where the excess liquid can be expelled through the bottom and top of the specimens. Following compaction, any excess liquid shall be removed with the aid of paper towels or cloth.

8.5.5 Adjustment and determination of Compaction Level to meet target Porosity

It has already been argued that CBEMs require heavier compaction and therefore compaction should preferably be conducted using heavy compaction effort.

Following compaction, the compacted samples are left to cure in their compaction moulds *for 24 hours at ambient room temperature.* After that the specimens shall be extruded. At that point in time, the specimens may still be too weak for handling as they may still contain some trapped water and therefore, the specimens require careful handling.

8.5.5.1 Determination of Dry Density

The volume of the compacted specimens are required, and since all compacted samples have uniform diameters (typically d = 100mm), the average height ($h_{avg.}$) and hence volume (V) of each sample can be calculated from measurements of heights across two perpendicular axis.

$$V = \tfrac{1}{4}.\pi.d^2.\ h_{avg},\ \text{in cm}^3 \qquad (8.4)$$

The wet bulk density (Wet Bulk D) is then calculated by dividing the mass of each wet sample (W_w) by its volume V.

$$\text{Wet Bulk D} = \frac{Ww}{V}\quad \text{gr/cm}^3 \qquad (8.5)$$

The specimens shall be broken down, and about 500 grams from each specimen shall be used for water content determination (w %) (by oven drying) at time of testing. The water content also allows the calculation of the dry bulk density at time of testing (Dry Bulk D):

$$\text{Dry Bulk D} = \frac{(100 + \text{RBC})}{(100 + \text{RBC} + w)} \times \text{ Wet Bulk D} \qquad (8.6)$$

where: RBC = residual bitumen content, w = water content at time of testing (%), [Asphalt Institute, MS-14, 1989; MPW-Indonesia, 1990].

8.5.5.2 Determination of Porosity (P)

Porosity is determined using the following well known formula:

$$\text{Porosity} \quad \text{P}\,(\%) = \left(1 - \frac{\text{DryBulkD}}{\text{SGmix}}\right) \times 100\%, \tag{8.7}$$

where the SG_{mix} is calculated using Equation 8.8.

$$\text{SGmix} = \frac{100}{\dfrac{\%\text{CA}}{\text{SGCA}} + \dfrac{\%\text{FA}}{\text{SGFA}} + \dfrac{\%\text{F}}{\text{SGF}} + \dfrac{\%\text{Binder}}{\text{SGBinder}}}, \text{ by weight of total dry mixture} \tag{8.8}$$

When the average porosity value of any one set of compacted specimens falls below the required design / specification limits, the compaction procedure must be repeated with an increased compaction effort. During the course of this investigation, it was observed that compaction efforts up to twice the conventional heavy compaction effort (75 blows Marshall Hammer) may be needed. It is *recommended that the Porosity values should lie between 5-10 %.*

8.5.6 Variation of Residual Bitumen Content (RBC)

Specimens shall be prepared with varying the RBC, which can only be achieved by varying the emulsion content. The RBC shall be varied in two increments above and two below the *initial residual bitumen content* (IRBC) in steps 0.5%. Samples shall be manufactured using the compaction effort selected in the previous stage (Section 8.5.5). For every RBC variation, a *minimum of three samples* shall be prepared.

8.5.7 Design Curing

The curing regime, referred to as **Design Curing,** consists of two curing stages followed by a Modified (room temperature) Marshall Stability Test of the S***oaked Samples.*** This procedure is carried out on specimens at varying residual bitumen contents (RBC). The design curing regime consists of the following:

- **Design Curing Stage A**; oven curing of compacted samples

This conditioning procedure consists of keeping the samples for 1 day in their moulds following compaction. Depending on the compaction level and the size of mould used, at early ages the samples in their moulds remain weak and care must be exercised to avoid disturbing the samples. Gyratory samples are normally cured in their compaction moulds whilst lying on their sides. The choice of whether the bottom and/or top compaction plates are kept in the moulds during the curing process depends mainly on the design of the compaction moulds. In the cases where the compaction process forces the samples to be fully seated in the compaction moulds on base plates, e.g. Marshall hammer, the samples can be cured standing upright in their compaction moulds whilst resting on the base plates. Wherever possible, similar exposure conditions to air of the curing mixtures (i.e. with or without end plates) should be maintained for all specimens. Following one day curing in the moulds, the samples are extruded from their moulds and kept in an oven for one more day at 40°C. The samples are then removed from the oven and stored for one extra day at room temperature.

- **Design Curing Stage B**; water conditioning samples for soaked stability test (capillary soaking):

After having been subjected to design curing stage A as explained earlier, the dry samples are water conditioned. In this procedure (also referred to as capillary soaking) half the thickness of each compacted specimen is soaked in water at room temperature for 24 hours, the specimen is then inverted and the other half soaked for a further 24 hours. During soaking, the samples shall rest on a bed of coarse sand (approximately 15 to 20mm thick to allow circulation of water under the test specimen). The samples are subsequently towel dried, their mass in air measured to determine the amount of *water absorption* and they are then tested for Marshall stability at room temperature. The Marshall stability test results obtained at this stage are thus referred to as **soaked stability** values.

8.5.8 Determination of Optimum Residual Bitumen Content (ORBC)

The ORBC is determined based on soaked samples by optimizing the two main **parameters** namely: **soaked stability**, and **dry bulk density.** Furthermore **porosity, water absorption, and bitumen film thickness (BFT)** values must be evaluated with reference to specifications. All parameters shall be *plotted in graphical format* against the Residual Bitumen Content (RBC).

8.5.9 Determination of Retained Stability

The retained stability, i.e. the ratio of dry stability to soaked stability is required *only for mixtures at ORBC.* A minimum of three samples are required for determination of an average dry stability value. These samples are obtained after conducting design curing stage A. The stability of the soaked samples that had been obtained in Section 8.5.8 are used for comparison. A minimum of *50% Retained Stability* shall be achieved following this design curing procedure.

8.5.10 Determination of the Ultimate Strength

The ultimate strength of the samples shall be determined using samples at ORBC. This is done by curing the samples at ORBC to a *full curing* condition.

8.5.10.1 Full Curing

In the 'full curing' procedure, a minimum of three specimens at ORBC are left in their compaction moulds for 1 day at room temperature before being extruded. The specimens are then placed in an oven at 40°C until a *constant mass* had been achieved (0.01 % weight difference). A typical time required to achieve full curing at 40°C for a 100mm diameter sample was approximately 14 days for samples having *porosities* in the range of 9-10%, and up to 21 days for samples with porosities in the range of 7-8%.

Finally the samples are left to cool down to room ambient temperature for 1 day. At the end of this curing procedure the specimens are tested either non-destructively for ITSM at 20°C, and/or destructively for Marshall stability after conditioning the samples at 60°C for 30 minutes in a water bath (as for conventional hot asphalt mixtures),.

8.5.10.2 Water Damage at full curing

Three fully cured samples shall be tested for water damage by totally soaking the samples in water at ambient temperature for 48 hours. The samples are then towel dried and weighed to assess water absorption. This is followed by testing the samples either non-destructively using the ITSM test at 20°C and/or destructively using the Marshall stability test after conditioning the samples at 60°C for 30 minutes in a water bath (as for hot mixtures).

8.5.10.3 Retained Ultimate Strength (RUS)

The retained ultimate strength (RUS) is the strength ratio of the fully cured samples after and before water conditioning as mentioned earlier in Section 8.5.10.2. It is recommended that the retained ultimate strength is a minimum of 90%. In the course of this investigation, it was found that such a target is achievable. In some cases, the RUS values were found to be greater than 100%. This was attributed to the development of pore water pressure within the sample during load application. Additionally, where CBEMs incorporate cementitious materials, hydration may still be ongoing during the water conditioning stage.

8.5.11 Incorporation of cementitious materials

In order to accelerate the rate of strength gain of CBEMs, and to improve and achieve the required ultimate strength, incorporation of 1-2% cementitious materials into the CBEMs at ORBC was found to be very beneficial. Incorporation of an excessive amount of cementitious materials may result in thermal cracking related problems and when used as a base layer, thermal cracks can rapidly propagate upwards into the surfacings layers. The retained ultimate strength (RUS) when incorporating cementitious or pozolanic materials must also be tested.

8.5.12 Conclusion

There are no universally accepted specifications for CBEMs currently available. The design procedure proposed in this chapter is believed to be very flexible in enabling the design of CBEMs that can satisfy the property requirements of existing specifications and recommendations, e.g. Specification of the Ministry of Public Work, Republic of Indonesia, 1990; Recommendations of Nikolaides.A.F, 1994; and Asphalt Institute Specifications, MS-19, 1997.

CHAPTER IX

MATERIALS AND METHODOLOGY DEVELOPED FOR FURTHER INVESTIGATIONS

9.1 General

One of the prime objectives of this study was to improve the performance of CBEMs incorporating waste / secondary materials. The main practical consideration was to achieve satisfactory *porosity targets* (*volumetric based design*), whilst simultaneously improving the rate of strength gain and the ultimate strength of the CBEMs. A wide range of materials and experimental techniques were devised and tested in this part of the investigation in order to achieve this objective. In the following parts of this chapter, it is demonstrated how different experimental techniques and strategies evolved and were modified in response to both positive and negative experimental results. The experimental methodology detailed in this chapter is presented in *chronological order.* The mixture design methodology used in all the experiments listed in this chapter was based on the Proposed CBEMs Design Procedure in Chapter VIII Section 8.5.

9.2 Experiments at Extra Heavy Compaction Effort Level 1 (EHC1): *210 revolutions* at 240 kPa

Referring to the results in Table 7.9, Chapter VII, section 7.13.3, and the Discussion in section 7.14, it was clear that the porosity values obtained when using the Nynas Emulsion were still unacceptably high, i.e. above 10%. The compaction effort used, which was classed as Heavy Compaction Effort was composed of: 120 revolutions using a Gyropac at 240 kPa axial pressure. Based on preliminary compaction trials and to satisfy the porosity targets, the number of compaction revolutions was increased from

120 to 210 revolutions. The new compaction effort (i.e. Gyropac, 210 revolutions, 240 kPa) was referred to as ***Extra Heavy Compaction Effort Level 1 (EHC1).***

9.2.1 Mixture Designation and Materials used

In order to narrow down the number of trials, further work that was carried out was *based on gradation WC I* (as detailed in Chapter VII). For ease of reference, the mixtures in this part of the investigation were re-named as shown in Table 9.1. The maximum nominal aggregate size of all mixtures was 12.7mm. The *coarse aggregates* for all mixtures were Carboniferous Limestone which was divided into the following fractions: (12.7-10.0mm, 10.0-5.0mm, 5.0-2.36mm). The filler was Pulverized Fuel Ash (PFA) from Eggborough Power Station, UK, and the emulsions used were a TotalFinaElf Emulsion and a Nynas emulsion.

The CBEMs were manufactured at 6% ORBC (same as in Table 7.9, Chapter VII for the WCI)) and were fully cured. Their properties are given in Section 9.2.5 Results and Discussion.

Table 9.1 WC I Mixture Designation at this stage. Compaction effort: Gyropac, 210 revolutions, 240 kPa (i.e. compaction level EHC1).

No	CBEMs	Coarse Aggregates	Fine Aggregates used	Mixture Designation
1	WCI-RPS graded *	Limestone	Graded red porphyry sand (RPS) *	WCI-RPSg
2	WCI-Synthetic Aggregates	Synthetic agg.: (10-5mm), The remainder of the coarse aggregate fractions were limestone	Graded red porphyry sand (RPS) *	WCI-SA
3	WCI-RPS un-graded **	Limestone	Ungraded red Porphyry sand (RPS) *	WCI-RPSug
4	WCI-Control Mixture	Limestone	Quartzitic asphalt sand	WCI-CM

* The Red Porphyry Sand (RPS) was graded into: (2.36-1.18mm, 1.18-0.6mm, 0.6-0.3mm, 0.3-0.075mm). ** Red Porphyry sand used as received (ungraded).

9.2.2 Waste Materials

At this stage of the investigation, the waste materials used were:

1. *synthetic aggregates* which were obtained from the waste produced by an on going project on 'production of synthetic aggregates from quarry waste' being carried out

at the School of Civil Engineering, Leeds University (see Section 4.3.3, Chapter IV). The particle size range of the synthetic aggregate used was from 10 to 5mm. The amount used in the asphalt mixtures was 25.65% by mass of total aggregates (to substitute by volume, all the limestone coarse aggregates of equal size).

2. the *shredded crumb rubber* supplied by Charles Lawrence Recycling Ltd.- Nottinghamshire, UK (see also Section 4.3.5, Chapter IV). Incorporation of 2% and 5% crumb rubber (10 to 5mm) by volume as aggregates substitute was also tried. This was equivalent to 0.9% and 2.1% mass substitution respectively. In an attempt to counteract the possible reduction in stiffness that would be caused by the rubber inclusion, all the specimens in this part of the investigation were manufactured with 2% added rapid setting cement.

9.2.3 Cement types used

Three types of cement were tested in this investigation, namely: ordinary portland cement (OPC), rapid setting/hardening portland cement, and natural cement. The objectives were: to improve mixture stiffness, to accelerate the rate of mixture strength gain, and for comparison purposes.

The main differences between the Rapid Setting cement and the OPC are a- rapid setting cement contains higher tricalcium silicate (C_3S) content (up to 70 %) as opposed to 59% for OPC and b- rapid setting cement has higher fineness requirements (minimum 325 m^2/kg), where as BS 12:1991 no longer specifies a minimum fineness requirement for OPC. These factors enable Rapid Setting cement to develop strength more rapidly (Neville & Brooks, 1997).

Natural Cements are produced by the calcination of a natural clay limestone at a temperature below sintering point and then grinding it to a fine powder. This cement is not used in exposed areas, and may be used as portland cement substitute in mortars and concretes that are not subjected to high stresses (Derucher et al., 1998).

In order to improve the mixture properties, and for comparison purposes, all three cement types mentioned above were incorporated in turn into CBEMs (WC I) in the amounts of 1% and 2% by mass of dry aggregates. Trials were conducted and it was found that incorporation of *2% OPC or 2% rapid setting cement* gave satisfactory results that met the stiffness targets set at the onset of the project. Meanwhile *natural* *cement did not meet expectations*. It was also found that the *rapid setting cement gave higher stiffness values and more rapid strength gain* when compared with the other two cements as shown in Fig. 9.1. The samples were cured outdoors (starting from 26-7-2001) with an average temperature of 13°C [Washington Post, 2001, URL-22]. It was therefore decided to use the *rapid setting cement* for the remainder of the investigation. OPC was incorporated in selected trials only for comparison, as it also gave good results.

It was observed that workability of the mixtures were slightly altered when cement was incorporated. However, the changes in workability and hence ease of mixing were not severe enough to adversely affect the degree of aggregate coating.

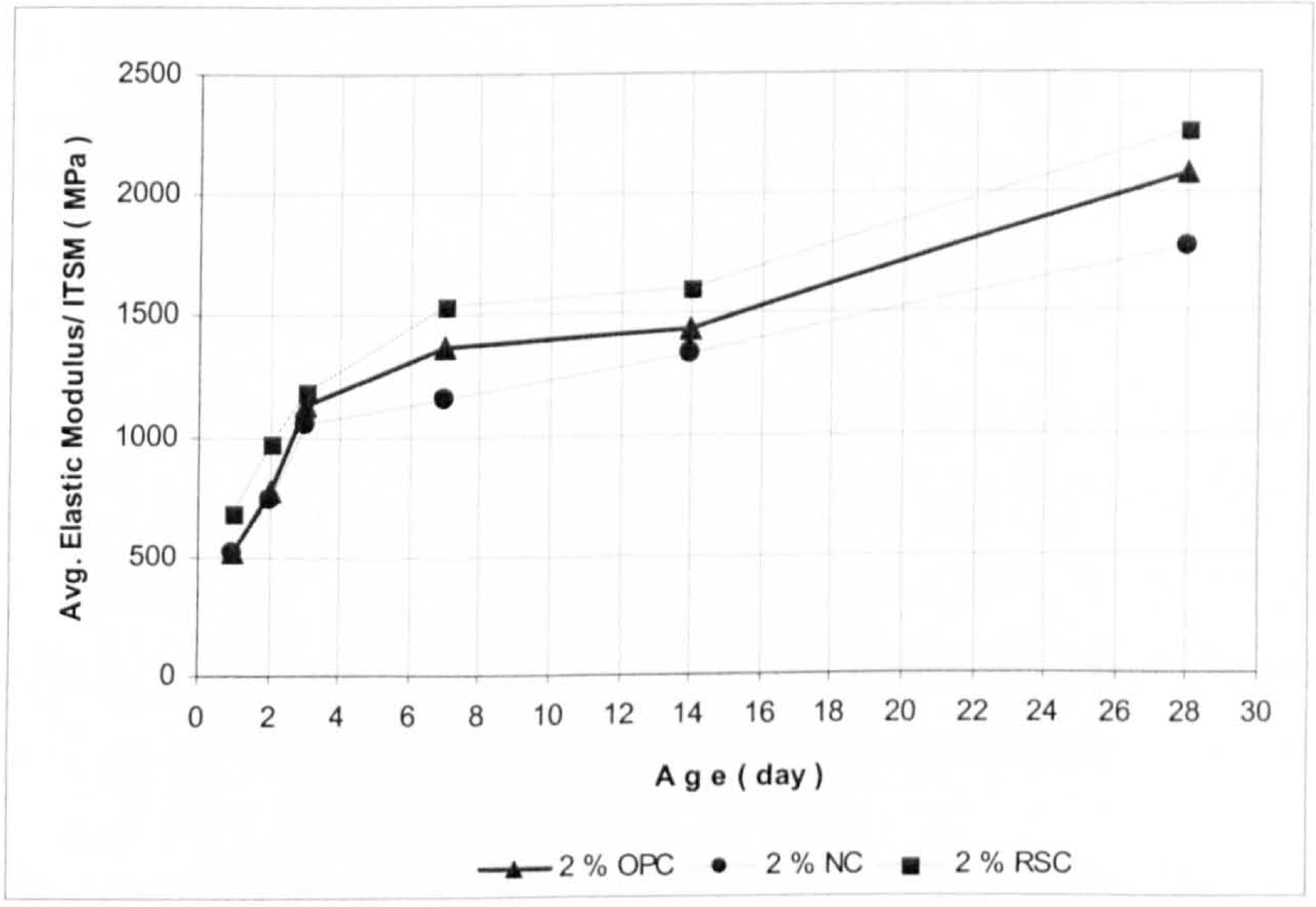


Fig. 9.1 CBEMs (WCI-CM) increase rate of strength incorporating cement, compacted with EHC1.

9.2.4 Mixture Production and Results

The CBEMs shown in Table 9.1 were designed based on the Proposed CBEMs Design Mixture as described in Chapter VIII, Section 8.5. The loose mixtures were prepared with OPWwc = 8% which resulted in a TLC = 15.71% (see Chapter VII, Fig. 7.11). The wet mixtures were air dried at room temperature (24°C), occasionally mixed until the mixture had just loosed with no balling of fines and binder. The loose mixtures were then compacted at compaction level (EHC1).

The results obtained up to this stage of investigation are presented in the following tables.

Table 9.2 Properties of (WC I – RPSg, and WCI-RPS) at full curing. Compaction effort: EHC1.

Type of Mixture	Porosity (%)	ITSM (MPa)
WCI-RPSg with Nynas Emulsion:		
• without Cement	7.8 *	1668
• + 2 % Rapid Setting Cement	8.6 *	2468 *
WCI-RPS with TotalFinaElf Emulsion:		
• without Cement	8.1 *	1318
• + 2 % Rapid Setting Cement	7.7 *	2352 *

* satisfy porosity ITSM target

Table 9.3. Properties of WC I – SA at full curing. Compaction effort: EHC1 (also refer to section 9.2.2).

Type of Mixture	Porosity (%)	ITSM (MPa)
WCI-SA with Nynas Emulsion:		
• without Cement	10.9	1412
• + 2 % Rapid Setting Cement	11.8	2548 *
WCI-SA with TotalFinaElf Emulsion:		
• without Cement	11.2	1428
• + 2 % Rapid Setting Cement	11.9	2511 *

* satisfy ITSM target

The size of the Synthetic Aggregate used was 10 to 5mm. The amount used was 25.65% by mass of total aggregates (to substitute by volume all the limestone coarse aggregates of the same size). Although the porosity values in Table 9.3 did meet Nikolaides Specification (6-12 %), they were nevertheless rather high. This was because the synthetic aggregates themselves were quite porous, see Table 4.11 in Chapter IV.

Table 9.4 Average Properties of the Control Mixture (WCI-CM) at full curing.

Type of Mixture	Porosity (%)	ITSM (MPa)
WCI-CM with Nynas Emulsion:		
• without Cement	9.2 *	1595
• + 2 % OPC	8.7 *	2581 *
• + 2 % Rapid Setting Cement	8.5 *	2593 *
WCI-CM with TotalFinaElf Emulsion:		
• without Cement	8.2 *	1346
• + 2 % OPC	7.8 *	2327 *
• + 2 % Rapid Setting Cement	7.9 *	2696 *

* satisfy ITSM targets

It can be seen from Table 9.4 that the ITSM values of the Control Mixture (WCI-CM) with and without cement were comparable to other WC I mixtures listed earlier. Although the ITSM of the WCI-CM without added cement was below the target of

2000 MPa, adding 2% OPC or Rapid Setting Cement guaranteed values in excess of 2000 MPa.

It was observed that the WCI-CM had better binder coating on all the aggregate particles when compared to the other WC I mixtures. It was evident that the *quartzitic fine asphalt sand* had *better affinity* to both emulsion types used in this investigation than the *red porphyry sand*.

9.2.5 Further Experiments

9.2.5.1 Effect of storage time on CBEMs

The CBEMs WCI-RPSg wet *loose mixtures were stored* (after mixing with the emulsion and lightly air drying the mixture to avoid clumping) in a *sealed container* for 0, 3, 6, 24, and 48 hours before compaction. The compacted samples were then fully cured in an oven at 40°C (until a constant mass was achieved). Two types of mixtures were tested: mixtures without cement and mixtures with 2% Rapid Setting Cement (RSC). Test results of *porosity* and *ITSM* values at 20°C of the *fully cured* compacted samples are presented in Figs. 9.2 and 9.3.

As the loose mixtures were lightly air dried before they were stored in a sealed container, it was observed that very little water was squeezed out during compaction.

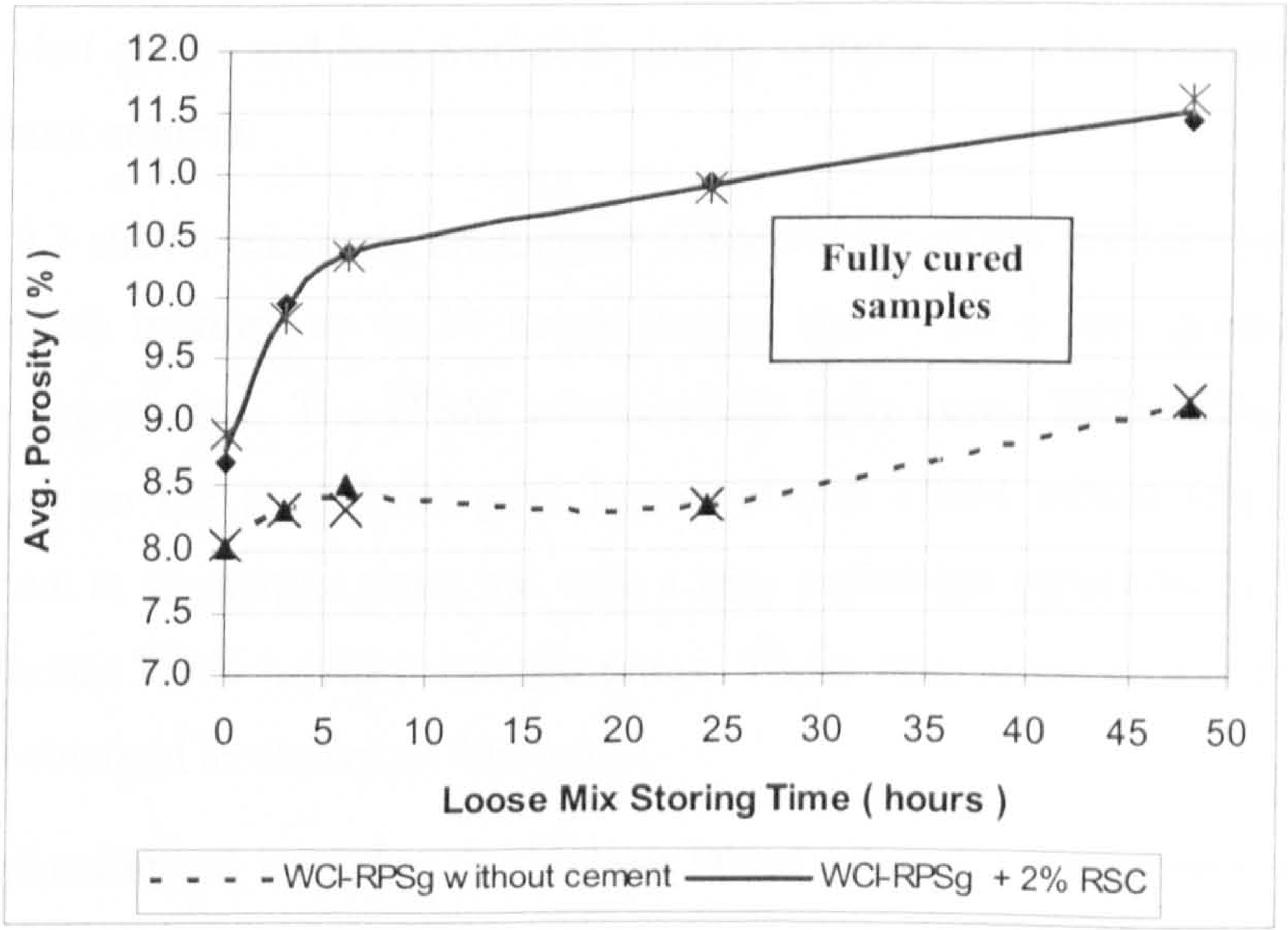


Fig. 9.2 Porosity of CBEMs (WCI-RPSg) v.s. Loose Mixture Storage Time.

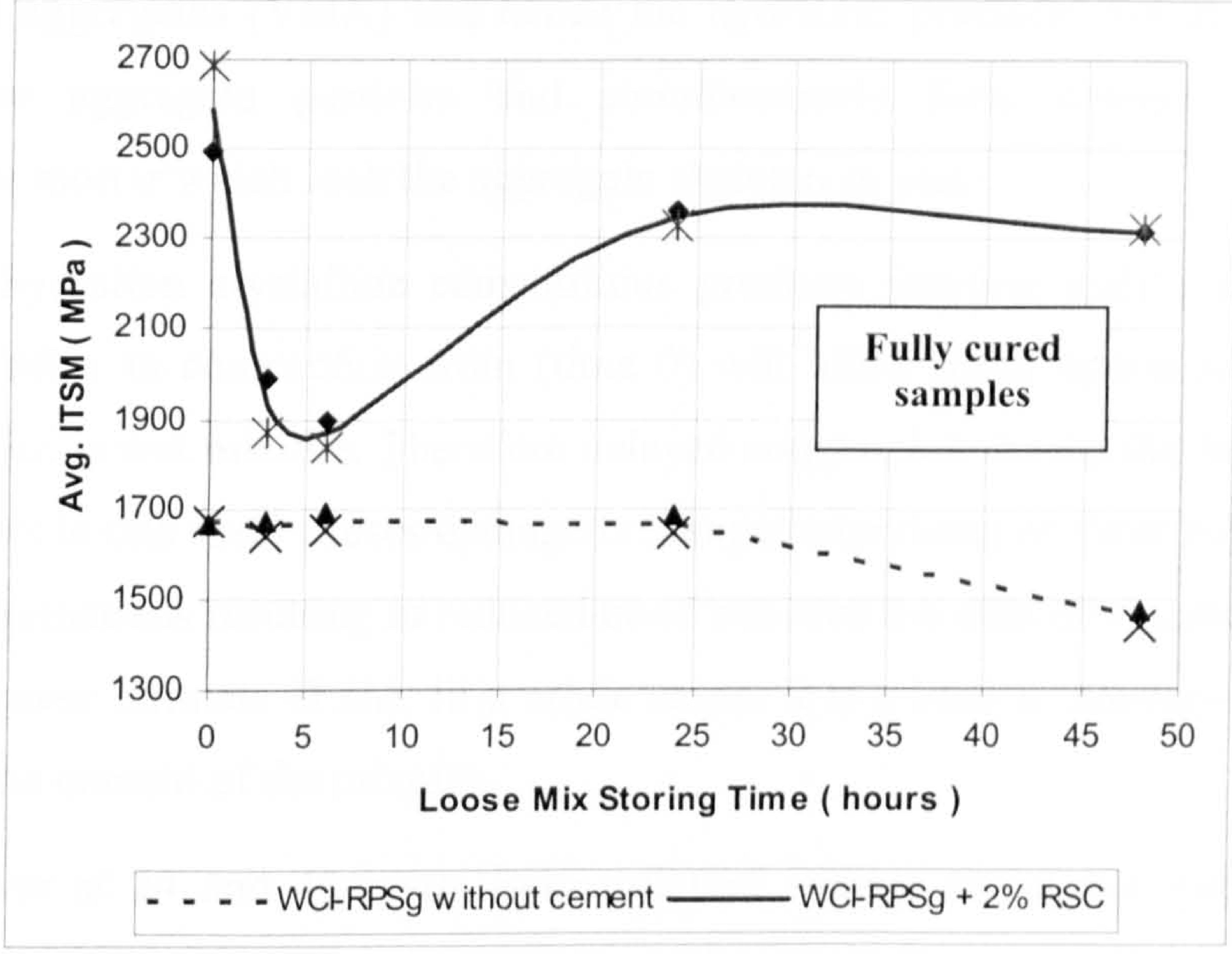


Fig. 9.3 ITSM of CBEMs (WCI-RPSg) vs. Loose Mixture Storage Time.

Fig. 9.2 shows the loose mixtures gradually becoming stiffer and hence less workable during compaction as the emulsion sets with increasing curing time, which resulted in increased porosity with increased storage time. The porosity of the WCI-RPSg + 2% rapid setting cement gave higher values at all storage times of the loose mixture with a smaller difference recorded within the first 6 hours. These higher values are due to the hydration of cement in the presence of moisture causing the loose mixture to be somewhat stiffer and less workable during compaction when compared with the mixture without cement.

Fig. 9.3 shows relatively unchanged ITSM values of the WCI-RPSg (fully cured with no cement) mixture up to 24 hours storage time with a very gradual reduction beyond one day storage. The ITSM values of the fully cured WCI-RPSg + 2% rapid setting cement on the other-hand gave higher overall ITSM values than the mixture without cement at all storage times but with a very noticeable reduction in ITSM values at 3 and 6 hours loose mixture storage times. These tests were repeated and similar results were obtained as shown in the figure.

It is interesting to note that the highest ITSM value for the mixture with cement was obtained when the wet mixture was compacted immediately following the mixing process (time 0). When the mixtures are compacted within very short times from adding in the cement, the cement hydration process would form in the available spaces between the mineral aggregates (VMA) and hence the hydration products would act as bonds between the aggregate particles and simultaneously form wedges of hydrated cement/fines mortar which lock the aggregate skeleton in place.

With hydration crystalline cementitious products forming mainly in the first 6 hours, any delay in compaction from (time 0) will allow more hydration products to form in the loose wet mixture. Therefore delayed compaction during the first few hours (up to 6 hours in this case) causes damage/breakage/pulverizing of these relatively weak crystalline formations resulting in reduced bond between the mineral aggregate particles and hence lower ultimate ITSM. In a crude sense, it is a kind to having increased the fine aggregate content of the mixture.

However at 24 and 48 hours loose mixture storage times, the majority of the cement hydration process would have taken place and the cementitious mortar thus formed becomes hard/tough enough that even if damage is still inflicted on the bonds between the crystalline phase and the mineral aggregates during compaction, less damage to the actual crystalline phase would occur during the compaction process. Since the hardened cement phase occurs (as irregularly shaped wedges) in the interstices between the larger mineral aggregate particles, it is logical that when these mortar wedges are partially broken in to medium sized particles, the increased roughness of the mixture (which was evident from the increased specimen porosities) would provide better interlock and a higher angle of internal friction, and hence a higher ITSM result.

9.2.5.2 Coating Test using the *Newly (third batch) Supplied* Nynas Emulsion

From this stage of the investigation onwards, it was decided to use only the *Nynas emulsion* instead of both the Nynas and the TotalFinaElf emulsions. This was in an attempt to narrow down the scale of the investigation and to guarantee a consistent supply of emulsion.

Samples of the Nynas emulsion were received at three different dates:

The first supply date was:	on 7th December 2000.
The second supply date was:	on 19th June 2001.
The third supply date was:	on 11th October 2001.

The CBEMs (which were produced using the Nynas emulsion) for the experiments described in Chapter VII were from the *first supply*. The experiments within this Chapter IX up to this point of the investigation (i.e. up to section 9.2.6.1) were manufactured using the Nynas emulsion from the *second supply*. Emulsion from the *first supply* was used during the initial 'coating tests' to obtain the optimum pre-*wetting water content (OPWwc)* of the *WCI-RPSg*, and the result obtained was 8% by mass of dry aggregate, which appeared to be rather high.

The samples which were produced with Nynas emulsion from the *second supply*, were pre-wetted with water at 8% OPWwc (based on coating test using emulsion from the first supply). The loose mixtures appeared to be excessively wet (sloppy), furthermore, as the added emulsion set during the mixing process, the changes in workability were not very clear and difficult to distinguish.

When a new (third) supply of the Nynas emulsion was *received and immediately used*, there was a very noticeable change in mixture workability, and as a result *new coating tests* had to be performed on WCI-RPSg, WCI-RPSug, and WCI-CM mixtures with results as shown in Table 9.5. An example of the appearance of the *WCI-RPSg* mixture during the coating test is presented in Fig. 9.4, which shows the difference in terms of degree of coating of the same Nynas emulsion but of different period of supply or age.

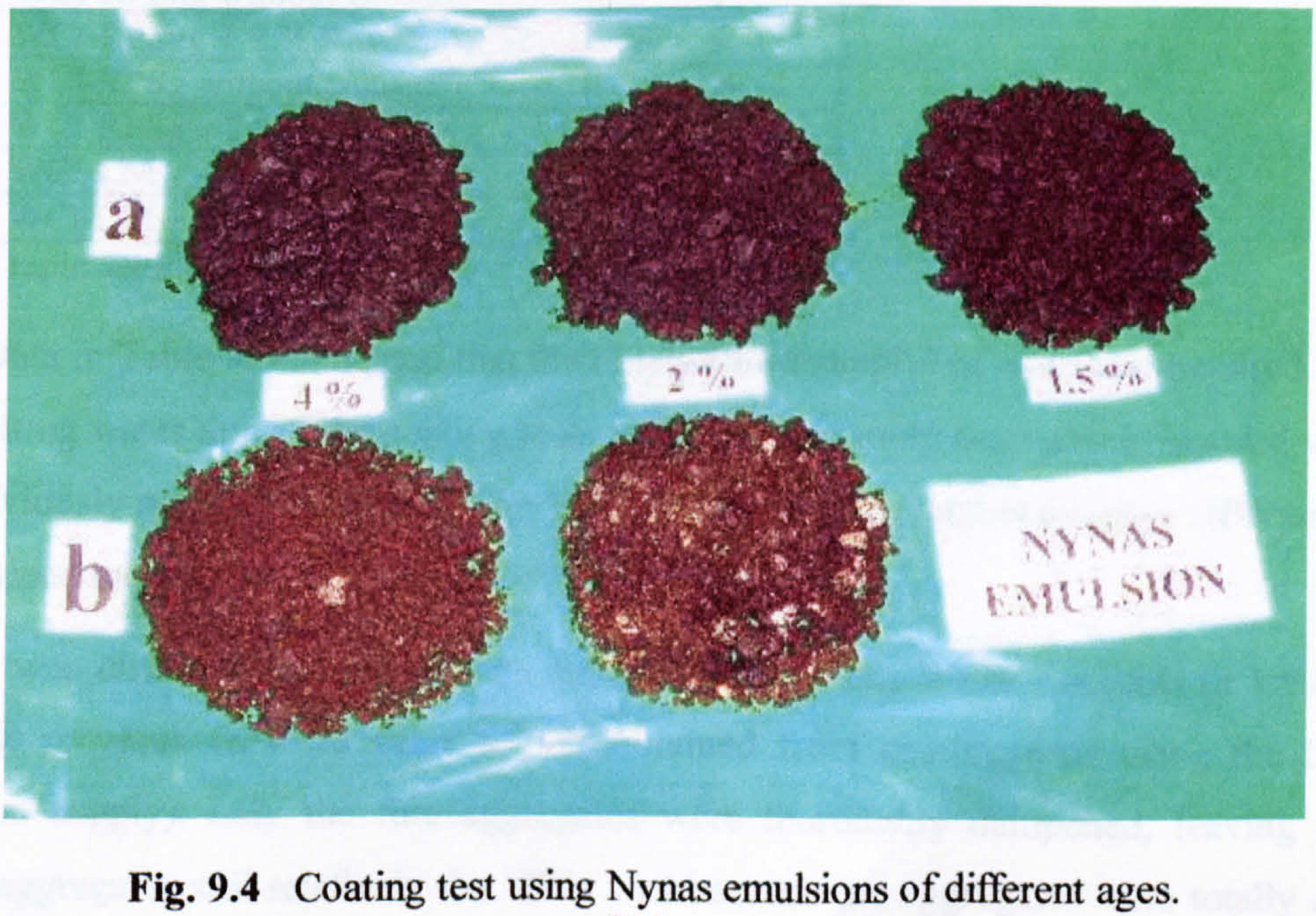


Fig. 9.4 Coating test using Nynas emulsions of different ages.

The coating test results shown in Fig. 9.4 clearly show the difference between the appearance of the CBEMs in the upper row (row a), which were coated with the newly (third) supplied emulsion, and the mixtures in the lower row (row b) which were coated using *the same emulsion but from the 'second supply'. The older (second supply) emulsion was about four months older than the newly (third) supplied emulsion* and although stored in air-tight containers, it had been frequently, though briefly, exposed to open air. The aggregates were pre-wetted with 4% and 2% water for both mixtures in rows 'a' and 'b'. At 2% pre-wetting water, the mixture in row 'b' was rather stiff with inadequate bitumen coating, but the mixture in row 'a' was still slightly on the wet side (sloppy). The mixture in row 'a' was hence pre-wetted with less water, i.e. 1.5%, which thus gave the optimum pre-wetting water content (OPWwc).

When determining the OPWwc of the WCI –RPS graded mixture *without cement*, the first supply of emulsion required 8% OPWwc. The second supply of emulsion may have required about 4% OPWwc or just over 4% (see Fig. 9.4 row 'b'). On the otherhand, the latest (third supply) emulsion required 1.5% OPWwc. All mixtures were mixed with and without incorporating cement and the OPWwc of all mixtures are shown in Table 9.5.

Table 9.5 Optimum Pre-wetting Water Content (OPWwc) using the latest (third supply) Nynas emulsion.

No.	Type of Mixture	OPWwc (%)
1	WCI-RPSug without Cement	4
2	WCI-RPSug + 2 % RSC *	5
3	WCI-RPSg without cement	1.5
4	WCI-RPSg + 2 % RSC *	2.5
5	WCI-CM	1.5
6	WCI-CM + 2 % RSC *	2.5

* RSC: rapid setting cement

Data in Table 9.5 indicated that the newly (third supply) of emulsion required less pre-wetting water during the coating tests and were also more workable compared with the previously supplied emulsion. The WCI-RPSug mixture required higher OPWwc as it contained more fines.

It was observed that at these amounts of Pre-wetting Water Contents (a lot less than the previous OPWwc value of 8% obtained from coating tests using the first emulsion supply), only the fine aggregates were thoroughly dampened, leaving the coarse aggregates still relatively dry. This was because the aggregates were totally dry prior to the wetting tests. This may not be the case on site trials, as materials stored exposed outdoors may already contain water (dampness) and hence may not require additional pre-wetting as is the case in the laboratory. *It is therefore essential that the coating test be preformed using actual materials obtained from site under ambient conditions.*

9.2.5.3 Comparison of Nynas Emulsion Water Contents

Water content tests were carried out using the older and newly supplied Nynas emulsions. From each batch of emulsion, two samples were prepared. A certain amount of emulsion was put into a metal container and then cured in an oven at 40°C until a constant weight was achieved (typically 10 days curing time). The weight before and after curing was measured and the test results are given below:

Table 9.6 Water Content in the emulsion.

Emulsion	Estimated Age on testing (months)	Water Content (%), by weight of emulsion
Second supply of Nynas Emulsion	6 *	37.35
New (third) supply of Nynas Emulsion	2	39.95

* Data not available on how long the emulsion had already been in stock before it was received.

According to the specifications, the water content of the emulsion should have been 40%. It is clear from Table 9.6 that there was some water loss with time, most likely due to exposure. Exposure to open air was inevitable, as the emulsion was be regularly stirred to maintain homogeneity. This loss of water could have affected workability and hence the OPWwc value. Additionally, a slight change in the percentage of the residual bitumen content by weight of total emulsion would have resulted. Changes in other properties of the emulsion may have also occurred due to the *evaporation of the volatile component* of the chemical constituents (surfactants) within the water phase of the system. Generally surfactants do not evaporate, except the 'amines'.

9.2.5.4 Re-production of CBEMs WCI-RPSug, and WCI-RPSg using the Newly Supplied Nynas Emulsion

From a practical point of view, it is very unlikely the Red Porphyry Sand (RPS) would be sieved on site, therefore samples were produced using the latest supply of emulsion (third batch) with both RPS-un-graded (RPSug) and RPS-graded (RPSg) mixtures at OPWwc as shown in Table 9.5. Samples were manufactured for both mixtures with and without cement. The compaction effort was 210 revolutions at 240 kPa. During compaction, less but darker liquid was squeezed out from the samples (when compared with previously supplied emulsion batches). The squeezed liquid contained some amount of fines. The extruded samples appeared to have less coating at the lower end of the samples, particularly so with WCI-RPSug. This appeared to have been caused by the stripping off the emulsion probably caused by the high pore water pressures developed during the compaction process. The results are given in Table 9.7.

Table 9.7 Porosity and ITSM of WCI-RPSug and WCI-RPSg, using the newly (third supply) Nynas Emulsion. Compaction Effort: 210 revs. at 240 kPa.

Type of Mixture	Porosity (%)	ITSM (MPa)
WCI-RPSug without Cement	11.0	2167.6
WCI-RPSug + 2 % Rapid Setting Cement	10.8	4559.5
WCI-RPSg without Cement	11.00	2154.5
WCI-RPSg + 2 % Rapid Setting Cement	10.2	4375.5

With 210 compactive revolutions at 240 kPa axial pressure, *the porosity values lie near the higher end of the target porosity range.* This is mainly because the mixture contained *less total liquid content* (compared with earlier mixtures, e.g. from the second supply emulsion batch, which were manufactured at a very high 8% OPWwc). The porosity of the mixture incorporating cement was slightly less than the mixture without cement. This is likely to be caused by the *higher total liquid content* of the mixture containing cement (see Table 9.7). Even with higher porosities, the ITSM values were somewhat greater than the results obtained using the second supplied batch of Nynas emulsion (see Table 9.4). The mixture without added cement did meet the ITSM target which may be due to the better adhesion property when using the new (third supply) emulsion.

In order to reduce porosity, additional tests were carried out on WCI-RPS-graded, with more specimens produced using a higher compaction effort, i.e. 240 revolutions instead of 210 revolutions with the axial pressure kept at 240 kPa and all at the same OPWwc as in Table 9.5. *Due to lower overall binder coating* on the WCI-RPSug mixture as discussed earlier, no further samples were produced with this mixture. The test results are shown in Table 9.8.

Table 9.8 Porosity and ITSM of WCI-RPSg, using the new (third supply) of NynasEmulsion. Compaction Effort: 240 revs. at 240 kPa.

Type of Mixture	Porosity (%)	ITSM (MPa)
WCI-RPSg without Cement	9.0	2708.4
WCI-RPSg + 2 % Rapid Setting Cement	9.3	6095.5

Table 9.8 shows that using a compaction effort of 240 revs. at 240 kPa, the porosity can be reduced to below the 10% target [MPW Indonesia, 1990]. Comparing the results with those in Table 9.7, it is clear that the ITSM values significantly increased with increased compaction effort. With added cement, a dramatic increase in ITSM values was obtained. *The time required to achieve full curing of CBEMs with a porosity value in the range of 8-9 % was about 21 days at 40°C.*

9.3 Experiments at Extra Heavy Compaction Effort Level 2 (EHC2): 240 revolutions at 240 kPa

9.3.1 Wearing Course (WCI) with other Fine Aggregate variations (WC-FA)

It was observed that the *red porphyry sand had less affinity* to the emulsions used and hence gave a reduced degree of coating in particular when used in the WCI-RPSug. Additionally, reduced coating was caused by *stripping during the compaction process.* Although the ITSM values on the mixtures using the newly supplied emulsion were generally high (comfortably met the 2000 MPa target), *the reduction in the degree of coating* did cause some concern, as it can result in long term durability problems.

In addition to the aforementioned reasons, the availability of the red porphyry sand (RPS) is limited. Its production depends on colored aggregate demand or for chipping applications. Therefore it became logical that combinations of the RPS with other sand types e.g. quartzitic asphalt sand should be tested.

New coating tests were carried out on three ratios of fine aggregates (FA) as shown in Table 9.9, in order *to improve the degree of coating.* For reducing the porosity, the compaction effort was also increased from 210 to 240 revolutions with the vertical pressure maintained at 240 kPa: Extra Heavy Compaction Effort Level 2 (EHC2).

Table 9.9 Properties of WCI mixtures with variations in proportions of fine aggregates (WC-FA) at full curing.

Mixture Type	Composition of Fine Aggregates	OPWwc (%)	Porosity (%)	ITSM (MPa)
Without cement				
WC-FA1	50% RPS * + 50% Asphalt Sand *	2	10.7	2138.9
WC-FA2	60% RPS * + 40% Asphalt Sand *	3	10.6	2276.2
WC-FA3	65% RPS ** + 35% Asphalt Sand *	2	10.8	2727.1
With 2% added Rapid Setting Cement				
WC-FA1C	50% RPS * + 50% Asphalt Sand *	3	9.1	5841.9
WC-FA2C	60% RPS * + 40% Asphalt Sand *	4	9.2	5973.3
WC-FA3C	65% RPS ** + 35% Asphalt Sand *	3	9.6	6117.7

Notes: RPS = red porphyry sand; * all passing 2.36mm; ** passing 2.36mm retained 0.3mm to reduce the amount of RPS fines.

The WC-FA1 and WC-FA2 mixtures contained fine aggregates (all passing 2.36mm), meanwhile the WC-FA3 mixture contained both asphalt sand (all passing 2.36mm) and RPS (only the fraction passing 2.36mm and retained on 0.3mm was used in this mixture). The use of fine aggregates in the WC-FA3 mixture was meant to eliminate the RPS fraction passing 0.3mm, which was observed to coat less well with binder. Practically it was also simpler and faster in the laboratory to sieve RPS that retains on 0.3mm rather than material that passes 0.3 mm. The proportion of fine aggregates in the WC-FA3 mixture was based on simple blending in order to keep the gradation very close to WCI. The coarse aggregates were properly graded as mentioned in Section 9.2.1.

As a result of the changes in the fine aggregate types and proportions, there were slight changes in the gradation of the fines portion of WC-FA1 and WC-FA2 mixtures as shown on Fig.9.5. Meanwhile the gradation lines of the coarse aggregate fractions remained unchanged.

As shown in Table 9.9, the porosity values of WC-FA mixtures incorporating 2% RSC were slightly lower than the mixtures without cement. The reduction in porosity is due to the cement particles filling up more of the void space available in the mixture, increasing packing, and therefore reducing porosity.

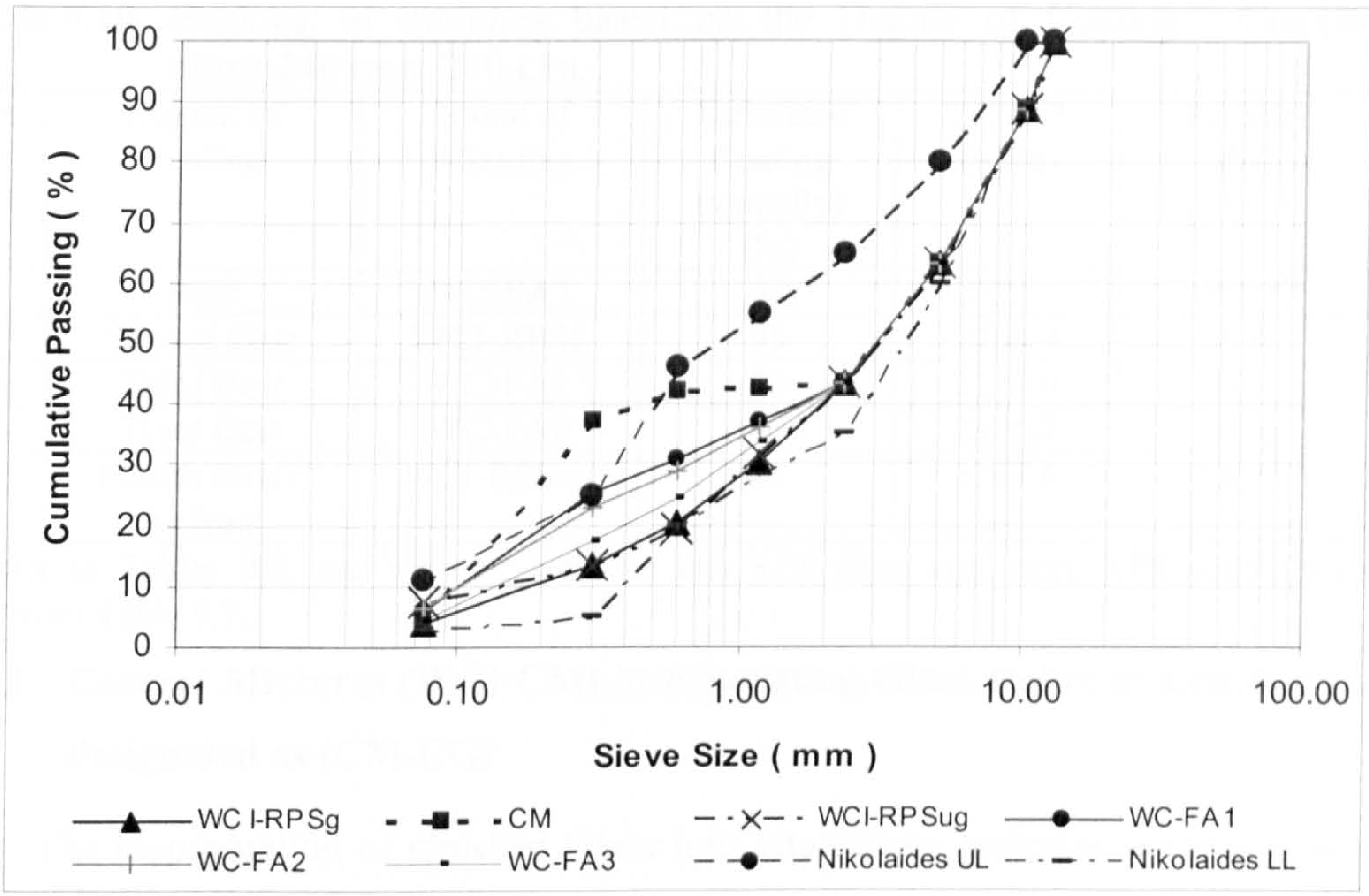


Fig. 9.5 Wearing course gradation, with variations in the fines fractions.

Referring again to Table 9.9, it can be seen that the stiffness (ITSM) of the mixtures that incorporate 2 % rapid setting cement increases as the porosity increases. This is due to the increased friction/interlock generated by the more angular RPS coarse fractions and less RPS fine fractions contains in the mixtures. The mixtures that contain more RPS fine fractions have less degree of coating as had been found that the RPS fine fractions have less affinity with the bitumen emulsion.

In terms of binder coating, all mixtures in shown in Table 9.9 had better coating when compared with WCI-RPSug. The *degree of coating* of the compacted samples was *visually assessed* which made it rather difficult to quantify. However with maximum care, the coating can be ranked as shown in Table 9.10, included in the Table are also the WCI-RPSg and WCI-RPSug mixtures and their ITSM values for comparison. It appeared that binder *affinity and the amount of fines in the mixture* played a major role in binder coating.

However, there is no general agreement yet available regarding the ideal amount of fines (including filler) to be used in cold bituminous mixtures. Further trials on binder coating and production of samples will give a clearer answer.

Table 9.10 Ranking of mixtures based on the Degree of Coating. Compaction effort:240 revs, 240 kPa.

No.	Degree of Coating	Name of Mixtures	Estimated Coating (visually) (%)	ITSM * (MPa)	Porosity (%)
1	Best	WC-FA3	98	2727.1	10.8 **
2	Second Best	WCI-RPSg	95	2708.4	9.0
3a	Third Best	WC-FA1	90	2138.9	10.7
3b	Third Best	WC-FA2	90	2276.2	10.6
4	Fourth Best / the least	WCI-RPSug	85	2167.6	11.0 ***

* refer to Tables 9.8, and 9.9 ** contains less RPS fines and more RPS coarse fractions
*** from Table 9.7.

9.3.2 Control Mixtures (WCI-CM) incorporating Glass Cullet or Crushed Glass, designated as (CM-CG)

The incorporation of Crushed Glass into bituminous mixtures is not a new idea. Replacement by up to 30% by mass of aggregates into *hot mixtures* (Glasphalt) had already been successfully tested on full-scale trials in the UK with encouraging performance results (Nicholls & Lay, 2002). In this investigation crushed glass was also incorporated into the *control mixture* in the following amounts: 2.5%, 5%, 10%, 15% and 20% substitution by volume. The gradation of the crushed glass consisted of 60% of 5-2.36mm, and 40% passing 2.36mm. Initially, the pre-wetting water content was kept the same as the control mixture at 1.5% (see also Table 9.5). But it was observed that when more crushed glass was incorporated, the mixtures became a bit sloppier during mixing. Wet (sloppy) mixtures were air dried *until the mixture had just adequate* for compaction *(still damp and loose)*. The test results are given in Table 9.11.

Table 9.11 Properties of the Control Mixture incorporating Crushed Glass (CM-CG) at full curing using the new (third) supply emulsion. Compaction effort: 240 revs. at 240 kPa.

No	Mixture Type	OPWwc (%)	Porosity (%)	ITSM (MPa)
Without cement				
1	CM without CG	1.5	7.2	2257.1
With 2% added Rapid Setting Cement				
2	CM without CG	2.5	7.3	4891.3
3	CM with 2.5% CG	2.5	7.4	4797.1
4	CM with 5% CG	2.5	7.5	4476.1
5	CM with 10% CG	2.5	8.0	4274.9
6	CM with 15% CG	2.5	8.2	3961.7
7	CM with 20% CG	2.5	8.4	3497.7

Note: CM = control mixture ; CG = crushed glass

Test results in Table 9.11 show consistent trends with porosity values slightly increasing as the crushed glass content was increased. This was likely to be the result of the constant OPWwc used for all mixtures (which was meant to keep the number of variables to a minimum). At the higher crushed glass content, there would have been more total liquid content freely available in the mixture to resist compaction, as glass will absorb almost no water. The porosity results influenced the stiffness as demonstrated in the ITSM results.

This data show once again that when using the newly supplied emulsion, the ITSM values obtained were higher than ITSM values of the samples manufactured earlier using the (second) supply of emulsion (as shown in Table 9.4) even at similar porosities.

9.3.2.1 Further experiments

After conducting trials to incorporate *crushed glass* by up to 20%, which showed very good results (all mixtures met porosity and ITSM targets), additional trials were carried out to incorporate 30% crushed glass into the CBEMs. *The optimum pre-wetting water content (OPWwc) was reduced slightly (from 2.5% to 2%) in order to maintain workability, otherwise the mixture would become too sloppy as the proportion of 'crushed glass' was increased.* The test results are shown in Table 9.12.

Trials were made to evaluate the *adequacy of the compaction level* (in terms of compacted porosity) at different *total liquid contents at compaction.* All mixtures *with 30% crushed glass* were pre-wetted with 2% OPWwc and were then air dried and compacted at three different levels of total liquid content (TLC) at compaction. The first mixture was manufactured and compacted soon after mixing (Table 9.12, No. 2). The second mixture was air dried until the mixture weight was reduced by a quarter of the OPWwc weight (Table 9.12, No. 3). The last mixture was dried until the weight was reduced by half of the OPWwc weight (Table 9.12, No.4). With these TLC adjustments at compaction, the porosity results were satisfactorily met. This small trial serves to reinforce the feasibility/viability of the proposed new CBEMs design procedure that had been covered earlier in Section 8.5, Chapter VIII.

As the mixtures incorporating 2% RSC appeared to be satisfactorily strong, additional trials were therefore carried out with 1% RSC at 1.5% OPWwc, which also gave satisfactory results as shown in Table 9.12.

Table 9.12 Properties of the Control Mixture incorporating Crushed Glass (CM-CG) at full curing. Compaction Effort: 240 revs. at 240 kPa.

No.	Mixture Type	OPWwc (%)	Porosity (%)	ITSM (MPa)	Notes
1	CM with 30% CG without cement	2	6.4	2475.2	Mixture rather sloppy, subse-quently air dried using a hair drier until just loose and dry enough for compaction.
2	CM with 30% CG + 2% RSC	2	8.1	4339.8	Sufficient workability, Mixture compacted soon after mixing.
3	CM with 30% CG + 2% RSC	2	7.8	4652.4	Mixture compacted following air drying to remove a quarter of the initially added OPWwc.
4	CM with 30% CG + 2% RSC	2	7.2	4851.8	Compacted following air drying to remove half the initially added OPWwc.
5	CM with 30% CG + 1% RSC	1.5	7.3	3290.7	Sufficient workability, with a slight air drying before compaction until just loose enough.

Note: CM = Control Mixture; CG = Crushed Glass, RSC = rapid setting cement.

For comparison, tests carried out on cores obtained from hot Glasphalt Mixtures incorporating 30% crushed glass (using a 100 pen. bitumen), laid at a trial site in Milton Keynes by RMC Aggregates Ltd. showed an average ITSM value of 1900 MPa. Meanwhile, on the same trial, the control mixture (without crushed glass) gave 2200 MPa. The average porosity for the Glasphalt and the control mixture were 4.9% and 4.7% respectively, [Nicholls & Lay, 2002].

Referring to the results in Table 9.12, the porosity of the CBEMs were somewhat higher than the hot mixture Glassphalt. On the other hand, the ITSM values of CBEMs with 30% crushed glass at *full curing* were comparable (somewhat stiffer) than Glasphalt, even without the incorporation of cement.

Due to its amorphous nature and its high silica content, when *finely ground,* glass possesses *pozzolanic properties* (Section 4.3.4, Chapter IV). The crushed glass used within this investigation contained about 0.85% passing 0.075mm (filler fraction). This might have further contributed to the improved strength of the CBEMs incorporating crushed glass.

9.3.3 The use of WC-FA2 gradation for further experiments (using the third supply of emulsion)

Considering the satisfactory test results obtained from the WC-FA2 mixture, (Tables 9.9, 9.10) where the composition of the fine aggregate fraction in this mixture (60% red porphyry sand-unsieved + 40% asphalt sand-unsieved) was considered practicable and it was therefore chosen for the further experiments.

9.3.3.1 Pre-coating the WC-FA2 mixture with hot 100 pen bitumen

The aim of this part of the investigation was *to eliminate the need for any pre-wetting water (PWwc) in CBEMs during the mixing stage.* The coarse and fine aggregate fractions of the WC-FA2 gradation were separately pre-coated with 100 pen. hot bitumen. Trials were initially carried out to obtain the correct amount of bitumen required, to avoid excessive binder drainage. The bitumen temperature selected for coating was in the range (140-160°C). The coarse aggregate (CA) component was proportioned separately, pre-heated and subsequently coated with 1.5% by weight using hot 100 pen. bitumen to give an even binder coating. The fine aggregate and filler (FA and F) fractions were also proportioned, combined, pre-heated up to 200°C and then coated with 3% hot bitumen (100pen.). The procedure of coating the fines was found to give good coating and sufficient workability. Soon after the coated fines had cooled down, although they were initially lightly stuck together, it was relatively easy to separate/loosen up the particles.

1. Mixture 1: Pre-coated WC-FA2 with 6% Residual Bitumen Content-RBC

Following pre-coating with hot bitumen, and in order to maintain the total residual bitumen content (RBC) of the WC-FA2 mixture at 6%, the usual amount of Nynas Emulsion added was thus reduced. Note: the value of RBC does not include any pre-wetting water, it purely refers to the actual amount of residual bitumen in the mixture when all other components are ignored. In this trial, it was observed that the mixture was **very rich in bitumen**. This was likely because the pre-coated aggregates absorbed almost no emulsion.

2. **Mixture 2: same as Mixture 1 but with less RBC**

As Mixture 1 was found to be too rich in bitumen, the added emulsion was reduced to give 5% total RBC including the pre-coated hot bitumen (i.e. RBC = 5% instead of 6% as in Mixture 1). In this case, it was found that the workability and coating were both satisfactory.

3. **Mixture 3: Mixture 1 + 2% Rapid Setting Cement without Pre-Wetting water**

This mixture contained 2% added Rapid Setting Cement (RSC) and since the added cement would require extra emulsion (due to the increased fines content in the mixture, otherwise the mixture would become less workable as the emulsion would quickly adhere to the fines) the RBC value was selected at 6% as in Mixture 1. The loose mixture was rather dry and no liquid was squeezed out during compaction. However the mixture appeared to be well coated with binder.

4. **Mixture 4: as Mixture 3 but with 6.5% RBC (0.5% extra RBC)**

The aim of this trial was *to improve coating and workability* of the previous mixture. Unfortunately, it was observed that the 0.5% extra RBC gave very little improvement. A very small amount of liquid was expelled during compaction.

5. **Mixture 5: as Mixture 3 with 1% PWwc**

With a similar objective of improving coating and workability, Mixture 5 also gave adequate coating and workability. In general, assessment of workability in this investigation was rather subjective, as it depended on the mixing method used, i.e. manual or with a mechanical mixer. CBEMs mixing workability can obviously be improved by increasing the PWwc, but the mixtures have to be subsequently air dried to a convenient state for optimum compaction. Results of the above trials are shown in Table 9.13.

Table 9.13 Porosity and ITSM at 20°C of WC-FA2 with pre-coated aggregates. Compaction effort for all mixtures: 240 revs. at 240 kPa.

Mixture Designation	Porosity (%)	ITSM (MPa)	Notes
Mixture 1	7.3	2056.2	6% RBC, without cement, without PWwc
Mixture 2	7.7	2710.6	5% RBC without cement, without PWwc
Mixture 3	7.5	4109.7	6% RBC + 2% RSC without PWwc
Mixture 4	7.2	3822.5	6.5% RBC + 2% RSC without PWwc
Mixture 5	7.6	3979.7	6% RBC + 2% RSC + 1% PWwc

These trials were very interesting, as they indicated several advantages of aggregate pre-coating with hot bitumen as follows:

- Pre-coating allows a reduction in the amount of pre-wetting water content, as the coated aggregates absorb much less water or liquid from the emulsion.
- Pre-coating eliminates the problem of variability in the 'affinity' between the aggregates and the emulsion (which depends on aggregate type), and which obviously affects coating.
- Pre-coating allows for more consistent workability, as depending on the actual gradation, the coated aggregates will require a more definite amount of pre-wetting water.
- Pre-coating results in a reduced total requirement for 'bitumen emulsion', as demonstrated earlier by the properties of Mixture 2, in Table 9.12.

On the negative side, from an environmental point of view, such a technique will inevitably require more heat energy as a result of the pre-coating process. In Table 9.13, Mixture 2 gave a higher ITSM than Mixture 1, as it contained lower (but the more appropriate amount) of RBC. Meanwhile Mixtures 3, 4 and 5 were generally comparable, with Porosity and ITSM values in close agreement.

9.3.4 WC-FA2 samples for outdoor curing (using the third supply of emulsion)

Samples of mixture WC-FA2 with 2% Rapid setting Cement (RSC) and without any cement were manufactured and cured in out-door exposure conditions (rain and real temperature profiles) in order to better simulate site conditions. This was carried out to evaluate the mixture's 'rate of strength gain' with time. Following compaction, the samples were kept in their moulds for 24 hours at 24°C room temperature prior to extrusion. The sides of the samples were then sealed with plastic adhesive tape and the taped samples were then placed on a flat metallic surface outdoor (on the roof of the School of Civil Engineering building, Leeds University), as shown in Fig. 9.6. This treatment was aimed to simulate site conditions where the evaporation of volatiles will predominantly occur through the surface of the mixture.

Fig. 9.6 Outdoor curing of CBEM samples.

Outdoor curing began on the 16th February 2002, with an average outdoor temperature of 10°C [Washington Post, 2002, URL-23]. Rain, which was a very frequent occurrence during the specimens curing period resulted in a slight wear of the surfaces of the samples. However the samples remained intact and in a good shape. At regular intervals, two samples were tested. Contrary to British Standard recommendations, each sample was tested for ITSM at 20°C only once, in order to avoid damage (as had been experienced with earlier mixtures) when a sample is subsequently tested at a later age. The samples (were) tested at ages: 1, 2, 4, 6, 8, 10, 12 and 24 weeks (i.e. 6 months). The rate of strength gain (ITSM at 20°C) of the samples cured outdoors are presented in Fig. 9.7.

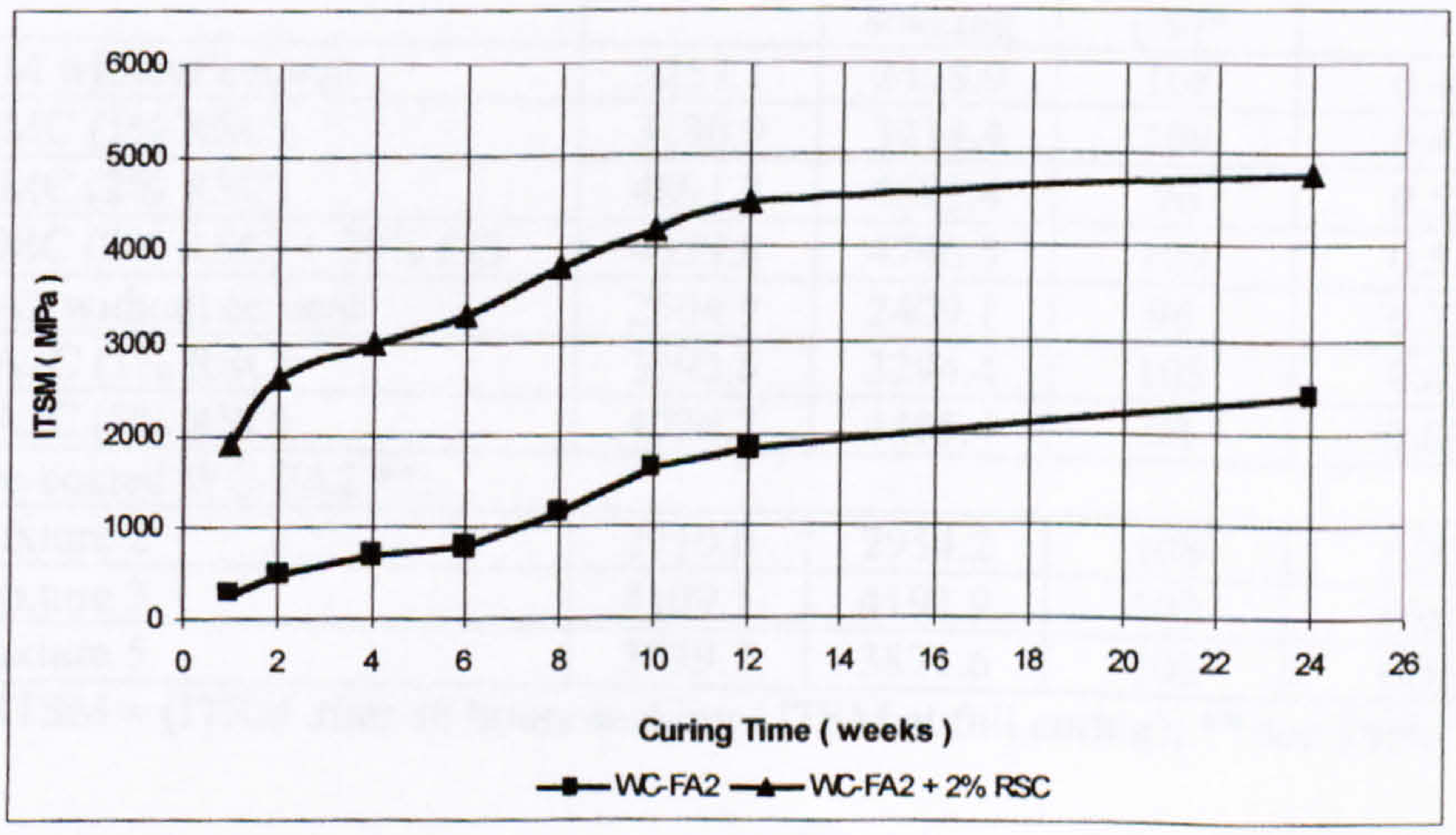


Fig. 9.7 Rate of strength gain of CBEM samples cured outdoors.

As shown in Fig. 9.7, the samples without cement gained strength slowly with time. The ITSM target of 2000 MPa was only achieved after an estimated 16 weeks of outdoor curing. On the other hand, *samples with 2% RSC gave a greatly improved performance, since they required only 2 weeks of curing to meet the ITSM target of 2000-2500 MPa.* Stiffness values were not tested beyond 24 weeks, but the trend shown in Fig. 9.7 does indicate a much more gradual but continued gain in stiffness.

In either case, the rate of gain of strength was 'relatively' fast when compared with some CBEM site trials without cement that required much longer curing times: 2 to 24 months [Leech D, 1994]. However, a direct comparison with other full scale investigations is not possible as the mixtures in the field are influenced by various factors, including; compaction level, climatic condition: rainfall, surface drainage, traffic conditions, etc.

9.3.5 Effect of water conditioning on Stiffness of fully cured CBEMs

Selected CBEM samples at full curing condition were soaked for 48 hours in water at room temperature (24°C). Following soaking, the samples were towel surface dried and tested for Water Absorption and ITSM at 20°C. The results are presented in Table 9.14.

Table 9.14 Retained ITSM of CBEMs at 20 °C.

No.	Type of Mixture	ITSM (MPa)			Water Absorption (%)
		at full curing	after 48 hours soaking	Retained ITSM (%)*	
1	CM without cement	2257.1	2428.9	108	0.40
2	CMC (1% RSC)	3130.9	3414.4	109	0.47
3	CMC (2% RSC)	4891.3	4691.4	96	0.52
4	CMC (2% RSC) + 30% CG	4339.8	4746.3	109	0.51
5	FA2 without cement	2504.9	2409.1	96	0.73
6	FA2C (1% RSC)	3193.6	3294.4	103	0.63
7	FA2C (2% RSC)	4776.7	4495.4	94	0.68
8	Pre-coated WC-FA2 **:				
8.1	Mixture 2	2710.6	2954.2	108	0.70
8.2	Mixture 3	4109.7	4191.9	102	0.61
8.3	Mixture 5	3979.7	3871.6	102	0.46

* Retained ITSM = (ITSM after 48 hours soaking / ITSM at full curing); ** see Table 9.12

Table 9.14 shows that all the Retained ITSM results were well above 90%. A number of samples even had Retained ITSM values exceeding 100%. This is probably the result of pore water pressure generated during the ITSM test by the trapped water within the voids when dynamic loading was applied. In general, all the fully cured samples maintained their strengths after 48 hours soaking.

9.3.6 Fatigue Tests and Dynamic Creep Tests

Fatigue tests and dynamic creep tests were carried out on mixtures at full curing condition. The mixture types selected for this part of the investigation were the control mixture and mixture WC-FA2C (the same type of mixture that was subjected to outdoor curing in Section 9.3.4. Mixture compositions are shown in Table 9.15. Fatigue tests were all carried out at 20°C, whilst dynamic creep tests were carried out at 40°C and 60°C.

Table 9.15 Mixture types used for Fatigue and Dynamic Creep Tests.

Type of Mixtures	Code/designation
CM without cement	CM without cement
CM + 2% Rapid Setting Cement	CM + 2% RSC
WC-FA2 (without Cement)	WC-FA2 without Cement
WC-FA2C (with 2% Rapid Setting Cement)	WC-FA2 + 2% RSC

For comparison of fatigue performance, selected types of hot mixtures fatigue test results were added as shown in Table 9.16.

Table 9.16 Selected types of hot mixtures used for comparison.

Type of hot mixtures	Optimum bitumen content (%)	Bitumen grade	Max. agg. size (mm)	Porosity (%)
20mm DBM *	4.7	100 pen.	20	5.1
A.C. **	5.0	100 pen.	19	5.0
HRA ***	5.0	50 pen.	14	3.7
A.C. ***	5.0	50 pen.	14	5.7

* [Read, and Brown, 1994] ** [Nikolaides, A.F, 1997] *** [Suparma, 2001]

9.3.6.1 Fatigue Tests

Fatigue failure of CBEM specimens not containing cement was accompanied by large irrecoverable horizontal deformations, the magnitude of which were very close to the maximum range detectable by the LVDTs. After careful consideration, it was decided to adopt the number of pulses at failure as that which would cause a strain value of 4.4% for the CM and 4.5% for the WC-FA2. Meanwhile the mixtures *incorporating 2% RSC* gave a clear indication of failure, with failure strains of 1.8% for the CM+2% RSC and 2.5% for the WC-FA2+2% RSC.

a. Results

Typical fatigue test results obtained from this investigation are given in Appendices 3.1, 3.2, 3.3, and 3.4. Power trend/regression lines were used to represent the fatigue lines. Fatigue line equations and correlation coefficient values (R^2) are presented in Table 9.17.

Table 9.17 Fatigue Lines Equations and Coefficient of Correlation (R^2).

Mixture	Equation based on Initial Strain	Equation based on number of cycles to failure	ε_i ($\mu\epsilon$) at $N_f = 10^6$ cycles	N_f (cycles) at $\varepsilon = 100\ \mu\epsilon$
CM without cement	$\varepsilon = 4091.0 \times N_f^{-0.3778}$ $R^2 = 0.9916$	$N_f = 3.0 \times 10^9 \varepsilon^{-2.63}$ $R^2 = 0.9916$	22	1.65×10^4
CM + 2% RSC	$\varepsilon = 679.99 \times N_f^{-0.1827}$ $R^2 = 0.9852$	$N_f = 2.0 \times 10^{15} \varepsilon^{-5.39}$ $R^2 = 0.9852$	54	3.32×10^4
WC-FA2 without cement	$\varepsilon = 3152.2 \times N_f^{-0.3397}$ $R^2 = 0.9898$	$N_f = 2.0 \times 10^{10} \varepsilon^{-2.91}$ $R^2 = 0.9898$	29	3.03×10^4
WC-FA2 + 2% RSC	$\varepsilon = 679.37 \times Nf^{-0.1774}$ $R^2 = 0.9899$	$N_f = 7.0 \times 10^{15} \varepsilon^{-5.58}$ $R^2 = 0.9899$	59	4.84×10^4
20mm DBM (100 pen. bitumen)	$\varepsilon = 2532.1 \times N_f^{-0.2406}$ $R^2 = 0.9950$	$N_f = 1.0 \times 10^{14} \varepsilon^{-4.14}$ $R^2 = 0.9950$	91	5.25×10^5
A.C. (100 pen. bitumen)	$\varepsilon = 1638.4 \times N_f^{-0.2567}$ $R^2 = 0.9991$	$N_f = 3.0 \times 10^{12} \varepsilon^{-3.89}$ $R^2 = 0.9991$	47	4.98×10^4
A.C. (50 pen bitumen)	$\varepsilon = 784.58 \times N_f^{-0.1529}$ $R^2 = 0.9221$	$N_f = 6.0 \times 10^{17} \varepsilon^{-6.03}$ $R^2 = 0.9221$	95	5.23×10^5
HRA (50 pen. bitumen)	$\varepsilon = 2399.6 \times N_f^{-0.2574}$ $R^2 = 0.9613$	$N_f = 6.0 \times 10^{12} \varepsilon^{-3.74}$ $R^2 = 0.9613$	69	1.99×10^5

The *coefficient of correlation* R^2 values were well above the required value of 0.90 as presented in Table 9.17.

The fatigue lines based on *initial strains* are presented in Fig. 9.8. *Strain-life relationships* are considered to be *a better basis for the comparison of different mixtures* compared to stress-life relationships, as had been described in Chapter V, Section 5.3.4.

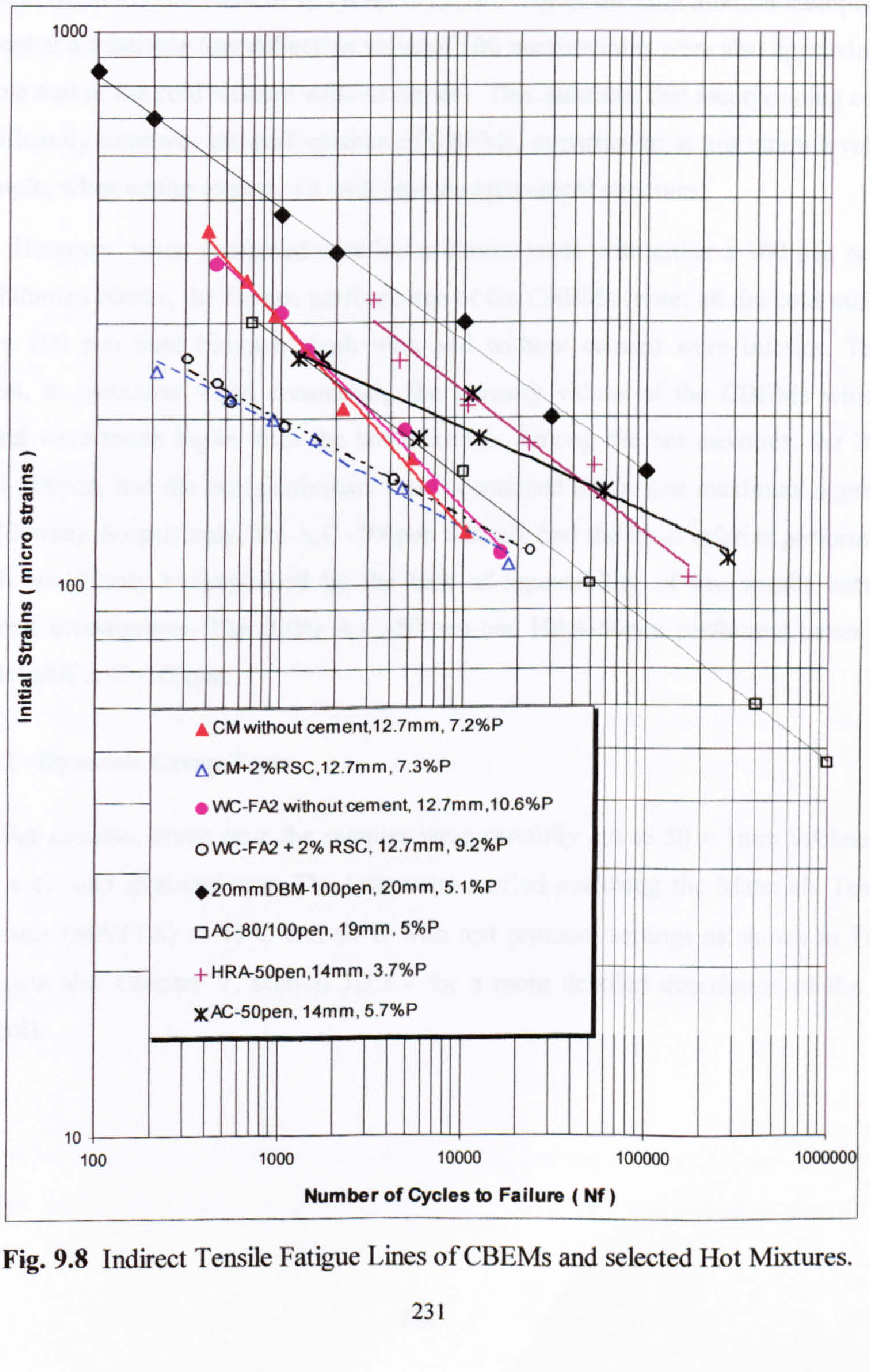


Fig. 9.8 Indirect Tensile Fatigue Lines of CBEMs and selected Hot Mixtures.

b. Discussion

Referring to Table 9.17 and Fig. 9.8, it can be clearly seen that the fatigue lines of the CBEMs+2% RSC were flatter than the cold mixtures without cement. At a projected life to failure of one million cycles, the CBEMs incorporating 2% Rapid Setting Cement (RSC) can withstand about twice the initial strain (deflection) of the CBEMs without cement. Similarly, *the number of cycles to failure* (N_f) of the cold mixture incorporating cement at a relatively low deflection value of 100 micro-strains were also approximately double that of the cold mixture without cement. This indicates that incorporating cement significantly improves the performance of CBEMs, in particular at low strain levels, for example, when acting as part of a well designed pavement structure.

However, when compared with hot mixtures made with either a 100 pen or a 50 pen bitumen binder, the fatigue performance of the CBEMs (note: all the cold mixtures had a 100 pen base bitumen) both with and without cement were inferior. This is logical, in particular when considering the porosity values of the CBEMs which in general were much higher than the hot mixtures. Among the hot mixtures, the 20mm DBM-100pen, had the best performance as it contained the largest maximum aggregate size (20mm). Surprisingly, the A.C.-100pen mixture had the most inferior performance which could only be explained by the lack of repeatability of test results between different investigators. The stiffer A.C.-50 pen and HRA-50pen performed better than the less stiff A.C.-100pen.

9.3.6.2 Dynamic Creep Tests

For *dynamic creep tests* the samples were carefully cut to 50 ± 1mm thicknesses using a circular diamond saw. The tests were carried out using the Materials Testing Apparatus (MATTA) at 40°C and 60°C with test protocol settings as shown in Table 9.18. (see also Chapter V, section 5.3.3.2 for a more detailed description of the test protocol).

Table 9.18 Setting of the MATTA for Dynamic Creep Test.

Settings	Value	Units
Pulse width	1000	(ms)
Pulse Period	2000	(ms)
Test Termination Strain	100000	microstrains
Terminal Pulse Count	3600	pulses
Conditioning Stress	10	kPa
Test Loading Stress	100	kPa
Conditioning Time	2	minutes
Pre-load Rest Time	1	minutes
Recovery Time	30	minutes

a. Cumulative axial strain

The relationship between cumulative axial strain and time of loading of the CBEM mixtures investigated compared with selected hot mixtures are presented in Fig. 9.9. Testing of CBEM samples at 60°C were carried out only on the mixtures incorporating 2% rapid setting cement. The CBEMs mixtures without cement failed before the end of the test duration (i.e. at 300 pulses for the CM and at 1847 pulses for the WC-FA2).

Fig. 9.9 illustrates that in general the CM and the WC-FA2 incorporating 2% Rapid Setting Cement had significantly lower average cumulative axial strains compared to the cold mixtures without cement. At 40°C these mixtures gave similar cumulative axial strains, i.e. 3349 and 3035 microstrains respectively. At 60°C, the WC-FA2 mixtures gave much lower cumulative axial strains than the CM mixtures. This is because at higher test temperatures, the effect of aggregate shape and angularity on creep deformation becomes more important as the stiffness of the binder reduces significantly. The red porphyry sand incorporated within the WC-FA2 had a rougher surface texture and a relatively more continuous gradation. This gave higher particle friction compared with the more rounded, relatively single sized particles of the asphalt sand in the CM. In general, at lower testing temperatures all mixtures gave lower axial strains.

When incorporating cement, the CBEM deformations were lower than typical hot mixtures (A.C. and HRA), even though the hot mixtures were manufactured with a harder 50 pen grade bitumen. This indicated that the incorporation of cement into CBEMs is vital and can dramatically increase the resistance to creep deformations.

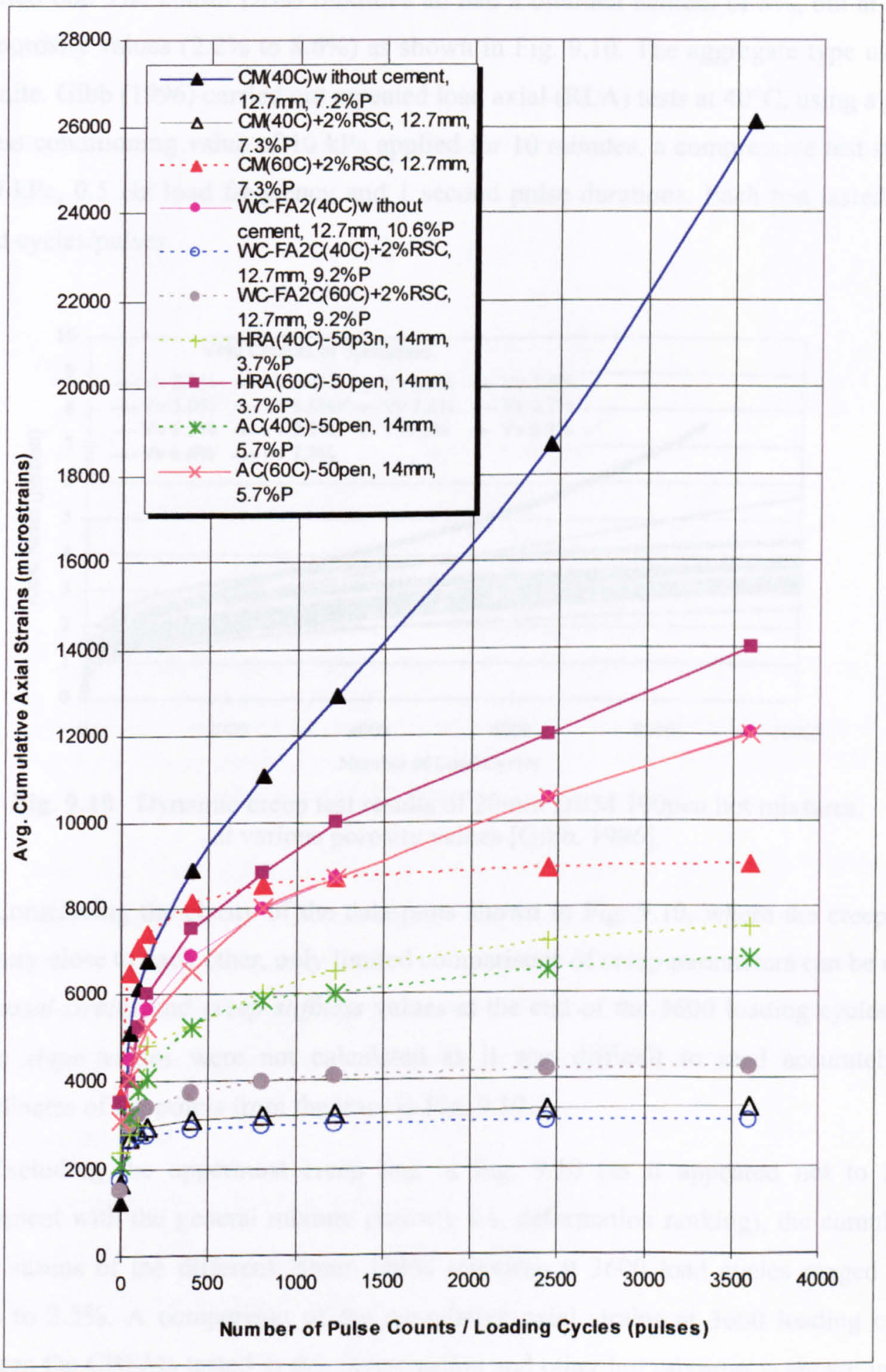


Fig. 9.9 Relationship between Cumulative Axial Strains and Pulse Counts for CBEMs and selected Hot Mixtures.

A comparison of the creep performance of the CBEMs developed in this investigation to 20mm DBM (100pen.) basecourse hot mixtures [Gibb, 1996] was also carried out. The 20mm DBM mixtures all had a bitumen content of 5%, but at a range of porosity values (2.2% to 8.6%) as shown in Fig. 9.10. The aggregate type used was granite. Gibb (1996) carried out repeated load axial (RLA) tests at 40°C, using a pre-test stress conditioning value of 10 kPa applied for 10 minutes, a compressive test stress of 100 kPa, 0.5 Hz load frequency and 1 second pulse durations. Each test lasted 10000 load cycles/pulses.

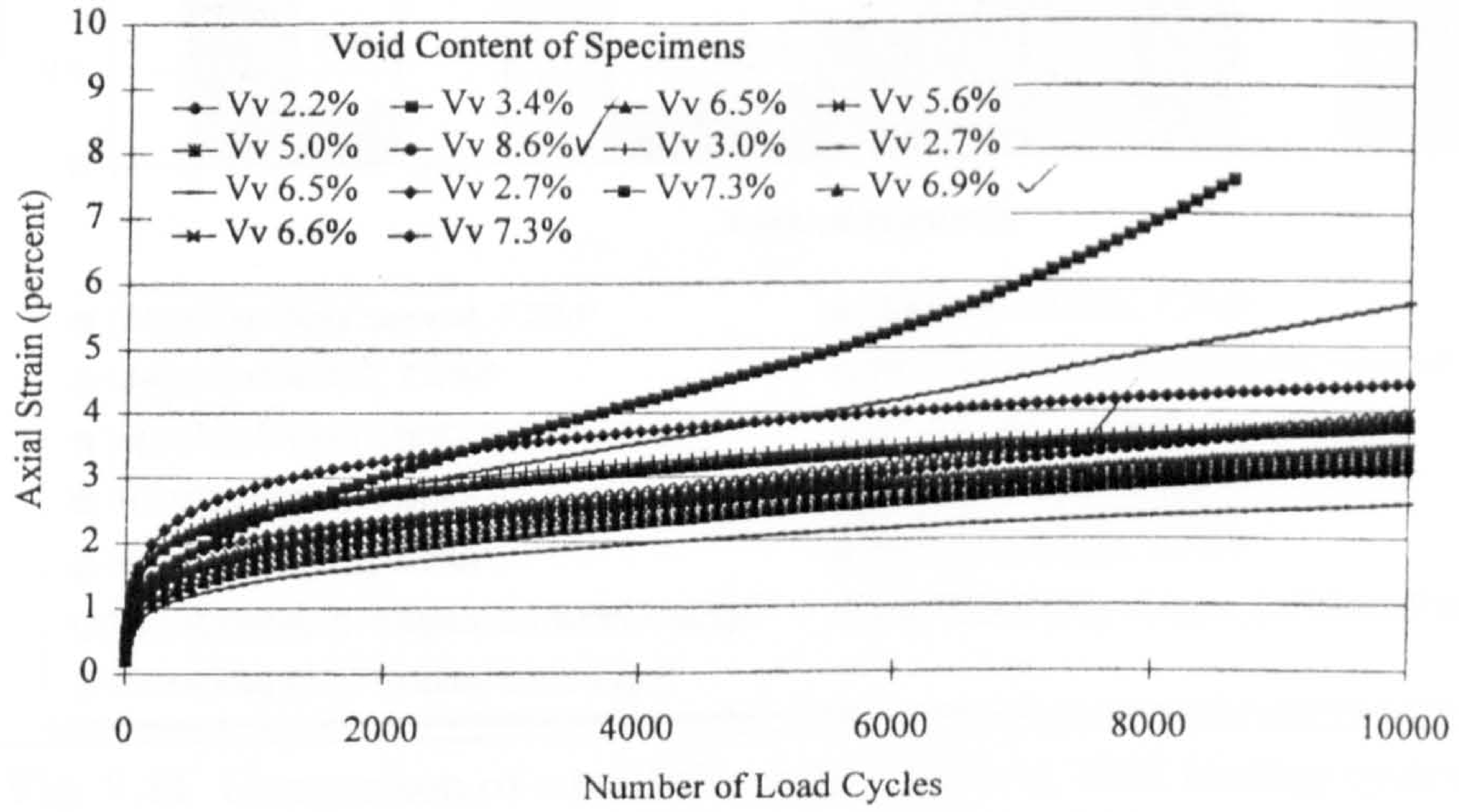


Fig. 9.10 Dynamic creep test results of 20mm DBM 100pen hot mixtures, at various porosity values [Gibb, 1996].

Considering the clarity of the data plots shown in Fig. 9.10, where the creep lines are very close to each other, only limited comparisons of creep parameters can be made, e.g. *axial strains* and *creep stiffness* values at the end of the 3600 loading cycles. The *creep slope* values were not calculated as it was difficult to read accurately the coordinates of the points from the lines in Fig. 9.10.

Excluding the uppermost creep line in Fig. 9.10 (as it appeared not to be in agreement with the general mixture porosity v.s. deformation ranking), the cumulative axial strains of the different 20mm DBM mixtures at 3600 load cycles ranged from 1.8% to 3.5%. A comparison of the cumulative axial strains at 3600 loading cycles between the CBEMs tested in this investigation and other hot mixtures is shown in Fig. 9.11 (refer also to Fig. 9.9).

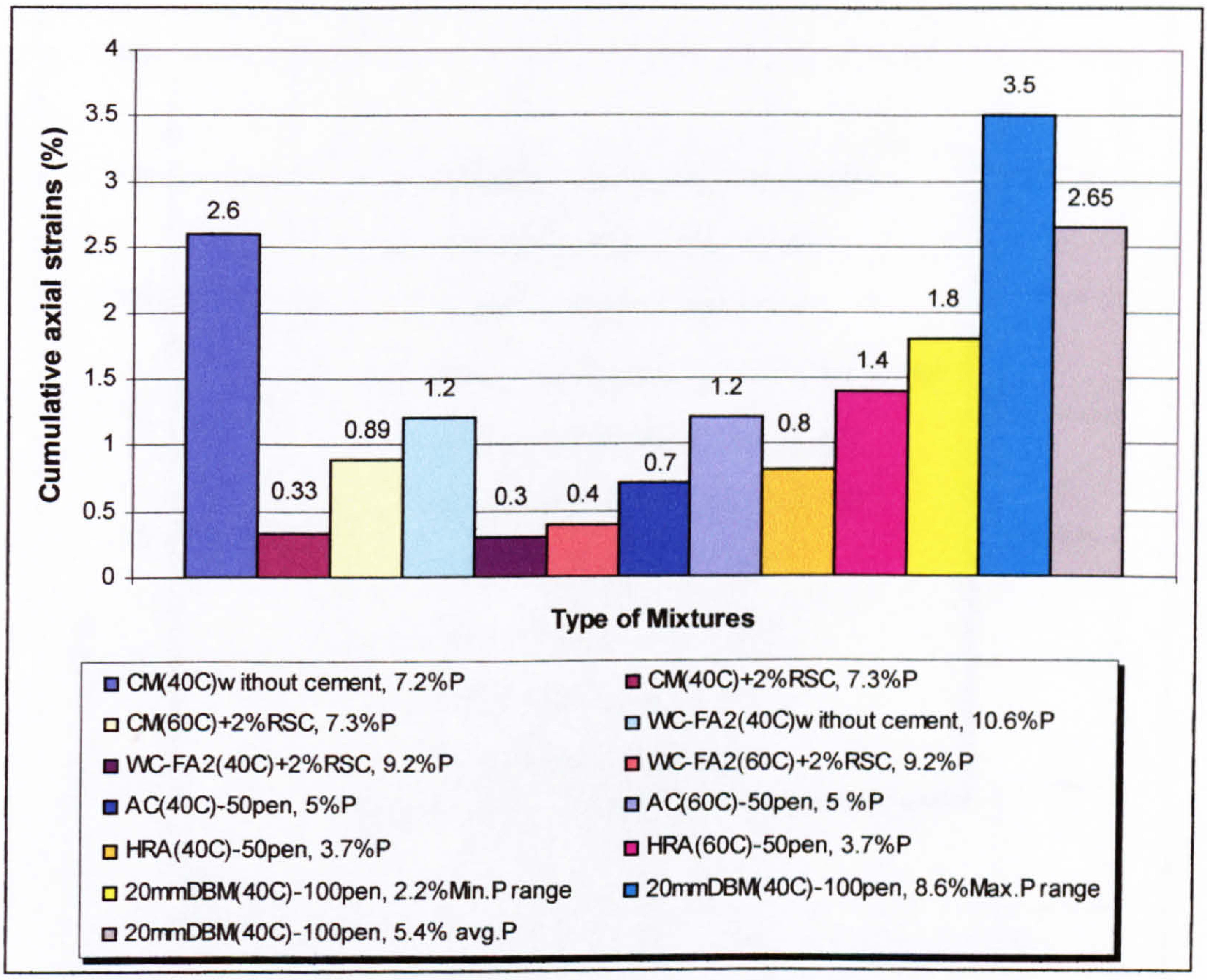


Fig. 9.11 Comparison of cumulative axial strains at 3600 loading cycles/pulses between CBEMs and other hot mixtures.

From Fig. 9.11, when focusing on the CBEMs and the 20mm DBM-100pen hot mixtures (the three bar charts on the far right hand side of the charts), it can be seen that except for the control cold mixture at full curing, all CBEMs can give somewhat lower deformations even at higher porosity values than hot mixtures of similar penetration grade base bitumen, in particular when the CBEMs incorporate cement.

b. Creep Stiffness

The average creep stiffness values of the mixtures are presented in Fig. 9.12. As calculated creep stiffness values are directly proportional to the measured permanent strains, it is only natural that the results agree with those shown earlier in Fig. 9.9.

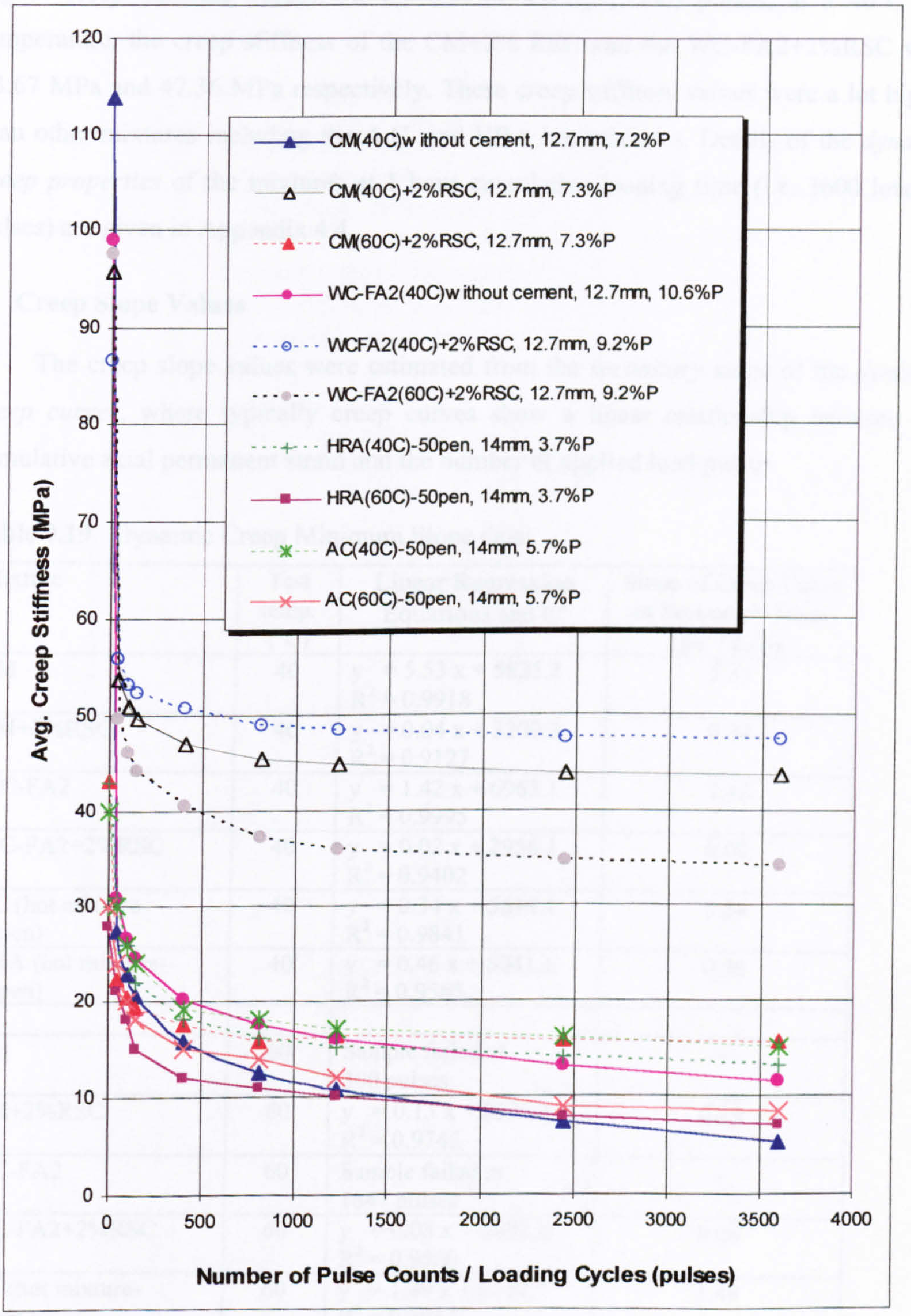


Fig. 9.12 Relationship between Creep Stiffness and Time of Loading for CBEMs and selected Hot Mixtures.

As expected, the CBEMs incorporating 2% rapid setting cement (RSC) show higher creep stiffness values. At the end of testing (3600) pulses, at a 40°C test temperature, the creep stiffness of the CM+2% RSC and the WC-FA2+2%RSC were 43.67 MPa and 47.36 MPa respectively. These creep stiffness values were a lot higher than other mixtures including the A.C. and HRA hot mixtures. Details of the *dynamic creep properties* of the mixtures at 1 hour cumulative loading time (i.e. 3600 loading pulses) are given in Appendix 4.4.

c. Creep Slope Values

The creep slope values were estimated from the *secondary stage* of the *dynamic creep curves*, where typically creep curves show a linear relationship between the cumulative axial permanent strain and the number of applied load pulses.

Table 9.19 Dynamic Creep Minimum Slope data.

Mixture	Test temp. (°C)	Linear Regression Equations and R^2	Slope of Creep Curve at Secondary Stage ($\mu\varepsilon$ / pulse)
CM	40	$y = 5.53x + 5825.2$ $R^2 = 0.9918$	5.53
CM+2%RSC	40	$y = 0.04x + 3200.3$ $R^2 = 0.9127$	0.04
WC-FA2	40	$y = 1.42x + 6963.1$ $R^2 = 0.9995$	1.42
WC-FA2+2%RSC	40	$y = 0.02x + 2956.1$ $R^2 = 0.9402$	0.02
AC (hot mixture-50pen)	40	$y = 0.34x + 5614.1$ $R^2 = 0.9841$	0.34
HRA (hot mixture-50pen)	40	$y = 0.46x + 6041.1$ $R^2 = 0.9565$	0.46
CM	60	Sample failed at 300 pulses	-
CM+2%RSC	60	$y = 0.13x + 8537.5$ $R^2 = 0.9745$	0.13
WC-FA2	60	Sample failed at 1847 pulses	-
WC-FA2+2%RSC	60	$y = 0.08x + 3993.6$ $R^2 = 0.9560$	0.08
AC (hot mixture-50pen)	60	$y = 1.49x + 6752.7$ $R^2 = 0.9957$	1.49
HRA (hot mixture-50pen)	60	$y = 1.58x + 7913.5$ $R^2 = 0.9997$	1.58

For ease of comparison, clarity and simplicity in deriving the dynamic creep slopes, the cumulative axial strains at three intervals were used, namely at 1200, 2400, and 3600 pulses.

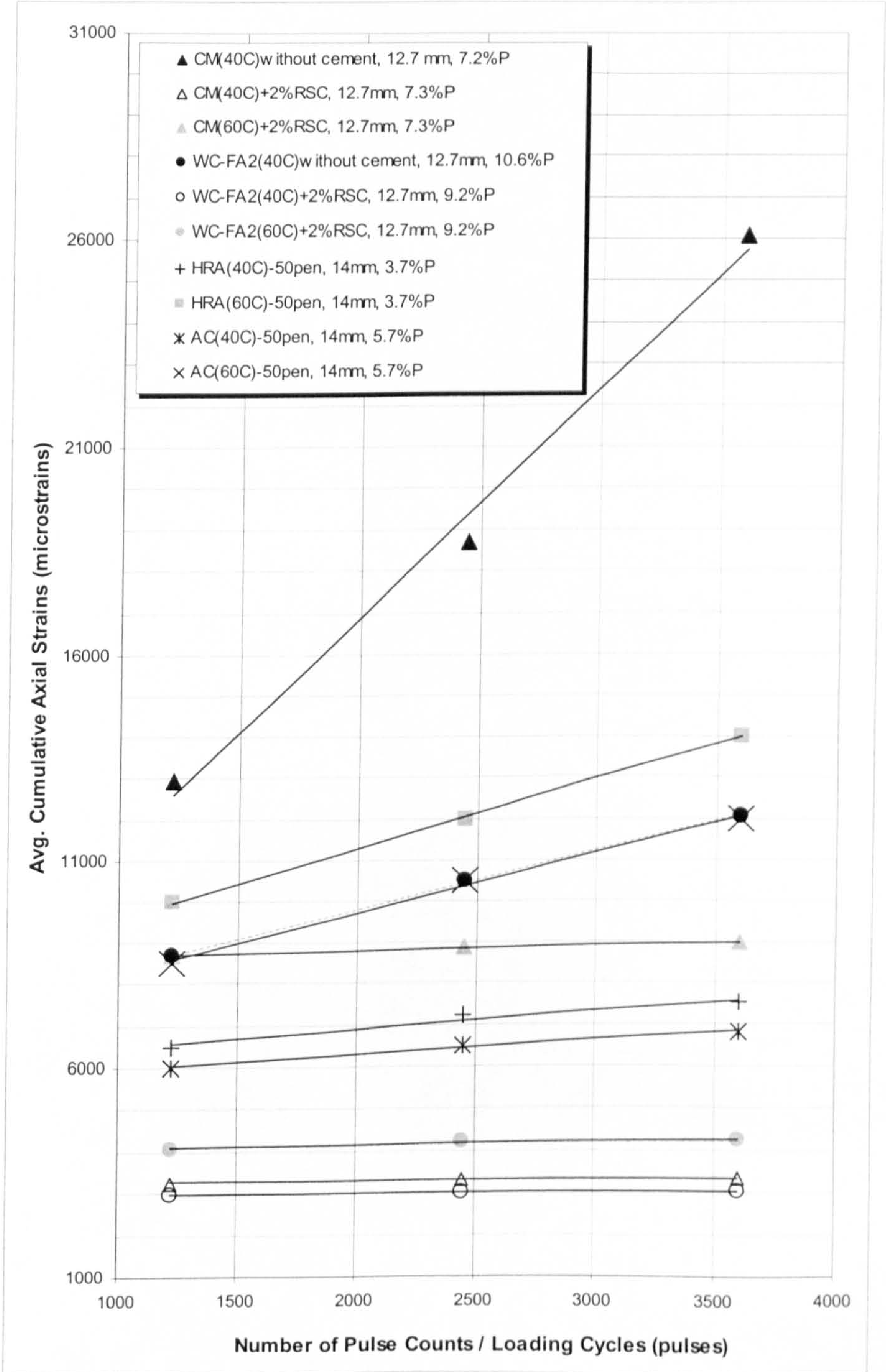


Fig. 9.13 Secondary Stage Dynamic Creep Slopes for CBEMs and selected Hot Mixtures.

The *dynamic creep slopes* curves for the secondary creep stages are presented in Fig. 9.13. The equation of each creep slope was expressed in the form y = ax + b, and the values of the constants 'a' (μm/pulse) are shown in Table 9.19.

Typical laboratory minimum dynamic creep slopes were given in Chapter V, Table 5.4. In general, lower slope values indicate stronger mixtures, more capable of resisting permanent deformations. The CBEMs without cement showed higher creep slope values. Comparing the slope values in Table 9.19 with typical laboratory minimum dynamic creep slopes in Table 5.4, it was revealed that the slopes of the CBEMs (with a 100 pen base bitumen) mixtures incorporating 2% RSC, ***at full curing,*** were relatively low, smaller even than the hot mixtures with the harder binder (50 pen). Therefore, theoretically CBEMs incorporating 2% Rapid Setting Cement at full curing, have excellent resistance to permanent deformation and are suitable even for heavy traffic conditions.

d. Creep Stiffness

The creep stiffness values, i.e. the ratio of applied stress to the permanent strain were calculated/estimated at the end of the 3600 loading cycles.

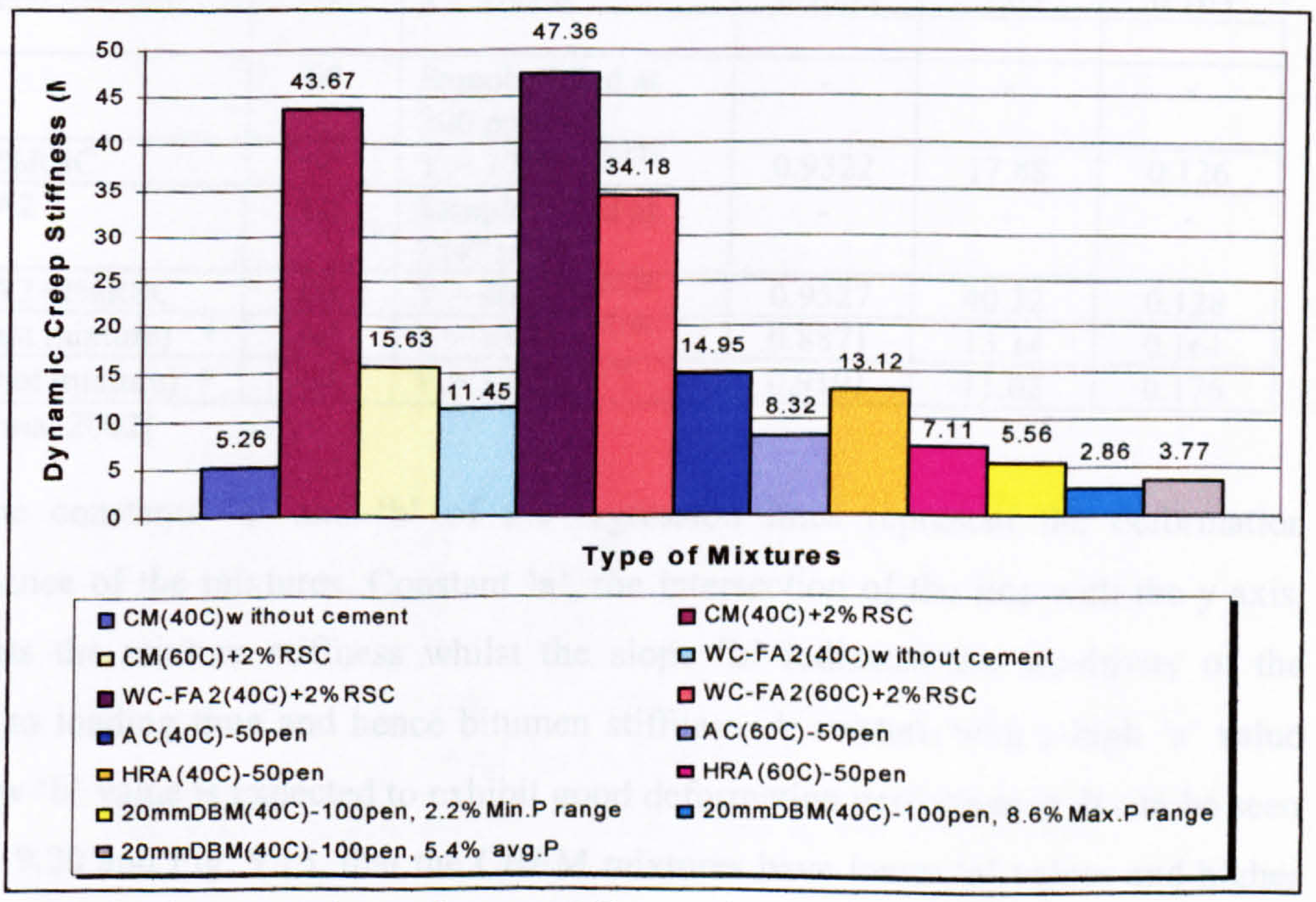


Fig. 9.14 Comparison of creep stiffness values at 3600 loading cycles/pulses. between CBEMs and hot mixtures.

Creep stiffness values are presented in Fig. 9.14. Typical calculations are given in Appendices 4.1, and 4.2.The creep stiffness values shown in Fig. 9.14 agree with the axial deformation values shown in Fig. 9.11. Mixtures with smaller creep deformations obviously result in higher creep stiffness values.

e. Relationship between stiffness of mixture (S_{mix}) and stiffness of bitumen (S_{bit})

This relationship was described earlier in Chapter V, section 5.3.3.2d and equation 5.15, where it was shown how the relationship between S_{mix} and S_{bit} can be expressed by a straight line on a double logarithmic scale using 'power' or regression lines. The S_{bit} values were calculated using Shell's *BANDS 2.0* software (see Appendix 4.3). The relationships for all mixtures are shown in Table 9.20 and Fig. 9.15.

Table 9.20 Regression Parameters for S_{mix}-S_{bit} relationship.

Mixture	Test Temp. (°C)	Power Regression Equations on a log-log scale (power lines)	R^2	Cons-tant 'a'	Cons-tant 'b'
CM	40	$y = 3.93\ x^{0.374}$	0.9865	3.93	0.374
CM+2%RSC	40	$y = 34.81\ x^{0.098}$	0.8944	34.81	0.098
WC-FA2	40	$y = 7.97\ x^{0.269}$	0.9921	7.97	0.269
WC-FA2+2%RSC	40	$y = 39.88\ x^{0.075}$	0.8957	39.88	0.075
A.C. (hot mixture) *	40	$y = 11.78\ x^{0.133}$	0.9574	11.78	0.133
HRA (hot mixture) *	40	$y = 9.65\ x^{0.163}$	0.9534	9.65	0.163
CM	60	Sample failed at 300 pulses	-	-	-
CM+2%RSC	60	$Y = 17.88\ x^{0.126}$	0.9322	17.88	0.126
WC-FA2	60	Sample failed at 1847 pulses	-	-	-
WC-FA2+2%RSC	60	$Y = 40.32\ x^{0.128}$	0.9527	40.32	0.128
A.C. (hot mixture) *	60	$Y = 13.14\ x^{0.161}$	0.8871	13.14	0.161
HRA (hot mixture) *	60	$Y = 1.02\ x^{0.176}$	0.9191	11.02	0.176

* [Suparma, 2002]

The constants 'a' and 'b' of the regression lines represent the deformation performance of the mixtures. Constant 'a', the intersection of the line with the y axis, represents the mixture stiffness whilst the slope 'b' indicates the sensitivity of the mixture to loading time and hence bitumen stiffness. A mixture with a high 'a' value and a low 'b' value is expected to exhibit good deformation performance. It can be seen in Table 9.20 and Fig. 9.15, that the CBEM mixtures have lower 'a' values and higher 'b' values than other mixtures, but when incorporating 2% RSC, the CBEMs gave significantly higher constant 'a' values and lower constant 'b' values, which meant that they had superior resistance to creep deformations.

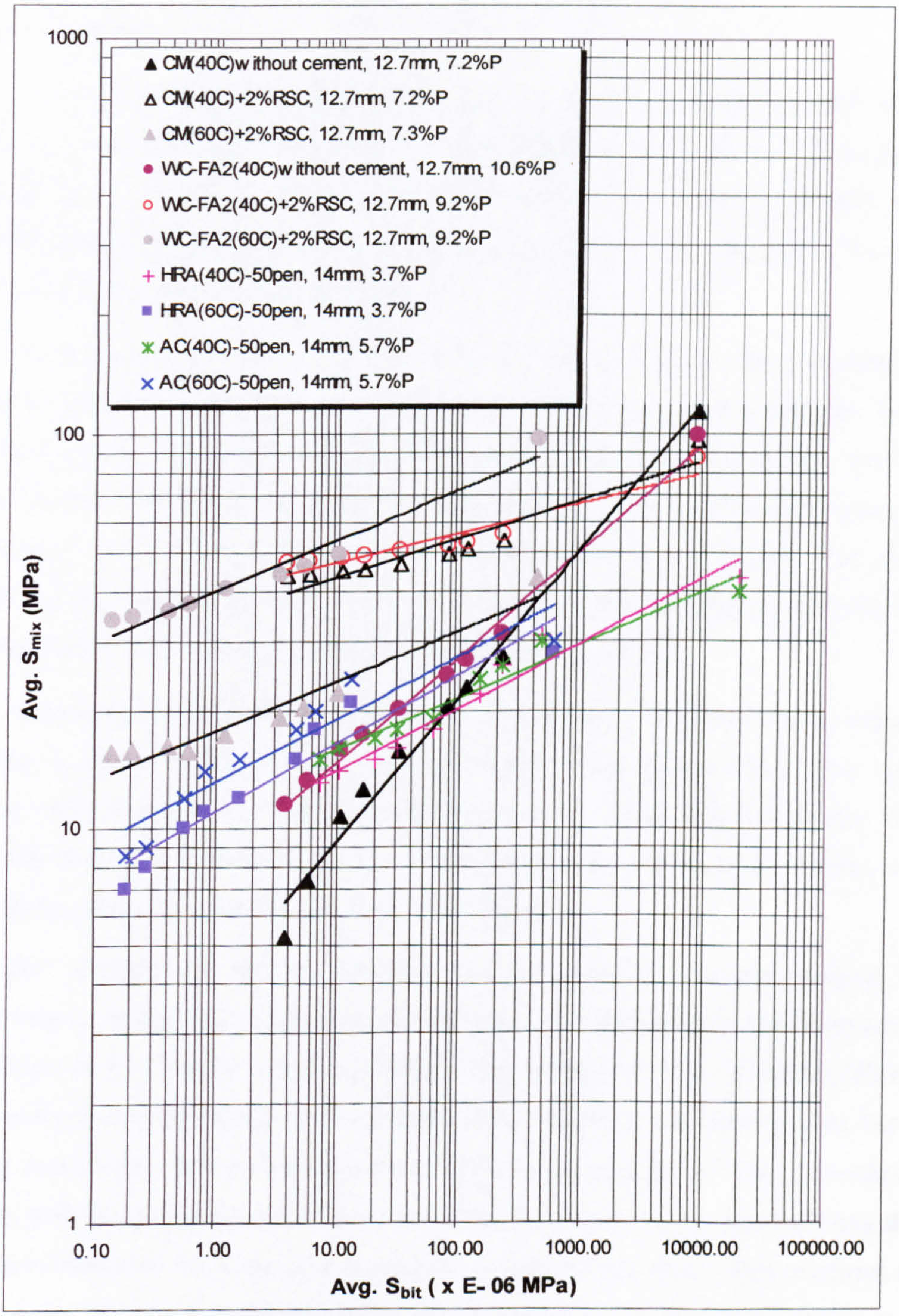


Fig. 9.15 Relationship between S_{mix} and S_{bit} for CBEMs and selected Hot Mixtures.

9.4 CBEMs Reinforced with 'plastic cells'

9.4.1 General

Reinforcement of *HOT asphaltic mixtures* by means of high tensile polymer grids has been investigated in the early 1980's [Brown et al., 2001]. When properly applied, grid reinforcement can enhance the cracking and rutting resistance of the asphalt concrete layers in pavements. It was realised that to locate the grid at the correct level within the asphalt layer was essential.

Research carried out at Nottingham University investigated the performance of asphaltic concrete layers with and without grid reinforcement. In the first case, the grid reinforcement was placed at mid depth of the asphalt layer and in the second case it was placed at the underside of an 80mm asphaltic concrete layer, both asphalt layers were constructed over a low stiffness granular base overlaying a subgrade. The asphalt concrete layer was subjected to simulated trafficking in the Nottingham Pavement Test Facility using a 9 kN wheel load for 200,000 repetitions at 20°C.

Positioning the grid at mid depth of the asphalt layer was based on the argument that this is approximately the zone where maximum shear strains which cause rutting are generally recognized as being located [Brown et al., 2001]. The second grid, which was placed at the underside of the asphalt layer was aimed at preventing crack propagation based on a traditional elastic layer theory.

The investigation results indicated that the asphaltic concrete without grid reinforcement failed comprehensively by cracking accompanied by a significant amount of rutting that was largely contributed by the supporting layer (Fig. 9.16-top). With the grid positioned at the center of the asphalt layer, cracking was still evident but the rutting was mainly due to the deflection of the supporting layer (Fig. 9.16-middle). Whilst, with the grid positioned at bottom of the asphalt layer (Fig. 9.16-bottom), there was no evidence of cracking, with much less overall deformations. This confirms that placement of the grid at the bottom is the correct position to encounter the tensile strains which cause cracking [Brown et al., 2001].

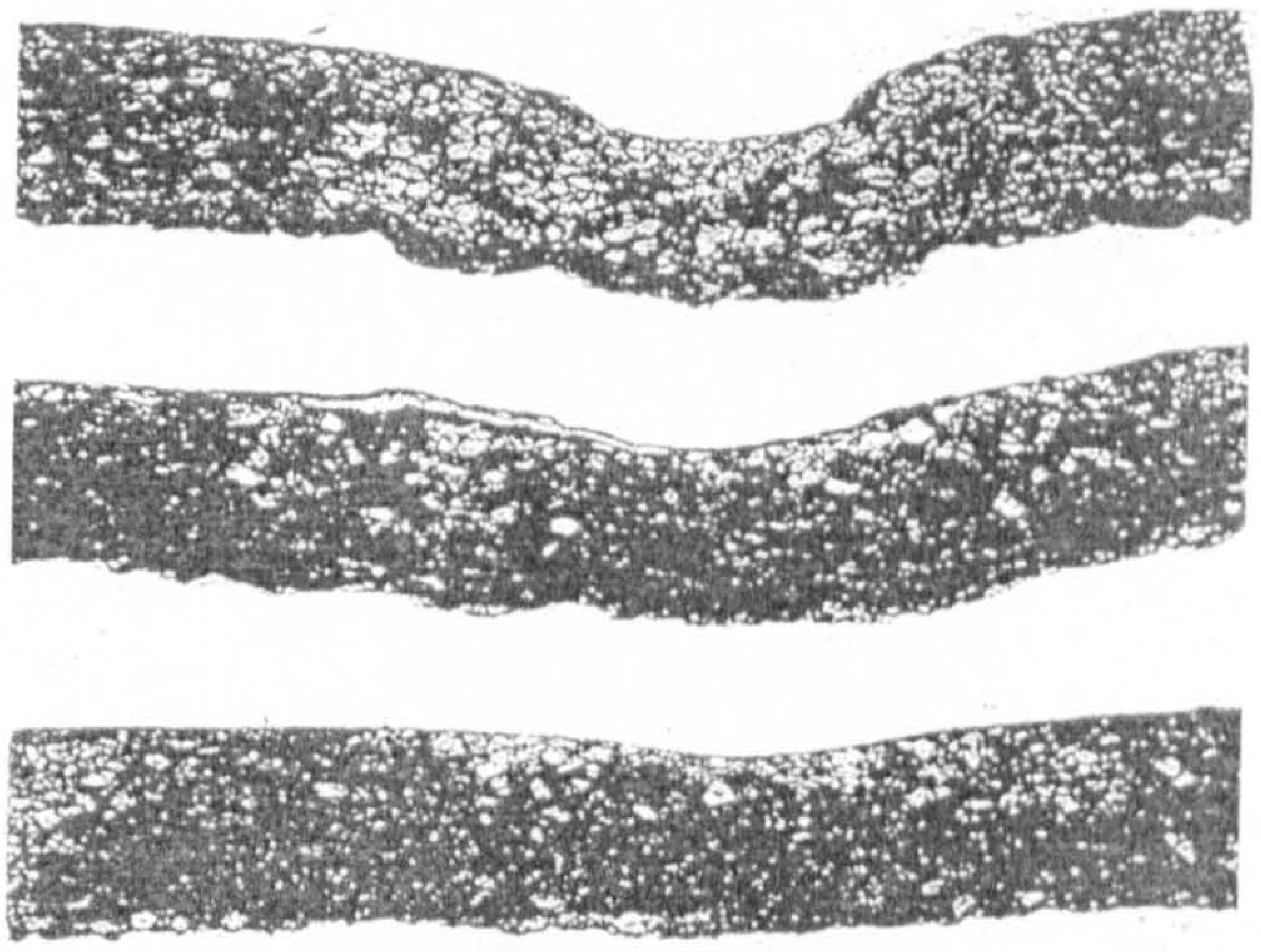


Fig. 9.16 Cross section of an asphalt pavement reinforced with geosynthetic grid following trafficking.
Top Figure - without grid, Middle Figure - grid at mid depth of asphalt layer, Bottom Figure - grid at underside of asphalt layer.

A critical factor that was identified early on in the research, was that there is a potential bond reduction between the asphalt layers above and below any geosynthetic reinforcement and that this was the case in a new construction or over an existing pavement [Brown et al., 2001].

The case of grid reinforcement described above was specifically applied in a *HOT asphalt mixture*. Naturally, the experiment was carried out when the hot asphalt concrete mixture was at optimum strength, i.e. soon after the hot mixture cooled down. Meanwhile, application of geosynthetic reinforcement in CBEMs, as is the case in this investigation is of a different nature. In this case the low early life strengths of the cold asphalt mixtures is very critical.

9.4.2 Initial investigations using 'plastic cells' reinforcement in CBEMs

The objective of the investigation described in this section was to reduce the susceptibility of CBEMs to irrecoverable vertical deformations (rutting) at early curing ages when the CBEMs are still weak, by using *'plastic cells'* reinforcement. The type of cold mixture selected was the **WC-FA2** (with fine aggregates: 60% Red Porphyry Sand + 40% asphalt sand), *without any added cement.*

For this investigation the 'plastic cells' used were 'specially produced', courtesy of 'Phi Design Ltd.', Northampton, UK. The 'plastic cells' or 'blocks of plastic strips' were made from *standard extruded Polyvinyl Chloride (PVC) sheets that had been cut into strips*. Each plastic strip was 20mm wide, 0.38mm thick and the strips were supplied 600mm in length. The plastic strips were 'slotted' or 'cut' with slits 1mm in width and 10mm in length (i.e. half of its 20mm total width) to enable the formation of 'plastic cells' with a typical cell size of 35×35mm, as shown in Fig. 9.17. The depth of the strips, dimensions of the individual cells and the number of cells that can be accommodated within one asphalt specimen was dictated to a large extent by the size of the asphalt specimens (150mm diameter) that can be produced in the laboratory.

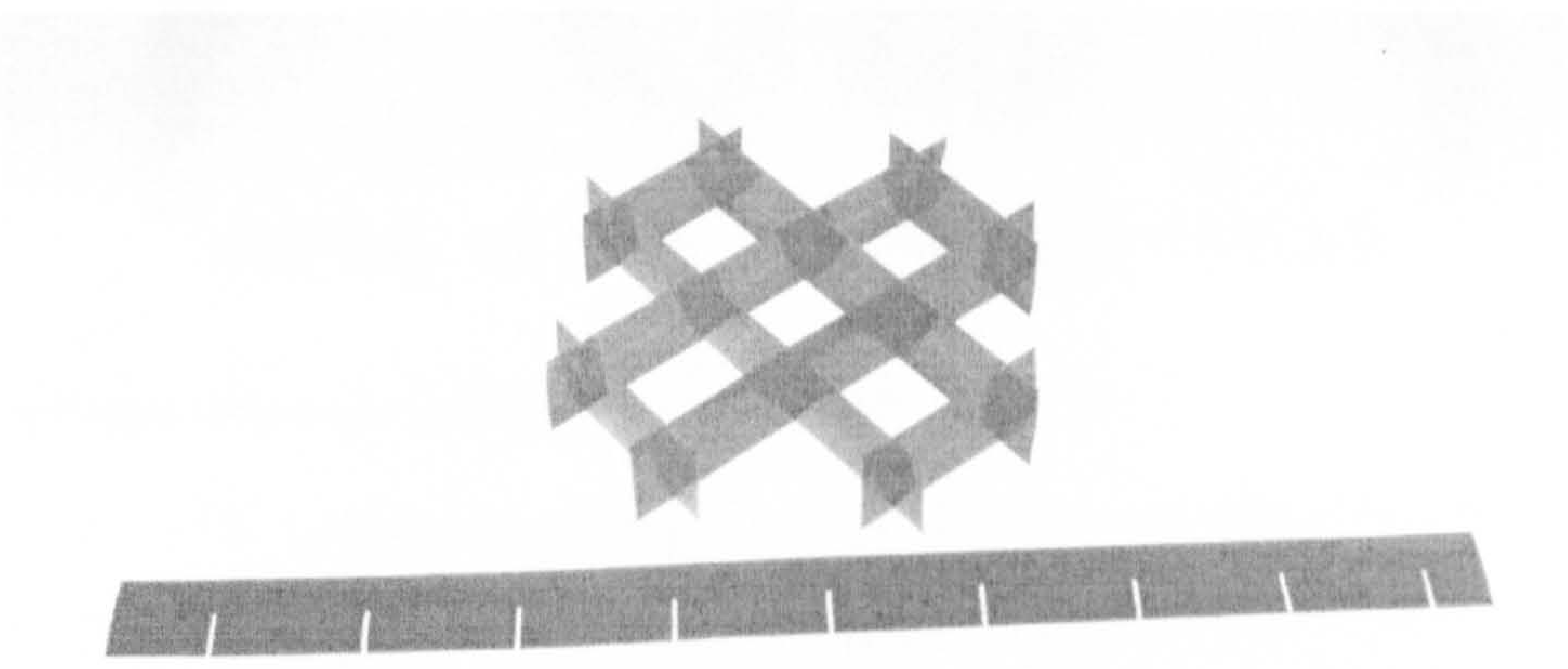


Fig. 9.17 Plastic Cells.

Two types of bituminous cold mixture samples were manufactured, namely samples without plastic cells (NPC), and samples with plastic cells (WPC). All samples were prepared using 150mm diameter compaction moulds (Fig. 9.18). The maximum aggregate size selected for the asphalt mixture was 5mm, which was considered to be a suitable size for optimum interaction / interlock with the plastic cells (35×35mm). To use larger aggregate sizes would require a much larger cell size, which was not possible due to limitations on the dimensions of the compaction moulds. The idea was produce a scaled down model of what is expected to be constructed in the field. The aggregate gradation was determined using a modified Fuller maximum aggregate packing equation (Equation 7.1 in Chapter VII). The materials used were similar to those used in cold mixture WC-FA2.

Fig. 9.18 Mould (150mm diameter) and Tamping Rod.

9.4.2.1 Preparation and compaction of the NPC samples

Two NPC samples were manufactured in the common way in which the loose mixture was poured into a lubricated compaction mould fitted with a steel base plate. The loose mixture was tamped with a 12mm diameter tamping rod 20 times at the edges and 15 times in the centre, and then covered with a top wearing steel plate.

Compaction of the 150mm diameter specimens was carried out using a Gyropac set at 240 revolutions and 540 kPa. This compaction effort was equal to two times the heavy compaction effort (i.e. Extra Heavy Compaction Level 2 - EHC2). The compacted samples were cured for *one day* in their moulds at room temperature (24°C) before being extruded and cured for a further 4 *days* at room temperature. This curing regime allowed the specimens to attain sufficient strength for handling. The samples were then tested for their bulk density and stiffness (ITSM) – see Section 9.4.2.9.

9.4.2.2 Capping of samples

Gyropac compacted samples do not have horizontal surfaces as a result of the pre-set angle of gyration. In order to obtain parallel, flat and horizontal surfaces, the *top end* of the samples were capped using an OPC mortar with a sand/cement ratio of 4:1.

Fig. 9.19 Capping of samples to obtain upper and lower parallel flat surfaces.

Fig. 9.20 Dynamic Loading of the capped CBEMs.

Capping was carried out by wrapping the sides of each sample with appropriate height of flexible aluminum sheet (Fig. 9.19). The sand cement mortar (of adequate workability) was then poured on the top surface of the specimen to fill the space created by the aluminum sheet. A heavily oiled flat glass plate was used to press the sand cement mortar with some rotational action until an even and horizontally flat surface was obtained. To prevent cracking of the capping mortar, the capped sample with the glass on top was covered with a damp towel and kept to cure for 2 days in a damp condition. The glass plate and aluminum sheet were subsequently removed.

For the Dynamic Creep tests, the capped samples were conditioned at the test temperature for 3 hours. During testing, the capped ends of the samples were used as the base, as shown in Fig. 9.20. The age of the asphalt samples at this stage was 7 days (i.e. 1 day in the compaction mould followed by extrusion, then 4 days curing, and finally 2 days for capping).

9.4.2.3 Preparation of the samples with 'plastic cells' at mid depth (WPC)

For *initial evaluation, two WPC samples* were manufactured with plastic cells positioned at *mid depth.* After conducting some trials, the amount of loose mixtures required was determined at 2400 grams total for obtaining a suitable sample thickness for ITSM testing. In order to position the plastic cells at about mid depth within the asphalt, the following protocol was developed;

1000g of the loose mixture was first poured into the mould, evenly distributed and then tamped 50 times using a tamping rod. The tip of the tamping rod was a cylindrical metal disc 15mm in thickness and 70mm in diameter (Fig. 9.18). The plastic cells were then placed on top of this first asphalt layer and a sufficient amount of loose mixture was then poured into the plastic cells and tamped 50 times until the mixture filled all the cells evenly. The remaining loose mixture (third layer) was then poured, tamped 50 times and compacted with a Gyropac. The compacted specimens were then cured and capped in a similar way to the NPC samples.

9.4.2.4 Dynamic Loading Setting 1

For comparing the performance of the cold asphalt specimens with and without plastic cells, dynamic creep loading was selected. The load intensity was set at a convenient level to generate a clear profile of cumulative vertical deformation v.s. number of load pulses. The test procedure (dynamic load setting 1) was set as listed below:

Total number of load pulses: 36,000 pulses
Conditioning Stress: 10 kPa
Loading Stress: 100 kPa
Conditioning Time: 2 minutes
Pre-load rest time: 1 minute
Recovery Time: 60 minutes

Test temperature: 20°C

The dynamic load was applied on the 150mm diameter samples using a cylindrical plate 100mm in diameter and 16mm in thickness as shown in Fig. 9.20.

9.4.2.5 Age of samples

For initial evaluation of CBEM specimens reinforced at mid depth, one sample was tested from each of the NPC and WPC mixtures at 7 and at 14 days curing. All samples were cured for one day in the compaction moulds, followed by exposed curing at room temperature. In each case, capping was only carried out 2 days before dynamic load tests were to be carried out, (i.e. 2 days before the 7 days and 14 days of curing).

9.4.2.6 Performance of the NPC and WPC sample (plastic cells positioned at mid depth) with Dynamic Load Setting 1

Initial trials indicated that at dynamic load setting 1, the samples underwent *very minor deformations* at 7 days curing and the deformations were negligible at 14 days curing. This meant that the specimens had not experienced enough plastic deformations to activate the plastic cell reinforcement. Therefore the samples were retested at dynamic load setting 2 as detailed below:

9.4.2.7 Dynamic Loading Setting 2

As testing the CBEM samples using dynamic load setting 1 gave very minor deformations, the loading was adjusted to dynamic load setting 2 as follows:

Total number of load pulses: 50,000 pulses
Conditioning Stress: 10 kPa
Loading Stress: 200 kPa (instead of 100 kPa)
Conditioning Time: 2 minutes
Pre-load rest time: 1 minute
Recovery Time: 60 minutes
Test temperature: 40°C (instead of 20°C)

9.4.2.8 Performance of the NPC and WPC samples (plastic cells positioned at mid depth) with Dynamic Load Setting 2

The performance of the samples at the higher dynamic stress level is shown in Fig. 9.21. It is clear from the figure that the NPC sample suffered from higher permanent axial strains and totally collapsed (see also Fig. 9.22) at 514 load cycles. On the otherhand, the test on the WPC was stopped at 1801 cycles when the sample's vertical deformation exceeded the allowable range of the LVDTs. Even at the highest recorded strain values, the WPC sample did not undergo total collapse. Much larger deformations were accommodated by the plastic cells (Figs. 9.23 and 9.24). The condition of the plastic cells after testing is shown in Fig.9.25.

By the time this test was carried out the CBEM samples were 4 weeks old. The samples were technically found to have achieved full curing. This was verified by the results of water content tests on the failed samples, which indicated very negligible retained moisture at 4 weeks curing.

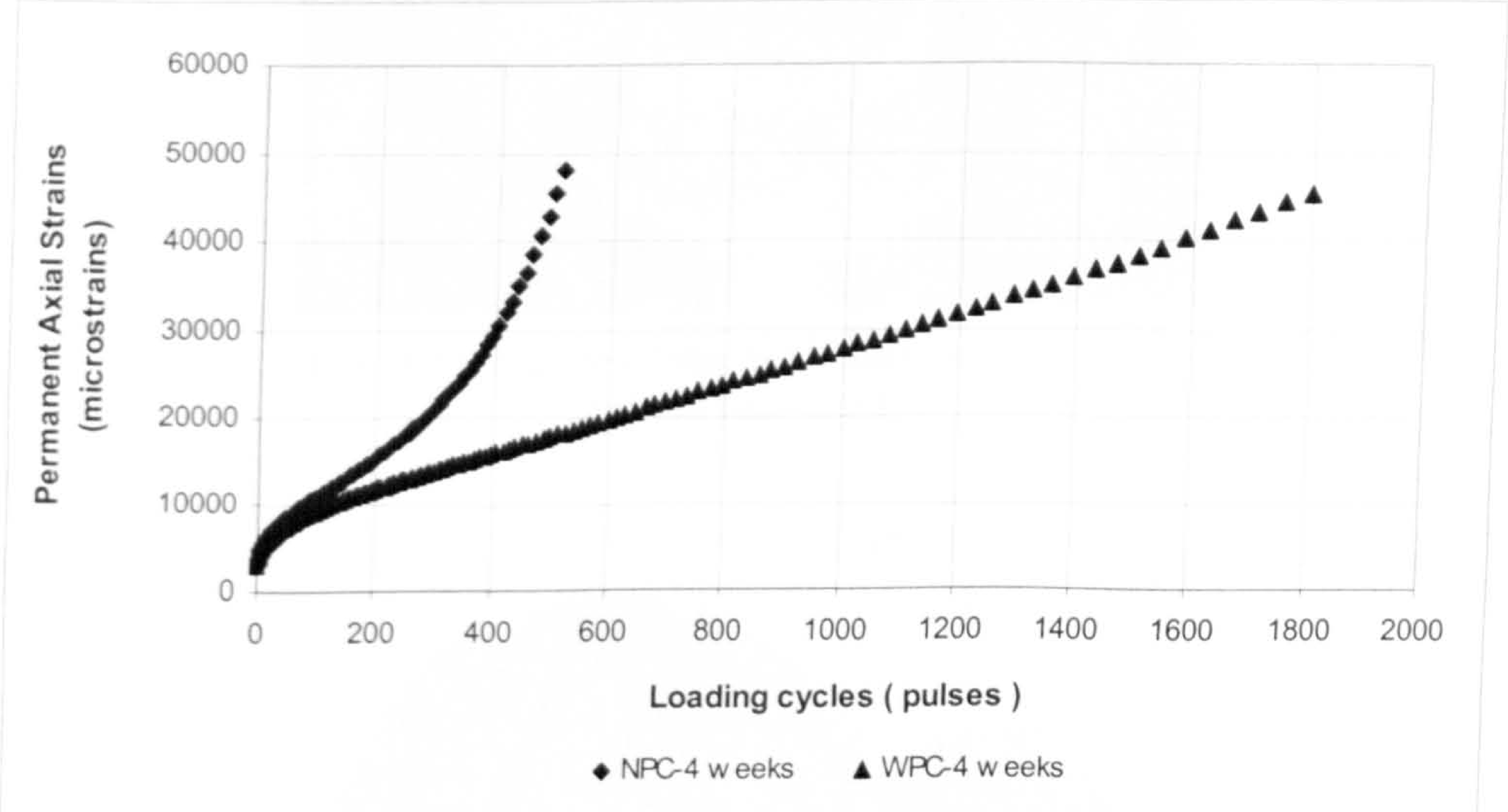


Fig. 9.21 Strain vs. loading cycles with Dynamic Loading Setting 2 .

Fig. 9.22 Total failure of the NPC sample.

Fig. 9.23 Failure of the WPC sample, with plastic cells at mid depth.

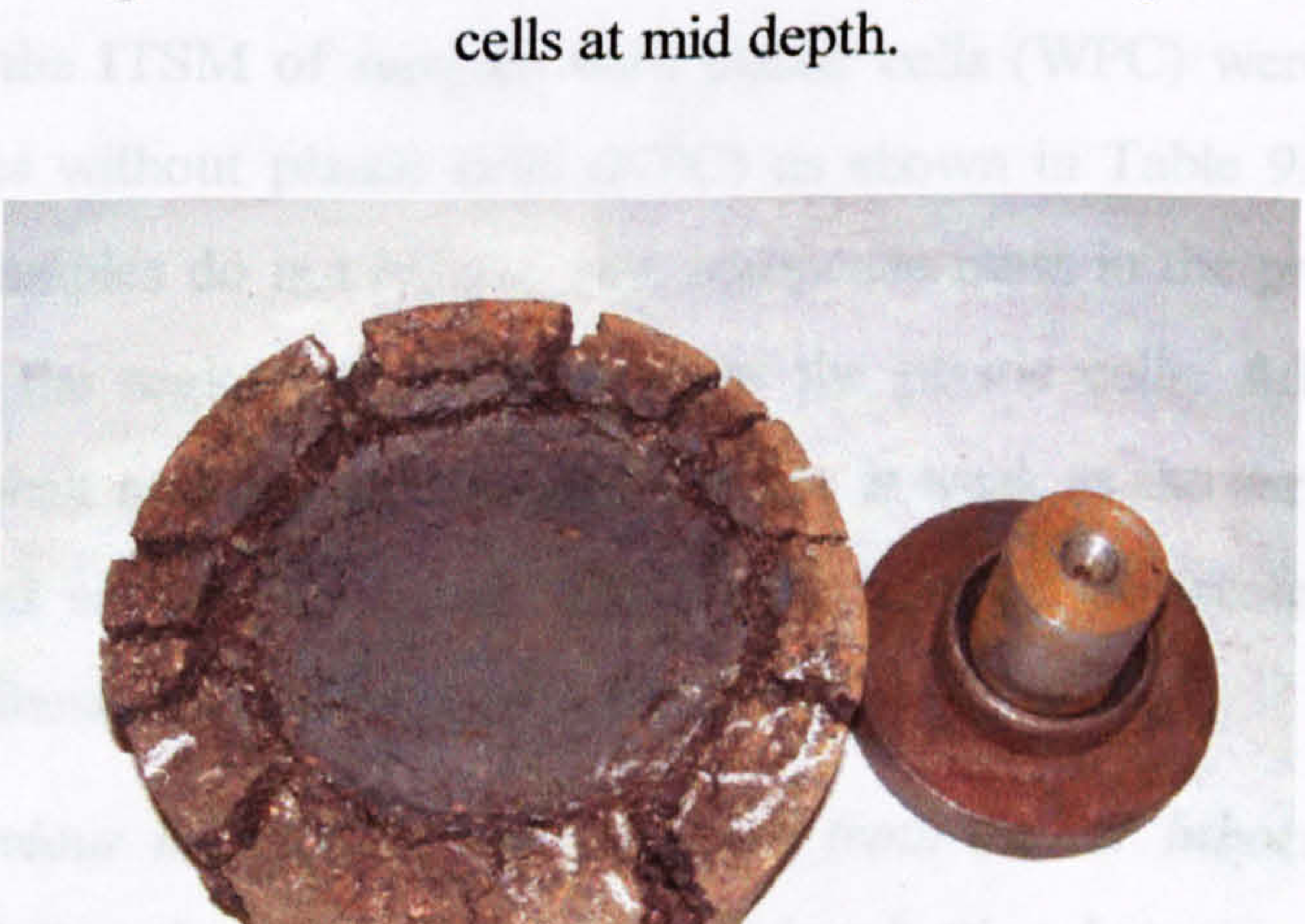


Fig. 9.24 Top view of the tested WPC sample as in Fig. 9.23.

Fig. 9.25 View of the plastic cells within the failed sample as in Fig. 9.23.

9.4.2.9 Dry density and stiffness of the samples initially investigated

It was found that the dry density value of the WPC sample was slightly lower than the NPC sample. This was believed to be caused by the asphalt material within the plastic cells being less free to adjust their positions during compaction as a result of the confinement provided by the cells walls. However, the difference was negligible (i.e. below 1%) as shown in Table 9.21 and the porosity was within targets (5-10%). Overall, compaction can be performed adequately on both specimen types (with and without plastic cell reinforcement).

However the ITSM of samples with plastic cells (WPC) were consistently lower than the samples without plastic cells (NPC) as shown in Table 9.22. It is very likely that the WPC samples do not behave as a composite mass in the presence of a possible discontinuity in the regions above and below the plastic cells. Additionally, the bond between the asphalt mixture and the plastic strips is weak as the surface of the plastic is very smooth and absorbs no liquid. These effects result in greater deformations and hence lower stiffness values during ITSM testing.

This behaviour is supported by evidence from earlier laboratory investigations which indicated that there is a potential bond reduction between the asphalt mixture above and below any geosynthetic reinforcement [Brown et al., 2001].

Table 9.21 Dry Density values of samples from the initial investigation.

Type of Mixture	Dry Density (g/cm^3)	Porosity (%)
NPC	2.20	7.9
WPC	2.18	8.8

Table 9.22 ITSM (at 20°C) of samples from the initial investigation.

Type of Mixture	ITSM (MPa) at curing age of		
	1 week	2 weeks	full curing
NPC	1178.9	1542.5	2302.6
WPC	863.24	1247.5	2117.4

9.4.3 Final investigation using 'plastic cells' reinforcement in CBEMs

As a consequence to first set of results, described above, modifications to the testing protocol was considered necessary. The following modifications were carried out:

9.4.3.1 Age of test samples

It was decided that the samples should be tested at the *earliest age practicable*, in order to evaluate the effectiveness of the plastic strip utilization. The samples were therefore extruded from their compaction moulds soon after compaction and it was found that with extra care, the samples can be handled. Next, the samples were *cured for 24 hours* at room temperature (24°C), and then tested for ITSM at 20°C. The samples were then capped and cured for a further 48 hours prior to dynamic loading. The overall age of the samples by the time it was subjected to dynamic loading was 3 days of curing.

9.4.3.2 The NPC sample

As sufficient experience had already been obtained from the earlier trials, only one NPC sample was prepared in the way as described earlier in Section 9.4.2.1.

9.4.3.3 WPC sample with 'perforated' plastic cells positioned *close to the surface*

Considering the unsatisfactory performance of the WPC with the plastic cells positioned at mid depth, test modifications were carried in two ways, namely:

- the walls of the plastic strips were punctured (two holes per side of plastic cell). This was aimed to provide improved interlock between the aggregates and the side plastic walls and the to allow more efficient lateral drainage of excess fluids during the compaction process. The holes were made using a paper puncher with diameter of 6mm as shown in Fig. 9.26.
- the 'perforated' plastic cells were positioned close to the upper surface of the specimen, in order to bring the cells closer to the area of highest stress. A higher positioned group of cells should also help arrest the downward propagation of tensile and shear cracks that are generated during the high shear strain dynamic loading.

Each sample was prepared using a total of 2400g loose asphalt mixture, which gave suitable a sample thicknesses of about 60 to 65mm. Initially about 300g of loose mixture was poured into each compaction mould, evenly distributed and then evenly tamped 50 times using the tamping rod shown in Fig. 9.18. This procedure gave a first tamped layer thickness of about 5-6 mm. The plastic grid was then placed on top of the first tamped layer. Approximately 1050g of loose mixture was then evenly poured into the mould to completely fill all the plastic cells and evenly tamped 50 times. The remaining 1050g loose mixture was poured and tamped 50 times, a greased steel wearing plate as then placed on top of the mixture and finally the whole specimen was compacted using the Gyropac. After compaction the samples were cured and capped in the manner previously described.

During testing, the capped end of each sample was used as the base and hence the plastic cells were in a position close to the upper surface of the test sample.

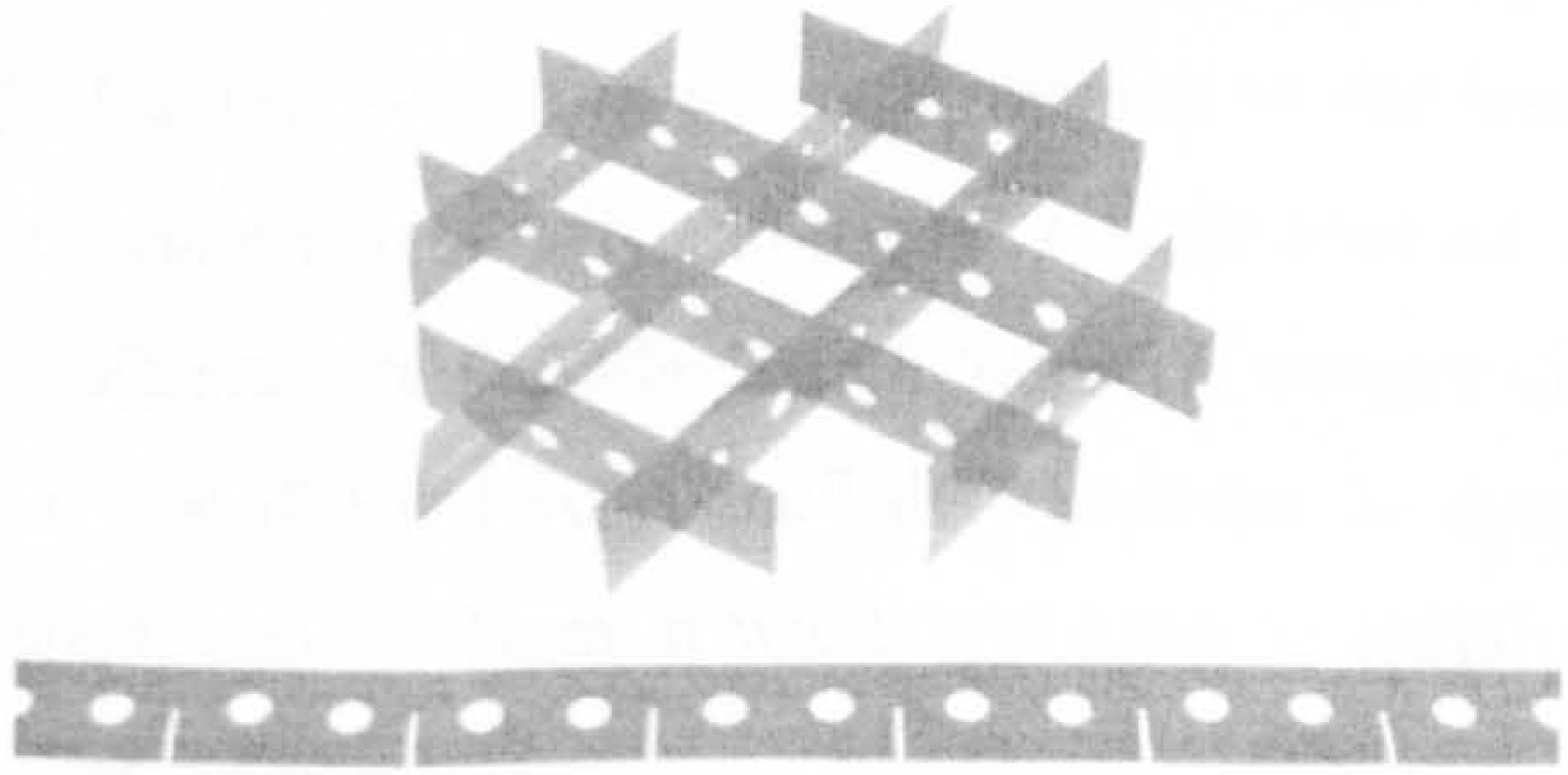


Fig. 9.26 Plastic Cells with two holes in each side wall.

9.4.3.4 Loading Setting 3

Within this final investigation the dynamic loading was set as follows:

Total number of load pulses: 100,000 pulses
Conditioning Stress: 10 kPa
Loading Stress: 100kPa
Conditioning time: 2 minutes
Pre-load rest time: 1 minute
Recovery time: 60 minutes
Test temperature: 40°C

9.4.3.5 Stiffness of the samples

The stiffness (ITSM) results of the samples produced at this stage of the investigation are shown in Table 9.23. As before, the WPC specimens showed lower stiffness values.

Table 9.23 ITSM at 20°C after 24 hours curing.

Sample	ITSM (MPa)
NPC	469.62
WPC	355.61

9.4.3.6 Results of the dynamic loading

Comparison of the permanent axial strains vs. number of load cycles of 3 days cured specimens containing no cells and specimens with cells positioned close to the upper surface of the specimens are presented in Fig. 9.27.

Fig. 9.27 shows that the NPC sample suffered higher permanent axial strains and collapsed at 68,000 loading cycles, in a similar manner to the results shown earlier in Fig. 9.22. The NPC sample gradually developed a large amount of cracks (Fig 9.28) leading up to total collapse. On the other hand, the WPC sample did not fail even after 100,000 load cycles, and showed very small deformations. In this case, much smaller cracks were developed, about 0.5mm in width and 6mm in length at the upper loaded face of the sample (Fig.9.29).

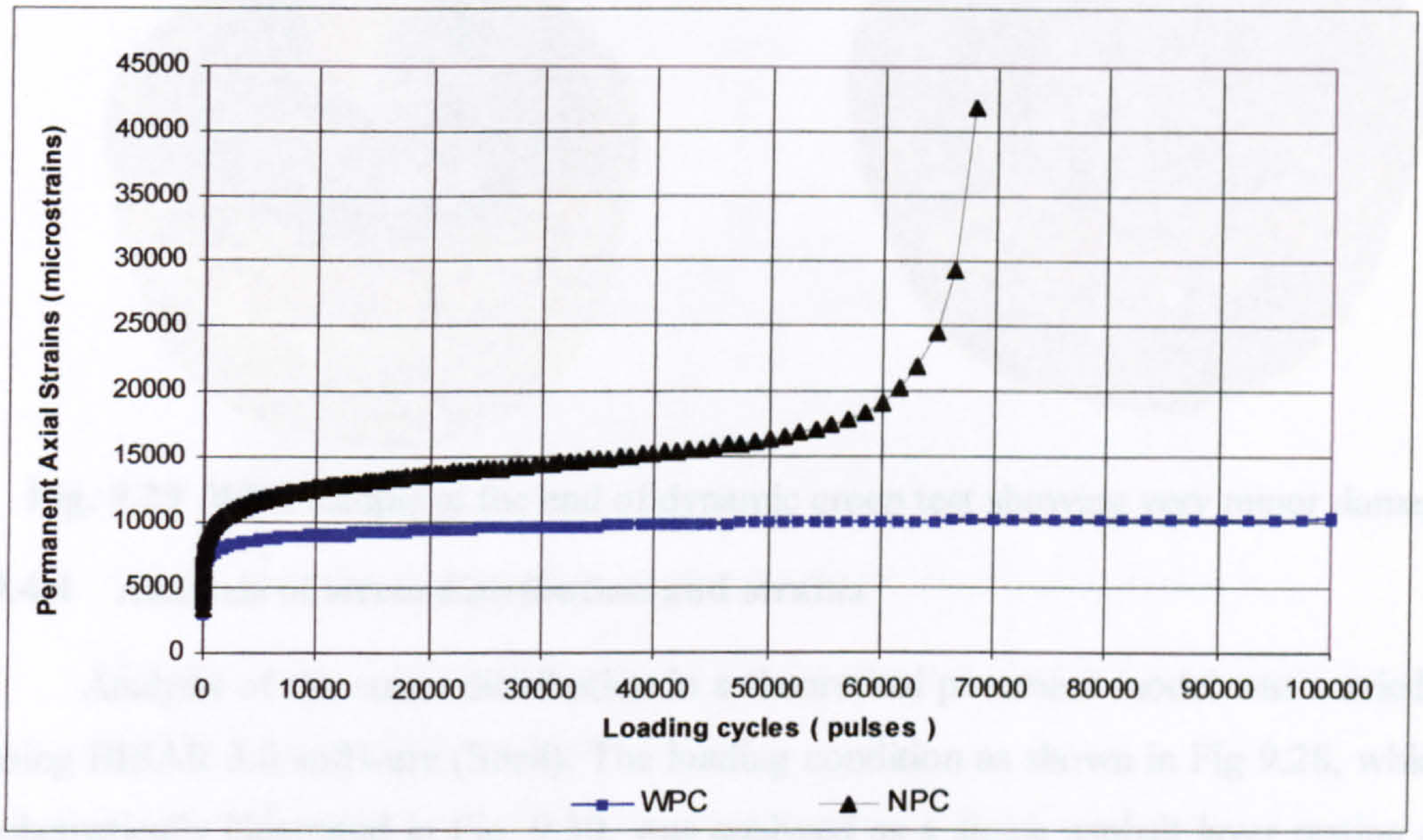


Fig. 9.27 Permanent axial strains vs. number of load cycles, at age 3 days. Note: WPC = with Plastic Cells; NPC = No (without) Plastic Cells.

Fig. 9.28 Propagation of cracks in the NPC sample during the final investigation.

Fig. 9.29 WPC sample at the end of dynamic creep test showing very minor damage.

9.4.4 Analysis of stress distribution and strains

Analysis of the stress distribution in a theoretical pavement model was carried out using BISAR 3.0 software (Shell). The loading condition as shown in Fig 9.28, which is schematically illustrated in Fig. 9.30, was analysed as a single asphalt layer resting on a semi-infinite base.

The object of this analysis was to estimate the stresses acting on the NPC sample which caused total failure (collapse) of the specimen during testing. These same stresses were on the other hand resisted by the plastic cells in the case of the WPC sample, which subsequently prevented the sample from failure. As none of the cells were observed to fail/rupture during the tests, it was clear that the tensile strength of the plastic cells was in fact higher than the tensile stresses that were produced during loading (see also Section 9.4.6).

During testing, the dynamic load was based on dynamic load setting 3, as described earlier in Section 9.4.3, with a loading stress level of 100 kPa (equivalent to 2.5 kN actual vertical force on a 100mm diameter loading plate). This loading scenario (force applied, thickness of asphalt, loaded area, etc.) was used as input parameters into the BISAR 3.0 software.

For BISAR analysis, the pavement structure was considered to consist of a 74mm thick CBEM (asphaltic) layer, resting on a very rigid base (in our experiments, this was a steel base). The modulus of elasticity used for the CBEMs and the steel base were 355.61 MPa (from Table 9.23) and 190 GPa respectively. The Poisson's ratio of the

CBEM and the steel base were 0.35 and 0.28 respectively. The property of the steel base was assumed to be typical of stainless steel [Umist, 2003, URL-21].

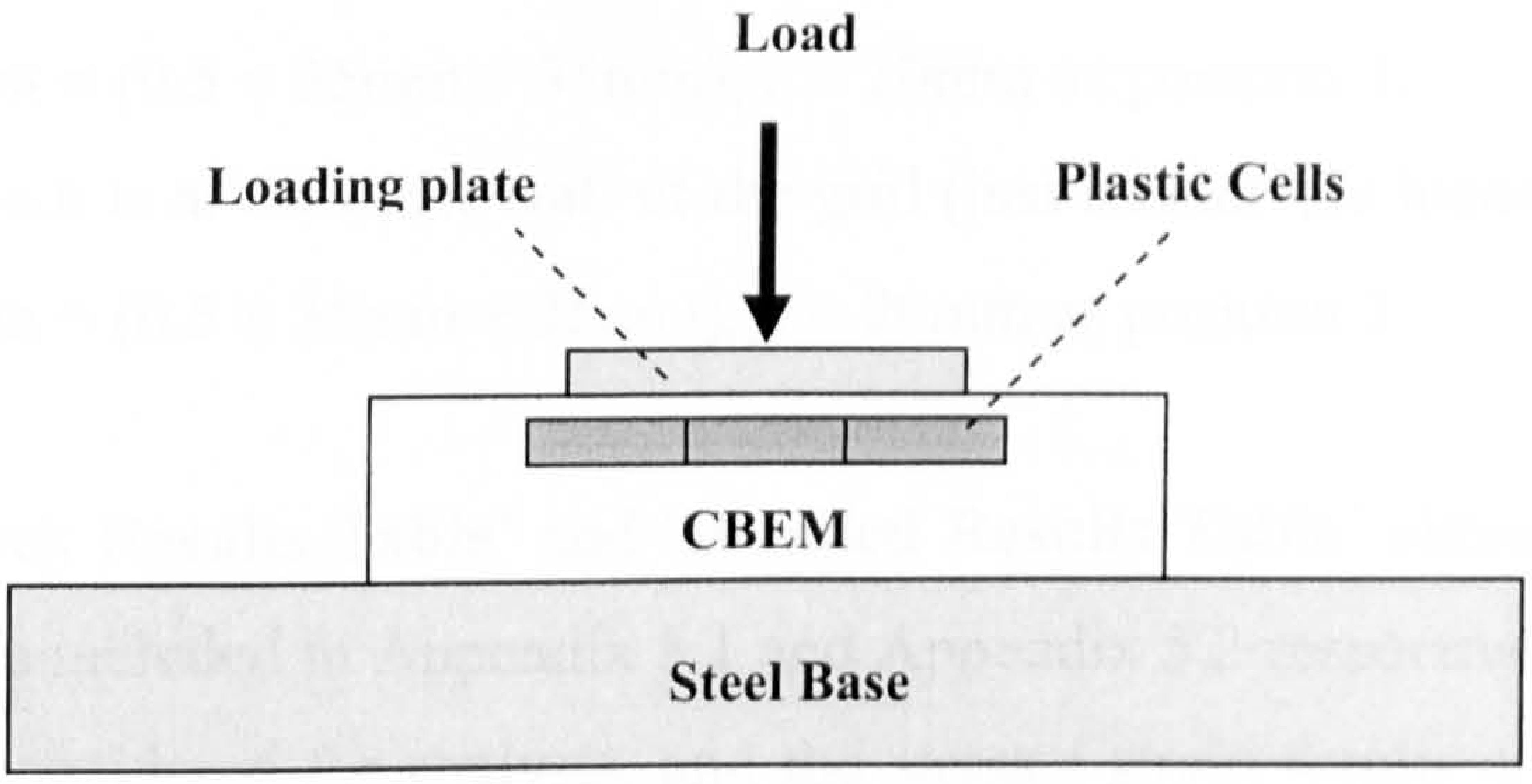


Fig. 9.30 Schematic loading for stress distribution analysis.

The positions of most interest for stress and strain analysis were selected taking the following conditions into consideration:

- distance of the upper part of the plastic cells from the surface: 5mm.
- height of the plastic cells grid 20mm.
- the size of a single cell: 35mm × 35mm.
- the plastic strips have been slit to half of their heights and therefore the effective height of the side wall of each plastic cell (that can effectively resist lateral forces) is half the height of the actual plastic strip: 0.5 × 20mm = 10mm (see Fig. 9.26).
- each plastic cell wall has two holes 6mm in diameter.
- the coordinates of the centre of the load at the sample's surface are: x = 0, y = 0, z = 0.

The coordinates or positions of the points of most interest for stress and strain analysis were as follows:

- Position No.1, is in the centre line of a cell, directly below the centre of load: x = 0mm, y = 0mm, z (depth) = 10mm, i.e. at 1/4 depth of the plastic grid (z from the surface = 5mm + 5mm).
- Position No.2, is in the centre line of a cell, directly below the centre of load: x = 0mm, y = 0mm, z (depth) = 20mm, i.e. at ¾ depth of the plastic grid (z from the surface = 5mm + 15mm).
- Position No.3, is at the centre line of the wall of a plastic cell, with x = y = 17.5mm = (0.5 × 35mm), z = 10mm as position 1.

- Position No.4, is at the centre line of the wall of a plastic cell, with x = y = 17.5mm = (0.5 × 35mm), z = 20mm as position 2.
- Position No.5, is at the outer wall of the grid (just outside the loaded area), with x = y = 52.5mm = (0.5 × 35mm + 35mm), z = 10mm as position 1.
- Position No.6 is at the outer wall of the grid (just outside the loaded area), with x = y = 52.5mm = (0.5 × 35mm + 35mm), z = 20mm as position 2.

The '**Block Results Table**' and '**Detailed Results Table**' obtained from BISAR 3.0 analysis are included in Appendix 5.1 and Appendix 5.2 respectively. The positions of the points considered for analysis, and the stress / strain results are summarised in Tables 9.24, and 9.25.

Table 9.24 The coordinates of the positions.

Position Number	Layer Number	X-Coordinate (m)	Y-Coordinate (m)	Depth (m)
1	1	0.00E+00	0.00E+00	1.00E-02
2	1	0.00E+00	0.00E+00	2.00E-02
3	1	1.75E-02	1.75E-02	1.00E-02
4	1	1.75E-02	1.75E-02	2.00E-02
5	1	5.25E-02	5.25E-02	1.00E-02
6	1	5.25E-02	5.25E-02	2.00E-02

Table 9.25 Stress and Strain distribution at the positions listed in Table 9.24.

Post. No.	Stress XX (MPa)	Stress YY (MPa)	Stress ZZ (MPa)	Strain XX (μstrain)	Strain YY (μstrain)	Strain ZZ (μstrain)	Max Shear Stress (MPa)
1	-1.45E-01	-1.45E-01	-3.19E-01	4.87E+01	4.87E+01	-6.10E+02	8.69E-02
2	-9.36E-02	-9.36E-02	-3.12E-01	1.36E+02	1.36E+02	-6.91E+02	1.09E-01
3	-1.33E-01	-1.33E-01	-3.15E-01	6.72E+01	6.72E+01	-6.23E+02	9.26E-02
4	-7.87E-02	-7.87E-02	-2.94E-01	1.45E+02	1.45E+02	-6.71E+02	1.11E-01
5	-4.29E-03	-4.29E-03	-2.93E-03	-4.96E+00	-4.96E+00	2.18E-01	8.84E-03
6	-1.94E-02	-1.94E-02	-1.44E-02	-2.13E+01	-2.13E+01	-2.19E+00	2.29E-02

Table 9.25 reveals that all the positions analysed were subjected to predominantly compressive stresses (negative sign) in all directions, i.e. horizontal direction (X), radial direction (Y), and vertical/depth direction (Z). This appears to be in line with elastic theory, i.e. when the asphaltic upper layer rests on a very much stronger base, *all or most* of the upper layer will be subjected to compressive stresses [Croney, 1977].

The application of dynamic loads onto the system caused the development of both compressive and shear stresses as shown in Table 9.25. The overall resultant effect of the combined compressive and shear stresses is the development of strains which then propagate to become cracks, finally causing the sample to fully collapse (Fig. 9.22).

As the WPC sample did not fail during testing, this meant that the plastic cells were able to resist / contain the compressive and shear stresses developed within the sample.

9.4.5 Estimation of the load acting on the walls of the plastic cells

The effect of the plastic cell reinforcement on the CBEMs was to create a horizontal zone/layer within the mixture where the asphalt is confined in cubical cells. Overall, this layer acts to redistribute the shear and compressive stresses propagating downwards from the load. Due to the confinement offered by the walls of each cell, the materials within each cell are more shielded from the direct influence of shear stress and are therefore less able to deform due to the wall confinement. With reduced deformations, cracks become less likely to propagate.

In the case of *asphaltic concrete beams* reinforced with geosynthetic grids, Brown et. al. (2001), found that the force generated in the reinforcement strand (F) at a given crack opening (2δ) may be estimated approximately from Boussinesq's equations for an elastic half space. The relevant equation is:

$$F = \frac{2rE\delta}{(1 - v^2)} \qquad (9.1)$$

where: F = the force generated in the reinforcement strand, 2δ = the crack opening, r = the radius or half thickness of the grid strand reinforcement, E = the stiffness of the asphaltic mixture (ITSM), and ν = the Poisson's ratio of the asphaltic mixture.

Equation 9.1 was applied to this investigation as shown in Figs 9.25 and 9.26, where the crack opening occurrence was because of the development of the shear and indirect tensile strains within the sample due to compressive loading. Test results based on Dynamic Load Setting Level 3 on WPC samples provided the following data: 2δ = 0.5mm, E = 355.61 MPa, r = 0.5×0.38mm = 0.19mm, and ν = 0.35. The value of F was found to be approximately 38 N. This value F = 38 N was felt to be a sensible value that could be resisted by the plastic cell grid reinforcement.

9.4.6 Tensile Strength of the plastic strips

As the plastic strips were supplied with no technical data to support the product, direct tensile strength tests were conducted on the individual strips. The tests were carried out using an Instron Hydraulic machine (100 kN capacity) at the Nottingham Centre for Pavement Engineering, The University of Nottingham (Fig. 9.31)

Fig. 9.31 Instron Hydraulic machine (100 kN capacity).

The plastic strips were tested in direct tension in two ways:

1- by forming a full loop from a single strip as shown in Fig. 9.32 (designated as test 1 and 2), and

2- by looping the upper and lower ends of a plastic strip around the loading bars as shown in Fig. 9.33 (designated as test 3 and 4).

In either case, super glue was used to glue together the ends of the plastic strips.

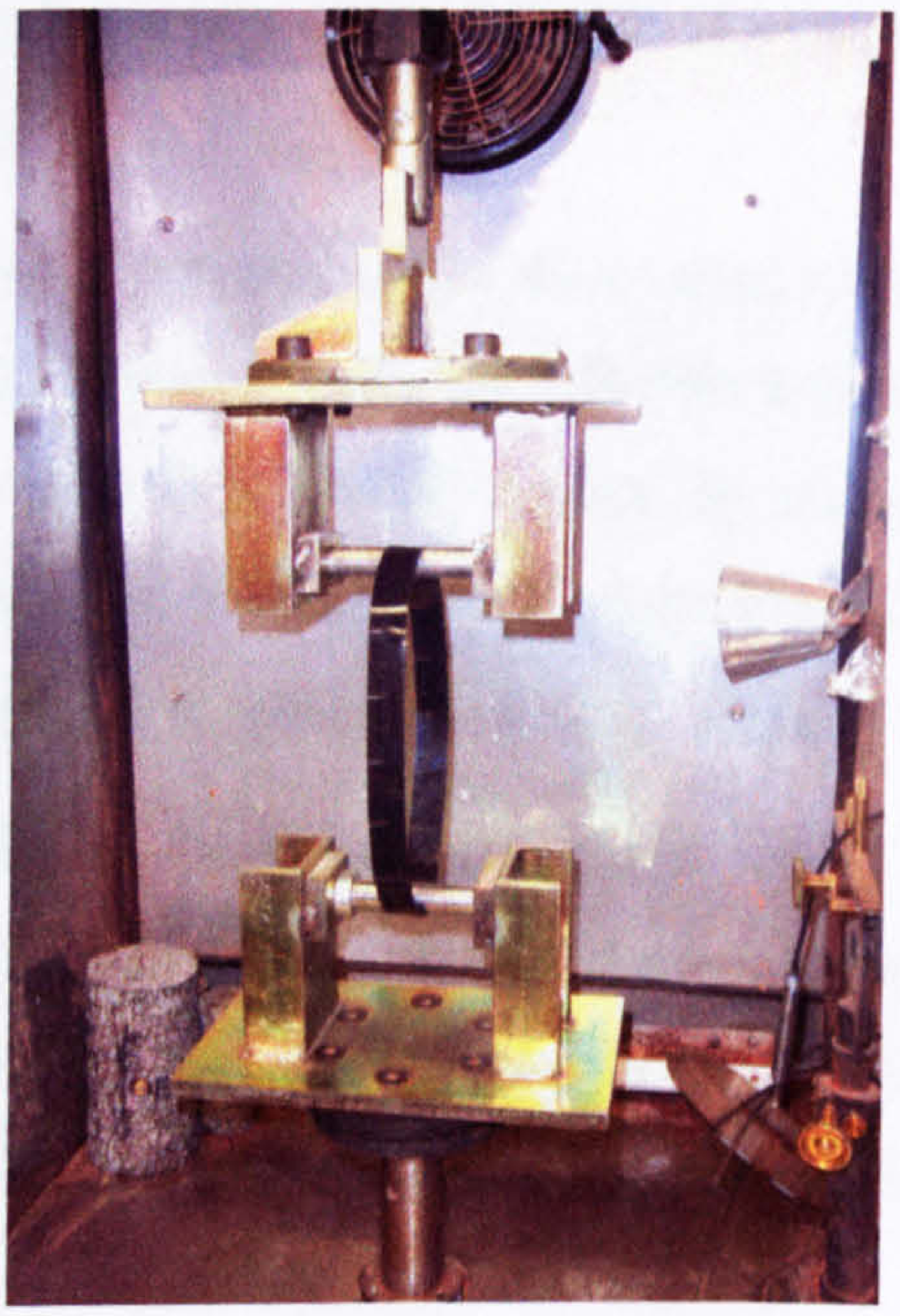


Fig 9.32 Arrangement of direct tension test (full loop).

Fig. 9.33 Arrangement of direct tension test (ends loop).

The Instron was set in a controlled strain mode of loading and the strain rate was selected at 1 mm/sec. The Instron is equipped with a temperature controlled cabinet and the test was carried out at 20°C.

Although the testing temperature of the direct tension test for the plastic strips was different from the testing temperature on the CBEMs sample with plastic cells (WPC sample), i.e. 40°C, this was confidently viewed to have no significant effect on deformation/strains, because the softening point of the plastic strips were expected to be a minimum of 140°C. Typical test results are shown in Fig. 9.34.

Since the cracks during the direct tension test would always propagate through the very sharp notches/slits that were introduced in the plastic strips, it was not felt necessary to introduce perforations into the plastic strips. The rounded holes made in the plastic strips generate much less stress intensity than the sharp notches.

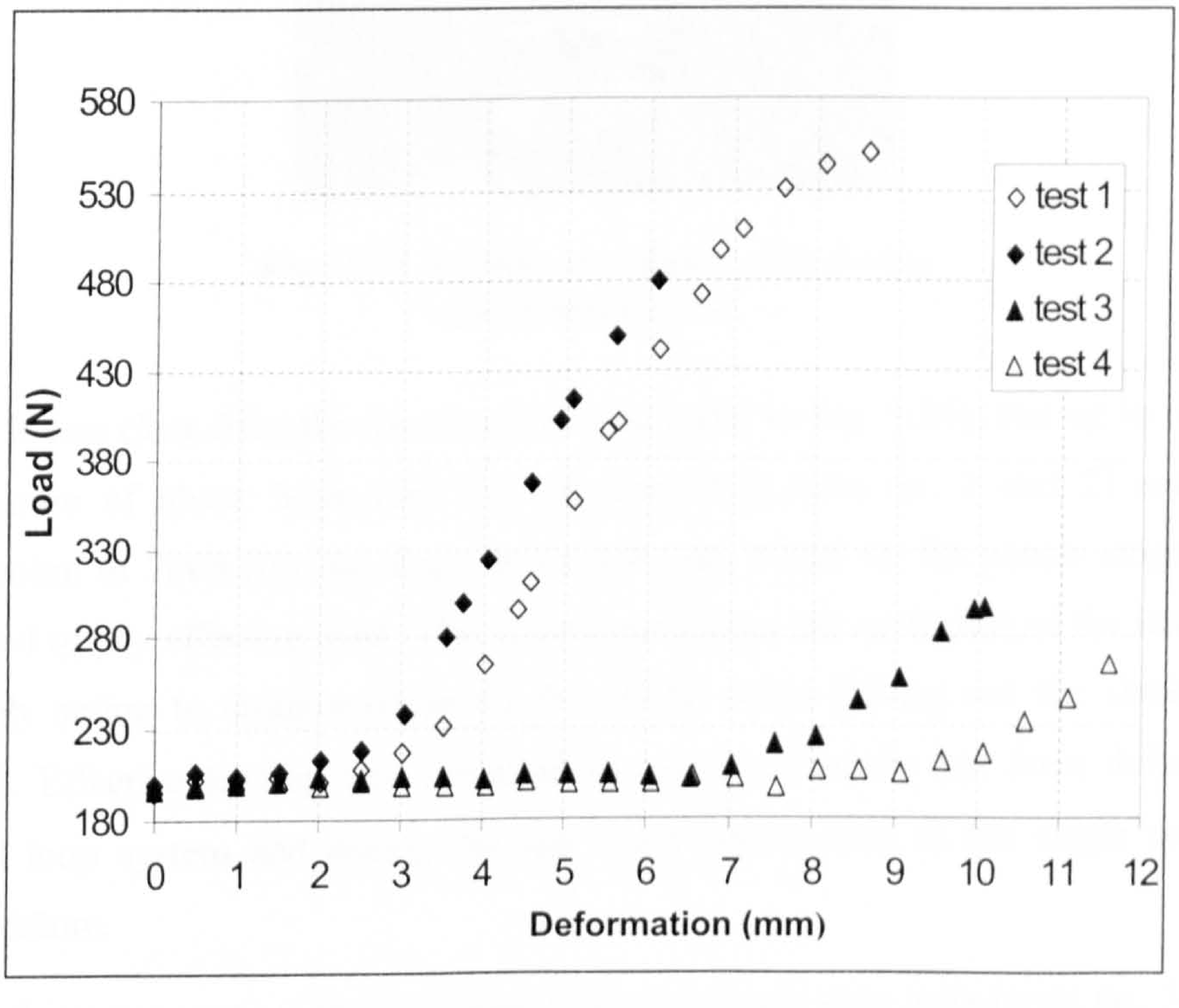


Fig. 9.34 Results of the plastic strips tensile testing.

As expected, tests 1 and 2 (test configuration as in Fig. 9.32, where the load is carried by 2 strips simultaneously) produced almost twice the failure load as tests 3 and 4 (test configuration as in Fig. 9.33, where the load is resisted by a single plastic strip).

As expected, cracks were initiated at a tip of a slit, halfway across the width of each strip, and rapidly propagated all the way across the width of the strip). No failures were observed within the glued sections (Fig. 9.35)

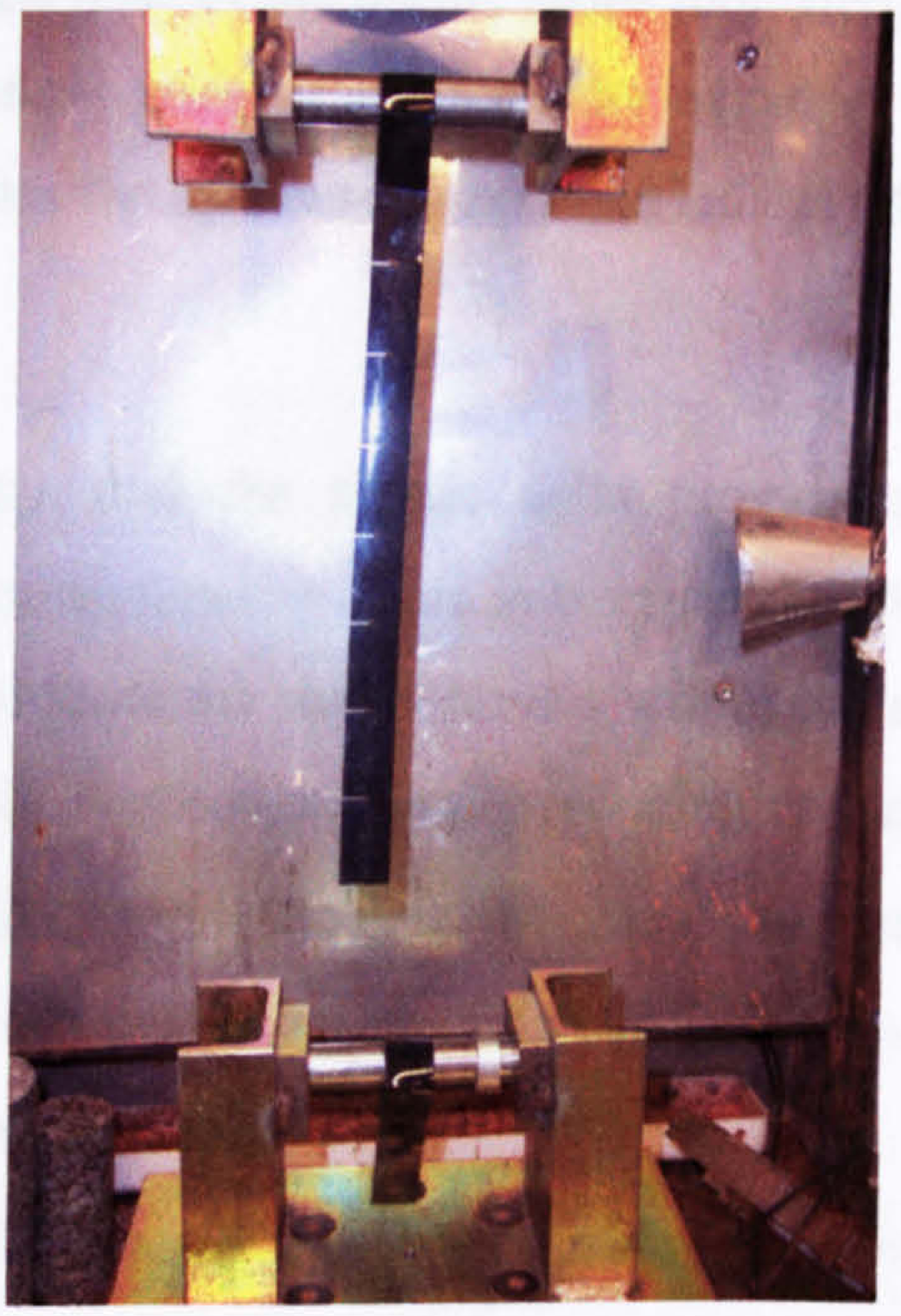


Fig. 9.35 Failure of a plastic strip during direct tension test.

It was clear from the direct tension test (refer to Fig. 9.34), that up to a measured deformation of about 3mm (for the full loop strip tests no. 1 and 2) and up to a deformation of 7mm (for the single strips tests no. 3 and 4), the plastic strips were not subjected to any effective load. This is because during the early part of the test, the load was only acting to straighten/tension the plastic strips (taking out the slack from the system). Effective loading was observed to occur within the last 5mm deformation in the full loop system and during the last 3mm deformation in the single strip system before failure.

Taking a very conservative value from the single strip tests (tests no. 3 and 4) as shown in Fig. 9.34, the tensile strength of a single strip can be estimated at approximately 250 N which is far greater that the F = 38 N calculated from Equation 9.1. As the original length of the strip was 400mm, the elongation to failure value of 3mm deformation was equivalent to 0.75%.

The effective cross sectional area of a single strip was (0.5×20mm×0.38mm) = 3.8mm^2, therefore the tensile stress capacity was 65 N/mm^2 (MPa) which is much greater than the calculated stresses as shown in Table 9.25.

The tensile strength tests of the plastic strips indicated clearly that the plastic cells can easily resist the tensile stresses predicted by equation 9.1. The failure stress of a typical strip also indicates the maximum stress that can be tolerated in the grid.

9.4.7 Discussion

Fig. 9.24 indicates that the plastic cells provide a substantial reduction to deformations under dynamic loading. This is because the plastic cells are able to hold the mixture intact, redistribute the shear stresses within the asphalt sample and *prevent the propagation of cracks*, hence significantly reducing vertical deformations.

On the negative side, the plastic cells do cause a reduction in the overall stiffness of the asphalt sample, however the reduction in stiffness was minor and within the repeatability of the ITSM test.

The incorporation of plastic cells in CBEM layers during their early life appears to be a promising idea. It appears that the most effective location for the plastic cells is higher up in the CBEM layer, close to the upper trafficked surface. Additional incorporation of cement will dramatically improve the performance of the whole system. Application of the plastic cell principle appears to be also attractive in road maintenance scenarios, e.g. trench reinstatements, and larger patches.

Single plastic strips used in this investigation (had been slotted as in Fig. 9.35) failed at approximately 0.75% (effective) elongation, which is a relatively small value. This value was determined by dividing the deformations (3mm) on effective loading condition to the original length of the plastic strip (400mm). The elongation estimated may be not be very accurate. This is due to the concentration of the stress at the notches, therefore the average elongation may underestimate the elongation at failure.

Most geosynthetic reinforcement materials require higher percentage of elongation at failure, which can be up to 5% elongation [Austin et al, 1993]. Therefore it is essential for future research to select the type of reinforcement material which would fail at a higher percentage of elongation.

9.5 Miscellaneous trials with CBEMs

This section deals with a range of additional trials carried out on CBEMs to further characterise the materials. The material types and compaction effort used were listed in each case as these were varied during the course of the investigation.

9.5.1 Water Resistance and Adhesion Tests

Referring to the latest edition of: The Asphalt Institute, 1997, *Basic Emulsion Manual*, Manual Series no. 19 (MS-19), 3rd Edition, Lexington, USA, the suitability of the emulsion must be tested by conducting the following tests: a- Coating Test, b- Water Resistance Test, and c- Adhesion Test.

a. Coating Test

The coating test was carried out following the test protocol detailed in Section 8.5.3 in Chapter VIII.

b. Water Resistance Tests

This test was conducted in accordance with the procedure recommended by the Asphalt Institute, 1997, *Basic Emulsion Manual*, Manual Series no. 19 (MS-19), 3rd Edition. The procedure is as follows:

1. Submerge fully in water a portion (100 grams) of the newly mixed asphalt mixture following the coating test.
2. Pour off the water and place the mixture on a flat surface.
3. Visually estimate the percentage or the degree of 'retained coating'.
4. If satisfactory, proceed to the adhesion test. *Otherwise choose another type or grade of emulsion.*

It was quickly realized that procedure No. 1 above was not clear, i.e. for how long should the coated loose mixture be kept before testing, and for how long should the mixture be submerged. Therefore, it was decided to cure the loose mixture at room temperature (following the coating test) for periods of: 1, 2, 3, 4, ... up to 10 hours by preparing several (ten) separate portions (about 25 grams each) of loose mixture. Trials were carried out to soak the loose mixture very soon after completion of the mixing procedure. It was observed that the emulsion was easily stripped off the aggregates, even when water was simply poured on to the mixture.

At the end of each curing period, a portion of the mixture was soaked in tap water for 5 minutes and was then agitated by stirring with a spoon 5 times at one second intervals. The mixture was then place on a paper towel on a flat surface/tray. The coating and the color of the water were then observed. The results are given in Table 9.26.

Table 9.26 shows that the CBEMs without cement were *unwashed* after 3 hours storage followed by 5 minutes of undisturbed soaking, and unwashed after 10 hours storage followed by 5 times stirring in water. Meanwhile CBEMs incorporating 2% RSC were *unwashed* after 2 hours storage followed by 5 minutes of soaking in water, and unwashed after 4 hours storage followed by 5 times stirring. A *general conclusion can be* therefore *be drawn*, that incorporation of cement can significantly reduce the stripping potential of the bitumen at a much shorter curing time. This test can be used a reference to assess the severity of exposing loose CBEMs to water attack. The author recommends that CBEMs shall be constructed whenever possible, in *dry weather* conditions.

Table 9.26 Water Resistance Test Results, in terms of estimated percentage of binder washed off.

Storage Time (hours)	WC-FA2 mixture (without cement)		WC-FA2C mixture (including 2% Rapid Setting Cement)	
	Soaked for 5 minutes	Stirred with a spoon 5 times	Soaked for 5 minutes	Stirred with a spoon
1	60% washed	70% washed	20% washed	40% washed
2	20% washed	30% washed	*unwashed*	10% washed
3	*Unwashed*	20% washed	*unwashed*	5% washed (less)
4	*Unwashed*	10% washed	-	*unwashed*
5	-	5% washed	-	*unwashed*
6	-	slightly washed (less)	-	-
7	-	slightly washed (less)	-	-
8	-	slightly washed (less)	-	-
9	-	slightly washed (less)	-	-
10	-	*unwashed*	-	-

c. Adhesion Test

The adhesion test was carried out as follows:

1. Cure 100 grams of mixture following the coating test in an oven at 60°C for 24 hours, then immediately continue to step no. 2.
2. Boil about 400 ml distilled water in a beaker, then immerse the oven cured mixture into the boiled water.
3. Bring back the water to boiling and stir at one revolution per second for 3 minutes.
4. Pour off the water and place the mixture on a piece of absorbent paper, and leave to dry.
5. Visually observe and estimate the degree of 'retained coating'.
6. If satisfactory, continue to use this emulsion/aggregate combination to produce the required number of samples. *Otherwise choose another type or grade of emulsion.*

The author observed that the adhesion test on WCI-RPSg mixture gave *satisfactory coating test results*, as shown in Figs. 9.36a and 9.36b.

a. WCI-RPSg mixture appearance after conducting the coating test and oven drying at 60°C for 24 hours

b. WCI-RPSg mixture appearance, after 3 minutes boiling in water.

Fig. 9.36 Performance of bitumen emulsion on adhesion test.

The Asphalt Institute, *The Basic Emulsion Manual*, Manual Series no. 19 (MS-19), 1997, 3rd Edition, recommends that the emulsion should not be used if it fails the tests listed above. As there are some unclear steps within the test procedure of the water resistance test, as were described earlier, this test *may not be suitable for use as a primary consideration when judging the suitability of a particular emulsion*. The test results revealed that the emulsion used in this investigation, in general, showed adequate performance.

9.5.2 Amount of liquid squeezed out during compaction

This matter was carefully observed by the author during several courses of trials. The amount of *Liquid in the Mixture* during mixing (**L in mixture**) consists of pre-wetting water and emulsion. The water content of the emulsion e.g. Nynas Emulsion was 40% water by weight of the emulsion. Depending upon the type of materials used and the compaction effort applied, the amount of liquid squeezed out during compaction can vary.

With Extra Heavy Compaction (240 revolutions at 240 kPa) using Gyropac compactor, the amount of liquid squeezed out was about 35-40% of the value '**L in mixture**'**.** Therefore about 60-65% of the original liquid remained within the compacted samples. This data were obtained by carrying out some compaction tests in which the loose mixtures were weighed before and after the compaction.

The liquid squeezed out during compaction contained a small amount of bitumen and fines, which was difficult to quantify accurately. It was noted that the squeezed liquid was dark brown in colour and rather sticky. Whilst being ejected from the samples during the compaction process, this may have caused some stripping of the compacted samples, in particular at the lower ends of the samples.

It is clear that mixtures would perform better if compacted in a drier loose state than immediately following emulsion coating. In this way, more bitumen globules within the emulsion would have set, which in turn would give better resistance to stripping. But as a consequence of the reduced workability, a heavier compaction effort would be required to achieve the target porosity.

9.5.3 Trials with WC-FA2 gradation + 2% RSC without Emulsion

The aim of this piece of work was to investigate the strength of the samples when bound only by 2% Rapid Setting Cement, (without bitumen emulsion). Two samples were produced. The aggregates (WC-FA2 gradation) and the RSC were mixed and pre-wetted with water (8% water by weight of dry aggregates resulted in maximum dampness of the aggregates without water drainage). The damp materials were then poured into the compaction moulds and initial compaction was delivered by tamping the mixture using a 12mm metal tamping rod, 15 times around the edge and 10 times in the middle. A metal wearing plate was then placed on top of the wet sample ready for compaction.

Based on practical reasons, compaction was carried out using a vibrating 'Kango Hammer', until the mixture appeared to have achieved optimum compaction. During compaction, a small amount of liquid (water + fines) was squeezed out through the upper wearing plate.

The samples were then cured in their moulds at room temperature 24°C for 7 days before extrusion. It was clear at that point that aggregate segregation was present, as more fines were present at the lower end of the samples. This was attributed to the effect of vibration during compaction, in which the slurry (of fines and water) will find their way to the lower end of the mixture. The samples were still rather damp when they were extruded, and they were therefore left to cure at room temperature for a further 24 hours. The average porosity of the samples at this stage of curing was 15.6%

The samples were then tested for ITSM. When using similar loads to those applied when testing fully cured CBEMs samples (i.e. 2000 to 2500 N), and the average ITSM value measured was 7688.1 MPa, with horizontal deformations significantly lower than those of CBEMs, (i.e. about *half* of the minimum horizontal deformation value of 5 ± 2μm recommended by BS DD213 for 100mm diameter samples).

It became clear that in the absence of a bituminous emulsion coating the mineral aggregates, samples with WC-FA2 gradation containing 2% RSC were very rigid and had lost most of their viscoelastic properties. Although high stiffness is an advantage, the list of disadvantage include: 1- it is well recognized that cementitious binders are more likely to develop cracks (in particular thermal shrinkage), 2- the reduced flexibility can be a disadvantage when the layer thickness is inadequate for the expected loading stresses, 3- the increased porosity would certainly make such mixtures less durable in a real pavement situation in particular with respect to moisture damage, 4- cementitious mixtures are susceptible to sulphate attack and carbonation.

Considering the performance of the CBEMs with a WC-FA2 gradation, it was very clear that the residual bitumen (rather than the hydrated cement phase) was controlling the mechanical behaviour of the cured mixture, and that the cementitious additives were not forming rigid links/bonds between the aggregate particles (it is more likely that they remain dispersed within the binder film and accelerate the breaking/curing process in addition to improving the stiffness of the resultant cured binder.

9.5.4 Multi Layer Compaction on WCI-RPSg mixture (WCI–ML)

Since curing of emulsion bound bituminous mixtures is such a critical parameter that influences performance, in particular moisture loss, it was hypothesized that a thinner layer would be capable of achieving a higher level of curing faster than a thicker layer. It was therefore decided in this part of the investigation to assess the mechanical properties of multi layered thin cold mixtures.

The first thin layer (approx. 25 to 30mm) was allowed to achieve a high level of curing at 40°C in an oven for 24 hours and then cooled down to room temperature for at least for 3 hours. Before constructing the second layer, a tack coat (50% emulsion : 50% water) was evenly applied onto the first layer in order to enhance the bond to the second layer. The key question was whether the two compacted thin layers (requiring a relatively short combined curing period) would behave at least as well as a fully cured (long curing period) single thick layer of equivalent thickness.

The samples were compacted using a Static Compression Machine as shown in Fig. 9.37 in order to enable the formation of two layered samples. Compacting such thin layers was not practical when using a Gyropac Compactor.

The first compacted layer was kept in the mould and cured in an oven at 40°C for 24 hours to simulate a high degree of curing prior to the application and compaction of the second layer. The combined specimens (2 layers each) were then cured again at 40°C until they achieved full curing (up to 12 days) prior to testing.

The samples were compacted at 180 kN for 3.5 minutes to approximate the porosity values achieved during extra heavy Gyropac compaction. The results of Porosity and ITSM values of all the specimens manufactured with and without cement addition are shown below in Table 9.27. The emulsion used was a Nynas Emulsion (obtained from the second supply).

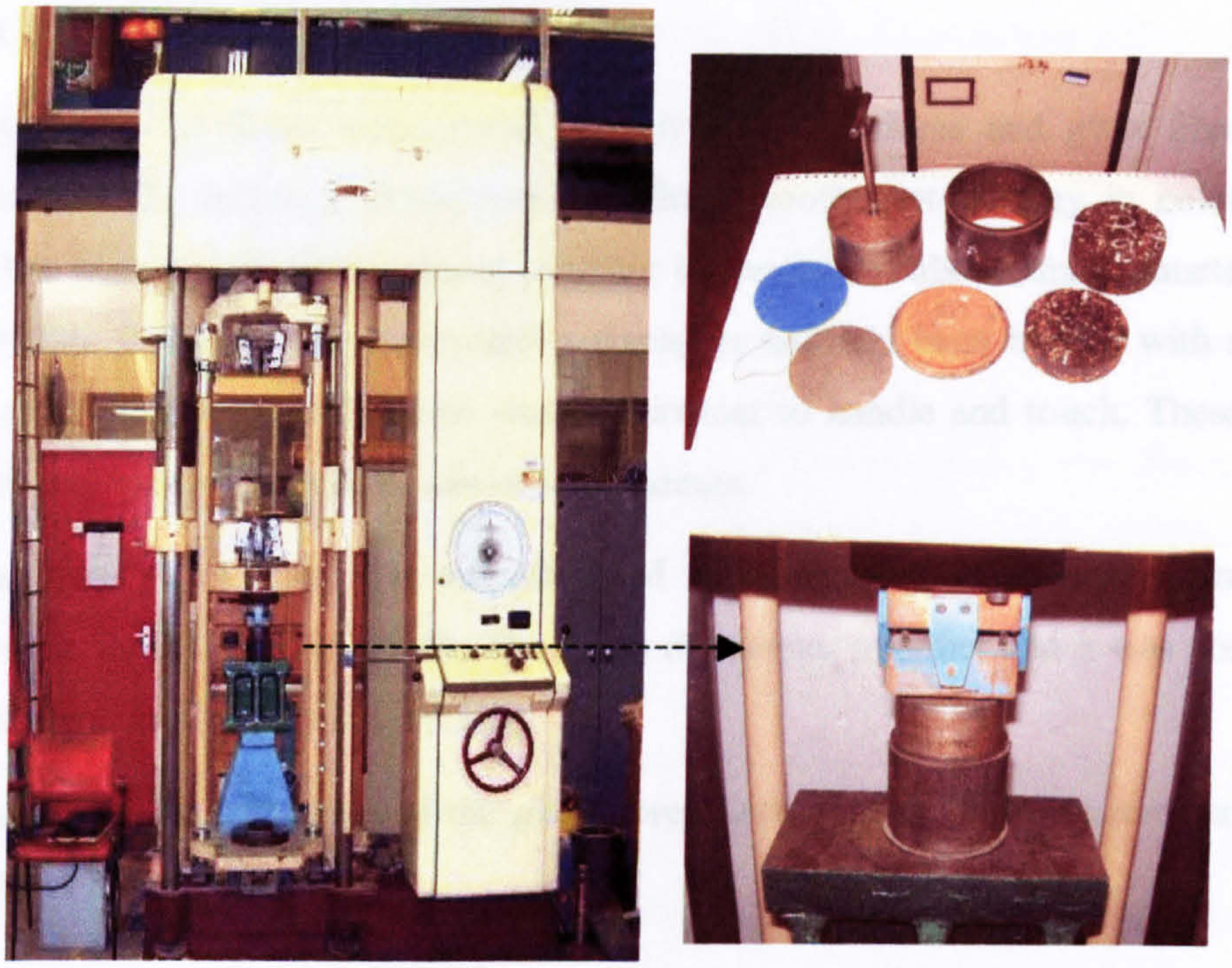


Fig. 9.37 Static Compression Machine, mould and samples for CBEMs multi layer compaction

Table 9.27 Average properties of WCI-ML at full curing, using Nynas emulsion.

Type of Mixture	Porosity (%)	ITSM (MPa)	Porosity Gyropac** (%)	ITSM Gyropac** (MPa)
WCI-ML without Cement	5.9 *	1564	7.8	1668
WCI-ML + 2% Rapid Setting Cement	6.2 *	2326 *	8.6	2468

* meets target ** Gyropac specimens compacted in the conventional manner, refer to Table 9.2 on Section 9.2.1.

It was noted that the porosity of the samples compacted using static compaction were somewhat lower than those compacted using a Gyropac (see Table 9.2, Section 9.2.1), however the ITSM values were slightly lower. Since the 2 layered specimens did not separate in half or show any signs of cracking along the line dividing the two layers, these results can be attributed to the differences in the modes of compaction. It is possible that the shear/kneading type compaction is very beneficial to the mechanical properties of the mixtures regardless of the degree of compaction. Additionally the Nynas Emulsion used for this investigation was from the earlier supply, which had been noted to give less bonding/adhesion. This should nevertheless not distract from the fact that multi layered cold mixtures are a very viable solution to the problem of curing.

9.5.5 CBEMs incorporating fibres

Two types of fibres were tested, namely cellulose fibres and glass fibres. The appearance of the cellulose fibres was like fine smooth cotton, grey in colour (Fig 9.38a). The fibre strands lightly clump together in bunches of about 2mm diameter. This cellulose fibre was made from recycled newspapers and had been treated with salts to provide pest and fire resistance and was non irritant to handle and touch. These fibres are commonly used as thermal insulation in buildings.

The glass fibres used was manufactured from waste glass. The very fine fibre threads were about 10mm in length. The fibres did adhere together and it was necessary to loosen them up manually (Fig. 9.38b).

The SG of the cellulose and the glass fibres, tested using Ultra Pycnometer 1000, were 2.4 and 2.85 respectively.

a. Cellulose Fibres. **b.** Glass Fibres.

Fig. 9.38 Type of fibres incorporated into CBEMs.

WCI-RPSg mixture incorporating fibres were manufactured using *Nynas emulsion from the second supply* with the addition of 2% Rapid Setting Cement. The amount of fibres was 0.3% by weight of total dry mixture. It was observed that the CBEMs were less workable when incorporating fibres. This was indicated by higher OPWwc requirement, i.e. 9% (instead of 8% for the WCI-RPSg, however the mixture's coating was satisfactory. Compared with the control mixture, the stiffness of the fully cured CBEMs incorporating fibres was reduced as shown in Table 9.28.

Table 9.28 Property of CBEMs incorporating fibres. Compaction Effort: Gyropac; 210revs.at 240 kPa.

Mixture composition	Porosity (%)	ITSM (MPa)
WCI +2% RSC *	8.6	2468
WCI +2% RSC+0.3% CF	10.9	1409.7
WCI +2% RSC+0.3% GF	10.3	1807.7

Note: * From Table 10.2; CF = cellulose fibre; GF = glass fibre.

Results in Table 9.28 indicate that both cellulose and glass fibres reduce workability, hence produce samples with higher porosities and lower stiffness values (ITSM).

This result was also confirmed when the author conducted trials on hot asphalt mixtures incorporating various amounts of fibres, i.e. on a BS594: 30/14 hot rolled asphalt (HRA) using limestone coarse aggregates, and asphalt concrete (A.C.- using the same materials and gradation as the WCI-RPSg). All mixtures were produced with 6% bitumen content of 100 pen grade. Both hot mixtures were manufactured with cellulose fibres (CF), and glass fibres (GF). The results are shown in Fig. 9.39. The author also observed that incorporating fibres into cold or hot asphalt mixtures requires a higher amount of bitumen content to improve workability and coating which had been confirmed by earlier investigations, e.g. Zoorob et.al., 1998.

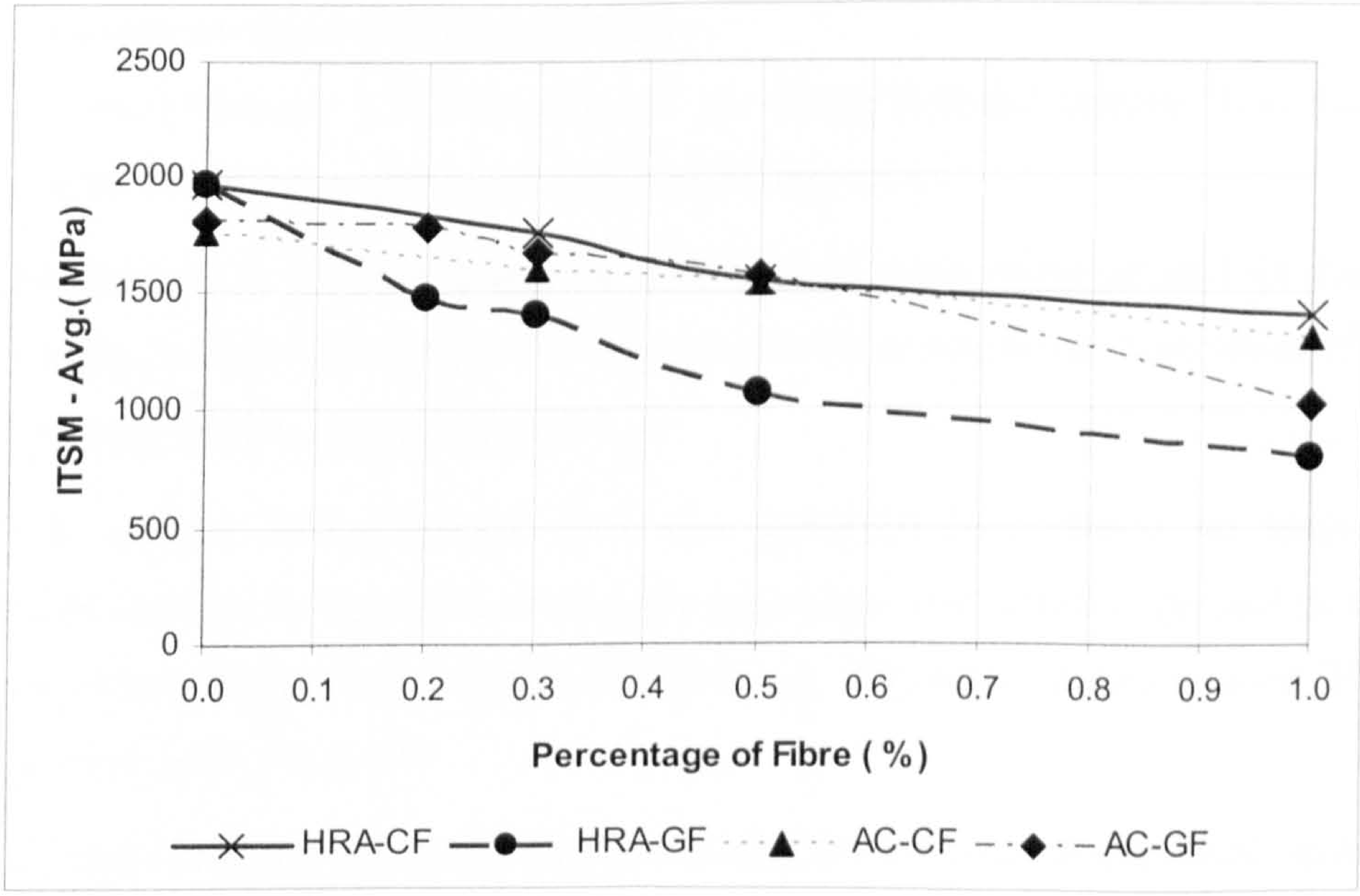


Fig. 9.39 ITSM values vs. percentage of fibres in hot mixtures.

9.5.6 Incorporation of Cement Accelerator

In an effort to improve rate of strength gain of CBEMs, trials were also carried out incorporating a cement accelerator. In theory any type of cement accelerator may can be used. In this trial *Sika3* additive (a rapid hardener for mortars, screeds, and renders) was incorporated into CBEMs due to ease of procurement. Sika3 is in liquid form with a dark green colour, with the following features quoted from the manufacturer's guidance:

- Utilization: for mortars, screeds, and renders.
- Function: to increase rate of hardening, to improve mechanical strength, to prevent cement mortars and concretes from freezing down to –2°C. Sika3 is also can be used as an *accelerator.*
- Application:
  - When used for sand cement mixtures, Sika3 should only be used with *sand cement* mixtures with a minimum ratio of 4 : 1.
  - When used in concrete mixtures, the concrete mixtures should contain a minimum of 300 kg cement per m^3.
  - When used for making cement paste, always add cement to Sika3 when mixing, i.e. add cement a little at a time into the Sika3 until the required dosage is met (point 4 below). This is to prevent the quick hardening of cement, and to maintain workability during mixing.
  - General dosage: 3.5 litres of Sika3 per 50 kg Portland cement. This may vary depending on specific needs after conducting trials.

Sika3 contains **Calcium Chloride ($CaCl_2$)**, and is not recommended for use when steel reinforcement is present. *$CaCl_2$ has been known to accelerate both the setting and the hardening of cement [Taylor, 1998].*

The amount of accelerator used was 'predetermined' based on engineering judgement, that is 1% by weight of total dry aggregates was selected, (as part of the 3% Optimum Pre-wetting Water Content-OPWwc), i.e. the added liquid becomes; 2% Pre-wetting water, plus 1% Sika3.

Trials were carried out in order to evaluate the influence of the Sika3 on the rate of strength gain of CBEMs. Mixture WC-FA2, *incorporating 2% rapid setting cement* was selected for trials. Four mixing procedures were tested, namely:

Mixture A: [{(W+S) + C} + E] + Agg.

Mixture B: {(S+C) + E} + (Agg.+W)

Mixture C: (Agg.+ C) + (W + S) + E

Mixture D: (Agg.+ C) + W only + E , without Sika3,

where: W = water; S = Sika3; C = rapid setting cement; E = Nynas emulsion; Agg. = aggregates. The mixture component within the inner brackets were mixed first.

The mixtures were compacted using a Gyropac (240 revs at 240 kPa) and cured at 24°C room temperature. Their rate of stiffness gain (ITSM at 20°C) was evaluated and the results are shown in Fig. 9.40.

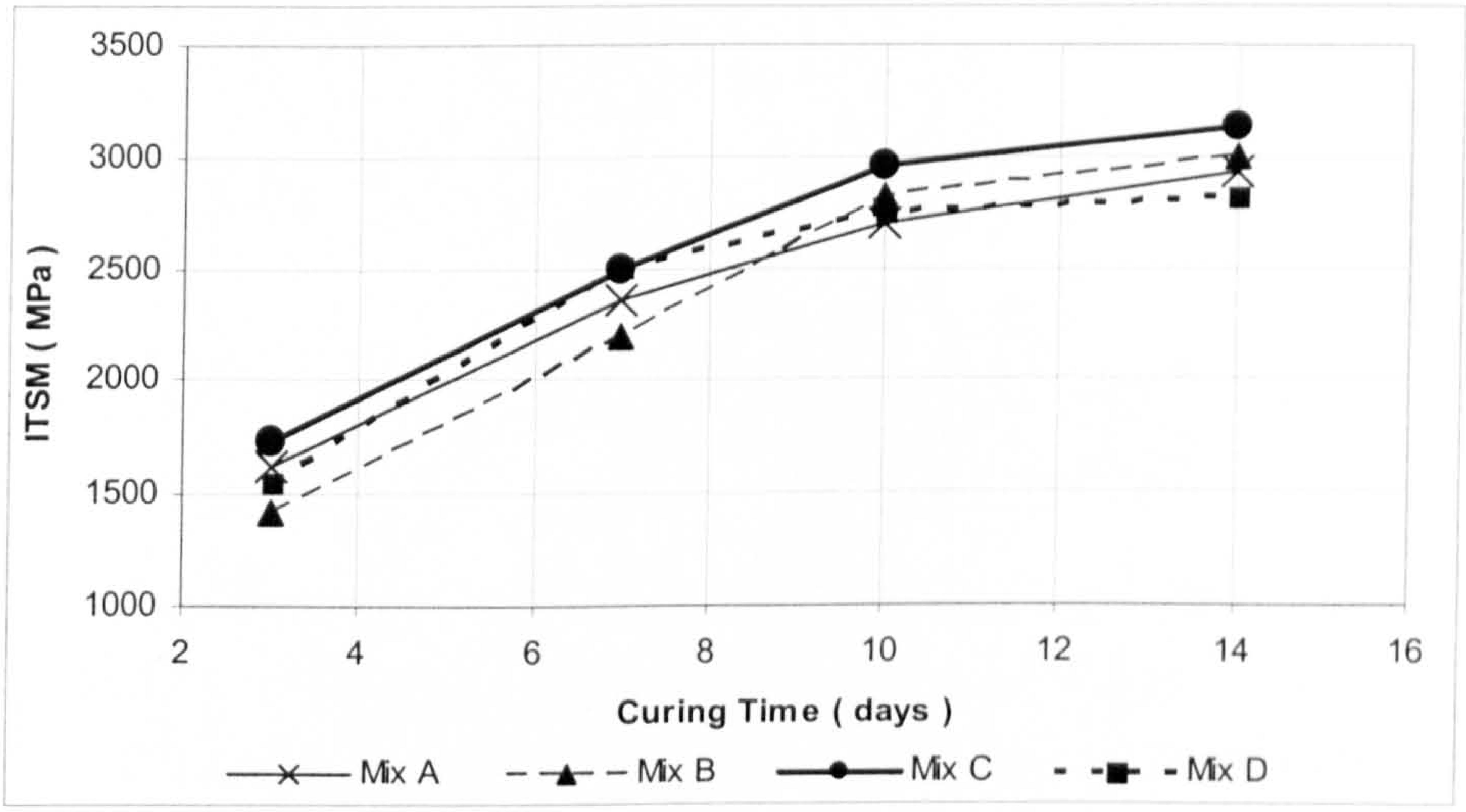


Fig. 9.40 Increase rate of strength of WC-FA2 + 2 % RSC incorporating Sika3.

In Fig. 9.40, Mixture C gave highest rate of stiffness gain. However, the rate of stiffness gain of all mixtures were not significantly different.

Further tests were carried out on WC-FA2 *without cement* for evaluating the effect of Sika3 on the strength of the samples. Due to the absence of cement, the OPWwc was reduced to 2%. The mixtures tested were:

Mixture E: agg. + 2% W + E

Mixture F: agg + (1% W + 1% Sika3) + E.

The test results are shown in Fig. 9.41, where the difference in the rate of stiffness increase was negligible. *At full curing*, the mixtures also had *very similar ITSM values* as shown in Table 9.29.

Table 9.29 ITSM values of Mixture E and Mixture F (as shown in Fig. 9.40) at full curing.

Mixture	ITSM at Full Curing (MPa)
Mixture E (without Sika3)	2535.8
Mixture F (with Sika3)	2500.0

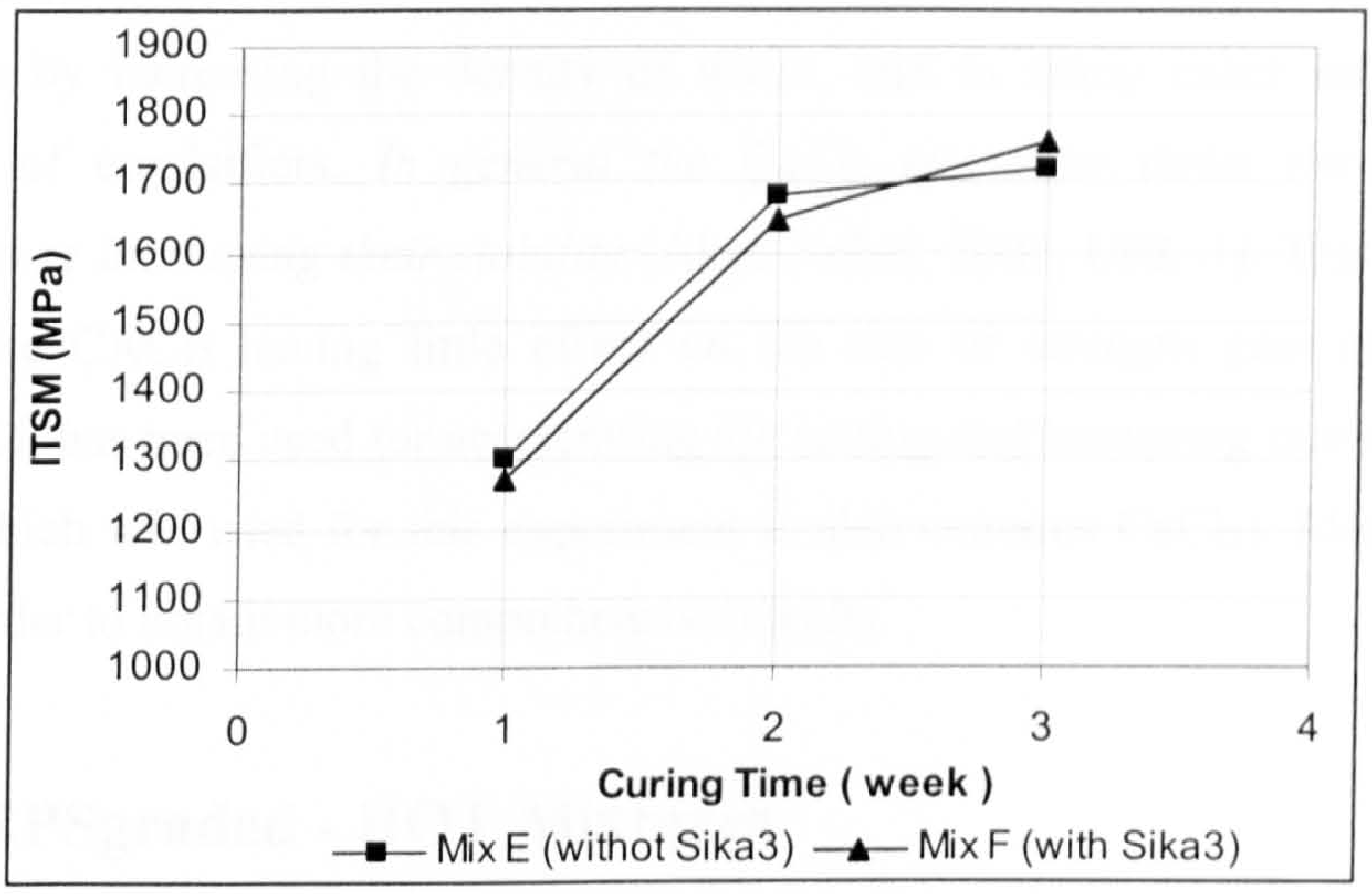


Fig. 9. 41 Effect of Sika3 on CBEMs without Cement.

Considering the above test results, it became clear that the incorporation of Sika3 into the formulation of CBEMs with or without cement, regardless of the method of incorporation, did not significantly affect the rate of CBEMs stiffness increase. The amounts of Sika3 incorporated were as follows:

- 1% out of the 3% total pre-wetting liquid (water +Sika3) in Mixtures A, B, C and D, i.e. about a third of the total pre-wetting liquid.
- 1% out of the 2% the total pre-wetting liquid (water +Sika3) in Mixtures E and F, i.e. about a half of the total pre-wetting liquid.

The Sika3 amounts incorporated as described above were quite *substantial amounts*, however no significant increase in the rate of strength gain was obtained. Utilizing a higher percentage of Sika3 may give a clearer effect, however this would certainly be uneconomical.

Referring to Chapter II, Section 2.3.3 on Stabilisers, where it was described that bitumens may contain left over salt from inadequate desalting of the crude oil. This salt can cause ***swelling*** of the bitumen droplets through '***osmosis***', i.e. when there is a high content of salt in the bitumen and water is pulled into the bitumen droplets. This can increase the emulsion viscosity and affect stability.

In a cationic system, about 0.05-0.2% **Calcium Chloride** ($CaCl_2$) or other soluble salts can be added into the water phase of the emulsion (as a stabiliser) to help in reducing osmosis of water into the bitumen. $CaCl_2$ can also reduce the rate of settlement of emulsions by increasing the density of water, and in many cases improves the performance of emulsifiers. *In general the $CaCl_2$ can slow down the setting of emulsions, hence increasing their stability* [Akzo Nobel, 2002, URL-1]. This may have resulted in the $CACl_2$ having little effect on the rate of strength gain of CBEMs, although $CaCl_2$ has been used for accelerating the setting and hardening rate of cements (the Sika3 which was used for this experiment is also contains $CaCl_2$). More work is required in order to obtain more comprehensive results.

9.6 WC-RPSgraded - HOT Mixtures

For comparison purposes, a WCI-RPS graded mixture with the same material composition and gradation as those used for CBEMs (Table 9.1, No.1, Section 9.2.1), was manufactured as a hot mixture using 100 pen bitumen, with a 6% bitumen content. The mixing temperature was 140°C, and the compaction temperature was 125°C [Whitoeak, 1991]. The compaction efforts were Medium and Heavy compaction with the following results obtained:

Table 9.30 Comparison of Properties of WCI-RPSgraded hot and cold mixtures at full curing.

WCI-RPSgraded	Hot Mixtures	
	Porosity (%)	ITSM (MPa)
Medium Compaction	4.7	1961.0
Heavy Compaction	3.4	2325.5
	Cold Mixtures *	
Extra Heavy Compaction	7.8 **	1668**
Extra Heavy Compaction	8.6 ***	2468 ***

* extra heavy compaction = Gyropac set at 210 revs. and 240 kPa . ** without cement.
*** with 2% rapid setting cement (see Table 9.2, Section 9.2.1).

Test results in Table 9.30 indicate that *hot mixtures*, when mixed and compacted at the correct temperatures can give good coating and good workability which is indicated by the lower porosity values achieved at lower compaction efforts than those required for cold mixtures.

Compared with the CBEMs stiffness values as shown in the previous sections, when properly designed and at full curing, CBEMs (even without cement) were comparable in stiffness to hot mixtures, although the porosity values were generally higher (e.g. see Table 9.14, Section 9.3.5).

CHAPTER X
OVERALL SUMMARY

10.1 General

The results and discussions presented earlier in Chapters VII and IX for each stage of the investigation were quite extensive. This chapter is intended to summarize and highlight the important findings from those discussions, without re-presenting in any detail the results of the investigations.

10.2 Coating Test

It was realised after many experiments, that the *coating test* was very essential and should form an integral part of the CBEMs design procedure. Occasionally, a coating retest may even be necessary prior to a full scale on-site production of a CBEM, particularly when there is a long gap between the time of designing and producing the CBEM.

Different emulsion types can have differences in storage stability, workability, and coating ability. The age and frequent exposure of the emulsion to open air was found to affect the quality and performance of the emulsions.

The type of aggregate materials used in the cold mixture, in particular the water absorption properties, were found to significantly affect the workability during the mixing and compaction stages. Furthermore, the degree of 'affinity' between the aggregates and bitumen emulsion can contribute towards achieving a satisfactory degree of binder coating. It is well documented in the literature that *calcareous/alkaline aggregates such as limestones and basalts* are positively charged, and are therefore more suitable for mixing with negatively charged anionic emulsions. On the other hand, siliceous/acidic aggregates such as granites and quartzites are negatively charged, and are therefore more suitable for mixing with positively charged cationic emulsions. In practice, it is found that cationic emulsions have good affinity with most aggregate types, e.g. as experienced by the author when mixing CBEMs using cationic emulsions and limestone aggregates with satisfactory results. This is because the cationic emulsifier acts as an adhesion promoter and an anti stripping agent.

Workability and hence the degree of coating can be improved by increasing the amount of pre-wetting water up to an optimum level, referred to as the Optimum Pre-wetting Water Content (OPWwc). In practice, pre-wetting water contents higher than the OPWwc may be required depending on the mixing equipment used. If the mixture becomes rather sloppy during the mixing stage due to excessive fluids in the system, the mixture must ideally be sufficiently air dried for optimum compaction, the extent of this air drying depends on the compaction equipment and compaction effort used.

For site applications, the requirement for pre-wetting the aggregates must take into consideration the existing water content of the aggregate materials. Under laboratory conditions however, dry aggregates are preferred because they are easier to proportion accurately. The author's experiments showed that the workability of CBEMs during the mixing stage were not significantly changed either by introducing the pre-wetting water content just before introducing the emulsion or by pre-dampening the aggregates to simulate wet site conditions.

10.3 Storage of loose CBEMs

When not required for immediate compaction, the best environment for storing a loose CBEM for any period of time prior to compaction is in a sealed environment. A sealed environment will delay the setting process of the bitumen emulsion as the water content of the system is prevented from evaporation. As described earlier in Chapter IX, Section 9.2.6.1, CBEMs without cement can be stored in a sealed environment for up to 24 hours prior to compaction without significantly affecting their workability. On the otherhand, for CBEMs incorporating cement, e.g. incorporating 2% Rapid Setting Cement (RSC), it is suggested that compaction be carried out soon after mixing to avoid any reduction in workability during compaction caused by the formation of hydrated cementitious products. Whenever possible, it is preferred that the loose mixture be stored with a slightly higher than optimum but not excessive amount of pre-wetting water content.

10.4 Compaction

A higher level of compaction effort for CBEMs than normally used for hot mixtures was found in this investigation to be essential for achieving the porosity targets. The correct level of compaction effort must be determined experimentally that would guarantee a satisfactory mixture density and hence porosity.

The water content of the loose CBEMs prior to compaction must be carefully selected and controlled for effective compaction. Gyratory compaction was found to be more effective than traditional impact type compaction (Marshall hammer). Only minor a modification to the Gyratory moulds was required to allow the dissipation of excess pore fluids from the system. The excess liquid present in the loose CBEMs can cause the Marshall hammer to bounce off the specimen during compaction, on the other hand, the combined shear and kneading type compaction is more effective at dissipating the pore fluid pressures developed in the specimens during compaction.

10.5 Bitumen Film Thickness (BFT)

The BFT requirement, which was included as part of the design procedure, is based on an empirical calculation which considers mainly the gradation of the various mineral aggregate fractions and assumes that the aggregate particles are evenly coated with the residual bitumen content. In reality, especially so with CBEMs, the residual bitumen does not evenly coat the aggregate surfaces and reduced coating is common on the coarse aggregate fractions. The bitumen emulsion generally preferably adheres to the fines during the mixing stage. In this investigation, better coating was experienced when the CBEMs were mixed with sufficient pre-wetting water content.

10.6 CBEMs using pre-coated aggregate with hot bitumen

Pre-coating the mineral aggregates with hot bitumen before introducing the emulsion was found to be very interesting. The main advantages of pre-coating are as follows: CBEMs require less pre-wetting water as the bitumen pre-coated aggregates absorb much less water / liquid from the emulsion, the usual problems of 'affinity' between the aggregates and the emulsion are eliminated, binder coating is improved; more consistent workability is guaranteed as the coated aggregates require a more consistent amount of pre-wetting water compared with uncoated aggregates, less 'bitumen emulsion' is required overall. However, on the negative side, pre-coating the mineral aggregates with hot bitumen will require heat energy.

10.7 Retained ITSM of CBEMs

Following water immersion tests (2×24 hours capillary soaking based on the design curing procedure), the *retained strength* of the CBEMs in terms of Marshall Stability and ITSM (at full curing) were well above 50%, which satisfactorily met most specifications. Some samples indicated retained ITSM values at full curing above 100% which is likely to be caused by the pore water pressure generated within the voids during load applications.

10.8 Compaction in Multi Layers for the WC I (WC I – ML)

Construction of CBEM specimens in Multi (Two) Layers gave satisfactory results. Compaction of the second layer was carried out after the first layer had achieved a sufficient level of curing. Good bond between the two layers was ensured by applying a proper tack coat. Multi layered cold mixtures are a very viable solution to the problem of curing.

10.9 Incorporation of Cement Accelerator

It is known that in general most cement accelerators can contain calcium chloride ($CaCl_2$) for accelerating the setting and hardening of OPC [Taylor, 1998]. In this investigation it was found that the incorporation of a cement accelerator in the CBEM formulation did not result in any significant acceleration in the rate of CBEM strength gain. This was thought to be because in bitumen emulsion technology, $CaCl_2$ is used to increase the bitumen emulsion stability and can slow down the rate of settlement of the of bitumen emulsion [Akzo Nobel, 2002, URL-1]. Additionally, the method of introducing the cement accelerator into the CBEMs in this investigation may not have been the optimum technique.

10.10 Incorporation of Crushed Glass (Glass cullet)

Incorporation of crushed glass in various applications including asphalt mixtures has been specified by the Texas Department of Transportation [Texas Dot, 2002,URL-

15]. The Texas Dot recommends that the percentage of glass cullet permitted in asphalt-stabilized bases and open-graded base courses should not exceed 5% of the total weight of aggregates (Table 4.16). UK full scale trials on a hot bituminous mixture containing 30% crushed glass **'Glasphalt'** have produced very satisfactory results [Nicholls & Lay, 2002].

In this investigation, the author had incorporated crushed glass into CBEMs at various levels up to 30% by mass then substituted by volume. At full curing, CBEMs incorporating 30% crushed glass gave satisfactory results, in particular when incorporating 1-2% cement (Table 9.12, in Chapter IX). When finely ground, glass has pozzolanic properties [Pascoe et al, 2001], which can be attractive for incorporation into CBEMs.

10.11 Fatigue and Dynamic Creep Tests

When a comparison is carried out on mixtures using the same residual binder grade, test results have revealed that at full curing CBEMs without cement are generally weak and have inferior mechanical properties compared to the hot mixtures. Typically the hot mixtures have roughly 3-4% lower porosity values compared to the CBEMs (without cement) produced in this investigation. The incorporation of cement significantly improved the fatigue performance of CBEMs. When fatigue life was plotted against initial tensile strains, CBEMs incorporating cement showed a significant improvement in fatigue life at strain values below 100 μs (Fig. 9.8, and Table 9.17).

At full curing, CBEMs without cement showed higher creep slope values than comparable hot mixtures. On the otherhand, compared with typical laboratory dynamic creep slope values of hot mixtures (Table 5.4), it was shown that by incorporating 2% RSC into CBEMs (with 100 pen base bitumen), the creep slope values of the CBEMs at full curing (Table 9.19), were smaller than the hot mixtures with a harder (50 pen.) binder. Therefore, theoretically CBEMs incorporating 2% Rapid Setting Cement, at full curing, have a very good resistance to permanent deformation and are suitable for heavy traffic.

10.12 CBEMs with Plastic Cells

The incorporation of plastic cells into CBEM mixtures provided a dramatic reduction in permanent deformation during dynamic creep tests. The plastic cells were able to hold the mixture intact and *prevent the propagation of radial cracks*, hence significantly reducing the vertical deformations as was shown in Fig. 9.27.

However, the plastic cells did cause a slight reduction in dynamic stiffness of the test specimens as a result of the partial loss of interlock within the mixture at the interfaces between the cells and the aggregate matrix.

The plastic cells should ideally be located as close as possible to the upper surface of the asphalt surfacing. By incorporating cement, the performance of CBEMs with plastic cells can be further improved. Applications where the idea of plastic cells can be attractive include maintenance operations, e.g. cold mixture asphalt patches, or trench reinstatements.

10.13 The Role of Cement in CBEMs

It is widely described in the literature that the incorporation of a small amount of cement into CBEMs can significantly improve the overall performance of CBEMs. In order to avoid the CBEMs becoming too stiff and susceptible to cracking, it is suggested that the amount of cement incorporated be limited to no more than 2%.

Cement in CBEMs behaves as an active filler by affecting the pH of the system. In the case of cationic bitumen emulsions, the system is acidic, generally with pH values < 7. The incorporation of cement (typically with pH levels between 11 to 13), the pH level of the CBEMs system will be affected (increased) causing the emulsion to become unstable and to set faster.

The production of CBEMs requires some amount of pre-wetting water and furthermore the emulsion itself consists of bitumen droplets in a water phase. The silicates in cement, i.e. the tricalcium silicates (C_3S) and the dicalcium silicates (C_2S) are the two most important compounds that are responsible for the strength of the hydrated cement paste. In the presence of water (pre-wetting water or water of the emulsion), the silicates and aluminates of the cement form hydration products, or hydrates which produce a firm and hard mass. In this process, the C_3S and C_2S of the cement reacts with water and form microcrystalline calcium silicate hydrates also known as C-S-H [Neville & Brooks, 1997].

With CBEMs incorporating cement, the presence of bitumen does not prevent the formation of cement hydration compounds which are responsible for the formation of bonds within the asphalt mixture. This was investigated by Giuliani [Giuliani, 2001] using X-ray diffraction. He was able to show diffraction patterns with peaks indicative of the crystalline structure of the hydrated cement, as shown in Fig. 10.1.

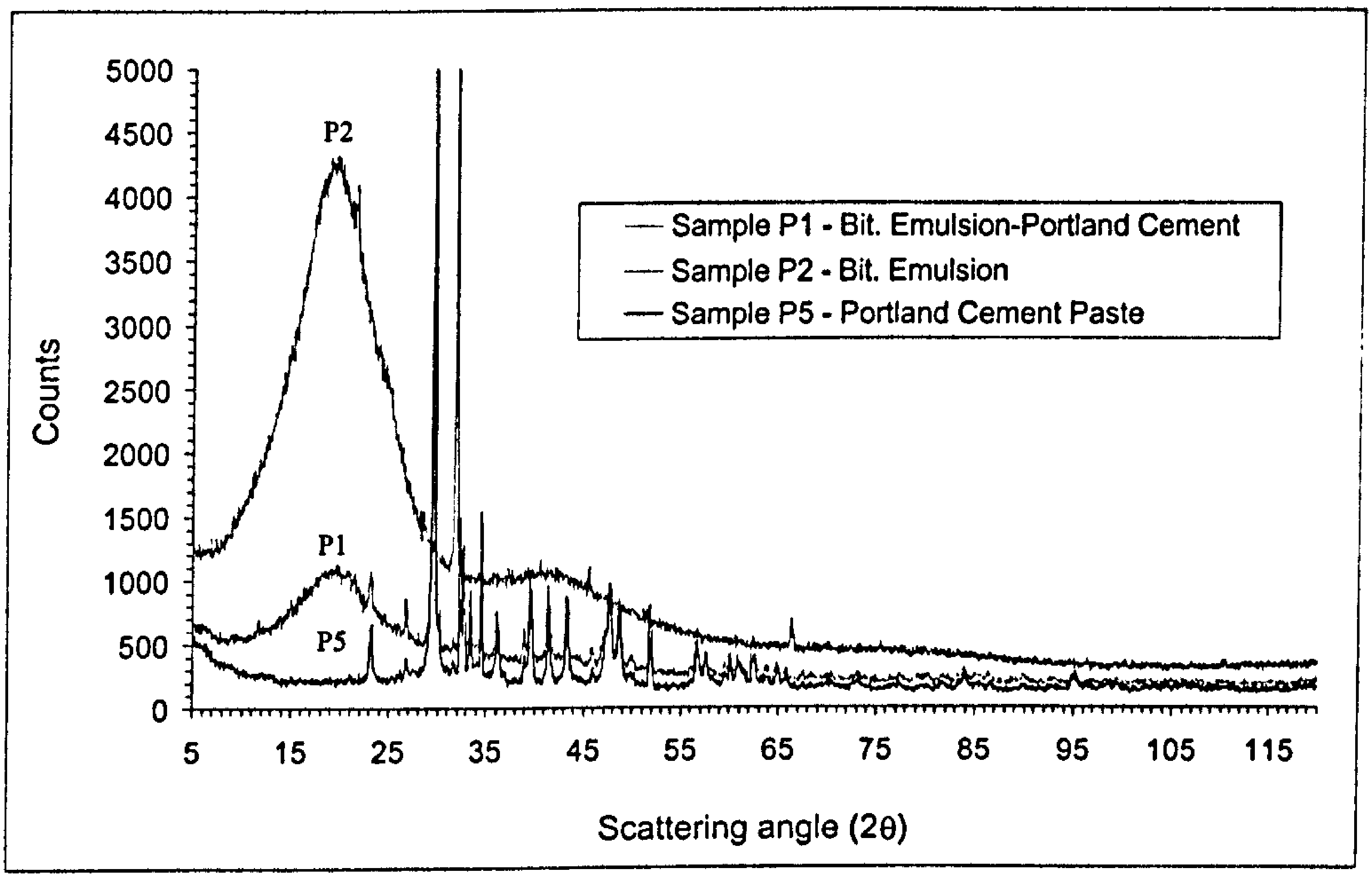


Fig. 10.1 X-ray diffraction pattern of samples.

10.14 The rate of strength gain of compacted CBEMs cured out doors

The author had evaluated the rate of increase of strength of CBEMs cured out doors at an average temperature of 10°C with frequent light rain. It was found that compacted CBEMs without and with 2% Rapid Setting Cement required 16 weeks and 2 weeks respectively for achieving the target ITSM range of 2000-2500 MPa. Naturally, in warmer dry temperatures, the CBEMs rate of strength gain would have been significantly faster.

10.15 Suitable filler materials for CBEMs

Due to the wet condition of freshly mixed CBEMs, **pozzolanic and cementitious** materials are most suitable for use as filler.

A **pozzolanic** material is a siliceous or siliceous and aluminous material, which in itself possesses little or no cementitious property, but which will, in finely divided (ground) form and in the presence of *moisture*, chemically react with *calcium hydroxide* at ordinary temperature to form compounds possessing cementing properties. *Volcanic ashes, calcined clays, and pulverized coal ash* from thermal power plants are among the most commonly used pozzolanic materials [Malhotra & Mehta, 1996]. A pozzolan *has to be mixed either with lime or with portland cement to develop cementing action.* When using cement, calcium hydroxide is one of the products of portland cement-water interaction.

A **cementitious** material is a finely divided (ground) and non crystalline or poorly crystalline material (similar to pozzolan), but *containing sufficient calcium to form compounds* which posses cementing properties after interaction with water. Examples of cementitious materials are; *high calcium/lime fly ash and granulated blast-furnace slag (GGBS).* High calcium fly ash is produced by the combustion of '*lignite coals and sub-bituminous coals*' from coal power generation plants.

CHAPTER XI

CONCLUSIONS AND RECOMMENDATIONS

11.1 Conclusions

In line with the main objectives of the investigation listed earlier in Chapter I, the following conclusions are the key outcomes from this investigation:

1. The proposed CBEMs design procedure, described in Chapter VIII section 8.5, which combines the best features from the following specifications: 1- Asphalt Institute's Asphalt Cold Mix Manual (MS-14, 1989), 2- Ministry of Public Works Indonesia (1990), and 3- Nikolaides A.F. (1990), is recommended for its simplicity and ease of application, in particular when applied in remote areas, using simple equipment, and less skillful workers.
2. The coating test, which assesses the affinity of the selected emulsion type to the mineral aggregates under investigation, plays a significant and essential role in the design of CBEMs.
3. The porosity values of cold bituminous emulsion mixtures can be successfully reduced to meet a pre-selected target simply by increasing the compaction effort.
4. Compaction effort is a significant variable that must be determined depending on the target porosity, mixture type, mixture storage conditions (sealed or unsealed), appropriate water content for compaction, and storage time prior to compaction.
5. The early life strength of cold bituminous emulsion mixtures can be improved and curing times significantly shortened by incorporating cementitious or pozzolanic binders.
6. The rate of increase in strength of cold bituminous mixtures incorporating cements depends on the type of cement used. Rapid Setting Cement (RSC) appears to provide the fastest rate of increase of strength, followed by OPC and finally Natural Cement.

7. At full curing the CBEMs incorporating cement can have comparable stiffness (indirect tensile and dynamic creep) properties to conventional hot mixtures.
8. Depending on the exposure conditions, loose CBEMs can be stored for much longer times than hot asphalt mixtures. The optimum storage condition for CBEMs is in a sealed environment.
9. In this investigation, it was found that selected waste materials, e.g. red porphyry sand, synthetic aggregates made from sintering quarry fines, crushed glass and the PFA can be readily incorporated into CBEMs in a range of quantities with acceptable mixture stiffness values in all cases.
10. Compressible materials such as *crumb rubber* were found to be not suitable for incorporation in CBEMs. Cracks were observed to occur in the freshly compacted mixtures, as a result the rebound of the highly elastic crumb rubber particles, soon after compaction.
11. The incorporation of steel slags (basic oxygen slag-BOS) as coarse aggregate into CBEMs can be risky. Cracks were observed to develop in samples incorporating steel slag during a 40°C curing regime, which was believed to be caused by the volume expansion of the steel slag at moist conditions. This did cause some concern regarding the durability of such mixtures on site. Unless the volumetric expansion properties of the steel slags can be controlled, the utilization of steel slags in CBEMs is not recommended.
12. Pre-coating the mineral aggregate fractions of the CBEMs with hot bitumen can provide several advantages including: reducing the amount of pre-wetting water, eliminating the problem of affinity between the aggregates and the bitumen emulsion, giving more consistent mixture workability, and reducing the amount of bitumen emulsion required.
13. Multi (two) layered cold mixture applications appear to give satisfactory mechanical properties. By laying and compacting a thick CBEM layer in several thinner lifts instead of the conventional single lift, the overall curing process is accelerated and a fully cured condition across the entire depth of the CBEM layer is more likely to be achieved much earlier.

14. It was shown experimentally that the utilization of plastic cells to confine the cold asphalt mixtures during the construction stage, can dramatically reduce the risk of rutting in CBEMs, which is a major concern at early life. The optimum location for the cells was found to be as close as practicable to the upper trafficked pavement surface.

11.2 Recommendations for Further Research

Based on the experiences gained by the author during the course of this investigation, some recommendations for future research are listed below:

1. In order to progress research into cold bituminous mixtures, researchers require better understanding of the basic chemical properties of the bitumen emulsions being investigated and any resultant aggregate/emulsion interactions. More openness in sharing information on the side of the bitumen and emulsion producers is essential in order to progress the current state of cold asphalt research.
2. A more precise evaluation of the changes that occur in bitumen emulsions during storage, handling, exposure to air in relation to the size of the bitumen droplets, their stability, rate of setting, coating ability and adhesion. Such data would be very useful for assisting the asphalt engineers at the mixture design stage (as proposed in Chapter VIII, Section 8.5) and in producing more consistent high quality CBEMs.
3. There is a need for evaluating the rheological properties of the residual bitumen from bitumen emulsions in particular with respect to short and long term ageing when compared to identical grades of penetration grade binders.
4. A more comprehensive comparison of the mechanical properties (ITSM and dynamic creep) of CBEMs at full curing compared with hot mixtures having the same penetration grade base bitumen and volumetric properties is required.
5. Development of laboratory and on-site air tight containers to store bitumen emulsions that can provide regular gentle circulation/agitation to maintain homogeneity is highly recommended.

6. The selection of suitable laboratory mixing equipment that can create adequate mixture coating and that are more representative of on-site operations should form part of any cold asphalt mixture investigation.
7. Modification of the standard compaction moulds to enable proper drainage of excess water during CBEMs compaction without draining/loosing the fine aggregate fractions is highly recommended.
8. Pre-coating the mineral aggregates with a small amount of hot bitumen is a very good solution to the problem of binder adhesion in particular for aggregates with less affinity to the bitumen emulsion. More detailed research into the optimum pre-coating binder contents and binder types is required.
9. Experimentation with the utilization of various types of pozzolanic materials (e.g. volcanic ash, calcined clays, metakaolin, risk husk, and silica fume), and cementitious materials (e.g. high calcium fly ash, ground granulated blast furnace slag-GGBS) in CBEMs is highly recommended.
10. Trials with different types and amounts of cement accelerators on CBEMs incorporating cement would be beneficial in accelerating the mixture curing times.
11. The idea of incorporating geo-synthetic grids/ cells appears to be very promising in laboratory tests and requires more detailed investigations into the mechanical and economical benefits.

REFERENCES

Abel F.(1978).*Foamed Asphalt Base Stabilisation.* 6th Annual Asphalt Paving Seminar, Colorado State University, **cited by Muthen K.M.**(1998). *Foamed Asphalt Mixes Mix Design Procedure.* Contract Report CR-98/077, Dec 1998, SABITA Ltd. & CSIR Transportek.

Ahmed R. and Klunderf .(1994). *Rubber Recycling* . 20th WEDC Conference , Colombo , Sri Lanka , 1994.

Aderson Allan .(1995). *Gyropac and Matta Training Course.* Australian Road Research Board Ltd.

Anderson et.al.(1992). *Rheological Properties of Mineral Filler-Asphalt Mastics and Its Importance to Pavement Performance.* Symposium on Effect of Aggregates and Mineral Fillers on Asphalt Mixture Performance, Editor : Richard C. Meininger, pp. 131- 153, ASTM STP 1147, Publication Code Number (PCN) 04-011470-08, Philadelphia , USA.

Asphalt Institute.(1977). *A Brief Introduction to Asphalt and Some of Its Uses. Manual Series No. 5 (MS-5)* , Seventh Edition.

Asphalt Institute. (1984). *Mix Design Methods for Asphalt Concrete and Other Hot Mix Types. Manual Series No. 2 (MS-2).*

Asphalt Institute. (1983). *Asphalt Cold-Mix Recycling.* Manual Series No. 21 (MS – 21) , Maryland 20740 , USA.

Asphalt Institute .(1989). Asphalt *Cold Mix Manual* . Manual Series No. 14 (MS – 14) , Third Edition , Lexington , KY 40512 – 4052 , USA.

Asphalt Institute.(1991). *Thickness Design of Asphalt Pavements for Highways & Streets.* Manual Series No. 1 (MS – 1) , Maryland 20740 , USA.

Austard J. and Troan A.K. (1997). *Cold Mix Recycling of Asphalt Pavements – Field Trials in Norway,* Proceedings of the Second European Symposium on Performance and Durability of Bituminous Materials , University of Leeds. pp. 337 to 350. Aedificatio Publishers , Zurich.

Austin Deron, N., Wu Kevin, J., and White David, F. (1993). The influence of test parameters and procedures on the tensile modulus of stiff grids. Geosynthetic Soil Reinforcement Tesing Procedures, ASTM, STP 1190, Editor: S.C. Jonathan Cheng, pp. 90 to 110, Philadelphia, PA 19103.

Barth, E.J.(1962). *Asphalt Science and Technology.* Gordon and Breach Science Publisher , London.

British Standard Institution.(1984). *BS 434 Part 1 and 2: Bitumen Road Emulsions (anionic and cationic)* , London.

British Standard Draft for Development DD – 213 .(1993). *Method for the Determination of the Indirect Tensile Stiffness Modulus of Bituminous Mixtures.*

British Standard Draft for Development, 2nd Draft-DD for Fatigue Characteristics. (2002). *DD ABF - Method for the Determination of the fatigue Characteristics of bituminous mixtures using indirect tensile fatigue.*

Button Joe, W. and Little Dallas N. (1987). *Asphalt Additives for Increased Pavement Flexibility.* Research Report 471-2F , Research Study 2-9-85-471, Texas Transportation Institute , Texas – USA.

Bicysco, S.J. (1994). *Performance of Cold Recycled Bituminous Material.* Proceedings of the First European Symposium on Performance and Durability of Bituminous Materials , University of Leeds , March 1994 , Editor : J.G. Cabrera and J.R. Dixon , pp. 270 to 277 , E & FN Spon , London.

Brown, S.F.,Thom, N.H., and Sanders, P.J.(2001)*A study of grid reinforced asphalt to combat reflection cracking,* Proceeding of the Association of Asphalt Paving Technologists (AAPT), Vol. 70, 2001, pp 543-571, Florida, USA.

Coomarasamy, A. & Walzak, T. L.(1995). *Effects of Moisture on Surface Chemistry of Steel Slag and Steel Slag-Asphalt Paving Mixes,* TRR1492, pp. 85-95, Transportation Research Board, Washington, D.C.

Cooper, K.E. , Brown, S.F. and Pooley, G.R. (1985). *The Design od Aggregate Gradings for Asphalt Basecourses* , Journal of The Association of Asphalt Paving Technologists , Vol. 54 , pp. 324 to 346.

Crick, J. (1994) . *Cold Mix Macadam Production* . Proceedings of the First European Symposium on Performance and Durability of Bituminous Materials , University of Leeds , March 1994 , Editor : J.G. Cabrera and J.R. Dixon , pp. 253 to 258 , E & FN Spon , London.

Cutbert, J. H. (1997). *An Investigation Into the Recycling of Bituminous Pavements.* Master of Engineering Final Year Report , Department of Civil Engineering , Civil Engineering Materials Unit , University of Leeds , UK.

Chapuis, R.P. and Legare, P.P. (1992). *A Simple Method for Determining the Surface Area of Fine Aggregates and Fillers in Bituminous Mixture* . Symposium on Effect of Aggregates and Mineral Fillers on Asphalt Mixture Performance , Editor : Richard C. Meininger , ASTM Publication Code Number (PCN) 04-011470-08 , Philadelphia , USA.

Chaignon Franccois. (1998). *Cold Mix – The French and European Experiences.* Conference on The Cold Road Ahead , Queen Elizabeth II Conference Centre , 24 March 1998 , London.

Croney David. (1977). *Design and Performance of Road Pavements.* Transport and Road Research Laboratory, London.

Croney Paul and Croney David. (1998). *The Design and Performance of Road Pavements* . McGraw-Hill , Third Edition , New York.

Derucher et al.(1998). *Materials for Civil & Highway Engineers*, Prentice Hall, New Jersey 07458.

Department of Scientific and Industrial Research Road Research Laboratory .(1962). *Bituminous Materials in Road Construction* . Her Majesty's Stationary Office , London.

Di Beneto and De La Roche .(1998). *State of the Art of Stiffness Modulus and Fatigue of Bituminous Mixtures.* In Bituminous Binders and Mixes , Rilem Report 17 , Belgian Road Research Centre , Brussels , Belgium , Edited by Francken L , pp. 137 – 180 , E & F Spon , London.

Dinnen Joe.(1998). *Use of Cold Emulsions in Recycling* . Conference on The Cold Road Ahead , Queen Elizabeth II Conference Centre , 24 March 1998 , London.

Dhir, R.K., Limbachiya, M.C., Dyer, T.D. (2001). *Glass Recycling: Combining Enhanced Recycling Methods and Alternative Route to Close Loop,* in Proceedings of the International Symposium, organised by the Concrete Technology Unit and held at University of Dundee, Scotland, UK on 19-20 March 2001, *Recycling and Reuse of Glass Cullet,* edited by **Dhir, R.K. et al.,** Thomas Telford Publishing, London.

Earland M and Nunn ME. (1998). *Foamix Technical Report.*TRL,PR/CE/158/98.
Emery J. J.(1982). *Slag Utilization in Pavement Construction, Extending Aggregate Resources,* ASTM STP 774, American Society for Testing and Materials, pp. 95-118.

Fordyce,D. , Khweir, K. and Newhall, M. (1999). *Advances in Low Energy Compaction System Using Emulsified Bitumen.* Proceedings of the Third European Symposium on Performance and Durability of Bituminous Materials and Hydraulic Stabilised Composites, University of Leeds , pp. 413 to 424 , Aedificatio Publishers , Zurich.

Fekpe Edward, S.K. (1988) . *Compaction of Bituminous Mixtures* . Thesis of Master of Science , Department of Civil Engineering , University of Leeds , UK.

Gaestel, C. (1968). The breaking Mechanism of Cationic Bitumen Emulsions. Sci Road and Building Materials Group. pp. 221-224. Cited by Needam, D. (1996). *Developments in Bitumen Emulsion Mixtures for Roads.* Thesis submitted to the University of Nottingham for the degree of Doctor of Philosophy.

Gibb, John M. (1996). *Evaluation of Resistance to Permanent Deformations in the Design of Bituminous Paving Mixtures.* Thesis submitted to the University of Nottingham for the degree of Doctor of Philosophy. Nottingham University, UK.

Giuliani, F. (2001). X-ray diffraction method for studying cement-modified bitumen emulsion mixtures in asphalt pavement cold recycling. 1St International Symposium on Subgrade Stabilisation and in-situ Pavement Recycling using cement, 1-4 October 2001, Salamanca (Spain).

Halleett, P. (2001). *Overview of Current Situation and Recommendations,* in Proceedings of the International Symposium, organised by the Concrete Technology Unit and held at University of Dundee, Scotland, UK on 19-20 March 2001, *Recycling and Reuse of Tyre,* edited by **Dhir, R.K. et al.**, Thomas Telford Publishing, London.

Heslop Martin. (1998). *Bond Coats – Their Use and Advantages* . Conference on The Cold Road Ahead , Queen Elizabeth II Conference Centre , 24 March 1998 , London.

Higgins R.A..(1993). *Engineering Metallurgy, Applied Physical Metallurgy* 6th edition, pp.140-150.

Highway Authorities and Utilities Committee (HAUC).(1992). *New Roads and Streetworks Act 1991 – Specification for the Reinstatement of Opening in Highways - HMSO.*

Horvli Ivar . (1999) . *Cyclic Load Triaxial Test on Cold Mix Asphalt* . Proceedings of Fifth International Conference on the Bearing Capacity of Roads and Airfields , pp. 1153 to 1162.

Horvli Ivar and Troan ,A.K. (1999) . *Gyratory Testing of Cold Mix Asphalt* . Proceedings of Fifth International Conference on the Bearing Capacity of Roads and Airfields , pp. 1143 to 1152.

Ibrahim, H.E. and Thom ,N.H. (1994) . *The Effect of Emulsion – Aggregate Mixture Stiffness on Both Mixture and Pavement Design* . Proceedings of the Second European Symposium on Performance and Durability of Bituminous Materials , University of Leeds , pp. 351 to 368 , Aedificatio Publishers , Zurich.

Kim ,Y.R. , Kim N. , and Khosla , N.P. (1992) . Effect of Aggregate Type and Gradation on Fatigue and Permanent Deformation of Asphalt Concrete. In Symposium on Effect of Aggregates and Mineral Fillers on Asphalt Mixture Performance , Editor : Richard C. Meininger , pp. 310-328, ASTM STP 1147 , Publication Code Number (PCN) 04-011470-08 , Philadelphia , USA.

Krebs, R.D. and Walker, R.D. (1971) . *Highway Materials* . McGraw-Hill Book Company.

Kett Irving (1999) . *Asphalt Materials and Mix Design Manual* . Noyes Publications , New Jersey – USA.

Khalid H.A. & Eta K.E. (1996), *Laboratory and Field Performance of Dense Emulsified Bitumen Macadam for Highway Reinstatement*, University of Liverpool, U.K.

Khalid, H.A. and Eta, K.E. (1997) . *Structural Support Values for Emulsified Bitumen Macadams in Highway Reinstatement* . Proceedings of the Second European Symposium on Performance and Durability of Bituminous Materials , University of Leeds , pp. 307 to 326 , Aedificatio Publishers , Zurich.

Kliewer J.E.,Bell C.A.,and Sosnovske D.A.(1995). *Investigation of the Relationship Between Field Performance and Laboratory Ageing Properties of Asphalt Mixtures (SHRP A – 003 A)* , Engineering Properties of Asphalt Mixtures and the Relationship to their Performance, ASTM STP 1265, Gerals A. Hubber and Dale S. Decker, Eds., American Society for Testing and Materials, Philadelphia.

Kennedy John .(1998). *Energy Minimisation in Road Construction and Maintenance Using Cold Emulsion Materials* . Conference on The Cold Road Ahead , Queen Elizabeth II Conference Centre , 24 March 1998 , London.

Kallas Bernard, F. (1979). *Elastic and Fatigue Behaviour of Emulsified Asphalt Paving Mixes* . Research Report No. 79 – 1 , The Asphalt Institute , Maryland 20740 , USA.

Lee, D.Y.(1981). *Treating marginal aggregates and soil with foamed asphalt.* In: Proceedings of the Association of Asphalt Paving Technologist, Vo. 50, pp 150-211.

Leech, D. (1994) . *Cold Bituminous Materials for Use in the Structural Layers of Roads* , Transport Research Laboratory , Project Report 75 , UK.

Leech, D. and Sexton, B. (1996). *Standardisation trial of performance tests for bituminous materials.* Department of Transport, Transport Research Laboratory, TRL Report 158. Crowthorne, cited by **Suparma, L.B. (2001).** *The Use of Recycled Waste Plastics in Bituminous Composites* , Thesis submitted to the University of Leeds for the degree of Doctor of Philosophy.

Leech, D. and Nun, M.E. (1997). *Deterioration mechanism in flexible roads, Performance and Durability of Bituminous Materials*, Proceedings of the Second European Symposium on Performance and Durability of Bituminous Materials , University of Leeds , pp 271-292., Aedificatio Publishers , Zurich.

Lewis, T.(1998). Personal Discussion. Pretoria. A.A.Loudon & Partners Consulting Engineers.

Logaraj, S. et all and Decanio, S.J. et all (2000). *Emulsification, A solution to Asphaltene Handling problems.* ISSA/AEMA 2nd Joint Conference, March 12-13, 2000, Amelia Island. Florida, USA.

Luminary and Fidato . (1988). State *of the Art Report on Mix Design* . In Bituminous Binders and Mixes , Rilem Report 17 , Belgian Road Research Centre , Brussels , Belgium , Edited by Francken L , pp. 69 – 101 , E & F Spon , London.

Maccaronne, S., Holleran, G., & Ky, A. (1995). *Cold Asphalt System as an Alternative to Hot Mix.* In: m9th AAPA International Asphalt Conference.

Ministry of Public Works Republic of Indonesia. (1990). *Paving Specifications Utilizing Bitumen Emulsions* , Jakarta.

McClory Mark .(1998). *Permanent Cold Lay Surfacing Materials.* Conference on The Cold Road Ahead , Queen Elizabeth II Conference Centre , 24 March 1998 , London.

Milton, L.J. and Earland, M.G. (1999). *Design Guide and Specification for Structural Maintenance of Highway Pavements by Cold in-situ Recycling* , Transport Research Laboratory Report 386 , UK.

Muthen K.M.(1998). Foamed Asphalt Mixes Mix Design Procedure. Contract Report CR-98/077, Dec 1998, SABITA Ltd. & CSIR Transportek.

Needam, D. (1996). *Developments in Bitumen Emulsion Mixtures for Roads.* Thesis submitted to the University of Nottingham for the degree of Doctor of Philosophy.

Neville & Brooks. (1997). *Concrete Technology,* Longman Ltd., Essex-England.

Nicholls J.C. (1998) . *Asphalt Surfacings.* TRL , E & FN Spon , London.

Nicholls, J.C. (1999). *European Standards for Test Methods on Asphalt Mixtures.* Proceedings of the Third European Symposium on Performance and Durability of Bituminous Materials and Hydraulic Stabilised Composites, University of Leeds , pp. 145 to 164 , Aedificatio Publishers , Zurich.

Nichols J.C. and Lay J.(2002). *Crushed Glass in Macadam for Binder Course and Road Base Layers.* Proceedings of 4th European Symposium on Performance of Bituminous and Hydraulic Materials in Pavements, BITMAT 4, University of Nottingham, U.K., 11-12 April 2002, pp. 197-212, A.A.Balkema Publishers, Rotterdam Netherlands.

Nikolaides, A.F. (1983). *Design of Dense Graded Bituminous Emulsion Mixtures and Evaluation of their Engineering Properties* , Ph.D Theses , University of Leeds , UK.

Nikolaides, A.F. (1994) . *A New Design Method for Dense Cold Mixtures* . Proceedings of the First European Symposium on Performance and Durability of Bituminous Materials , University of Leeds , March 1994 , Editor : J.G. Cabrera and J.R. Dixon , pp. 259 to 269 , E & FN Spon , London.

Nikolaides , A.F. (1994) . *Construction and Performance of Dense Cold Bituminous Mixtures as Strengthening Overlayer and Surface Layer* . Proceedings of the First European Symposium on Performance and Durability of Bituminous Materials , University of Leeds , March 1994 , Editor : J.G. Cabrera and J.R. Dixon , pp. 259 to 269 , E & FN Spon , London.

Nikolaides , A.F. (1997) . *Effect of binder content variation on the stiffness and fatigue of asphaltic concrete* . Proceedings of the Second European Symposium on Performance and Durability of Bituminous Materials , University of Leeds , April 1997 , Editor : J.G. Cabrera, pp. 227 to 240 , Aedificatio Publishers, Zurich.

Nick Nuttall. (2000) . *Recycling Crisis Over Green Glass Mountain* . The times , Edition Monday July 17 , 2000 , UK.

Oliver, J.W.H. and Alderson, A.J. (1995) . *The Effect of Air Void Content on Resilient Modulus , Dynamic Creep , and Wheel Tracking Result.* In Result of Laboratory Program Associated with ALF Asphalt Deformation Trial , Austroads Pavement Research Group (APRG) Report No. 12 , pp. 2.1-2.11 , Australian Research Board Ltd , Victoria Australia.

Pascoe, R.D., Barley, R.W., Child, P.R. (2001). *Autogeneus Grinding of Glass Cullet in a Stirred Mill,* in Proceedings of the International Symposium, organised by the Concrete Technology Unit and held at University of Dundee, Scotland, UK on 19-20 March 2001, *Recycling and Reuse of Glass Cullet,* edited by **Dhir, R.K. et al.**, Thomas Telford Publishing, London.

Phillips, P. (1997) . *Cold Mix Asphalt Comparison Between Field Results and Laboratory Simulation.* Proceedings of the Second European Symposium on Performance and Durability of Bituminous Materials , University of Leeds , pp. 161 to 172 , Aedificatio Publishers , Zurich.

Phillips, P. (1998) . *Evaluation , Prediction and Enhancement of HAUCphalt – a Cold Mix Asphalt Technology* , Theses of Master of Philosophy , University of Ulster , North Ireland , UK.

Plotnikova, I.A. (1993). *Control of the interaction process between emulsion and mineral aggregates by means of physico-chemical modification of their surfaces.* First World Congress on Emulsion, Paris. pp. 2-10-176/01-06, cited by **Needam, D.** (1996).

Developments in Bitumen Emulsion Mixtures for Roads. Thesis submitted to the University of Nottingham for the degree of Doctor of Philosophy.

Puzinauskas, V.P. (1983) . *Fillers in Asphalt Mixtures* , Research Report No. 69-2 , RR-69-2 , The Asphalt Institute , Maryland - USA

Read, J.M. and Collop, A.C. (1997). *Practical Fatigue characterisation of bituminous paving mixtures.* Proceeding of the Association of Asphalt Paving Technologists. Vol. 66. pp. 75-108, cited by **Suparma, L.B. (2001).** *The Use of Recycled Waste Plastics in Bituminous Composites* , Thesis submitted to the University of Leeds for the degree of Doctor of Philosophy.

Read, J.M. and Brown, S.F. (1994). *Fatigue Characterisation of Bituminous Mixes Using a Simplified Test Method.* Proceedings of the First European Symposium on Performance and Durability of Bituminous Materials , University of Leeds , March 1994 , Editor : J.G. Cabrera and J.R. Dixon , pp. 158 to 172 , E & FN Spon , London.

Robinson, H.L. (1997) . *Thin Asphalt Surfacings Using Cold Mix Technology* . Proceedings of the Second European Symposium on Performance and Durability of Bituminous Materials , University of Leeds , pp. 369 to 384 , Aedificatio Publishers , Zurich.

Robinson, H.L. (1998) . *The Latest Maintenance Solutions May Be Cold Rather than Hot* . Conference on The Cold Road Ahead , Queen Elizabeth II Conference Centre , 24 March 1998 , London.

Roque, R. and Buttlar, W.G. (1992). "The development of a measurement and analysis system to accurately determine asphalt concrete properties using the indirect tensile mode". *Proceeding of the Association of Asphalt Paving Technologists.* Vol. **61.** pp. 304-331, cited by **Suparma, L.B. (2001).** *The Use of Recycled Waste Plastics in Bituminous Composites* , Thesis submitted to the University of Leeds for the degree of Doctor of Philosophy.

Ruckel, P.J.et.al. (1979). *Foamix Asphalt Advances, Asphalt Pavement Construction : New Materials and Technique.* ASTM STP 724 , 1979 , pp. 93 – 109.

Ruckel, P.J., Acott, S.M/ & Bowering, R.H. (1982). *Foamed-asphalt mixtures:preparation of design mixes and treatment of test specimens.* In: Asphalt materials, mixtures, construction, moisture effect and sulphur. Washington DC: Transport Research Board (Transport Research Record 911), pp88-95.

Sakr, H.A.&Manke, P.G.(1985)*Innovations in Oklahoma foamix design procedures.* In: Asphalt materials, mixes, Construction and quality. Washington, DC: Transportation Researh Board. (Transpotattion Research Record 1034), pp 26-34.
Sherwood, P.T.(1995). Alternative Materials in Road Construction. Thomas Telford, London.

Singleton ,T.M. (2000). *Characterisation of Impact Absorbing Asphalt*, PhD Thesis, School of Civil Engineering, The University of Nottingham, June 2000.

Suhaibani , Mudaiheem , and Al-Fozan . (1992) . *Effect of Filler Type and Content on Properties of Asphalt Concrete Mixes* . Symposium on Effect of Aggregates and Mineral Fillers on Asphalt Mixture Performance ,Richard C. Meininger , Editor , pp. 107 – 130 , ASTM Publication Code Number (PCN) 04-011470-08 , Philadelphia , USA.

Suparma, L.B.(1998) . *The Use of Plastic Waste in Bituminous Composites*, First Year Report, Submitted for the fulfilment of the requirement of transfer to Ph.D. program , Civil Engineering Department , Leeds University.

Suparma, L.B. (2001). *The Use of Recycled Waste Plastics in Bituminous Composites* , Thesis submitted to the University of Leeds for the degree of Doctor of Philosophy.

Taylor, M.B. (1997) . *Assessment of Cold Mix Emulsions* . Proceedings of the Second European Symposium on Performance and Durability of Bituminous Materials , University of Leeds , pp. 399 to 408 , Aedificatio Publishers , Zurich.

Taylor Mike. (1998). *Cold Mix Technology and Advancemet of Binders* . Conference on The Cold Road Ahead , Queen Elizabeth II Conference Centre , 24 March 1998 , London.

Taylor, H.F.W.(1998). *Cement Chemistry.* 2nd edition, Thomas Telford Publishing Services Ltd., London.

Thanaya I.N.A.(2000). *Improving The Performance of Cold Bituminous Emulsion Mixtures (CBEMs) Incorporating Waste Materials.* First Year Report submitted for the requirement of Transfer to Ph.D. Program. School of Civil Engineering, University of Leeds, U.K.

Thanaya, I.N.A. and Zoorob, S.E. (2002). Improved Mix Design Procedure for Cold Asphalt Mixtures. Proceedings of 5th Malaysia Road Conference, 7th – 9th October 2002, Kuala Lumpur.

Thom, N.H. & Choi, Y.K, Collop, A.C. (2002). *Top-down cracking, damage and hardening in practical flexible pavement design.* A PDF file document, School of Civil Engineering, Nottingham University, 2002.

Turner-Fairbank Highway Research Center.(2000). *User Guidelines, Steel Slag,* U.S. Department of Transportation, Federal Highway Administration, 27th March 2001,

Van de Loo, P.J. (1978). *The creep test: A Key Tool in Asphalt Mix Design and in the Prediction of Pavement Rutting.* Proceeding of the Association of Asphalt Paving Technologists. Vol. 47. pp. 522-555, cited by **Suparma, L.B. (2001).** *The Use of*

Recycled Waste Plastics in Bituminous Composites , Thesis submitted to the University of Leeds for the degree of Doctor of Philosophy.

Wainwright P.J. & Creswell D.J.F.(2000). *Synthetic aggregates from waste using an innovative rotary kiln*, Proceedings Int. Conference Science & Engineering of recycling for environmental protection (WASCON 2000) Harrogate, England, June 2000.
Wates, J.M., James, A.D. (1993). *Zeta-potential measurements on bitumen emulsions and roads aggregates.* First World Congress on Emulsions. Paris. pp 1-40-089/01-06. Cited by **Needam, D.** (1996). *Developments in Bitumen Emulsion Mixtures for Roads.* Thesis submitted to the University of Nottingham for the degree of Doctor of Philosophy.

Whitoeak, D.(1991). *The Shell Bitumen Hand Book*. Shell Bitumen – UK.
Wirtgen.(1998). *Wirtgen Cold Recycling Manual.* A.A. Loudon &Partners Consulting Engineers.

Woodside, A. , Phillips, P. and Mills, A. (1999). *Maximisation of Recycled Asphalt Use in Cold Mix and Hot Mix Environment.* Proceedings of the Third European Symposium on Performance and Durability of Bituminous Materials and Hydraulic Stabilised Composites, University of Leeds , pp. 401 to 412 , Aedificatio Publishers , Zurich.

Zoorob, S.E. (1995). *The Effect of Pulverised Fuel Ash on the Properties and Performance of Hot Rolled Asphalt.* Ph.D Thesis , University of Leeds , UK.

Zoorob, S.E., Cabrera, J.G., and Kamarudin, I. (1998). *Design of Fibre Reinforced Hot Rolled Asphalt for Thin Overlay Surfacings.* Proceedings of First World Conference on Highway Surfacing, May 11-12, 1998, Hotel Atrium Hyatt-Budapest.

Zoorob S.E. (2000). *Pavement Materials and Design & Pavement Design and Maintenance* , Course Notes , Civil Engineering Department , Leeds University.

Zoorob S.E.(2002). *Design and Construction of Hot-mix Bituminous Surfacings and Roadbases,* in *Highways, The Location, Design, Construction & Maintenance of Pavements,* **Edited by C.A. O'Flaherty,** 4th Edition, Butterworth-Heinemann, Oxford.

Zoorob S.E. & Thanaya I.N.A. (2002). *Improving The Performance of Cold Bituminous Emulsion Mixtures (CBEM's) Incorporating Waste Materials.* Proceedings of 4th European Symposium on Performance of Bituminous and Hydraulic Materials in Pavements, BITMAT 4, University of Nottingham, U.K., 11-12 April 2002, pp. 237-249, A.A.Balkema Publishers, Rotterdam Netherlands.

Zizi, Z. , Benhassaine, A. and Sainton, A. (1997) . *The Composite Emulsion , Filler , Cement and Aggregates. Improvement of the Performance of the Cold Mixture* , Proceedings of the Second European Symposium on Performance and Durability of Bituminous Materials , University of Leeds , April 1997 , pp. 121 to 131 , Aedificatio Publishers , Zurich .

Universal Resource Locater (URL) References

URL-1 Akzo Nobel, 2002.
http://www.asphalt-akzonobel.com/chemicals/bc/morebc.htm
Basic emulsion know-how. Visited 17/06/2002.

URL-2 Holleran Glynn, 2002.
http://www.asphalt.com/equipment/emulsifier.html
Size, Distribution Affect Emulsifier. Visited 16/06/2002.

URL-3 Silver Colliods, 2002.
http://www.silver-colloids.com/Tutorials/Intro/html.
Tutorials. An Introduction to Zeta Potential and its Measurement. Visited 10-07-02.

URL-4 Orhan, E.C., 2002.
http://www.geocities.com/CapeCanaveral/Hangar/555/zeta.htm
Mineral Processing site. Electrical Double Layer (Counter-Ion Layer). Visited 10-07-02.

URL-5 Brookhaven Instruments Corporation - BIC, 2002.
http://www.bic.com/ztheory1.htm
Zeta Potential and Stability. Visited 10-07-02.

URL-6 Malvern Instruments Ltd.,2002.
http://www.malvern.co.uk/Laboratory/zettheo.htm
Zeta Potential Theory. Visited 10-07-02.

URL-7 Arlon, 2002.
http://www.arlonmed.com/Everything/Materials/What%20is%20Dielectric%20Constant.htm
What is Dielectric Contant. Visited 11-07-02.

URL-8 EAPA, 2002.
http://www.eapa.org/home.htm
Statements. The Use of Bitumen Emulsion in Europe. Visited 17-07-02.

URL-9 Instarmac, 2002.
http://www.instarmac.co.uk.
Ultracrete-IRR, Instant Road Repair. Visited 09-08-02.

URL-10 Akzo Nobel, 2002.
http://www.asphalt-akzonobel.com/applications/ge/ge.htm
Grave Emulsion. Visited 09-08-02.

URL-11 EDP Consultant, 2002.
http://www.edpconsultants.com/NewFiles/tech.html
Pavement Recycling. Visited 03-06-02

URL-12 Infratech Polymer Inc., 2002.
http://www.infratech.com/tag8000.htm
Products Pothole Repairs. Visited 03-06-02

URL-13 A. Fillipone, 2002.
http://www.aerodyn.org/Drag/additives.html
Aerodynamic Drags, Surface Additives. Visited 27-09-02.

URL-14 Sunilbhangale, 2002.
http://www.sunilbhangale.tripod.com/epoxy.html
Epoxy Resins. Visited 27-09-02.

URL-15 Texas Department of Transportation(2000).
http://www.dot.state.tx.us/insdtdot/orgchart/gsd/recycle/mat.htm.
Materials. Visited 23-07-2000.

URL-16 Ollie Recycles UK (2000).
http://www.ollierecycles.com/uk/html/glass_facmor.html
Explore Recycle Glass Facts. Visited 21-07-2000.

URL-17 Minesota Office of Environmental Assistance, 2000.
http://w.w.w.moea.state.mn.us/lc/glass.cfm.
Recycle Glass Products. Visited 20-08-2000.

URL-18 Hossain, M et.al. (2000).
http://w.w.w.engg.ksu.edu/HSRC/95proceed/hossain.html
A Study of Chunk Rubber from Recycled Tires as Road Construction Materials. Visited 06-08-2000.

URL-19 Steel – UK (2001).
http://www.steel.org.uk
Steel Slag. Visited 27-03-2001

URL-20 Wrap Fact Sheet (2002).
http://www.wrap.org.uk
Fact Sheet Recycle glass for short blasting. Visited16-11-2002.

URL-21 Wrought martensitic stailess steel (2003)
http://www2.umist.ac.uk/material/teaching/year2/ml2211/iccpin/results/materia/martensit.htm
ML2211 : Design I. Visited 11-02-03.

URL-22 Washington Post (2001)
http://www.washingtonpost.com/wp-srv/weather/longterm/historical/data/leeds_united kingdom.htm
Weather Post. Visited 12-11-01

URL-23 Washington Post (2002)
http://www.washingtonpost.com/wp-srv/weather/longterm/historical/data/leeds_united kingdom.htm
Weather Post. Visited 16-02-02

APPENDICES

APPENDIX I	**Worksheets of the Initial CBEMs Design Procedure Developed During the investigation**
APPENDIX II	**Volumetric Calculation of selected CBEMs**
APPENDIX III	**Indirect Tensile Fatigue Test (ITFT) Results**
APPENDIX IV	**Dynamic Creep Test Results**
APPENDIX V	**BISAR 3.0 Analysis**

APPENDIX I

Worksheets of the Initial CBEMs Design Procedure Developed During the Investigation

Appendix 1.1

Mixture Proportion for Optimum Total Liquid Content (OTLC) at Compaction Based on Initial Residual Bitumen Content (IRBC) Cold Bituminous Emulsion Mixtures (CBEMs) for Wearing Course I (WC I)

Emulsion Type : Nynas Emulsion

No	Weight of Dry Agg.		Bit. Cont.	IRBC (%)	Dry Weight of Tot. Mix	Emulsion Required		Water added/ remains **		Total Weight of Mix in mixing	Total Liquid Content (TLC) in the Mix	
	(gram)	(%) by wtdm*	(%) of bit. em.	by wtdm	(gram)	(%) by wtdm	(gram)	(%) by wda**	(gram)	(gram)	(%) by wtm-m***	(gram)
a	b	c	d	e	f = b+(e/c)xb	g = (100/d)xe	h = f x g	i	j = i x b	k = b + h + j	l = ((h+j)/k)x100	m = l x k = h+j
1	1200	94	60	6	1276.60	10.00	127.66	5	60	1387.66	13.52	187.66
2	1200	94	60	6	1276.60	10.00	127.66	6	72	1399.66	14.26	199.66
3	1200	94	60	6	1276.60	10.00	127.66	7	84	1411.66	14.99	211.66
4	1200	94	60	6	1276.60	10.00	127.66	8	96	1423.66	15.71	223.66
5	1200	94	60	6	1276.60	10.00	127.66	9	108	1435.66	16.41	235.66

*: by wtdm = by weight of total dry mix

**: by wda = by weight of dry aggregates after air drying with steps of 1 % (as necessary)

***: by wtm-m = by weight of mix in mixing

Note : weight of total dry mix = weight of RBC + weight of dry Aggregate
RBC (Residual Bitumen Content) : residual asphalt content in the mixture (by wtdm)
The Initially estimated RBC was 6 % by wtdm
TLC = (added water+emulsion) by wtm-m
The Optimum Pre-wetting water Content (OPWwc) was at : 8 % added water , or at 15.71 % Total Liquid Content (TLC) by wtm-m

Appendix 1.2

Loss of Weight Form for Optimum Total Liquid Content at Compactionm (OTLC) Based on Initial Residual Bitumen Content (IRBC)

Cold Bituminous Emulsion Mixtures (CBEMs) for Wearing Course I (WC I)

Emulsion Type **: Nynas Emulsion**

No	E Type	IRBC by wtdm* (%)	Water added/remains by wda** (%)	Total Liquid Content in Mix by wtm-m*** (%)	Weight of Mix in Mixing		Container	T o t a l
					1 sample (gram)	2 samples (gram)	(gram)	(gram)
a	b	c	d	e	f	g = 2 x f	h	i = g + h
1	Nynas E	6	5	13.52	1387.66	2775.32		
2	Nynas E	6	6	14.26	1399.66	2799.32		
3	Nynas E	6	7	14.99	1411.66	2823.32		
4	Nynas E	6	8	15.71	1423.66	2847.32		
5	Nynas E	6	9	16.41	1435.66	2871.32		

*: by wtdm = by weight of total dry mix

**: by wda = by weight of dry aggregates after air drying with steps of 1 % (as necessary)

***: by wtm-m = by weight of mix in mixing

Note:

Refer to Appendix 1.1

Mixing was done at 8 % added water (by wda) or at 15.71 % Total Liquid Content (TLC) in the mix (by wtm), then to be air dried for OTLC test

Optimum Pre-wetting water content (OPWwc) for best coating based on 6 % Initial RBC

Mixing at higher OPWWc can be directly compacted after mixing if practicable

Emulsion required to produce 2 samples : 2 x 127.66 = 255.32 gram

Added Water required to produce 2 samples : 2 x 96.00 = 192.00 gram (at 8 % Added Water)
: 2 x 108.00 = 216.00 gram (at 9 % Added Water)
: 2 x 120.00 = 240.00 gram (at 10 % Added Water)

Appendix 1.3

Determination of Dry Bulk Density for Optimum Total Liquid Content at Compaction (OTLC)

Emulsion Type : **Nynas Emulsion** **Percent of Emulsion : 10.00%**
Initial Residual Bitumen Content (IRBC) : **6%**
Bitumen Content in Emulsion : **60%**
Compactive Effort : **Heavy Compaction (Gyropac , 120 rev , 240 kPa)**

No	Pre-Wetting Water		TLC	Weight of Specimen			Vol. Of	Bulk D	Liquid Content (LC) at Testing						Dry Bulk Density
	at mixing by wda (%)	at comp. by wda (%)	at comp. by wtm-m (%)	in Air (gram)	in water (gram)	in SSD cond. (gram)	Spec. (cm3)	Comp. Mix	Cont. (gram)	Cont + Wet Samp. (gram)	Cont + Dry Samp. (gram)	W. of Dry Samp. (gram)	W. of Water (gram)	LC by wtdm (%)	
a	b	c	d	e	f	g	h = g - f	i = e / h	j	k	l	m = l - j	n = k - l	o = n / m	p = i (100+NBC) / (100+NBC+ o)
1a	8	5	13.52	1111.50	598.21	1114.30	516.09	2.154	117.39	477.00	459.40	342.01	17.60	5.15	2.054
1b	8	5	13.52	1112.50	595.21	1113.30	518.09	2.147	114.39	400.50	387.40	273.01	13.10	4.80	2.054
Avg.															**2.054**
2a	8	6	14.26	1106.20	592.40	1106.70	508.49	2.175	118.94	476.80	458.00	339.06	18.80	5.54	2.067
2b	8	6	14.26	1101.50	590.30	1103.80	508.59	2.166	115.94	450.60	435.20	319.26	15.40	4.82	2.072
Avg.															**2.069**
3a	8	7	14.99	1093.00	582.30	1093.60	501.20	2.181	118.16	539.80	517.80	399.64	22.00	5.50	2.073
3b	8	7	14.99	1090.20	581.40	1091.50	501.20	2.175	112.20	500.50	481.30	369.10	19.20	5.20	2.073
Avg.															**2.073**
4a	8	8	15.71	1088.30	584.80	1089.40	507.10	2.146	117.45	511.90	493.30	375.85	18.60	4.95	2.050
4b	8	8	15.71	1080.20	580.70	1086.70	505.30	2.138	107.50	515.00	497.20	389.70	17.80	4.57	2.049
Avg.															**2.050**
5a	9	9	16.41	1076.70	574.10	1090.00	505.20	2.131	118.84	529.30	508.60	389.76	20.70	5.31	2.030
5b	9	9	16.41	1069.87	572.00	1080.90	500.20	2.139	117.90	450.00	432.30	314.40	17.70	5.63	2.031
Avg.															**2.030**

Note: Refer to Appendix 1.1.
Weight in SSD is the weight of towel-dried specimen after it is weighed in water.
by wtdm = by weight of total dry mix ; by wda = by weight of dry aggregates ; by wtm-m = by weight of mix in mixing

Appendix 1.4

Mixture Proportion for the Production of Specimens
Cold Bituminous Emulsion Mixtures (CBEMs) for Wearing Course I (WC I)

Emulsion Type : Nynas Emulsion

No	Weight of Dry Agg. by wtdm		Bit. Cont.	RBC (%) by wtdm	Weight of Tot. Mix (wtdm)	Emulsion Required by wtdm		Pre-Wetted Water * - OPWwc (by wda)		Total Weight of Mix in mixing	Total Liquid Content in the Mix (by wtm-m)	
	(gram)	(%)	(%)		(gram)	(%)	(gram)	(%)	(gram)	(gram)	(%)	(gram)
a	**b**	**c**	**d**	**e**	**f = b+(e/c)xb**	**g = (100/d)xe**	**h = f x g**	**i**	**j = i x b**	**k = b + h + j**	**l = ((h+j) / k) x 100**	**m = l x k or m=h+j**
1	1200	95.0	60	5.0	1263.16	8.33	105.26	8	96	1401.26	14.36	201.26
2	1200	94.5	60	5.5	1269.84	9.17	116.40	8	96	1412.40	15.04	212.40
3	1200	94.0	60	6.0	1276.60	10.00	127.66	8	96	1423.66	15.71	223.66
4	1200	93.5	60	6.5	1283.42	10.83	139.04	8	96	1435.04	16.38	235.04
5	1200	93.0	60	7.0	1290.32	11.67	150.54	8	96	1446.54	17.04	246.54

Note : weight of total mix = weight of RBC + weight of dry Aggregate
RBC : Residual Bitumen Content , in the mixture (by by wtdm)
by wtdm = by weight of total dry mix

by wda = by weight of dry aggregates
by wtm-m = by weight of mix in mixing
* According to the Coating Test Result

Appendix 1.5

Calculation of Total Weight of Mix at OTLC
Cold Bituminous Emulsion Mixtures (CBEMs) for Wearing Course I (WC I)

Type of Emulsion	**: Nynas Emulsion**
Compactive Effort	**: Heavy Compaction (Gyropac , 120 rev , 240 kPa)**
Optimum Total Liquid Content (OTLC) at compaction	**: 14.8 % (achieved by air drying from the OPWwc condition)**

No	Weight of Dry Agg. by wtdm		Bit. Cont.	RBC by wtdm	Weight of Tot. Mix (wtdm)	Emulsion Required by wtdm		Pre-Wetted Water remains from OPWwc to OTLC after air drying		Total Weight of Mix in mixing (see App.1.4)	Total Liquid Content in the Mix at OTLC (by wtm-m)		Weight of mix at OTLC after air dring
	(gram)	(%)	(%)	(%)	(gram)	(%)	(gram)	(%)	(gram)	(gram)	(%)	(gram)	(gram)
a	b	c	d	e	f = b+(e/c)xb	g = (100/d)xe	h = f x g	i	j = i x b	k	l = ((h+j) / k) x 100	m = h+j	n=b+h+j
1	1200	95	60	5	1263.16	8.33	105.26	-	96	1401.26	14.36 *	201.26	1401.26
2	1200	94.5	60	5.5	1269.84	9.17	116.40	-	92.63	1412.40	14.80	209.03	1409.03
3	1200	94	60	6	1276.60	10.00	127.66	-	83.04	1423.66	14.80	210.70	1410.70
4	1200	93.5	60	6.5	1283.42	10.83	139.04	-	73.35	1435.04	14.80	212.39	1412.39
5	1200	93	60	7	1290.32	11.67	150.54	-	63.55	1446.54	14.80	214.09	1414.09

Note :

weight of total mix = weight of RBC + weight of dry Aggregate
RBC : Residual Bitumen Content , in the mixture (by by wtdm)
by wtdm = by weight of total dry mix

by wda = by weight of dry aggregates
by wtm-m = by weight of mix in mixing
* does not require air drying as it lies below OTLC 14.8 %, hence directly to be compacted

Appendix 1.6

Loss of Weight Form for Compaction at Optimum Total Liquid Content at (OTLC) Cold Bituminous Emulsion Mixtures (CBEMs) for Wearing Course I (WC I)

Type of Emulsion : **Nynas Emulsion**
Compactive Effort : **Heavy Compaction (Gyropac , 120 rev , 240 kPa)**
Optimum Total Liquid Content (OTLC) at compaction : **14.8 %**

No	Emulsion Type	RBC (%)	Pre-Wetted Water (OPWwc) (%)	TLC * in Mix (%)	Weight of Mix in Mixing at OPWwc 1 sample (gram) **	Container (gram)	Tot. Weight : Samps. + Cont. at OPWwc (gram)	Final Weight of samples at OTLC	
								1 sample ** (gram)	1 samps + cont (gram)
a	b	c	d	e	f	g	h=f+g	i	j=i+g
1	Nynas	5.0	8	14.36	1401.26			1401.26	-
2	Nynas	5.5	8	15.04	1412.40			1409.03	
3	Nynas	6.0	8	15.71	1423.66			1410.70	
4	Nynas	6.5	8	16.38	1435.04			1412.39	
5	Nynas	7.0	8	17.04	1446.54			1414.09	

* TLC = Total Liquid Content ** Refer to Appendix 1.5

No	Emulsion Type	RBC (%)	Emulsion Required for 1 sample (gram)
1	Nynas	5.0	105.26
2	Nynas	5.5	116.40
3	Nynas	6.0	127.66
4	Nynas	6.5	139.04
5	Nynas	7.0	150.54

Note:

- Mixing is done at 8 % Pre-Wetted Water (by wda) or at 15.71 % TLC in mix (by wtm-m), i.e. at the best coating based on 6 % Initial Residual Bitumen Content (IRBC)

Appendix 1.7

Volumetric Calculation
Cold Bituminous Emulsion Mixtures (CBEMs) for Wearing Course I (WC I)

Type of Emulsion : **Nynas Emulsion**
Bitumen Content in Emulsion : **60 %**
Compactive Effort : **Heavy Compaction (Gyropac , 120 rev , 240 kPa)**

MATERIALS COMPOSITION									Note
Materials	**By wt. of dry agg (%)**	**Multifying Factor**	**By weight of total mix (by wt. of total mix)**					**Specific Gravity**	
			(%)	(%)	(%)	(%)	(%)		
a	b	ci	d1 = b*ci	d2 = b*ci	d3 = b*ci	d4 = b*ci	d5 = b*ci	e	
Coarse Agg. (CA)	56.61	(100-RBCi)/100	53.78	53.50	53.21	52.93	52.65	2.630	Limestone
Fine Agg. (FA)	39.39	(100-RBCi)/100	37.42	37.22	37.03	36.83	36.63	2.540	Red P Sand
Fille (F)	4.00	(100-RBCi)/100	3.80	3.78	3.76	3.74	3.72	2.260	PFA
Bitumen (RBCi)	-	-	**5.00**	**5.50**	**6.00**	**6.50**	**7.00**	1.020	100 pen
Sum	100.00	-	100.00	100.00	100.00	100.00	100.00	-	
SG Agg *	-	-	2.577	2.577	2.577	2.577	2.577	-	
SG Mix **	-	-	2.394	2.378	2.361	2.345	2.328	-	
D-Bulk (gr/cm3) - Avg.	-	-	**2.074**	**2.086**	**2.095**	**2.087**	**2.076**	-	
Porosity (%) ***	-	-	**13.381**	**12.262**	**11.263**	**10.983**	**9.859**	-	
VMA (%) ****	-	-	**23.547**	**23.510**	**23.586**	**24.283**	**25.103**	-	
VFB (%) of the VMA *****	-	-	**43.175**	**47.844**	**52.249**	**54.769**	**60.726**	-	

Note:

* Refer to Equation 5.8 in Chapter V
** Refer to Equation 5.5 in Chapter V
*** Refer to Equation 5.4 in Chapter V
**** Refer to Equation 5.6 in Chapter V
***** Refer to Equation 5.7 in Chapter V

Appendix 1.8

Calculation of Aggregate Surface Area (ASA)*

Cold Bituminous Emulsion Mixtures (CBEMs) for Wearing Course I (WC I)

Type of Emulsion : **Nynas Emulsion**
Bitumen Content in Emulsion : **60 %**
Compactive Effort : **Heavy Compaction**
(Gyropac , 120 rev , 240 kPa)

Sieve Size (mm)	% Passing by Mod.Fuller's Curve (F = 4 , n = 0.45)	Surface Area Factor *	Agg. Surface Area (m2/kg)
a	b	c	d = b x c
12.7	100.00	0.41	0.410
10. 00 , and above	89.13	-	-
5.0	63.48	0.41	0.260
2.36	43.39	0.82	0.356
1.18	30.00	1.64	0.492
0.60	20.40	2.87	0.585
0.30	13.17	6.14	0.809
0.075	4.00	32.77	1.311
			4.223 **(m2 / kg)**

* The Asphalt Institute , Manual Series No.2 (MS - 2) , May 1984 Edition , " Mix Design Methods for Asphalt Concrete and other hot mix types ". , page : 36.

Note :
Refer to Section 6.2.4, Chapter VI.

Appendix 1.9

Calculation of
Bitumen Film Thickness (T)

Cold Bituminous Emulsion Mixtures (CBEMs)
for Wearing Course I (WC I)

Type of Emulsion : **Nynas Emulsion**

Bitumen Content in Emulsion : **60 %**

Compactive Effort : **Heavy Compaction**
(Gyropac; 120 rev; 240 kPa)

$$T = \frac{\%\,\text{Binder}}{100 - \%\,\text{Binder}} \times \frac{1}{\text{SG Binder}} \times \frac{1}{\text{Agg Surface Area}}$$

RBC ** (%)	SG of Binder	Aggregate Surface Area *** (A S A , m2 / kg)	Bitumen Film Thickness (T) * (mm)	(mm)
5.00	1.020	4.223	0.01222	12.22
5.50	1.020	4.223	0.01351	13.51
6.00	1.020	4.223	0.01482	14.82
6.50	1.020	4.223	0.01614	16.14
7.00	1.020	4.223	0.01747	17.47

* Whiteoak, D.,(1991) , The Shell Bitumen Hand Book , pp. 332

** RBC = Residual Bitumen Content

*** see Appendix 1.8

Note :
Refer to Section 6.2.4, Chapter VI.

Appendix 1.10

Design Mix Worksheet of CBEMs [MPW-RI, 1990]

Cold Bituminous Emulsion Mixtures (CBEMs) for Wearing Course I (WC I)

Type of Emulsion	:	**Nynas Emulsion**
Compactive Effort	:	**Heavy Compaction (Gyropac , 120 rev , 240 kPa)**
Optimum Total Liquid Content (OTLC)	:	**14.8 % by weght of total mix in mixing**

Spec No	Mix Proportion by by weight of total mix (by wtm , %)			Emulsion by wtm (%)	RBC by wtm (%)	Max.SG of Mix	Weight of Specimen			Vol. of Spec. (cm 3)	Bulk D of Comp. Mix	W/C at Testing (%)	Dry Bulk D of Comp. Mix	Porosity or Total Voids (%)	Wt. of Sp after 48 h soaking * (gram)	Water abs of Comp. mix. (%)	Dry Stability			Soaked Stability			Retained Stability (%)
							in air (gram)	in water (gram)	in SSD * (gram)								Meas. kN	cf	Adjust kN	Meas. kN	cf	Adjust kN	
	CA	FA	FF	A	B	C	D	E	F	G = F - E	H = D / G	I	J = H (100+ B) / (100+B+I)	K = ((C-J) / C)*100	L	M = (L-D) / D * 100	N	O	P = N*O	Q	R	S = Q*R	T = (S / P) * 100
Dry 1.1	53.78	37.42	3.80	8.33	5.00	2.394	1062.8	556.1	1064.3	508.2	2.091	0.807	2.075	13.310	-	-	27.82	1.04	28.933	-	-	-	
1.2	53.78	37.42	3.80	8.33	5.00	2.394	1083.0	566.0	1084.1	518.1	2.090	0.896	2.073	13.423	-	-	28.80	1	28.800	-	-	-	
S 1.1	53.78	37.42	3.80	8.33	5.00	2.394	1106.3	584.00	1107.3	523.30	-	-	-	-	1119.8	1.220	-	-	-	19.30	0.96	18.528	64.04
1.2	53.78	37.42	3.80	8.33	5.00	2.394	1096.8	579.30	1097.8	518.50	-	-	-	-	1110.9	1.286	-	-	-	19.25	1	19.250	66.84
Dry 2.1	53.50	37.22	3.78	9.17	5.50	2.378	1063.6	557.2	1064.2	507.0	2.098	0.639	2.085	12.313	-	-	25.50	1.04	26.520	-	-	-	
2.2	53.50	37.22	3.78	9.17	5.50	2.378	1082.8	568.3	1084.0	515.7	2.100	0.625	2.087	12.224	-	-	25.80	1.00	25.800	-	-	-	
S 2.1	53.50	37.22	3.78	9.17	5.50	2.378	1080.0	567.40	1081.0	513.60	-	-	-	-	1091.8	1.093	-	-	-	20.96	1	20.960	79.03
2.2	53.50	37.22	3.78	9.17	5.50	2.378	1100.8	582.20	1102.3	520.10	-	-	-	-	1113.0	1.108	-	-	-	20.56	1	20.560	79.69
Dry 3.1	53.21	37.03	3.76	10.00	6.00	2.361	1087.4	572.7	1088.2	515.5	2.109	0.657	2.096	11.207	-	-	24.58	1	24.580	-	-	-	
3.2	53.21	37.03	3.76	10.00	6.00	2.361	1083.3	569.9	1084.0	514.1	2.107	0.651	2.094	11.295	-	-	24.52	1	24.520	-	-	-	
S 3.1	53.21	37.03	3.76	10.00	6.00	2.361	1080.0	570.90	1080.9	510.00	-	-	-	-	1090.4	0.963	-	-	-	22.13	1.00	22.130	90.03
3.2	53.21	37.03	3.76	10.00	6.00	2.361	1080.2	570.80	1081.7	510.90	-	-	-	-	1090.5	0.954	-	-	-	22.70	1.00	22.700	92.58
Dry 4.1	52.93	36.83	3.74	10.83	6.50	2.345	1067.4	560.6	1068.1	507.5	2.103	0.898	2.086	11.059	-	-	21.85	1.04	22.724	-	-	-	
4.2	52.93	36.83	3.74	10.83	6.50	2.345	1107.1	582.4	1108.1	525.7	2.106	0.903	2.088	10.949	-	-	23.50	0.96	22.560	-	-	-	
S 4.1	52.93	36.83	3.74	10.83	6.50	2.345	1082.9	570.40	1084.1	513.70	-	-	-	-	1092.5	0.887	-	-	-	21.96	1	21.960	96.64
4.2	52.93	36.83	3.74	10.83	6.50	2.345	1094.5	576.10	1095.6	519.50	-	-	-	-	1104.3	0.895	-	-	-	21.58	1	21.580	95.66
Dry 5.1	52.65	36.63	3.72	11.67	7.00	2.328	1081.6	565.0	1082.3	517.3	2.091	0.753	2.076	9.814	-	-	21.14	1	21.140	-	-	-	
5.2	52.65	36.63	3.72	11.67	7.00	2.328	1066.7	557.0	1067.2	510.2	2.091	0.820	2.075	9.874	-	-	21.21	1	21.210	-	-	-	
S 5.1	52.65	36.63	3.72	11.67	7.00	2.328	1084.3	573.00	1085.2	512.20	-	-	-	-	1093.6	0.858	-	-	-	20.80	1.00	20.800	98.39
5.2	52.65	36.63	3.72	11.67	7.00	2.328	1091.7	573.90	1092.6	518.70	-	-	-	-	1101.1	0.861	-	-	-	20.85	1.00	20.850	98.30

Note: * towel dried after capillary soaking D = Density Dry = dry sample S = soaked sample

APPENDIX II

Volumetric Calculation of selected CBEMs

Appendix 2.1

Calculation of Density, SGmix and Porosity for the WC-FA2 mixes

a. WC-FA 2 : 60 % RPS unsieved passing 2.36 mm + 40 % Asphalt Sand, without cement

Density

Sample	Dia	Height (cm)					Volume	Weight *	Dry Bulk
	(cm)	H1	H2	H3	H4	Havg.	(cm3)	(gram)	Density (g/cm3)
Sample 1	10.00	6.35	6.25	6.2	6.25	6.26	491.61	1045.65	2.13
Sample 2	10.00	6.45	6.35	6.35	6.4	6.39	501.42	1063.01	2.12
								Avg.	**2.12**

* at full curing

SGmix and Porosity

Materials	by wda (%)	Weight (gram)	by wtdm (%)	SG	SG Mix *	Density (gr/cc)	Porosity ** (%)
CA: limestone (12.7 - 2.36)	56.61	1358.64	53.21	2.63	**2.37**	**2.12**	**10.60**
FA: 60 % RPS	60 % (39.39)	567.22	22.22	2.54			
FA: 40 % Asphalt Sand	40 % (39.39)	378.14	14.81	2.64			
Filler	4.00	96.00	3.76	2.26			
Total	**100.00**	**2400.00**					
RSC : 0 % by wda		0.00	0.00	3.10			
RBC : 6 % by wtdm		153.19	6.00	1.02			
SUM		**2553.19** (gram)	**100.00** (%)				

Note: see Table 9.9 (Chapter IX)

wda: weight of dry sggregate

wtdm: weight of total dry mix

* Refer to Equation: 5.5 (Chapter V)

** Refer to Equation: 5.4 (Chapter V)

Appendix 2.1 (cont'd)

Calculation of Density, SGmix and Porosity for the WC-FA2 mixes

b. WC-FA 2 : 60 % RPS unsieved passing 2.36 mm + 40 % Asphalt Sand + 2 % Rapid Setting Cement (RSC)

Density

Sample	Dia	Height (cm)					Volume	Weight *	Dry Bulk
	(cm)	H1	H2	H3	H4	Havg.	(cm3)	(gram)	Density (g/cm3)
Sample 1	10.00	6.45	6.45	6.35	6.3	6.39	501.42	1089.50	2.17
Sample 2	10.00	6.35	6.35	6.35	6.35	6.35	498.48	1075.01	2.16
								Avg.	**2.16**

* at full curing

SGmix and Porosity

Materials	by wda (%)	Weight (gram)	by wtdm (%)	SG	SG Mix *	Density (gr/cc)	Porosity ** (%)
CA (12.7 - 2.36)	56.61	1358.64	52.23	2.63	**2.38**	**2.16**	**9.20**
FA : 60 % RPS	60 % (39.39)	567.22	21.81	2.54			
FA : 40 % Asphalt Sand	40 % (39.39)	378.14	14.54	2.64			
Filler	4.00	96.00	3.69	2.26			
Total	**100.00**	**2400.00**					
RSC : 2% by wda		48.00	1.85	3.10			
RBC : 6 % wtdm		153.19	5.89	1.02			
SUM		**2601.19** (gram)	**100.00** (%)				

Note: see Table 9.9 (Chapter IX)

wda: weight of dry sggregate

wtdm: weight of total dry mix

* Refer to Equation: 5.5 (Chapter V)

** Refer to Equation: 5.4 (Chapter V)

Appendix 2.2

Calculation of Density, SGmix and Porosity for the Control Mixes (CM)

a. CM without cement

Density

Sample	Dia	Height (cm)					Volume	Weight *	Dry Bulk
	(cm)	H1	H2	H3	H4	Havg.	(cm3)	(gram)	Density (g/cm3)
Sample 1	10.00	6.45	6.35	6.38	6.45	6.41	502.99	1115.35	2.22
Sample 2	10.00	6.45	6.35	6.35	6.40	6.39	501.42	1113.80	2.22
								Avg.	**2.22**

* at full curing

SGmix and Porosity

Materials	by wda (%)	Weight (gram)	by wtdm (%)	SG	SG Mix *	Density (gr/cc)	Porosity ** (%)
CA: limestone (12.7 - 2.36)	56.61	1358.64	53.21	2.63	**2.39**	**2.22**	**7.2**
FA: Asphalt Sand	39.39	945.36	37.03	2.64			
Filler: PFA	4.00	96.00	3.76	2.26			
Total	**100.00**	**2400.00**					
RSC : 0 % by wda		0.00	0.00	3.10			
RBC : 6 % by wtdm		153.19	6.00	1.02			
SUM		**2553.19** (gram)	**100.00** (%)				

Note: see Table 9.11 (Chapter IX)

wda: weight of dry sggregate

wtdm: weight of total dry mix

* Refer to Equation: 5.5 (Chapter V)

** Refer to Equation: 5.4 (Chapter V)

Appendix 2.2 (cont'd)

Calculation of Density, SGmix and Porosity for the Control Mixes (CM)

b. CM + 2 % Rapid Setting Cement (RSC)

Density

Sample	Dia	Height (cm)					Volume	Weight *	Dry Bulk
	(cm)	H1	H2	H3	H4	Havg.	(cm3)	(gram)	Density (g/cm3)
Sample 1	10.00	6.25	6.35	6.35	6.30	6.31	495.53	1100.50	2.22
Sample 2	10.00	6.38	6.35	6.35	6.35	6.36	499.06	1116.01	2.24
								Avg.	**2.23**

* at full curing

SGmix and Porosity

Materials	by wda (%)	Weight (gram)	by wtdm (%)	SG	SG Mix *	Density (gr/cc)	Porosity ** (%)
CA (12.7 - 2.36)	56.61	1358.64	52.23	2.63	**2.40**	**2.23**	**7.3**
FA : Asphalt Sand	39.39	945.36	36.34	2.64			
Filler	4.00	96.00	3.69	2.26			
Total	**100.00**	**2400.00**					
RSC : 2% by wda		48.00	1.85	3.10			
RBC : 6 % wtdm		153.19	5.89	1.02			
SUM		**2601.19** (gram)	**100.00** (%)				

Note: see Table 9.11 (Chapter IX)

wda: weight of dry sggregate

wtdm: weight of total dry mix

* Refer to Equation: 5.5 (Chapter V)

** Refer to Equation: 5.4 (Chapter V)

APPENDIX III

Indirect Tensile Fatigue Test (ITFT) Results

Appendix 3.1

Fatigue Test Results and Analysis for the CBEMs: Control Mix (CM)

$$\sigma x,\max = \frac{2P}{\pi d t} \qquad \varepsilon x,\max = \frac{\sigma x,\max}{\text{Smix}}(1 + 3\upsilon) \times 1000 \qquad \upsilon = 0.35$$

No	Sample	Height	$\sigma x,\max$ = $\sigma applied$	S mix = ITSM	ex, max * (x 10^6)	No of Cycles at failure	At failure	
							Displa- cement	Strain **
		(mm)	(kPa)	(MPa)	(micro strain)		(mm)	(%)
Type of Mix : Control Mix (CM) without Cement								
1	CM -1	44.3	500	2336.5	439	398	4.40 ***	4.40
2	CM - 2	44.0	425	2455.8	355	650	4.40 ***	4.40
3	CM - 3	44.7	350	2326.8	308	920	4.40 ***	4.40
4	CM - 4	44.8	250	2468.8	208	2180	4.40 ***	4.40
5	CM - 5	44.9	200	2429.8	169	5328	4.40 ***	4.40
6	CM - 6	44.8	150	2469.1	125	10288	4.40 ***	4.40
Type of Mix : Control Mix (CM) incorporate 2 % Rapid Setting Cement (RSC)								
1	CMC - 1	41.0	678	5685.4	244	212	1.5	1.5
2	CMC - 2	43.4	600	5643.1	218	534	1.5	1.5
3	CMC - 3	39.8	550	5662.7	199	904	2.1	2.1
4	CMC - 4	38.7	500	5562.3	184	1524	2.6	2.6
5	CMC - 5	40.6	400	5567.1	147	4608	1.4	1.4
6	CMC - 6	40.3	300	5653.3	109	17952	1.6	1.6
						Avg:	**1.8**	**1.8**

Note :

* εx, max is the max horizontal tensile strain at the centre of the specimen under fatigue loading

** strain : displacement divided by the specimen diameter of 100 mm

*** the samples did not totally fail at max detectable dispalcement, therefore for uniformity , failure was taken at 4.4 % strain.

t = height of the sppecimen d = diameter of the specimen = 100 mm

σx , max = max tensile stress at the centre of the specimen

Appendix 3.2

Fatigue Test Results and Analysis for the CBEMs: WC-FA2

$\sigma x, \max = \frac{2P}{\pi dt}$ $\varepsilon x, \max = \frac{\sigma x, \max}{Smix}(1 + 3\upsilon) \times 1000$ $\upsilon = 0.35$

No	Sample	Height (mm)	$\sigma x, \max$ = *σapplied* (actual avg.peak stress) (kPa)	S mix = ITSM (MPa)	εx, max * (x 10^6) (micro strain)	No of Cycles at failure	At failure	
							Displa-cement (mm)	Strain ** (%)
Type of Mix : WC-FA2 (60 % RPS + 40% Asphalt Sand) without Cement								
1	WC-FA2.1	41.0	500	2701.5	379	450	4.50 ***	4.50
2	WC-FA2.2	40.2	425	2798.5	311	1028	4.50 ***	4.50
3	WC-FA2.3	41.4	350	2697.2	266	1434	4.50 ***	4.50
4	WC-FA2.4	40.0	250	2700.7	190	4800	4.50 ***	4.50
5	WC-FA2.5	42.5	200	2734.4	150	6848	4.50 ***	4.50
6	WC-FA2.6	42.4	150	2680.6	115	16256	4.50 ***	4.50
Type of Mix : WC-FA2C (60 % RPS + 40% Asphalt Sand) + 2% Rapid Setting Cement (RSC)								
1	WC-FA2C.1	40.0	664	5308.5	256	314	2.5	2.5
2	WC-FA2C.2	43.4	600	5324.5	231	456	2.6	2.6
3	WC-FA2C.3	41.5	550	5269.6	214	536	2.6	2.6
4	WC-FA2C.4	41.9	500	5298.0	193	1048	2.5	2.5
5	WC-FA2C.5	42.4	400	5292.9	155	4192	2.7	2.7
6	WC-FA2C.6	42.6	300	5341.5	115	23520	2.4	2.4
						Avg:	2.6	2.6

Note :

* εx, max is the max horizontal tensile strain at the centre of the specimen under fatigue loading

** Strain : displacement divided by the specimen diameter of 100 mm

*** the samples did not totally fail at max detectable dispalcement, therefore for uniformity , failure was taken at 4.5 % strain.

t = height of the sppecimen d = diameter of the specimen = 100 mm

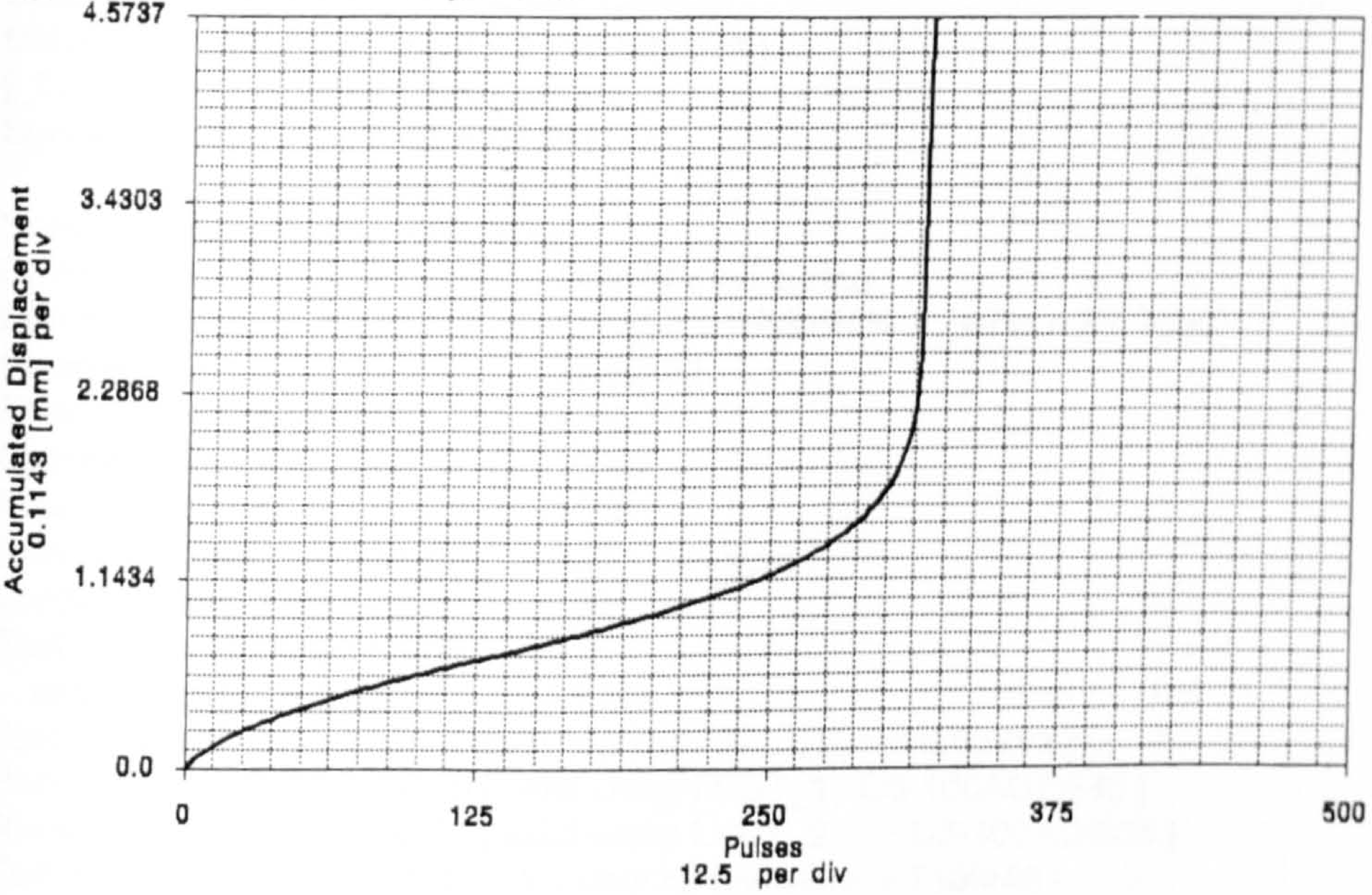
σ x , max = max tensile stress at the centre of the specimen

Appendix 3.3

An Example Print Out of Fatigue Test Graph

Leeds University (Civil Engineering CEMU)

[_030] V1.00 DD ABF IT Fatigue Test



```
         specimen identification: FA2Cr
                         comment: Cold Mix , full curing
                         comment: CA+(60% RPS+40% A.Sand)+ 2% RSC
                         comment: 24-02-02
             specimen length (mm): 40.0
           specimen diameter (mm): 100.0
                pulse period (ms): 1500
    rise time [ 30ms ... 130ms]: 120
   loading tensile stress (kPa): 700
   test termination displ. (mm): 9.0
     ... or terminal pulse count: 25000
       force load cell cal index: 6  [ force load cell #2    STC2000/65291 ]
        radial LVDT #1 cal index: 10 [ axial creep LVDT_1    D5-100AG/5543 ]
        radial LVDT #2 cal index: 11 [ axial creep LVDT_2         D5-100AG/555 ]
 temperature probe 1 cal index: 1  [ core temperature  PT100/46 ]
 temperature probe 2 cal index: 2  [ skin temperature  PT100/43 ]
        archive binary file name: C:\DD213\ARYA\FA2CR.BIN
    ASCII output data file name: C:\DD213\ARYA\FA2CR.CSV
        test variant description:

final reading of measured parameters:
     loading time (hh:mm:ss) 00:07:57     total loading pulse count     317
     accumulated displ. (mm)   4.5737     peak loading stress (kPa)  224.0
         LVDT 1 range (% FS)      100        peak loading force (N) 1410.0
         LVDT 2 range (% FS)      100     0_peak load rise time (ms)    119
                                            loading pulse width (ms)    350
                                            temperature probe 1 (°C)  19.91
                                            temperature probe 2 (°C)  19.96

INDUSTRIAL PROCESS CONTROLS Ltd.,   Universal Testing Machine (UTM V2.00BETA)
```

Appendix 3.4

An Example Print Out of Fatigue Test Parameters

```
Leeds University (Civil Engineering CEMU)
Universal Testing Machine (UTM V2.00BETA)
[_030] V1.00 DD ABF IT Fatigue Test
Sunday February 24, 2002 4:56 PM

Specimen identification: WC- FA2Cr
Comment                        : Cold Mix full curing
Comment                        : CA+(60% RPS+40% A.Sand)+ 2% RSC
Comment                        : 24-02-02
Specimen length (mm)           : 40.0
Specimen diameter (mm)         : 100.0
Pulse period (ms)              : 1500
Rise time [ 30ms ... 130ms]    : 120
Loading tensile stress (kPa)   : 700
Test termination displ. (mm)   : 9.0
... or terminal pulse count    : 25000
Force load cell cal index: 6  [ force load cell #2   STC2000/65291 ]
Radial LVDT #1 cal index: 10 [ axial creep LVDT_1   D5-100AG/5543 ]
Radial LVDT #2 cal index: 11 [ axial creep LVDT_2      D5-100AG/555 ]
Temperature probe 1 cal index: 1  [ core temperature  PT100/46 ]
Temperature probe 2 cal index: 2  [ skin temperature  PT100/43 ]
Archive binary file name       : C:\DD213\ARYA\FA2CR.BIN
ASCII output data file name    : C:\DD213\ARYA\FA2CR.CSV
Test variant description:
Final reading of measured parameters:
Loading time (hh:mm:ss) 00:07:57   total loading pulse count      317
Accumulated displ. (mm)  4.5737    peak loading stress (kPa)      224.0
LVDT 1 range (% FS)      100       peak loading force (N)         1410.0
LVDT 2 range (% FS)      100         0_peak load rise time (ms)   119
                                 loading pulse width (ms)       350
                                 temperature probe 1 (øC)       19.91
                                 temperature probe 2 (øC)       19.96

 Pulse   PkStress   Displ't   TempPrb1    Pulse   PkStress   Displ't   TempPrb1
 Count    (kPa)      (mm)       (øC)      Count    (kPa)      (mm)       (øC)
   0       669        0        19.88        18      684       0.181      19.91
   2       674      0.028      19.88        20      682       0.195      19.91
   4       679      0.053      19.88        22      681       0.209      19.91
   6       683      0.076      19.88        24      683       0.223      19.91
   8       683      0.097      19.88        26      683       0.235      19.93
  10       684      0.115      19.91        28      681       0.249      19.93
  12       683      0.133      19.91        30      681       0.26       19.93
  14       683      0.15       19.91        32      680       0.273      19.93
  16       684      0.166      19.91        34      680       0.284      19.93
```

Appendix 3.4 (cont'd)

An Example Print Out of Fatigue Test Parameters

Pulse Count	PkStress (kPa)	Displ't (mm)	TempPrb1 (øC)	Pulse Count	PkStress (kPa)	Displ't (mm)	TempPrb1 (øC)
36	681	0.295	19.93	120	665	0.644	19.96
38	679	0.307	19.93	122	667	0.651	19.93
40	680	0.317	19.93	124	664	0.657	19.93
42	680	0.328	19.93	126	668	0.664	19.93
44	680	0.339	19.93	128	665	0.67	19.93
46	678	0.349	19.93	130	665	0.677	19.93
48	679	0.359	19.93	132	665	0.684	19.93
50	676	0.369	19.93	134	667	0.691	19.93
52	678	0.379	19.93	136	666	0.697	19.93
54	679	0.388	19.93	138	666	0.704	19.96
56	674	0.397	19.93	140	666	0.711	19.96
58	676	0.407	19.93	142	665	0.717	19.96
60	675	0.416	19.93	144	665	0.726	19.96
62	674	0.425	19.93	146	665	0.733	19.96
64	674	0.433	19.93	148	666	0.739	19.96
66	675	0.443	19.93	150	665	0.746	19.96
68	675	0.45	19.93	152	664	0.754	19.96
70	673	0.459	19.93	154	665	0.76	19.96
72	675	0.467	19.93	156	665	0.768	19.96
74	674	0.475	19.93	158	664	0.775	19.96
76	672	0.483	19.93	160	663	0.781	19.96
78	672	0.49	19.93	162	665	0.788	19.96
80	673	0.498	19.93	164	663	0.796	19.96
82	672	0.507	19.93	166	662	0.803	19.96
84	669	0.514	19.93	168	661	0.81	19.96
86	674	0.522	19.93	170	663	0.817	19.93
88	672	0.53	19.93	172	662	0.824	19.93
90	672	0.537	19.96	174	660	0.831	19.93
92	671	0.544	19.96	176	661	0.839	19.93
94	670	0.552	19.96	178	661	0.845	19.93
96	671	0.559	19.96	180	661	0.853	19.93
98	669	0.566	19.96	182	661	0.861	19.93
100	670	0.574	19.96	184	658	0.867	19.93
102	671	0.581	19.96	186	661	0.874	19.93
104	669	0.589	19.96	188	659	0.883	19.93
106	668	0.596	19.96	190	661	0.891	19.93
108	668	0.603	19.96	192	662	0.898	19.93
110	667	0.611	19.96	194	661	0.906	19.93
112	666	0.617	19.96	196	658	0.913	19.93
114	665	0.624	19.96	198	660	0.921	19.93
116	664	0.63	19.96	200	660	0.929	19.93
118	667	0.637	19.96	202	659	0.937	19.93

Appendix 3.4 (cont'd)

An Example Print Out of Fatigue Test Parameters

Pulse Count	PkStress (kPa)	Displ't (mm)	TempPrb1 (øC)	Pulse Count	PkStress (kPa)	Displ't (mm)	TempPrb1 (øC)
204	659	0.946	19.93	262	655	1.25	19.93
206	659	0.954	19.93	264	653	1.265	19.93
208	659	0.962	19.93	266	653	1.28	19.91
210	660	0.97	19.93	268	653	1.296	19.91
212	657	0.978	19.93	270	650	1.314	19.91
214	659	0.987	19.93	272	652	1.33	19.91
216	658	0.995	19.93	274	652	1.349	19.91
218	658	1.005	19.93	276	652	1.368	19.91
220	658	1.014	19.93	278	653	1.388	19.91
222	657	1.022	19.93	280	651	1.408	19.91
224	657	1.032	19.93	282	652	1.43	19.91
226	659	1.041	19.93	284	650	1.453	19.91
228	655	1.051	19.93	286	648	1.478	19.91
230	659	1.06	19.93	288	649	1.504	19.91
232	658	1.07	19.93	290	649	1.531	19.91
234	657	1.081	19.91	292	647	1.56	19.91
236	657	1.091	19.91	294	649	1.591	19.91
238	657	1.101	19.91	296	647	1.626	19.91
240	656	1.112	19.91	298	648	1.663	19.91
242	655	1.124	19.91	300	645	1.704	19.91
244	656	1.134	19.91	302	643	1.751	19.91
246	657	1.146	19.91	304	643	1.803	19.91
248	654	1.158	19.91	306	642	1.866	19.91
250	654	1.17	19.93	308	642	1.946	19.91
252	655	1.183	19.93	310	637	2.053	19.91
254	654	1.195	19.93	312	634	2.212	19.91
256	654	1.209	19.93	314	627	2.518	19.91
258	653	1.222	19.93	* 316	561	4.573	19.91
260	653	1.236	19.93	* 318	224	4.573	19.91

Note: avg. stress up to 314 pulses : 664 kPa
* sample had failed

APPENDIX IV

Dynamic Creep Test Results

Appendix 4.1

Dynamic Creep stiffness of CBEMs: Control Mixtures (CM)

a. Control Mixtures (CM) without cement

Testing Temperature : 40 ° C

Samples Dia : 100 mm Samples Area (A) : 0.00785 m2

	CM40m				CM40n				Avg.			
Pulse Count	PermStrn (εm/m)	TotVFrc (N)	Stress (kPa)	S Mix (MPa)	PermStrn (εm/m)	TotVFrc (N)	Stress (kPa)	S Mix (MPa)	PermStrn (εm/m)	TotVFrc (N)	Stress (kPa)	S Mix (MPa)
a	b	c	d= c/A	e=d/b	b	c	d= c/A	e=d/b	b	c	d= c/A	e=d/a
1	1076	1077	137.20	127.51	1343	1055	134.39	100.07	1209.5	1066.00	135.80	113.79
50	4424	1077	137.20	31.01	5725	1070	136.31	23.81	5074.5	1073.50	136.75	27.41
100	5331	1080	137.58	25.81	6833	1067	135.92	19.89	6082	1073.50	136.75	22.85
150	5929	1080	137.58	23.20	7588	1070	136.31	17.96	6758.5	1075.00	136.94	20.58
408	7812	1085	138.22	17.69	9961	1070	136.31	13.68	8886.5	1077.50	137.26	15.69
814	9735	1083	137.96	14.17	12392	1065	135.67	10.95	11063.5	1074.00	136.82	12.56
1227	11379	1080	137.58	12.09	14510	1060	135.03	9.31	12944.5	1070.00	136.31	10.70
2448	16381	1075	136.94	8.36	20961	1040	132.48	6.32	18671	1057.50	134.71	7.34
3600	22808	1075	136.94	6.00	29363	1040	132.48	4.51	26085.5	1057.50	134.71	5.26

Appendix 4.1 (cont'd).

Dynamic Creep stiffness of CBEMs: Control Mixtures (CM), (cont'd).

b. Control Mixtures (CM) + 2 % Rapid Setting Cement (RSC) without cement

Testing Temperature : 40 ° C

Samples Dia : 100 mm Samples Area (A) : 0.00785 m2

Pulse Count	CMC40m				CMC40n				Avg.			
	PermStrn (εm/m)	TotVFrc (N)	Stress (kPa)	S Mix (MPa)	PermStrn (εm/m)	TotVFrc (N)	Stress (kPa)	S Mix (MPa)	PermStrn (εm/m)	TotVFrc (N)	Stress (kPa)	S Mix (MPa)
a	b	c	d= c/A	e=d/b	b	c	d= c/A	e=d/b	b	c	d= c/A	e=d/a
1	2269	1075	136.94	60.35	1045	1075	136.94	131.05	1657.00	1075.00	136.94	95.70
50	3379	1073	136.69	40.45	2050	1080	137.58	67.11	2714.50	1076.50	137.13	53.78
100	3566	1075	136.94	38.40	2161	1077	137.20	63.49	2863.50	1076.00	137.07	50.95
150	3644	1075	136.94	37.58	2231	1080	137.58	61.67	2937.50	1077.50	137.26	49.62
408	3841	1075	136.94	35.65	2352	1075	136.94	58.22	3096.50	1075.00	136.94	46.94
814	3988	1075	136.94	34.34	2422	1075	136.94	56.54	3205.00	1075.00	136.94	45.44
1227	4037	1073	136.69	33.86	2452	1075	136.94	55.85	3244.50	1074.00	136.82	44.85
2448	4136	1075	136.94	33.11	2523	1080	137.58	54.53	3329.50	1077.50	137.26	43.82
3600	4175	1075	136.94	32.80	2523	1080	137.58	54.53	3349.00	1077.50	137.26	43.67

Appendix 4.1 (cont'd).

Dynamic Creep stiffness of CBEMs: Control Mixtures (CM),A49 (cont'd).

b. Control Mixtures (CM) + 2 % Rapid Setting Cement (RSC) without cement

Testing Temperature : 60 °C

Samples Dia : 100 mm **Samples Area (A) : 0.00785 m2**

Pulse Count	CMC60k				CMC60l				Avg.			
	PermStrn (εm/m)	TotVFrc (N)	Stress (kPa)	S Mix (MPa)	PermStrn (εm/m)	TotVFrc (N)	Stress (kPa)	S Mix (MPa)	PermStrn (εm/m)	TotVFrc (N)	Stress (kPa)	S Mix (MPa)
a	b	c	d= c/A	e=d/b	b	c	d= c/A	e=d/b	b	c	d= c/A	e=d/a
1	4888	1063	135.41	27.70	2352	1080	137.58	58.49	3620.00	1071.50	136.50	43.10
50	7541	1095	139.49	18.50	5354	1095	139.49	26.05	6447.50	1095.00	139.49	22.28
100	8071	1097	139.75	17.31	6033	1103	140.51	23.29	7052.00	1100.00	140.13	20.30
150	8347	1093	139.24	16.68	6417	1103	140.51	21.90	7382.00	1098.00	139.87	19.29
408	8990	1090	138.85	15.45	7224	1103	140.51	19.45	8107.00	1096.50	139.68	17.45
814	9337	1093	139.24	14.91	7746	1007	128.28	16.56	8541.50	1050.00	133.76	15.74
1227	9459	1093	139.24	14.72	7904	1097	139.75	17.68	8681.50	1095.00	139.49	16.20
2448	9612	1095	139.49	14.51	8150	1095	139.49	17.12	8881.00	1095.00	139.49	15.81
3600	9724	1095	139.49	14.34	8248	1095	139.49	16.91	8986.00	1095.00	139.49	15.63

Appendix 4.2

Dynamic Creep stiffness of CBEMs: WC-FA2

a. WC-FA2 without cement

Testing Temperature : 40 ° C

Samples Dia : 100 mm Samples Cross Section Area (A) : 0.00785 m2

Pulse Count	WC-FA240a				WC-FA240b				Avg.			
	PermStrn (εm/m)	TotVFrc (N)	Stress (kPa)	S Mix (MPa)	PermStrn (εm/m)	TotVFrc (N)	Stress (kPa)	S Mix (MPa)	PermStrn (εm/m)	TotVFrc (N)	Stress (kPa)	S Mix (MPa)
a	b	c	d= c/A	e=d/a	b	c	d= c/A	e=d/a	b	c	d= c/A	e=d/a
1	1173	1080	137.58	117.29	1711	1083	137.96	80.63	1442.00	1081.50	137.77	98.96
50	4102	1083	137.96	33.63	4871	1085	138.22	28.38	4486.50	1084.00	138.09	31.00
100	4806	1085	138.22	28.76	5596	1087	138.47	24.74	5201.00	1086.00	138.34	26.75
150	5224	1083	137.96	26.41	6039	1085	138.22	22.89	5631.50	1084.00	138.09	24.65
408	6429	1080	137.58	21.40	7307	1083	137.96	18.88	6868.00	1081.50	137.77	20.14
814	7459	1083	137.96	18.50	8424	1085	138.22	16.41	7941.50	1084.00	138.09	17.45
1227	8184	1075	136.94	16.73	9189	1080	137.58	14.97	8686.50	1077.50	137.26	15.85
2448	10020	1077	137.20	13.69	10960	1087	138.47	12.63	10490.00	1082.00	137.83	13.16
3600	11561	1077	137.20	11.87	12560	1087	138.47	11.02	12060.50	1082.00	137.83	11.45

Appendix 4.2

Dynamic Creep stiffness of CBEMs: WC-FA2

b. WC-FA2 + 2% Rapid Setting Cement (RSC)

Testing Temperature : 40 ° C

Samples Dia : 100 mm Samples Cross Section Area (A) : 0.00785 m2

	WC-FA2C40a				WC-FA2C40b				Avg.			
Pulse Count	PermStrn (εm/m)	TotVFrc (N)	Stress (kPa)	S Mix (MPa)	PermStrn (εm/m)	TotVFrc (N)	Stress (kPa)	S Mix (MPa)	PermStrn (εm/m)	TotVFrc (N)	Stress (kPa)	S Mix (MPa)
a	b	c	d= c/A	e=d/a	b	c	d= c/A	e=d/a	b	c	d= c/A	e=d/a
1	2242	1080	137.58	61.36	1229	1080	137.58	111.94	1735.50	1080.00	137.58	86.65
50	3237	1080	137.58	42.50	2000	1085	138.22	69.11	2618.50	1082.50	137.90	55.81
100	3338	1077	137.20	41.10	2117	1083	137.96	65.17	2727.50	1080.00	137.58	53.14
150	3378	1080	137.58	40.73	2156	1080	137.58	63.81	2767.00	1080.00	137.58	52.27
408	3499	1080	137.58	39.32	2224	1085	138.22	62.15	2861.50	1082.50	137.90	50.73
814	3569	1077	137.20	38.44	2312	1077	137.20	59.34	2940.50	1077.00	137.20	48.89
1227	3620	1080	137.58	38.01	2341	1080	137.58	58.77	2980.50	1080.00	137.58	48.39
2448	3670	1077	137.20	37.38	2371	1075	136.94	57.76	3020.50	1076.00	137.07	47.57
3600	3690	1077	137.20	37.18	2380	1075	136.94	57.54	3035.00	1076.00	137.07	47.36

Appendix 4.2

Dynamic Creep stiffness of CBEMs: WC-FA2

c. WC-FA2 + 2% Rapid Setting Cement (RSC)

Testing Temperature : 60 ° C

Samples Dia : 100 mm Samples Cross Section Area (A) : 0.00785 m2

	FA2C60x				FA2C60y				Avg.			
Pulse Count	PermStrn (εm/m)	TotVFrc (N)	Stress (kPa)	S Mix (MPa)	PermStrn (εm/m)	TotVFrc (N)	Stress (kPa)	S Mix (MPa)	PermStrn (εm/m)	TotVFrc (N)	Stress (kPa)	S Mix (MPa)
a	b	c	d= c/A	e=d/a	b	c	d= c/A	e=d/a	b	c	d= c/A	e=d/a
1	1816	1080	137.58	75.76	1154	1080	137.58	119.22	1485.00	1080.00	137.58	97.49
50	3801	1080	137.58	36.20	2189	1085	138.22	63.14	2995.00	1082.50	137.90	49.67
100	4139	1077	137.20	33.15	2338	1083	137.96	59.01	3238.50	1080.00	137.58	46.08
150	4327	1080	137.58	31.80	2428	1080	137.58	56.66	3377.50	1080.00	137.58	44.23
408	4764	1080	137.58	28.88	2647	1085	138.22	52.22	3705.50	1082.50	137.90	40.55
814	5002	1077	137.20	27.43	2935	1077	137.20	46.75	3968.50	1077.00	137.20	37.09
1227	5081	1080	137.58	27.08	3075	1080	137.58	44.74	4078.00	1080.00	137.58	35.91
2448	5240	1077	137.20	26.18	3174	1075	136.94	43.15	4207.00	1076.00	137.07	34.66
3600	5300	1077	137.20	25.89	3224	1075	136.94	42.48	4262.00	1076.00	137.07	34.18

Appendix 4.3

Results of S_{bit} Analysis using BANDS 2.0 Software

a. Data :

Bitumen	100 Pen	50 pen
Soft Point	45 °C	48.5 °C
Pen at 25 deg. C	98 (x 0.1mm)	54 (x 0.1mm)

Input required :

time of loading/pulse count, test temperature softening point, penetration temperature and penetration value

Pulse Count	40 deg. C		60 deg. C		40 deg. C		60 deg. C	
	S Bit 100 pen (MPa)				S Bit 50 pen (MPa)			
	Real	x E +6	Real	x E +6	Real	x E +6	Real	x E +6
1	8.60E-03	8600.00	4.17E-04	417.000	1.89E-02	18900.00	5.91E-04	591.000
50	2.22E-04	222.00	9.83E-06	9.830	4.60E-04	460.00	1.30E-05	13.000
100	1.12E-04	112.00	5.22E-06	5.220	2.19E-04	219.00	6.59E-06	6.590
150	7.80E-05	78.00	3.26E-06	3.260	1.46E-04	146.00	4.54E-06	4.540
408	3.17E-05	31.70	1.17E-06	1.170	5.98E-05	59.80	1.56E-06	1.560
814	1.62E-05	16.20	5.96E-07	0.596	3.09E-05	30.90	8.20E-07	0.820
1227	1.07E-05	10.70	3.98E-07	0.398	2.04E-05	20.40	5.54E-07	0.554
2448	5.71E-06	5.71	2.03E-07	0.203	1.03E-05	10.30	2.72E-07	0.272
3600	3.76E-06	3.76	1.37E-07	0.137	7.10E-06	7.10	1.85E-07	0.185

Appendix 4.3 (cont'd)

Results of S_{bit} Analysis using BANDS 2.0 Software vs. S_{mix}

b. Graphs data at 40deg.C

Pulse Count	Sbit 100 pen (MPa)	S Mix (100 pen) - MPa				Sbit 50 pen (MPa)	S Mix (50 pen) - MPa	
		CM40	CMC40	FA240	FA2C40		AC(40)	HRA(40)
1	8600.00	113.79	95.70	98.96	86.65	18900.00	40	43
50	222.00	27.41	53.78	31.00	55.81	460.00	30	31
100	112.00	22.85	50.95	26.75	53.14	219.00	26	26
150	78.00	20.58	49.62	24.65	52.27	146.00	24	22
408	31.70	15.69	46.94	20.14	50.73	59.80	19	18
814	16.20	12.56	45.44	17.45	48.89	30.90	18	16
1227	10.70	10.70	44.85	15.85	48.39	20.40	17	15
2448	5.71	7.34	43.82	13.16	47.57	10.30	16	14
3600	3.76	5.26	43.67	11.45	47.36	7.10	15	13

Appendix 4.3 (cont'd)

Results of S_{bit} Analysis using BANDS 2.0 Software vs. S_{mix}

c. Graphs data at 60deg.C

Pulse Count	S_{bit} 100 pen (MPa)	S_{Mix} (100pen)-MPa		S_{bit} 50 pen (MPa)	S_{Mix} (50 pen)-MPa	
		CMC60	FA2C60		AC(60)	HRA(60)
1	417.000	43.10	97.49	591.000	30	28
50	9.830	22.28	49.67	13.000	24	21
100	5.220	20.30	46.08	6.590	20	18
150	3.260	19.29	44.23	4.540	18	15
408	1.170	17.45	40.55	1.560	15	12
814	0.596	15.74	37.09	0.820	14	11
1227	0.398	16.20	35.91	0.554	12	10
2448	0.203	15.81	34.66	0.272	9	8
3600	0.137	15.63	34.18	0.185	8.5	7

Appendix 4.4

Dynamic Creep Properties at The End of Loading (3600 pulses)

Test Loading Stress **: 100 kPa**

Sample	Loading Strain (Microstr)	Recovery strain after unloading	Peak Load Stress (kPa)	Resilient Strain * (Microstr)	Resilient Modulus * (MPa)	Recoverable Axial Strain	Creep Recovery (%) **	
								Avg.
a	b	c	d	e	f	g = b-c	h = g/b	
TEST TEMPERATURE : 40 °C								
Control Mix (CM) without cement:								
CMm	22808	22668	137	408	335.4	140	0.61	
CMn	29363	29196	132	374	352.4	167	0.57	0.59
Control Mix (CM) + 2% Rapd Setting Cement:								
CMCm	4175	4000	137	217	629	175	4.19	
CMCn	2523	2424	137	222	618.4	99	3.92	4.06
WC-FA2, without cement								
FA240a	11561	11357	136	362	375.9	204	1.76	
FA240b	12560	12339	138	367	376.2	221	1.76	1.76
WC-FA2 + 2% Rapd Setting Cement:								
FA2C40a	3690	3569	138	192	717	121	3.28	
FA2C40b	2380	2263	137	205	668	117	4.92	4.10
TEST TEMPERATURE : 60 °C								
Control Mix (CM) without cement:								
CMk60	failed at 300 pulses							
Control Mix (CM) + 2% Rapid Setting Cement:								
CMCk60	9724	9571	139	444	313.4	153	1.57	
CMCl60	8248	8071	139	486	286	177	2.15	1.86
WC-FA2, without cement:								
FA260x	failed at 1847 pulses							
WC-FA2 + 2% Rapid Setting Cement:								
FA2C60x	5300	5181	138	419	328.1	119	2.25	
FA2C60y	3224	3124	138	379	362.5	100	2.26	2.25

* refers to Appendix 4.7 ** refers to Equation 5.14 in Chapter V.

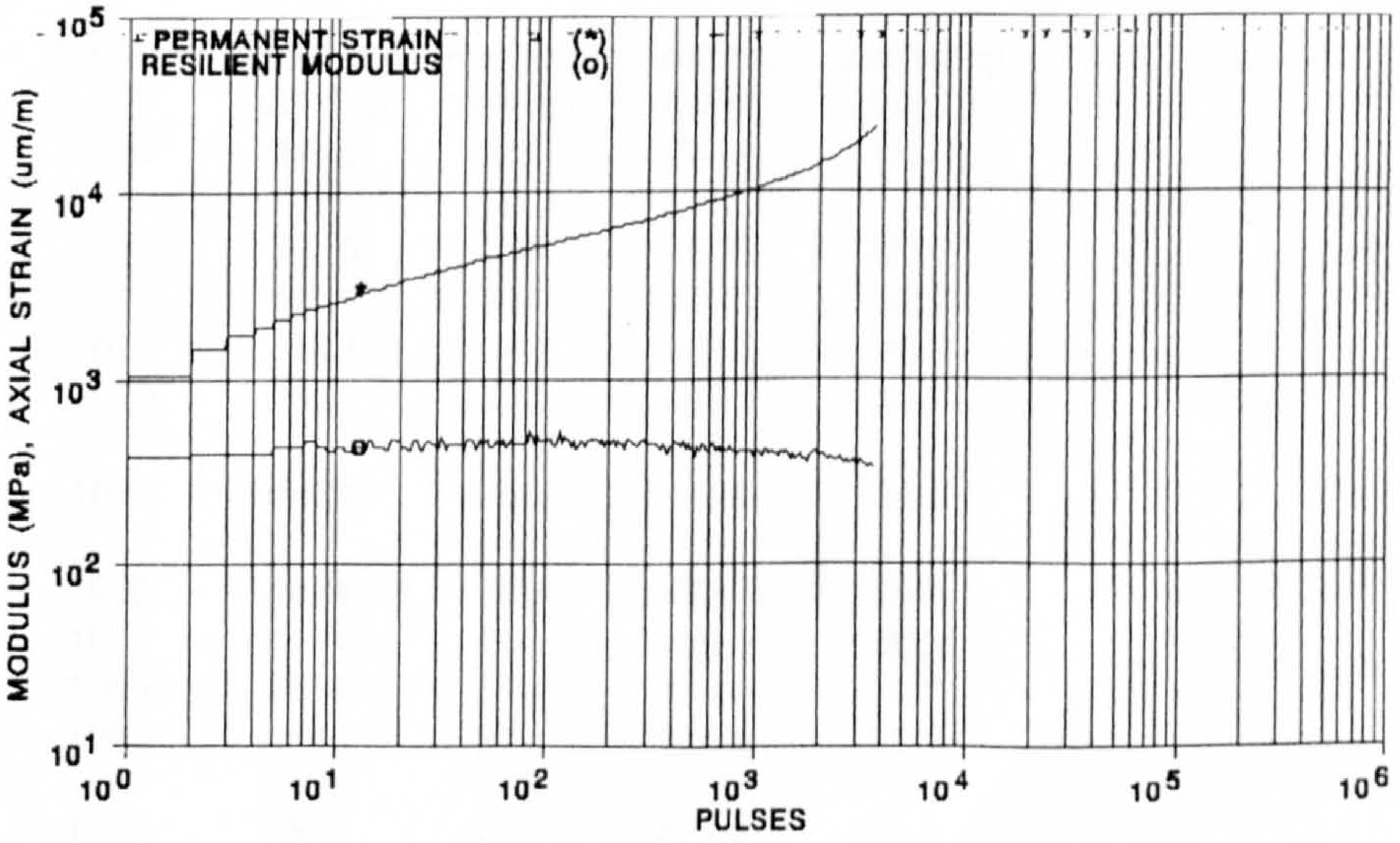
Appendix 4. 5

An Example Print Out of Dynamic Creep Test Graph

```
INDUSTRIAL PROCESS CONTROLS Limited      REPEATED LOAD ASPHALT CREEP TEST
Universal Material Testing Apparatus     Thursday February 14, 2002  2:03 AM

      specimen identification: CMm
                      comment: Control Mix ( CM ) without cement
                      comment: full curing
                      comment: 16-02-02
         specimen length (mm): 50.18
       specimen diameter (mm): 100.0
         set temperature (°C): 40.0
             pulse width (ms): 1000
            pulse period (ms): 2000
 test termination strain (µε): 100000
  ... or terminal pulse count: 3600
    conditioning stress (kPa): 10
    test loading stress (kPa): 100
  conditioning time (minutes): 2
  preload rest time (minutes): 1
      recovery time (minutes): 30
             binary file name: C:\UMAT\ARYADCREEP\CMM.BIN
  ASCII output data file name: C:\UMAT\ARYADCREEP\CMM.CSV

final reading of measured parameters:
    conditioning strain (µε)       638    condition time (hh:mm:ss) 00:02:00
     preload rest strain (µε)      598         rest time (hh:mm:ss) 00:01:00
          loading strain (µε)    22808      loading time (hh:mm:ss) 02:00:00
         recovery strain (µε)    22668     recovery time (hh:mm:ss) 00:30:01
   conditioning stress (kPa)        10    total loading pulse count     3600
   peak loading stress (kPa)       137         skin temperature (°C)    40.1
        resilient strain (µε)      408         core temperature (°C)    40.5
     resilient modulus (MPa)     335.4
```



Appendix 4.6

An Example Print Out of Dynamic Creep Test Parameters

INDUSTRIAL PROCESS CONTROLS Limited REPEATED LOAD ASPHALT CREEP TEST
Universal Material Testing Apparatus Saturday February 16 2002 2:03 AM
Specimen identification: CMm

Comment	: Control Mix (CM) without cement
Comment	: full curing
Comment	: 16-02-02
Specimen length (mm)	: 50.18
Specimen diameter (mm)	: 100.0
Set temperature (øC)	: 40.0
Pulse width (ms)	: 1000
Pulse period (ms)	: 2000
Test termination strain (æî)	: 100000
... or terminal pulse count	: 3600
Conditioning stress (kPa)	: 10
Test loading stress (kPa)	: 100
Conditioning time (minutes)	: 2
Preload rest time (minutes)	: 1
Recovery time (minutes)	: 30
Binary file name	: C:\UMAT\ARYADCREEP\CMM.BIN
ASCII output data file name	: C:\UMAT\ARYADCREEP\CMM.CSV

Final reading of measured parameters:

Conditioning strain (æî)	638	condition time (hh:mm:ss)	00:02:00
Preload rest strain (æî)	598	rest time (hh:mm:ss)	00:01:00
Loading strain (æî)	**22808**	loading time (hh:mm:ss)	02:00:00
Recovery strain (æî)	**22668**	recovery time (hh:mm:ss)	00:30:00
Conditioning stress (kPa)	10	total loading pulse count	3600
Peak loading stress (kPa)	137	skin temperature (øC)	40.0
Resilient strain (æî)	367	core temperature (øC)	40.5
Resilient modulus (MPa)	371.7		

Pulse Count	TotVFrc (N)	PermStrn (εm/m)	ResiStrn (εm/m)	ResiMod (MPa)	SkinTemp (degC)
1	1077	**1076**	359	381.8	38.9
2	1075	1475	339	403.2	38.9
3	1075	1734	340	403.1	38.9
4	1077	1943	340	403.9	38.9
5	1075	2122	310	441.9	38.9
50	1077	**4424**	300	456.6	39.2
51	1080	4474	290	473.5	39.2
52	1077	4494	280	489.2	39.2
54	1085	4534	310	444.9	39.3
55	1085	4544	310	444.9	39.3
100	1080	**5331**	291	473.1	38.9

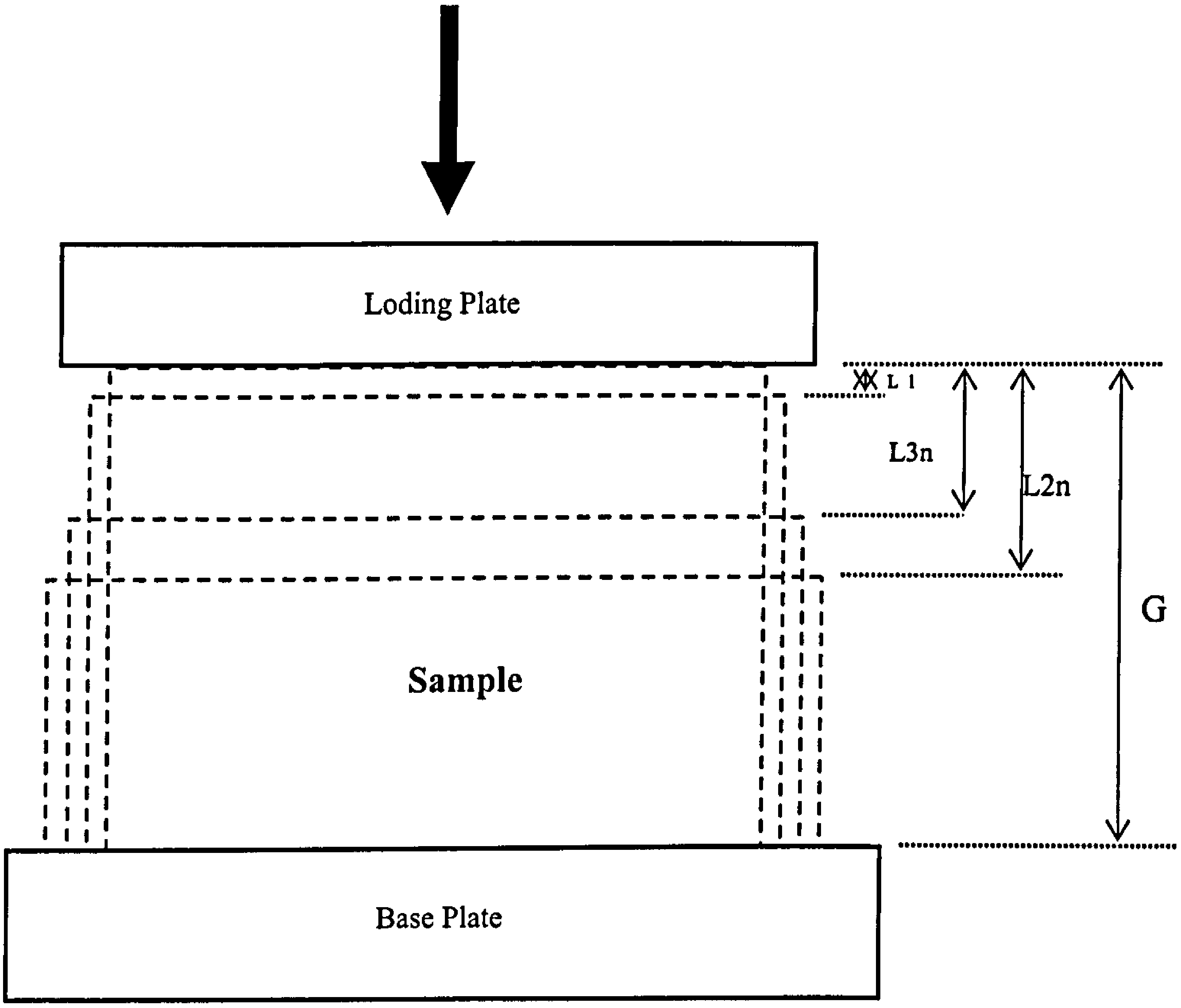
Appendix 4. 6 (cont'd)

An Example Print Out of Dynamic Creep Test Parameters

Pulse Count	TotVFrc (N)	PermStrn (um/m)	ResiStrn (um/m)	ResiMod (MPa)	SkinTemp (degC)
100	1080	**5331**	291	473.1	38.9
102	1080	5361	281	489.9	39.1
105	1083	5411	281	491	39.1
107	1077	5420	301	456.2	39.1
110	1083	5460	311	443.5	39.2
150	1080	**5929**	301	457	39.6
154	1083	5959	311	443.3	39.6
158	1083	6018	291	473.8	39.6
162	1080	6068	281	489.6	39.6
166	1085	6088	291	474.9	39.7
408	1085	**7812**	291	474.1	40.7
419	1083	7882	301	457.2	40.8
429	1083	7931	301	457.1	40.8
441	1083	7981	312	442.4	40.8
452	1083	8061	291	472.8	40.8
814	1083	**9735**	342	402.6	40.6
836	1085	9825	332	415.7	40.5
857	1085	9924	332	415.7	40.5
879	1080	10014	332	413.8	40.5
902	1083	10114	332	414.7	40.4
1227	1080	**11379**	343	401	40.2
1258	1080	11509	343	401	40.2
1291	1077	11628	353	388.6	40.2
1325	1077	11778	333	412.1	40.2
1359	1080	11887	343	400.8	40.2
2448	1075	**16381**	365	375.1	40.2
2511	1075	16690	355	385.7	40.2
2576	1075	16999	365	374.9	40.2
2643	1075	17318	375	364.6	40.2
2712	1073	17666	365	373.7	40.1
3600	1075	**22808**	408	335.4	40.1

Appendix 4.7

Dynamic Creep Data Calculation Principle



Formula:

$$\epsilon p = (L_{3n}-L_1)/G$$
$$\epsilon r = (L_{2n}-L_{3n})/(G-(L_{3n}-L_1))$$
$$Er = \sigma / \epsilon r$$
$$\sigma = F / A$$

where:

ϵp: is the permanent axial strain, ϵr : is the resilient axial strain

Er : is the resilient modulus, σ : is the peak vertical stress

F : is the peak vertical force, A : is the cross sectional ara of the sample,

L_1: is the initial zero reference displacement of the tranducer without vertical stress applied (at pre load rest condition),

L_{2n}: is the maximum displacement of the tranducer with stress applied for pulse ' n '.

L_{3n}: is the final displacement level of the tranducer for pulse ' n ' just prior to the application of the stress for pulse ' n+1 '.

Source : UMMATTA (Universal Materials testing Apparatus) for Asphalt and Unbound Specimens, Reference and Operating Manual, 1992.

APPENDIX V

BISAR 3.0 Analysis

Appendix 5.1

BISAR 3.0 , Block Results Table

Project : **CBEMs with Plastic Cells Reinforcement**
Calculated : **26/05/2003 14:47:59**

System: 1: Single Circular load

Layer Number	Thickness (m)	Modulus Elasticity (MPa)	Poisson's Ratio
1	0.074	3.56E+02	0.35
2		1.90E+05	0.28

Load Number	Vertical Load (kN)	Vertical Stress (MPa)	Horz. (Sheai Load (kN)	Horz. (Shear) Stress (MPa)	Radius (m)	X-Coordinate (m)	Y-Coordinate (m)	Shear Angle (Degrees)
1	2.50E+00	3.18E-01	0.00E+00	0.00E+00	5.00E-02	0.00E+00	0.00E+00	0.00E+00

Position Number	Layer Number	X-Coordinate (m)	Y-Coordinate (m)	Depth (m)	Stress XX (MPa)	Stress YY (MPa)	Stress ZZ (MPa)	Strain XX µstrain	Strain YY µstrain	Strain ZZ µstrain	Displacement UX (µm)	Displacement UY (µm)	Displacement UZ (µm)
1	1	0.00E+00	0.00E+00	1.00E-02	-1.45E-01	-1.45E-01	-3.19E-01	4.87E+01	4.87E+01	-6.10E+02	0.00E+00	0.00E+00	3.85E+01
2	1	0.00E+00	0.00E+00	2.00E-02	-9.36E-02	-9.36E-02	-3.12E-01	1.36E+02	1.36E+02	-6.91E+02	0.00E+00	0.00E+00	3.20E+01
3	1	1.75E-02	1.75E-02	1.00E-02	-1.33E-01	-1.33E-01	-3.15E-01	6.72E+01	6.72E+01	-6.23E+02	1.00E+00	1.00E+00	3.48E+01
4	1	1.75E-02	1.75E-02	2.00E-02	-7.87E-02	-7.87E-02	-2.94E-01	1.45E+02	1.45E+02	-6.71E+02	2.47E+00	2.47E+00	2.82E+01
5	1	5.25E-02	5.25E-02	1.00E-02	-4.29E-03	-4.29E-03	-2.93E-03	-4.96E+00	-4.96E+00	2.18E-01	1.45E+00	1.45E+00	3.12E+00
6	1	5.25E-02	5.25E-02	2.00E-02	-1.94E-02	-1.94E-02	-1.44E-02	-2.13E+01	-2.13E+01	-2.19E+00	2.78E+00	2.78E+00	3.14E+00

Appendix 5.2

BISAR 3.0 , Detailed Results Table

Project **: CBEMs with Plastic Cells Reinforcement**
Calculated **: 26/05/2003 , 11:02:02**

System: **1: Single Circular load**

Layer Number	Thickness (m)	Young's Modulus (Pa)	Poisson's Ratio	Shear Spring Compliance (m^3/N)	Load Number	Normal Stress (Pa)	Shear Stress (Pa)
1	0.074	3.56E+08	0.35	0.00E+00	1	3.18E+05	0.00E+00
2	-	1.90E+11	0.28				

Radius of Loaded Area (m	Load Position X (m)	Load Position Y (m)	Shear Direction (°)
5.00E-02	0.00E+00	0.00E+00	0.00E+00

Appendix 5.2 (cont'd)

BISAR 3.0 , Detailed Results Table

Position Number: 1 Layer Number: 1 X Coordinate (m 0.00E+00

Load No.	Distance to Load Axis (m)	Theta (°)	Radial Displacement (m)	Tangential Displacement (m)	Vertical Displacement (m)	Radial Stress (Pa)	Tangential Stress (Pa)	Vertical Stress (Pa)
1	0.00E+00	0.00E+00	0.00E+00	0.00E+00	3.85E-05	-1.45E+05	-1.45E+05	-3.19E+05

Total Stresses (Pa)	XX:	-1.45E+05	YY:	-1.45E+05	ZZ:	-3.19E+05	YZ:	0.00E+00
Total Strains	XX:	4.87E-05	YY:	4.87E-05	ZZ:	-6.10E-04	YZ:	0.00E+00
Total Displacements (m)	UX:	0.00E+00	UY:	0.00E+00	UZ:	3.85E-05		

Y Coordinate (m): 0.00E+00 Z Coordinate (m): 1.00E-02

Rad./Tang. Stress (Pa)	Rad./Vert. Stress (Pa)	Tang./Vert. Stress (Pa)	Radial Strain	Tangential Strain	Vertical Strain	Rad./Tang. Strain	Rad./Vert. Strain	Tang./Vert. Strain
0.00E+00	0.00E+00	0.00E+00	4.87E-05	4.87E-05	-6.10E-04	0.00E+00	0.00E+00	0.00E+00

XZ:	0.00E+00	XY:	0.00E+00
XZ:	0.00E+00	XY:	0.00E+00

Appendix 5.2 (cont'd)
BISAR 3.0 , Detailed Results Table

Principal Value and Direction of Total Stresses and Strains

	Normal Stress (Pa)	Normal Strain	Shear Stress (Pa)	Shear Strain	X Comp.	Y Comp.	Z Comp.
Maximum:	-1.45E+05	4.87E-05			1	0	0
Minimax:	-1.45E+05	4.87E-05			0	1	0
Minimum:	-3.19E+05	-6.10E-04			0	0	1
Maximum:			8.69E+04	3.30E-04	0.7071	0	-0.7071
	-2.32E+05				0.7071	0	0.7071
MiniMax:			8.69E+04	3.30E-04	0	0.7071	-0.7071
	-2.32E+05				0	0.7071	0.7071
Minimum:			8.59E-02	3.26E-10	0.7071	-0.7071	0
	-1.45E+05				0.7071	0.7071	0

Strain Energy (J): 9.02E+01
Strain Energy of Distortion (J): 3.82E+01

Position Number: 2 Layer Number: 1 X Coordinate (m 0.00E+00

Load No.	Distance to Load Axis (m)	Theta (°)	Radial Displacement (m)	Tangential Displacement (m)	Vertical Displacement (m)	Radial Stress (Pa)	Tangential Stress (Pa)	Vertical Stress (Pa)
1	0.00E+00	0.00E+00	0.00E+00	0.00E+00	3.20E-05	-9.36E+04	-9.36E+04	-3.12E+05

Total Stresses (Pa)	XX:	-9.36E+04	YY:	-9.36E+04	ZZ:	-3.12E+05	YZ:	0.00E+00
Total Strains	XX:	1.36E-04	YY:	1.36E-04	ZZ:	-6.91E-04	YZ:	0.00E+00
Total Displacements (m)	UX:	0.00E+00	UY:	0.00E+00	UZ:	3.20E-05		

Appendix 5.2 (cont'd)
BISAR 3.0 , Detailed Results Table

Y Coordinate (m):	0.00E+00		Z Coordinate (m): 2.00E-02					
Rad./Tang. Stress (Pa)	Rad./Vert. Stress (Pa)	Tang./Vert. Stress (Pa)	Radial Strain	Tangential Strain	Vertical Strain	Rad./Tang. Strain	Rad./Vert. Strain	Tang./Vert. Strain
0.00E+00	0.00E+00	0.00E+00	1.36E-04	1.36E-04	-6.91E-04	0.00E+00	0.00E+00	0.00E+00
XZ:	0.00E+00	XY:	0.00E+00					
XZ:	0.00E+00	XY:	0.00E+00					

Principal Value and Direction of Total Stresses and Strains

	Normal Stress (Pa)	Normal Strain	Shear Stress (Pa)	Shear Strain	X Comp.	Y Comp.	Z Comp.
Maximum:	-9.36E+04	1.36E-04			0	1	0
Minimax:	-9.36E+04	1.36E-04			1	0	0
Minimum:	-3.12E+05	-6.91E-04			0	0	1
Maximum:			1.09E+05	4.13E-04	0	0.7071	-0.7071
	-2.03E+05				0	0.7071	0.7071
MiniMax:			1.09E+05	4.13E-04	0.7071	0	-0.7071
	-2.03E+05				0.7071	0	0.7071
Minimum:			3.71E-01	1.40E-09	-0.7071	0.7071	0
	-9.36E+04				0.7071	0.7071	0

Strain Energy (J):	9.50E+01
Strain Energy of Distortion (J):	6.01E+01

Appendix 5.2 (cont'd)
BISAR 3.0 , Detailed Results Table

Position Number:	**3**		Layer Number:	1		X Coordinate (m	1.75E-02		
Load No.	Distance to Load Axis (m)	Theta (°)	Radial Displacement (m)	Tangential Displacement (m)	Vertical Displacement (m)	Radial Stress (Pa)	Tangential Stress (Pa)	Vertical Stress (Pa)	
1	2.48E-02	4.50E+01	1.42E-06	0.00E+00	3.48E-05	-1.30E+05	-1.35E+05	-3.15E+05	
Total Stresses (Pa)	XX:	-1.33E+05	YY:	-1.33E+05	ZZ:	-3.15E+05	YZ:	-5.34E+03	
Total Strains	XX:	6.72E-05	YY:	6.72E-05	ZZ:	-6.23E-04	YZ:	-2.03E-05	
Total Displacements (m)	UX:	1.00E-06	UY:	1.00E-06	UZ:	3.48E-05			
Y Coordinate (m):	1.75E-02		Z Coordinate (m):	1.00E-02					
Rad./Tang. Stress (Pa)	Rad./Vert. Stress (Pa)	Tang./Vert. Stress (Pa)	Radial Strain	Tangential Strain	Vertical Strain	Rad./Tang. Strain	Rad./Vert. Strain	Tang./Vert. Strain	
0.00E+00	-7.55E+03	0.00E+00	7.71E-05	5.73E-05	-6.23E-04	0.00E+00	-2.86E-05	0.00E+00	
XZ:	-5.34E+03	XY:	2.61E+03						
XZ:	-2.03E-05	XY:	9.90E-06						

Appendix 5.2 (cont'd)
BISAR 3.0 , Detailed Results Table

Principal Value and Direction of Total Stresses and Strains

	Normal Stress (Pa)	Normal Strain	Shear Stress (Pa)	Shear Strain	X Comp.	Y Comp.	Z Comp.
Maximum:	-1.30E+05	7.82E-05			0.7065	0.7065	-0.0408
Minimax:	-1.35E+05	5.73E-05			-0.7071	0.7071	0
Minimum:	-3.15E+05	-6.24E-04			0.0289	0.0289	0.9992
Maximum:			9.26E+04	3.51E-04	0.4792	0.4792	-0.7354
	-2.22E+05				0.52	0.52	0.6777
MiniMax:			8.99E+04	3.41E-04	-0.5204	0.4796	-0.7065
	-2.25E+05				-0.4796	0.5204	0.7065
Minimum:			2.76E+03	1.05E-05	0.9996	-0.0004	-0.0289
	-1.33E+05				-0.0004	0.9996	-0.0289

Strain Energy (J):	8.94E+01
Strain Energy of Distortion (J):	4.21E+01

Position Number: **4** Layer Number: 1 X Coordinate (m 1.75E-02

Load No.	Distance to Load Axis (m)	Theta (°)	Radial Displacement (m)	Tangential Displacement (m)	Vertical Displacement (m)	Radial Stress (Pa)	Tangential Stress (Pa)	Vertical Stress (Pa)
1	2.48E-02	4.50E+01	3.49E-06	0.00E+00	2.82E-05	-7.76E+04	-7.98E+04	-2.94E+05

Total Stresses (Pa)	XX:	-7.87E+04	YY:	-7.87E+04	ZZ:	-2.94E+05	YZ:	-1.74E+04
Total Strains	XX:	1.45E-04	YY:	1.45E-04	ZZ:	-6.71E-04	YZ:	-6.60E-05
Total Displacements (m)	UX:	2.47E-06	UY:	2.47E-06	UZ:	2.82E-05		

Appendix 5.2 (cont'd)

BISAR 3.0 , Detailed Results Table

Y Coordinate (m):	1.75E-02	Z Coordinate (m):	2.00E-02

Rad./Tang. Stress (Pa)	Rad./Vert. Stress (Pa)	Tang./Vert. Stress (Pa)	Radial Strain	Tangential Strain	Vertical Strain	Rad./Tang. Strain	Rad./Vert. Strain	Tang./Vert. Strain
0.00E+00	-2.46E+04	0.00E+00	1.49E-04	1.41E-04	-6.71E-04	0.00E+00	-9.33E-05	0.00E+00

XZ:	-1.74E+04	XY:	1.10E+03
XZ:	-6.60E-05	XY:	4.18E-06

Principal Value and Direction of Total Stresses and Strains

	Normal Stress (Pa)	Normal Strain	Shear Stress (Pa)	Shear Strain	X Comp.	Y Comp.	Z Comp.
Maximum:	-7.49E+04	1.60E-04			0.7027	0.7027	-0.1116
Minimax:	-7.98E+04	1.41E-04			-0.7071	0.7071	0
Minimum:	-2.97E+05	-6.81E-04			0.0789	0.0789	0.9937
Maximum:			1.11E+05	4.21E-04	0.4411	0.4411	-0.7816
	-1.86E+05				0.5527	0.5527	0.6237
MiniMax:			1.08E+05	4.11E-04	-0.5558	0.4442	-0.7027
	-1.88E+05				-0.4442	0.5558	0.7027
Minimum:			2.49E+03	9.42E-06	0.9969	-0.0031	-0.0789
	-7.74E+04				-0.0031	0.9969	-0.0789

Strain Energy (J):	8.94E+01
Strain Energy of Distortion (J):	6.08E+01

Appendix 5.2 (cont'd)
BISAR 3.0 , Detailed Results Table

Position Number:	**5**		Layer Number:	1		X Coordinate (m	5.25E-02	
Load No.	Distance to Load Axis (m)	Theta (°)	Radial Displacement (m)	Tangential Displacement (m)	Vertical Displacement (m)	Radial Stress (Pa)	Tangential Stress (Pa)	Vertical Stress (Pa)
1	7.43E-02	4.50E+01	2.05E-06	0.00E+00	3.12E-06	-1.29E+04	4.28E+03	-2.93E+03
Total Stresses (Pa)	XX:	-4.29E+03	YY:	-4.29E+03	ZZ:	-2.93E+03	YZ:	-1.67E+03
Total Strains	XX:	-4.96E-06	YY:	-4.96E-06	ZZ:	2.18E-07	YZ:	-6.34E-06
Total Displacements (m)	UX:	1.45E-06	UY:	1.45E-06	UZ:	3.12E-06		
Y Coordinate (m):	5.25E-02		Z Coordinate (m):	1.00E-02				
Rad./Tang. Stress (Pa)	Rad./Vert. Stress (Pa)	Tang./Vert. Stress (Pa)	Radial Strain	Tangential Strain	Vertical Strain	Rad./Tang. Strain	Rad./Vert. Strain	Tang./Vert. Strain
0.00E+00	-2.37E+03	0.00E+00	-3.75E-05	2.75E-05	2.18E-07	0.00E+00	-8.97E-06	0.00E+00
XZ:	-1.67E+03	XY:	-8.57E+03					
XZ:	-6.34E-06	XY:	-3.25E-05					

Appendix 5.2 (cont'd)

BISAR 3.0 , Detailed Results Table

Principal Value and Direction of Total Stresses and Strains

	Normal Stress (Pa)	Normal Strain	Shear Stress (Pa)	Shear Strain	X Comp.	Y Comp.	Z Comp.
Maximum:	4.28E+03	2.75E-05			0.7071	-0.7071	0
Minimax:	-2.39E+03	2.24E-06			-0.1558	-0.1558	0.9754
Minimum:	-1.34E+04	-3.95E-05			0.6897	0.6897	0.2203
Maximum:			8.84E+03	3.35E-05	0.0123	-0.9877	-0.1558
	-4.56E+03				0.9877	-0.0123	0.1558
MiniMax:			5.50E+03	2.09E-05	-0.5979	-0.5979	0.5339
	-7.90E+03				0.3776	0.3776	0.8455
Minimum:			3.34E+03	1.27E-05	0.6102	-0.3898	-0.6897
	9.43E+02				0.3898	-0.6102	0.6897

Strain Energy (J):	3.21E-01
Strain Energy of Distortion (J):	3.02E-01

Position Number: **6** Layer Number: 1 X Coordinate (m 5.25E-02

Load No.	Distance to Load Axis (m)	Theta (°)	Radial Displacement (m)	Tangential Displacement (m)	Vertical Displacement (m)	Radial Stress (Pa)	Tangential Stress (Pa)	Vertical Stress (Pa)
1	7.43E-02	4.50E+01	3.93E-06	0.00E+00	3.14E-06	-3.90E+04	1.71E+02	-1.44E+04

Total Stresses (Pa)	XX:	-1.94E+04	YY:	-1.94E+04	ZZ:	-1.44E+04	YZ:	-1.02E+04
Total Strains	XX:	-2.13E-05	YY:	-2.13E-05	ZZ:	-2.19E-06	YZ:	-3.86E-05
Total Displacements (m)	UX:	2.78E-06	UY:	2.78E-06	UZ:	3.14E-06		

Appendix 5.2 (cont'd)
BISAR 3.0 , Detailed Results Table

Y Coordinate (m):	5.25E-02		Z Coordinate (m):	2.00E-02

Rad./Tang. Stress (Pa)	Rad./Vert. Stress (Pa)	Tang./Vert. Stress (Pa)	Radial Strain	Tangential Strain	Vertical Strain	Rad./Tang. Strain	Rad./Vert. Strain	Tang./Vert. Strain
0.00E+00	-1.44E+04	0.00E+00	-9.55E-05	5.29E-05	-2.19E-06	0.00E+00	-5.46E-05	0.00E+00

XZ:	-1.02E+04	XY:	-1.96E+04
XZ:	-3.86E-05	XY:	-7.42E-05

Principal Value and Direction of Total Stresses and Strains

	Normal Stress (Pa)	Normal Strain	Shear Stress (Pa)	Shear Strain	X Comp.	Y Comp.	Z Comp.
Maximum:	1.71E+02	5.29E-05			0.7071	-0.7071	0
Minimax:	-7.72E+03	2.30E-05			-0.296	-0.296	0.9082
Minimum:	-4.56E+04	-1.21E-04			0.6422	0.6422	0.4186
Maximum:			2.29E+04	8.68E-05	0.0459	-0.9541	-0.296
	-2.27E+04				0.9541	-0.0459	0.296
MiniMax:			1.90E+04	7.19E-05	-0.6634	-0.6634	0.3462
	-2.67E+04				0.2448	0.2448	0.9382
Minimum:			3.94E+03	1.50E-05	0.7093	-0.2907	-0.6422
	-3.77E+03				0.2907	-0.7093	0.6422

Strain Energy (J):	2.67E+00
Strain Energy of Distortion (J):	2.27E+00