STIFFNESS AND DEFORMATION PROPERTIES OF ASPHALT OPEN GRADED FRICTION COURSE CONTAINING CEMENT COATED NATURAL AGGREGATES

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A thesis submitted in partial fulfillment of the requirements of
Liverpool John Moores University
for the Degree of Doctor of Philosophy

June 2010

DECLARATION

I declare that this thesis work entitled:

"STIFFNESS AND DEFORMATION PROPERTIES OF ASPHALT OPEN GRADED FRICTION COURSE CONTAINING CEMENT COATED NATURAL AGGREGATES"

is the result of my own research except as cited in the references. The thesis has not been accepted for any degree and is not concurrently submitted in candidature of any other degree.

Signature:

Name: Abbas Musa ABBAS

Date: June 2010

DEDICATION

"To the soul of my ever beloved father, to my kindest mother,

to my lovely wife

and

my dearest children.

for their support and encouragement…"

ACKNOWLEDGEMENTS

First and foremost, I would like to express my deep gratitude and most genuine thanks to the Almighty "ALLAH" (SWT) for His Blessing, for lightening up my heart with the torch of knowledge and for seeing me throughout my lifetime.

With utmost respect and pleasure, I would like to express my sincere thanks and appreciation to my academic supervisors, Dr English R., Dr. Rothwell G. and Professor Nageim H. who continuously guided me throughout my thesis work and generously shared their time and knowledge with me.

My gratitude and deep appreciation go to the Directors of Tarmac ltd Professor Robinson H. and Professor Ghazireh N. for allowing access to their testing facilities, at Wolverhampton's laboratories site, to assist in the completion of the research work and orienting me in the correct research direction.

My special thanks must be extended to all the staff of the Built environment's faculty for having a friendly environment within the department and special thanks goes to the technical staff members at the LJMU for their collaboration In particular, Mr. Feegan M. and Mr. Sinclair J. for their dedicated assistance while carrying out my laboratory work.
I would like to express special great words of thanks to my family, who tirelessly encouraged and supported me in countless ways to pursue my PhD Degree.

DEFINITIONS

AASHTO	(American Association of State Highway and Transportation Officials) is a nonprofit making independent association that represents highway and transportation departments in the USA
AAV	Aggregate Abrasion Value
ACV	Aggregate Crushing Value
Aggregate	Granular material used in construction. Aggregate may be natural, manufactured or recycled
AIV	Aggregate Impact Value
APT	Asphalt Pavement Test
ASTM	(American Society for Testing and Materials) Based in the USA. ASTM is one of the largest voluntary standards development organizations in the world. Their standards are followed worldwide
ATPB	Asphalt treated permeable bases, surface course, and upper course of a pavement which is in contact with the traffic
AUSTROADS	the Association of Australian and New Zealand Road Transport and Traffic Authorities
BS	(British Standards) is the National Standards Body of the UK, responsible for facilitating, drafting, publishing and marketing British Standards and other guidelines

Bulk Density	Is the mass of material in either a compacted or un-compacted state in a given volume and is expressed in kg/m^3
CEN	Is the European Committee for Standardisation. At this time the CEN TC227 WG1 is rationalising the draft product standards for bituminous mixtures.
Coarse Aggregate	Designation given to the larger aggregate sizes with average Diameter of $2 \text{ mm} \leq D \leq 45 \text{ mm}$
FWD	Falling Weight Deflectometer: is a testing device used to evaluate the physical properties of pavement, designed to impart a load pulse to the pavement surface which simulates the load produced by a rolling vehicle wheel
EVT	Equiviscous Temperature, The temperature at which bitumen attains the proper viscosity
Fine Aggregate	Designation given to the smaller aggregate sizes with D less than or equal to 2 mm and containing particles which mostly are retained on a 0.063 mm sieve
Flow	Deformation of the moulded specimen in millimetres (mm) at maximum load less the nominal deformation obtained by extrapolation the tangent of the graph of load against deformation back to zero load
Geophone	A device which converts ground movement (displacement) into voltage
GMB	Bulk specific gravity of a compacted specimen

Grading Particle	Size distribution expressed as the percentage by mass passing a specified number of sieves
Gravel	Product of the erosion of bedrock and surface materials
HAC	High Alumina Cement
HMA	Hot Mix Asphalt
kPa	kilopascal (kN/m^2)
LHPC	Low Heat Portland Cement
LoCATE	Low Cost Asphalt Testing Equipment
LTPP	(Long-Term Pavement Performance)
LVDT	linear variable differential transformer
MPa	Mega Pascal (MN/m^2)
NAT	Nottingham Asphalt Tester
Natural Aggregate	Aggregate from mineral sources which has been subjected to nothing more than mechanical processing
NMAS	Nominal Maximum Aggregate Size
OGFC	Open Graded Friction Course
OPC	Ordinary Portland Cement
Pavement	Structure composed of one or more courses, to assist the passage of traffic over terrain.

Pa s	In the SI system the dynamic viscosity units are Pa s, N s/m^2 or kg/m s where: 1 Pa s = 1 N s/m^2 = 1 kg/m s
PD	Permanent deformation
PEN	Penetration
Porous Asphalt	Bituminous material with bitumen as a binder prepared so as to have a very high content of interconnected voids which allow passage of water and air in order to provide the compacted mixture with drain and noise reducing characteristics
PSV	Polished Stone Value
PTF's	Pavement Testing Facilities
RHPC	Rapid Hardening Portland Cement
Rice test	Rice Density or Rice Specific Gravity. A test used to determine the maximum specific gravity or the zero voids density of a mixture used in calculating the volume of air voids in a compacted Hot Mix Asphalt (HMA) mixture
RLAT	Repeat Load Axial Test
RLIT	Repeat Load Indirect Tensile Test
Rutting	Surface depressions in the wheel path of a pavement
SHRP	(Strategic Highway Research Program) SHRP was a five year plan started in 1987 to improve highways in the USA through development of new technology

SRPC	Sulphate Resisting Portland Cement
SSD	Saturated Surface Dried
Stability	Maximum resistance to deformation, in kilonewtons (kN), of a moulded asphalt specimen
TFV	Ten Percent Fines Value
TMD	Theoretical Maximum Density
UK	United Kingdom of Great Britain and Northern Ireland
UTM	Universal Testing Machine, the Servo-pneumatic Universal Testing Machine CRT-UTM-NU is a development of the NAT which was developed by Keith Cooper and Professor Steven Brown at the University of Nottingham
Wearing course	Also called the surface course, the top course of asphalt pavements, which is in contact with the traffic
Workability	Regarding HMA, a term that refers to an HMA's ability to be placed and compacted. Workable mixes are easy to place and compact and are generally more viscous than mixes with poor workability.

ABSTRACT

The primary objective of this research study was to observe the changes in the physical properties of cement coated secondary aggregates, namely surface roughness and angle of internal friction and how these properties are related to the performance of Open Graded Friction Course asphalt mixtures, OGFC made from these materials under laboratory Uni-axial, deformation and rutting testing simulating the traffic loading and temperature of the materials in service.

The secondary aggregates, namely Croxden natural gravel aggregates, which is rejected by road engineers for use in road surfacing was upgraded with a specifically designed cement coating. Asphalt mixtures mechanical properties in terms of stiffness modulus, deformations and rutting were determined and analyzed using state-of-the-art laboratory test equipment and supported by technical literature from different information and resources.

Three aggregate types were considered in this study; Arcow rock crushed aggregates (as a control aggregate), Croxden aggregate (as a secondary or low quality aggregates) and cement coated Croxden aggregate (as an upgraded material). One aggregate gradation was designed for all mixture types and the adopted aggregate gradation had a nominal maximum aggregate size (NMAS) of 14mm. Two binder types were selected, 100/150 pen binder and 40/60 pen binder to represent the soft and hard binders respectively that are commonly used by road engineers. This research also documents a survey of literature review that led to the design of a novel machine that is eagerly needed by the pavement industry to boost the knowledge and understanding of real tyre-road interaction. Computer software, SOLIDWORKS, was used to create a 3-dimentional model of the machine that was jointly named by the Highway Agency and Tarmac ltd., as Low Cost Asphalt Tasting Equipment (LoCATE). The machine was designed for use and for the first time to investigate the performance of the said mixtures under real traffic loading and environmental conditions and to correlate its results with those obtained using Repeated

Load Indirect Tensile Test (RLIT) and Repeated Load Axial Test (RLAT), which were carried out at 10^0C, 20^0C, 30^0C and 45^0C. At an advanced stage of this research work and when the cost of fabricating LoCATE was found impossible to be met by LJMU, wheel tracking tests were done on the materials studied as a replacement of LoCATE and this part of the research program was achieved at Wolverhampton's bituminous testing laboratories in collaboration with Tarmac Ltd.

In general, the results of this research investigation indicate that the introduction of cement coated aggregates to OGFC led to an outstanding improvement in the chemical properties of the coated natural aggregates and the stiffness and deformation properties/resistance of the road mixtures containing them. This was in comparison with that made from the uncoated aggregates in all the tests that have been carried out within this research study. Also, the effects of elevated temperature were investigated and it was found to have significant influence on the engineering properties, Indirect Tensile Stiffness Moduli (ITSM) and permanent deformation/rutting characteristic of the mixtures were also investigated.

Outstanding new results were achieved, using LJMU cement coating paste for coating the waste natural aggregate and as thus this will hopefully eliminate the restriction imposed by road engineers on the use of natural gravel aggregates in road pavement surfacing layers. This is an outstanding knowledge extension in this field and marks a starting point for more research in this ever growing industry i.e. the use of waste and recycled aggregates in construction industry.

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INTRODUCTION

1.1 Introduction

Flexible pavements are a conglomeration of materials, namely aggregates and bituminous binders. These materials, their associated properties, and their interactions determine the properties of the resultant pavement. Thus a good understanding of these materials, how they are characterised and how they perform, is fundamental to understanding pavement engineering properties and its interaction with traffic loading and environmental conditions.

Throughout today's transportation industry there is a continuous need to improve the quality of the materials used for the construction of flexible pavements. These pavements constitute a very large part of the transportation networks, therefore the performance and durability of bituminous material needs a constant upgrading to meet the socio-economic and environmental demands of the transportation network's users.

Bituminous pavements consist mainly from graded crushed rock aggregates, sands and asphalt binders. In many countries there are insufficient supplies of crushed aggregates suitable for high stability, low deformation bituminous road surfacing mixes. Often these countries need to import expensive aggregates to meet their construction demands. It follows then, that road construction and maintenance can become a burden on local and national economies.

Poorer quality bituminous surfacing materials made from uncrushed poor surface quality natural gravel or reclaimed waste materials are no longer acceptable by road engineers, for they can fail under the stresses applied by modern vehicles. The use of these materials could therefore be costly in repair bills, and in the resulting traffic delays. Many countries guidelines, specifications and regulations rule out the use of these materials in all but the least severe applications such as secondary roads.

This research work focused on the improvement of gravel aggregate's surface quality so they can be used in a wider range of applications in road surfacing material by testing a coating paste made of cement and additives to improve the property of natural aggregate and eliminate the restriction in using them in bituminous materials road surface applications.

1.2 Project background

The project was developed from a PhD research work completed in 1999 at Liverpool John Moores University (JMU) by Vaughan (1999), which was composed of an investigation of cement coating on the properties of Porous asphalt containing natural aggregates, especially the bonding and chemical adhesion between cement paste and natural aggregate particles. The aim of Vaughan's work was to understand and improve the bonding at chemical and microstructure level between cement coated aggregates and asphalt binder to increase the overall mechanical strength of asphalt specimens. The author, after discussion with the supervision team, has recommended that; to give confidence in the new upgraded natural aggregate for use in road pavements, Vaughan's work needs to be extended to cover the prediction of the material deformation resistance according to the British-European standards, which is the primary aim of most highway agencies, so it became the focus of this research work and thus provide a fundamental study predicting the performance of the materials at different binder's stiffness and traffic-road interaction conditions. The above aim was also coupled with the development of a new testing facility named "Low Cost Asphalt Testing Equipment (LoCATE).

The same type of low quality aggregate materials used in the Vaughn's research work will be subjected to stiffness and deformation (rutting) resistance testing according to the British-European standards but with different type of mix, namely, Open Graded Friction Coarse. The three mixes are:

1. Mix with Arcow type aggregate (as a control mix),
2. Mix with natural aggregate (Croxden aggregate) coated with cement paste and
3. Mix with plane Croxden natural aggregate.

The effect of the coating paste on improving the deformation resistance will be mapped against those of control mixes accepted by the road engineers.
The idea of cement coating arises from the:

1) Preliminary studies of Curtis (1992), Vaughan (1999) and Al Nageim and Robinson (2006) which showed that the performance of bituminous pavement depends upon the bonding between the bitumen binder and the aggregate particles, and this bonding is a function of both physical and chemical properties of the aggregate particles.

2) Chemical properties of different types of aggregates have been studied by many researchers, especially in respect of their mechanical strength, adsorption and desorption of bitumen on the aggregate surface. Chemical tests were also used to determine aggregate-bitumen pairings that were likely to produce a high performance bituminous pavement material. These studies gave substance to the hypothesis that bitumen share an affinity with calcareous, lime-rich, high surface area microtexture aggregates. They also indicated methods of improving the surface characteristics of poor quality aggregates by using cement dry powder coating as cement being lime-rich and having a crystalline, high surface area microtexture (Kandhal et al, 1998a, Brannan et al, 1991).

3) Very limited research work and to a lesser depth of application and laboratory work was carried out by Daoud et al (1980b) on the use of coated primary aggregates in flexible pavement, compared with Vaughan's work (Vaughan, 1999), which recommended that further work on providing fundamental study in predicting the performance of the materials at different binder's stiffness and traffic interaction conditions are necessary to give confidence in the use of the cement coated aggregates in pavement engineering and specially surface friction coarses.

1.3 Project Aims & Objectives

1.3.1 The Aim of the Research Project

The aim of this research project is to investigate the performance of a new Open Graded Friction Course (OGFC) asphalt mixes containing coated natural waste aggregates, at different traffic and road environmental conditions by means of testing laboratory prepared samples for material stiffness and deformation responses using new Low Cost Asphalt Testing Equipment (LoCATE) and comparing its results with those resulting from the use of The Nottingham Asphalt Tester (NAT) and The Wheel Tracking Testing Facilities available as laboratory testing facilities in the pavement industry.

LoCATE has been designed in this research work to operate at a real tyre/road surface environment interaction. The new testing facility will allow the researcher to extend the knowledge by defining in fundamental terms tyre/road interaction in both laboratory and real road pavement environments. By doing so i.e. using the same testing facility to test in situ samples and similar samples extracted from the same road section and tested in laboratory conditions by the same facility. This will eliminate many discrepancies that might arise from testing the samples in two different testing facilities and prepared at different conditions. This also will allow the researchers to optimize the design of the Hot Mix Asphalt (HMA) in general and the design of HMA containing the new upgraded aggregate in particular.

1. 3. 2 The Project Objectives

The following objectives seek to define the scope of the research, and have provided the framework for the experimental programme and subsequent analysis:

1. To reveal by literature review, those aggregate properties considered necessary for good bonding with bitumen and compare those properties with the properties of the new coating paste.

2. Compare the bitumen affinity between:

- Coated aggregates, primary aggregate and
- Uncoated aggregates.

3. To measure the deformation resistance of mixes containing control aggregates, natural aggregate and cement coated aggregates at different binder stiffness and temperatures using NAT and Wheel Tracking testing facilities.

4. To create a 3-dimentional model of a new pavement testing facility, the "Low Cost Asphalt Testing Equipment (LoCATE)", through the use of the computer software (SOLIDWORKS version 2006 SP4.1) available at LJMU, School of Engineering, followed by the assembly and calibration of the LoCATE machine.

5. To measure the deformation resistance of the three mixes mentioned in 3 above using LoCATE and comparing the results with those tested using Wheel Tracking test facilities.

6. Critical analysis and assessment of the new mixes in terms of their mechanical properties and suitability for use in road pavement industry.

7. Publication of conference and journal papers.

8. Writing up the thesis.

1.4 Methodology

The methodology adopted to meet the objectives of this study involves:

1. A literature review which includes:

1.1 Identifying all the factors and the properties of aggregates considered necessary for use in road surface layers, 1.2 Studying the influence of the properties of the new cement coated aggregates on:

   i- Their adhesion with binder and

   ii- The permanent deformation response of pavement mixtures containing coated aggregates compared with control mixes

1.3 Testing methods that are used to characterize permanent deformation property of asphalt mixtures.

2. Laboratory investigations which involves:

Manufacturing laboratory specimens, made with two levels of binder hardness (100/150 pen graded and 40/60 pen graded) and three types of mixes (mixes containing natural low quality aggregates, coated aggregates and crushed primary aggregates), compacted by the Marshall compacter, and tested mainly at different temperatures (10^0C, 20^0C, 30^0C and 45^0C) for all the mixtures.
All the mixes will be prepared at their optimum binder contents according to the BS DD 232: 2005 procedures.

i. Evaluating stiffness modulus and deformation of the mixes containing untreated aggregates using the NAT testing machine.
ii. Evaluating rutting (rut depth) of the mixes containing untreated aggregates by using the Wheel tracking testing machines
iii. Evaluating stiffness and deformation / rutting (rut depth) of the mixes containing treated aggregates by using both the NAT and Wheel tracking testing machines
iv. Evaluating deformation and rutting (rut depth) of the mixes containing treated and untreated aggregates by using LOCATE, NAT and Wheel tracking testing machines
v. Analyzing/ Evaluating the effect of cement coated aggregates on the stiffness modulus, permanent deformation and rutting responses of all mixtures studied.

3. Critical analysis of the results and evaluation of the suitability of the new developed materials for use in road pavement construction.

1.5 Sequence of Thesis Chapters

The thesis has been constructed in nine chapters:

1- Chapter One: **Introduction.**

This chapter covers the project background, project aims and objectives, research methodology and the sequence of thesis chapters.

2- Chapter Two: **Literature Review.**

This chapter will be concentrating on the literature review on the use of primary and natural aggregates in Hot Mix Asphalt (HMA), and on why natural aggregates are not considered suitable by road engineers for use in HMA. This chapter will also be covering bitumen-aggregates adhesion and the introduction of the upgrading cement paste to natural aggregates. The limited previous research work on the evaluation of the chemical and mechanical properties of coated aggregates will be discussed in this chapter as well as the evaluation of the mechanical properties of HMA containing coated aggregates.

3- Chapter Three: **Materials Design.**

Chapter three will have a detailed review of the:

- Description of the cement coating concept,
- Choice of cement coating type and amount,
- Surface roughness measurement of utilized aggregates,
- Asphalt binders utilized in the research work,
- Determination of mixes volumetric properties,
- Mixture design procedures and
- Aggregates gradation modification.

4- Chapter Four: **Design of OGFC for Performance tests (NAT and Wheel Tracking Testing).**

This chapter will be discussing the mix design procedures to compose the Open Graded Friction Course (OGFC) asphalt mixes, and this will include:

- Aggregate types and bituminous binders utilized for mixes preparations,
- Binder drainage test,
- Specimens preparation for the NAT facility (RLIT and RLAT tests),
- Specimen volumetric properties,
- Specimens preparation for the Wheel tracking tests,
- Results of volumetric properties and
- Graphical and summary of results

5- Chapter Five: **Determination of the Indirect Tensile Stiffness Modulus for OGFC Mixes.**

This chapter will be concentrating on the evaluation of the stiffness moduli for the three types of mixtures under consideration, and the chapter will include the following topics:

- Introduction
- The Repeat Load Indirect Tensile testing method (RLIT),
- (RLIT) Testing procedures,
- RLIT results and discussion
- Factors affecting resilient modulus values
- Summary of findings

6- Chapter Six: **Evaluation of Deformation by Repeat Load Axial Tests.**

This chapter will focus on the (RLAT) testing procedures that were carried out in order to investigate the axial deformation found in laboratory manufactured specimens containing the three different types of aggregates and the two types of binders.

7- Chapter Seven: **Evaluation of Deformation by Wheel Tracking Tests.**

This chapter will be focusing on the testing procedures of wheel tracking tests' that were carried out at Wolverhampton's laboratories, in collaboration with Tarmac ltd.

8- Chapter Eight: **The Concept led to the Creation of the LoCATE Facility.**

This chapter will be discussing the industrial and educational needs for a new novel facility developed in this research work which has the characteristics of being mobile, versatile and affordable by research institutes as well as private contractors worldwide, so they can investigate road bituminous problems more easily and frequently.

It will also be covering the following topics:

- Impact of traffic loading & environmental conditions,
- Road maintenance cost,
- Accelerated pavement testing facilities,
- Summary of the weaknesses of the existing pavement tasting facilities,
- The development of the Low Cost Asphalt Testing Equipment (LoCATE) and
- Detailed specifications of the LoCATE Testing Facility.

9- Chapter Nine: **Conclusions and Further Recommendations.**

Conclusions of results and recommendations for further works will be dealt with in this chapter.

CHAPTER 2

Background and Literature Review

2.1 Introduction

As was mentioned in chapter one, flexible pavements are conglomeration of materials and these materials include aggregates (coarse and fine), asphalt binders and binder modifiers. A wide range of rocks of igneous, sedimentary and metamorphic origins and a number of artificial aggregates are used in bituminous mixes (Hunter, 2000). The general requirements for aggregates are to be durable (i.e. resistant to abrasive wear), strong (i.e. resistant to slow crushing loads and to rapid impact loads) and where appropriate, i.e. in surface course and surface dressing applications, resistant to the polishing action of traffic (Smith and Collis, 2001). An additional property required especially for surface course aggregates is that they should show good quality adhesion with bituminous binders (Mick et al, 2001).

It is important to have suitable proportions of asphalt and aggregates in HMA so as to develop mixtures that have desirable properties associated with good performance. These performance measures include the resistance to the three primary HMA distresses: permanent deformation, fatigue cracking, and low temperature cracking. Permanent deformation refers to the plastic deformation of HMA under repeated loads. This permanent deformation can be in the form of rutting (lateral plastic flow in the wheel-path) or consolidation (further compaction of the HMA after construction). Aggregate interlock is the primary component that resists permanent deformation with the binder having only a minor role. Angular, rough-textured aggregates will help reduce permanent deformation. To a significantly lesser extent, stiffer binder may also provide some minor benefit (Fwa, 2006). As highway pavements consist, mainly of multi-layers of selected and processed materials whose main function is to impose and spread the applied vehicle loads to the sub-grade, the most important aim is to guarantee that the transmitted stress has been adequately reduced and would not top the supporting stress capacity of the sub-grade. Highway pavement layers mostly consist of surfacing layer, road-base and sub-base. Each of these layers may be subdivided into two more layers. The surfacing layer includes a wearing course (surface course) and a binder-course. The base may be divided into an upper and lower road-base, each of which may contain the same or different materials. The road base may be divided into two layers namely road base and sub-base (Fig.2-1).

Surface layers	**Surface Course**
	Binder Course
Road base layers	**Sub-base Course or Road Base Materials**
	Capping Layer (Optional)
	Sub-grade (Existing Soil)

Fig.2-1: Typical Flexible Pavement Structure

The wearing course (surface course) is one of the main pavement layers; its functions are to resist the deformation by traffic as well as the effects of weather, abrasion and fatigue. It should also give sufficient skid resistance in wet weather as well as allowing a relaxing vehicle journey. Since it is the prime pavement components, that distribute the applied loads to the lower layers, they must be strong enough to protect the lower layers of the pavement. This requires a list of requirements for wearing courses to compromise in terms of the component material types used, in particular the performance of aggregates.

Aggregates are the essential component in bituminous mixtures. They must have specific properties to withstand the stress imposed on the road surface layer. The main part of designing successful bituminous mixtures is the selection of a physically powerful aggregate. The best aggregate for producing a successful bituminous mix would have suitable gradation and size, be strong and tough, and be angular in particle shape whilst providing good adhesion properties with the binder. The high quality aggregates applied in bituminous mixture would provide high resistance to premature failure, such as rutting and cracking during the pavement's service life (Patrick, 2003). The emphasis in this research will be upon the use of upgraded natural rounded aggregates type (i.e. naturally occurring aggregates).

2.2 Types of Hot Mix Asphalt (HMA)

HMA can be designed and produced from many different aggregate types, a wide range of aggregate size combinations and a variety of binder types. Every mix has its specific properties which make it suitable for particular conditions of use. All these mixes can be produced using a blend of new and reclaimed materials. An asphalt mix is normally modified with some additives to improve one or more of the engineering properties such as durability, fatigue resistance and resistance to plastic deformation, and are normally distinguished by aggregate grading; and they are in general classified into three categories (Figure 2-2):

a- Dense graded,

b- Open graded and

c- Gap graded mixes

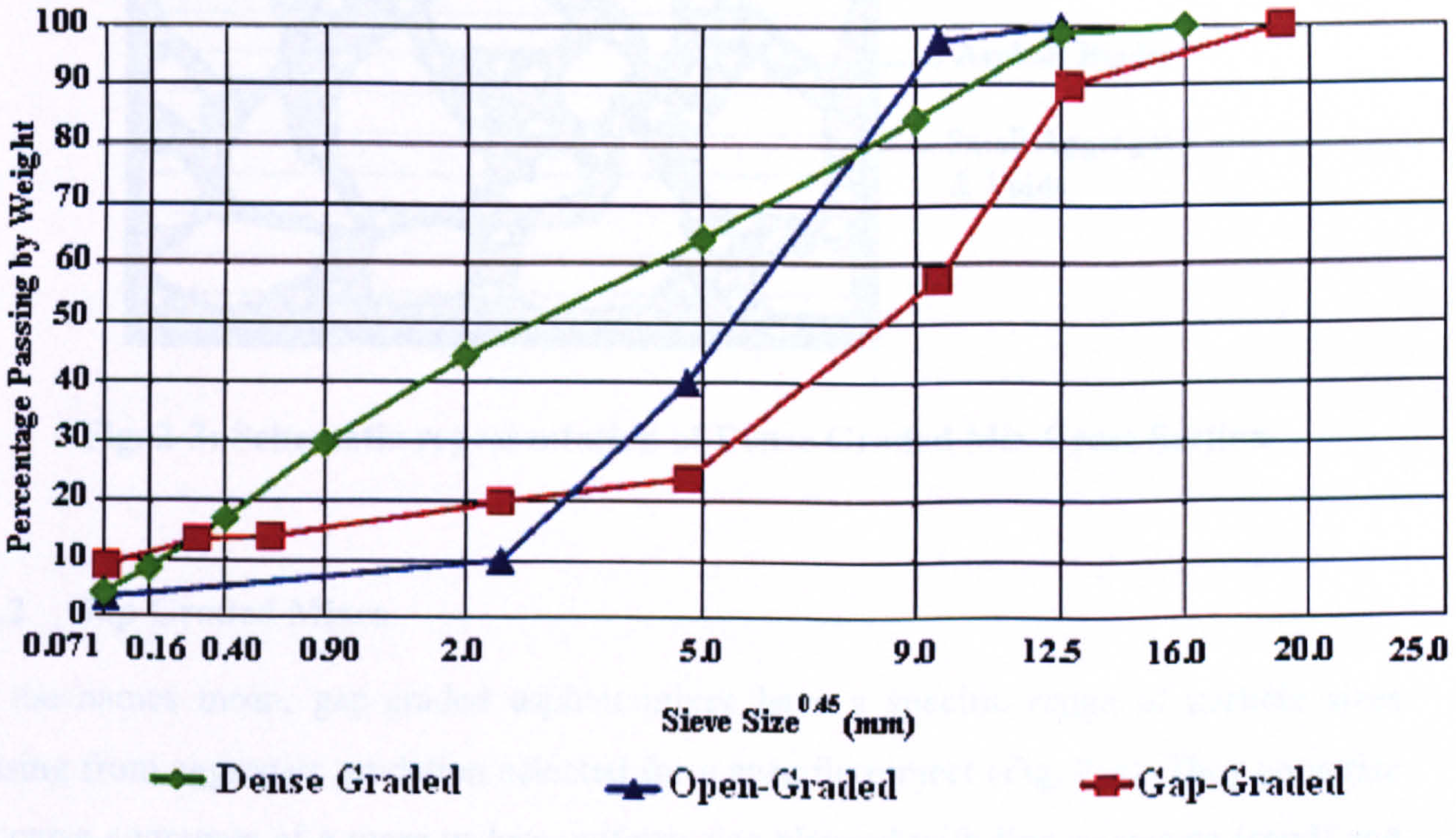


Fig.2-2: Aggregate Gradations of Common Asphalt Mix Types (Fwa, 2006)

2.2.1 Dense Graded Mixes

A dense-graded mix is a well-graded HMA and it is relatively impermeable nominal stone sizes of up to 40 mm are commonly used, and this type of mix derives its high resistance to plastic deformation from the well graded coarse aggregates (Fig. 2-3). They can be classified as fine-graded or coarse-graded. Fine-graded mixes have more fine and sand sized particles compared to coarse-graded mixes. This mix can suit all pavement layers and all traffic conditions. Dense graded mix involves a combination of well graded aggregate and asphalt binder. The comparatively low binder content of these types of mixes, together with the macrostructure roughness of the aggregate, make these mixes more suitable for base coarse and inappropriate for use in surfacing (SABITA, 2001).

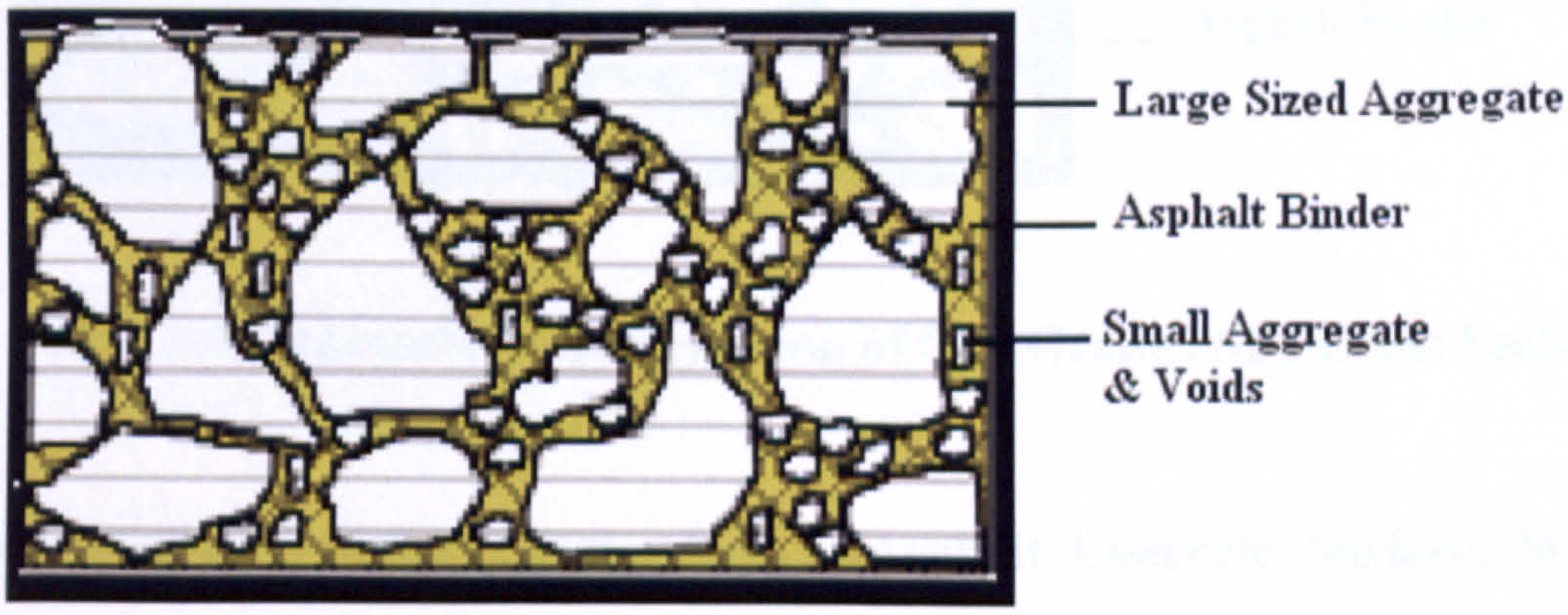


Fig. 2-3: Schematic representation of Dense Graded Mix Cross Section

2.2.2 Gap Graded Mixes

As the names mean, gap-graded asphalt mixes have a specific range of particle sizes missing from aggregate gradation selected for a specific project (Fig. 2-4). They comprise of coarse aggregate of a more or less uniform size blended with fine aggregate (sand) and filler. Research studies have pointed to that gap-graded mixes are better than densely graded mixes with regard to fatigue resistance (SABITA, 2001; CSRA, 1987). In practice, however, this will depend on the grouping of layer thicknesses. For some layer thickness combinations the strains in more continuously-graded mixes may be lower, due to their increased stiffness. This can result in similar or higher fatigue performance for densely graded mixes than for gap-graded mixes. Recent in-service experience and controlled accelerated trafficking by the Heavy Vehicle Simulator (HVS) (seen in section 8.3.1.2.a of this thesis) have indicated that with some materials it may be difficult to design gap-graded mixes that have sufficient resistance to plastic deformation under heavy traffic conditions. The quality of sand (or fine aggregate) is of particular importance in this regard. Gap-graded asphalt has slightly higher voids in the mineral aggregate and a lower tensile strength than the more densely graded mix types (Mick et al, 2001).

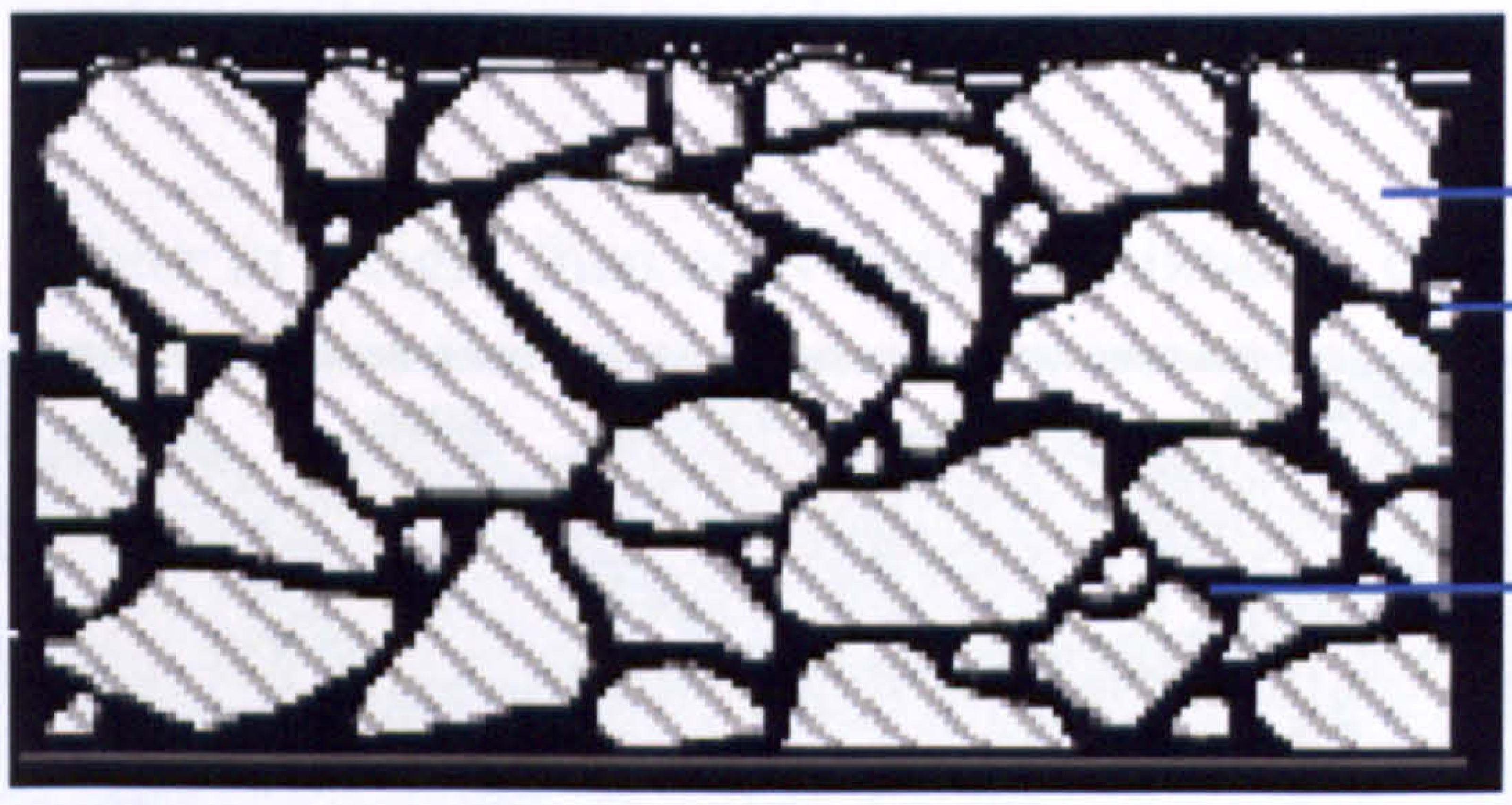


Fig. 2-4: Schematic Representation of Gap Graded Mix Cross Section

2.2.3 Open Graded Friction Course Mixes (Asphalt Concrete Surface, Wearing Course)

Open graded porous asphalt mixes developed and used, by the military airfield runway pavements, in UK, in the early 60s as porous friction course material to avoid aquaplaning and skidding in wet weather (Khalid and Jimenez, 1996). In later years, as the technology was developed for road surfaces, different names were used based on the application and its looks, such as popcorn mix, whisper mix, drainage mix, as well as OGFC mix.

An open-graded HMA mixture is featured for its high permeability (Fig. 2-5). Open-graded mixes combines crushed stone (or gravel) and a small percentage of manufactured sands. The two most typical open-graded mixes are Open Graded Friction Course (OGFC) and Asphalt Treated Permeable Bases (ATPB). OGFC mix is used only at surface course and its air voids is more than 15% and this reduces splash/spray during downpour and result in smooth surface and reduces tyre noise. (ATPB) is less efficient and used only with dense-graded HMA and Stone Mastic Asphalt (SMA) for drainage. Research, in the United States of America by Kuennen (1996), has shown that OGFC provides immediate reduction in tyre-pavement noise level by as much as 5 decibels (dBA), although the effect may diminish with time but these benefits can come at an extra cost, with OGFC costing up to 35% per ton more than same bulk of conventional asphalt pavements. Interestingly, in the UK stringent complaints from residents have encouraged Highway Agency to reduce noise level by overlaying all major highways with Asphalt Surface Friction Coarse, by 2010 (Assoc. of British Drivers, 2001).

OGFC are lighter in weight than conventional mixtures, due to their high level of porosity, an

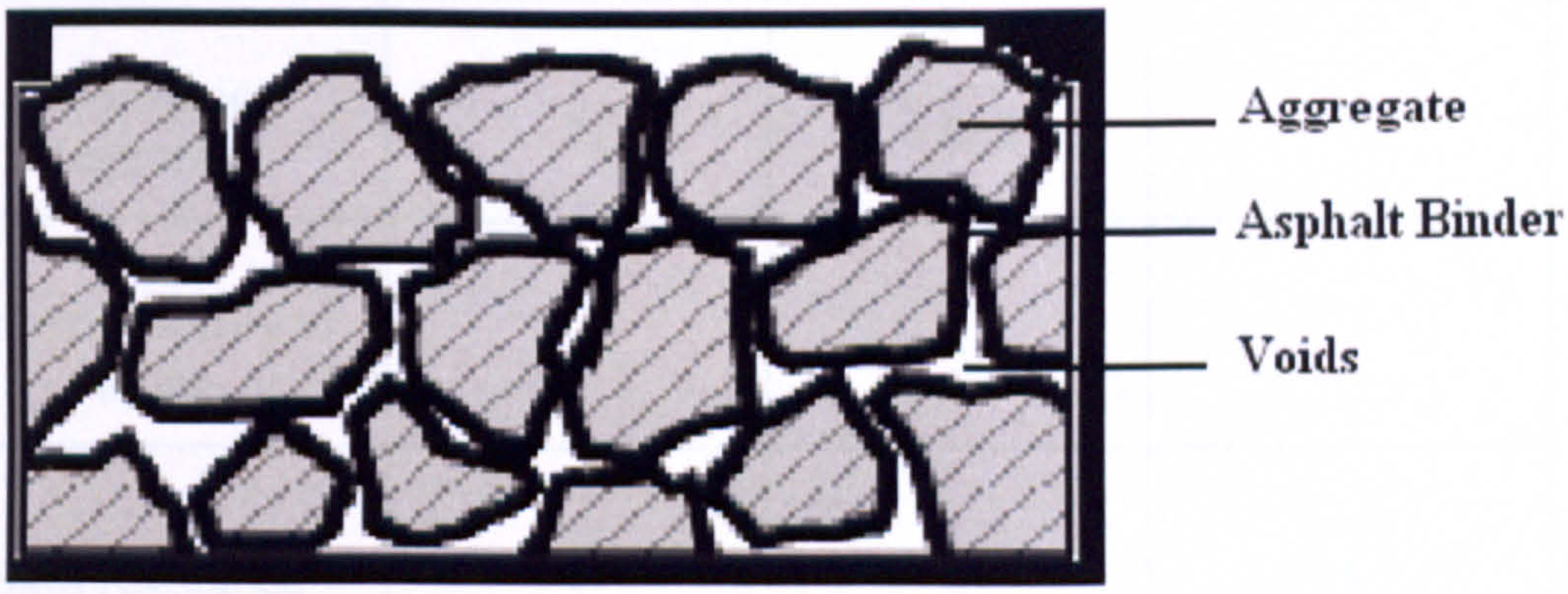


Fig. 2-5: Schematic Representation of Open Graded Mix Cross Section

equivalent amount of OGFC material can cover more square metres of pavement surface than a conventional mixes. But even the cost disadvantage is far overcome when long-term life-cycle costing is used, both in terms of reduced future maintenance and in reduced delay costs to highway users during maintenance operations. And although that some previous research work indicated that OGFC may disturb winter snow and ice control, with higher levels of salt required, Oregon, a leader in OGFC research and use in the USA, has indicated that the use of OGFC does not prevent the use of sand as friction material in winter conditions (Kuennen, 1996). Table 2-1 shows the typical rating of asphalt types in terms of their engineering properties.

Grading / Mix property:	Dense -graded	Gap-graded	Open-graded
Durability	3	3 to 4	1
Tensile strength	4	3	1
Fatigue Resistance	2	4 to 5	4
Deformation Resistance	5	2 to 3	3
Impermeability to water	2	5	1
Skid resistance	-	4	5
Workability	2	4	5
Stiffness	5	3	1
Shrinkage	5	3	4
(1 = poor and 5= good)			

Table 2-1: Engineering Properties of common Asphalt Mixtures (CSRA Committee of State Roads Authorities, 1987)

2.3 Open Graded Porous Asphalt Mixes

Unlike conventional dense asphalt mix that normally has low void contents; porous asphalt is designed with void content of up to 25%. These interconnected air voids allow rain water to infiltrate vertically through the layer, which is normally constructed on an impermeable layer, and drains out side way according to the cross fall of the pavement to the road edge into appropriate drainage system. Porous asphalt was first developed and originally used by the military airfield runway pavements in the early 60s in UK as porous friction course material to avoid aquaplaning and skidding in wet weather (Khalid and Jimenez, 1996).

In later years, as the technology was developed for road surfaces, different names were used based on the application and its looks, such as Popcorn mix, Whisper mix, Drainage mix, etc. With the development in binder technology and mix design methods, porous asphalt is gaining recognition especially for road surface application. In the last decade, open graded friction course, has been used extensively worldwide, especially in European countries (where it is typically called porous asphalt as well as OGFC) since the 1980's in many countries such as Germany, Netherlands, France, Italy, United Kingdom, Belgium, Spain, Switzerland, and Austria (Kandhal, 2002). It is also being used in Japan, Singapore and some states in the USA, such as Florida and Georgia (Kandhal et al, 1998b), for its benefits in noise reduction and improved safety under wet conditions, namely:

1. Reduced spray levels from vehicle tires in wet conditions,
2. Prevention of aquaplaning,
3. Improved skid resistance in wet conditions,
4. Reduction in vehicle induced traffic noise generated at the tire/ road interface,
5. Reduction in headlight glare from oncoming vehicles on wet pavements and
6. Improved night time wet weather visibility.

2.4 Characteristics of Open Graded Porous Asphalt Mixes

In general, porous asphalt consists of large proportion of coarse aggregate, generally 75%-85%, 4-5% binder and the rest fine aggregate and filler. It is also designed to contain more than 15% and up to 25% air voids by mean of specially selected aggregate grading. With the high air voids content that are interconnected, the wearing course with this porosity will allow water to seep and drain away.

2.5 Predicting Mixtures Performance

There are three general varieties of laboratory mixture design and evaluation tests:

1. Performance related,
2. Performance based, and
3. Index tests.

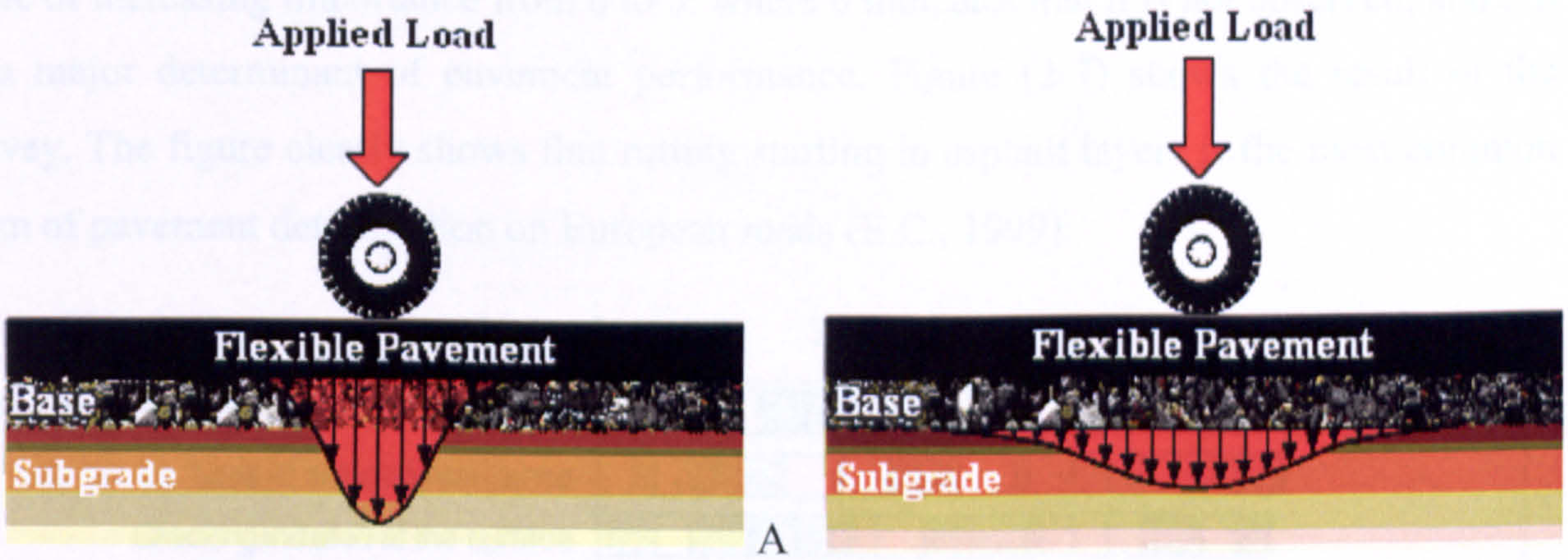
Performance-related tests measure properties or responses related to mixture performance (density, air voids, etc.) but separately are inadequate to drive a performance prediction model. Performance based tests evaluate material properties such as (resilient modulus, complex modulus, etc.) that can be used in fundamental response models to predict mixture response to applied vehicle and environmental loads.

Index tests, also known as torture tests, are experimental and apply very severe loading conditions on the test sample to evaluate a type of failure condition in a tested mix (FHWA , 1998). As stated previously, there is an increasing need for a simple performance test that can accurately predict the mixture's ability to resist permanent deformation (rutting).

Witczak et al (2002a) defined the simple performance test as follows: *"A test method(s) that accurately and reliably measures a mixture response characteristic or parameter that is highly correlated to the occurrence of pavement distress (e.g., cracking and rutting) over a diverse range of traffic and climatic conditions"*.

2.6 Permanent Deformation (Rutting) in Flexible Pavements

Permanent Deformation (Rutting) is one of the main distress mechanisms in flexible pavements, and it can be defined as a longitudinal surface depression in the wheel path of road traffic (Fig. 2-6). It can occur in all layers of the pavement structure and results from lateral distortion and densification. Moreover, rutting represents a continuous accumulation of incrementally small permanent deformations from each load application (Ralph and Haas, 1982; Werkmeister et al, 2004). There are different causes of rutting depending on configuration and structural capacity of the different layers and environmental conditions, in most cases, by pavement upheaval along the sides of the rut.



A B

Poorly designed Pavement well designed Pavement

Fig. 2-6: Flexible pavement load distribution **(Washington DOT, 2003)**

Significant rutting can lead to serious structural failure and hydroplaning, which is a safety hazard. Permanent deformation is composed of two different mechanisms (Roberts et al, 1996): densification and repetitive shear deformation. The densification takes place at the early stage of cyclic loading, whereas the shear deformation is a long term process. Lytton (2000) showed that permanent deformation relates to micro-cracking and defined a transition point where micro-cracking starts [•].

[•] As pavements expand and contract, internal stresses are produced. Aggregates have different expansion characteristics than the asphalts. As asphalts age, the internal stresses allow for pavements to form micro cracks.

When an asphalt mix is subjected to repeat loading, it hardens with accumulating plastic deformation. If there is no micro-crack preventer, nor the material heals rapidly, it will reach a point where it is stiff enough for micro-cracks to initiate and grow. The asphalt mix starts accumulating more plastic deformation after the start of micro-cracks, where is commonly called (tertiary flow) (Witczak and Kaloush, 2002b).

Because of the increase in tyre pressure and axle loads in recent years (COST 347, 2001), rutting has become the dominant mode of failure of flexible pavements in many countries. In Europe, a survey was conducted, under the COST 333 programme (E.C., 1999), to determine the most common type of pavement deterioration. Accordingly, countries were asked to rate the most common forms of deterioration observed on their roads using a rising scale of increasing importance from 0 to 5: where 0 indicates that it is not observed; and 5 it is a major determinant of pavement performance. Figure (2-7) shows the result of the survey. The figure clearly shows that rutting starting in asphalt layers is the most common form of pavement deterioration on European roads (E.C., 1999).

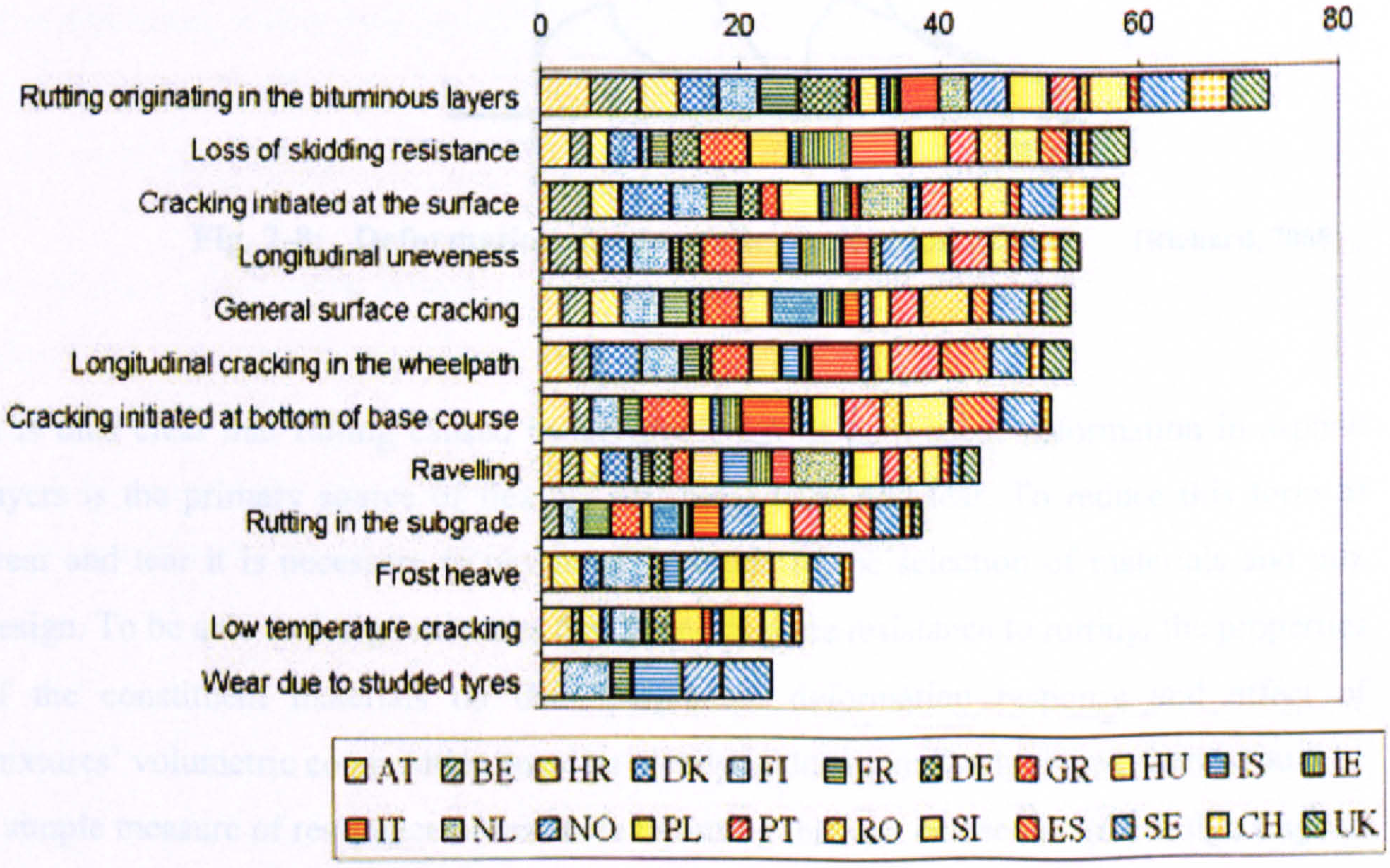


Fig. 2-7: Rutting as a main deterioration in EU countries **(E.C., 1999)**

As shown in Figure (2-8), rutting is in fact a repeated creep mechanism with sinusoidal loading pulses. In this case the pavement layer is not forced back to zero deflection but would recover some deformation due to elastic energy stored in the material of the layers. Under this type of loading, the energy is dissipated in damping, also called visco-elasticity. The damping energy is mostly recoverable but requires time to be effectively dissipated. The energy related to the permanent flow, however, is lost and thus called permanent. The permanent part of the dissipated energy is believed to be the main contributor to the rutting behaviour of asphalt mixtures and pavement layers (Richard, 2008).

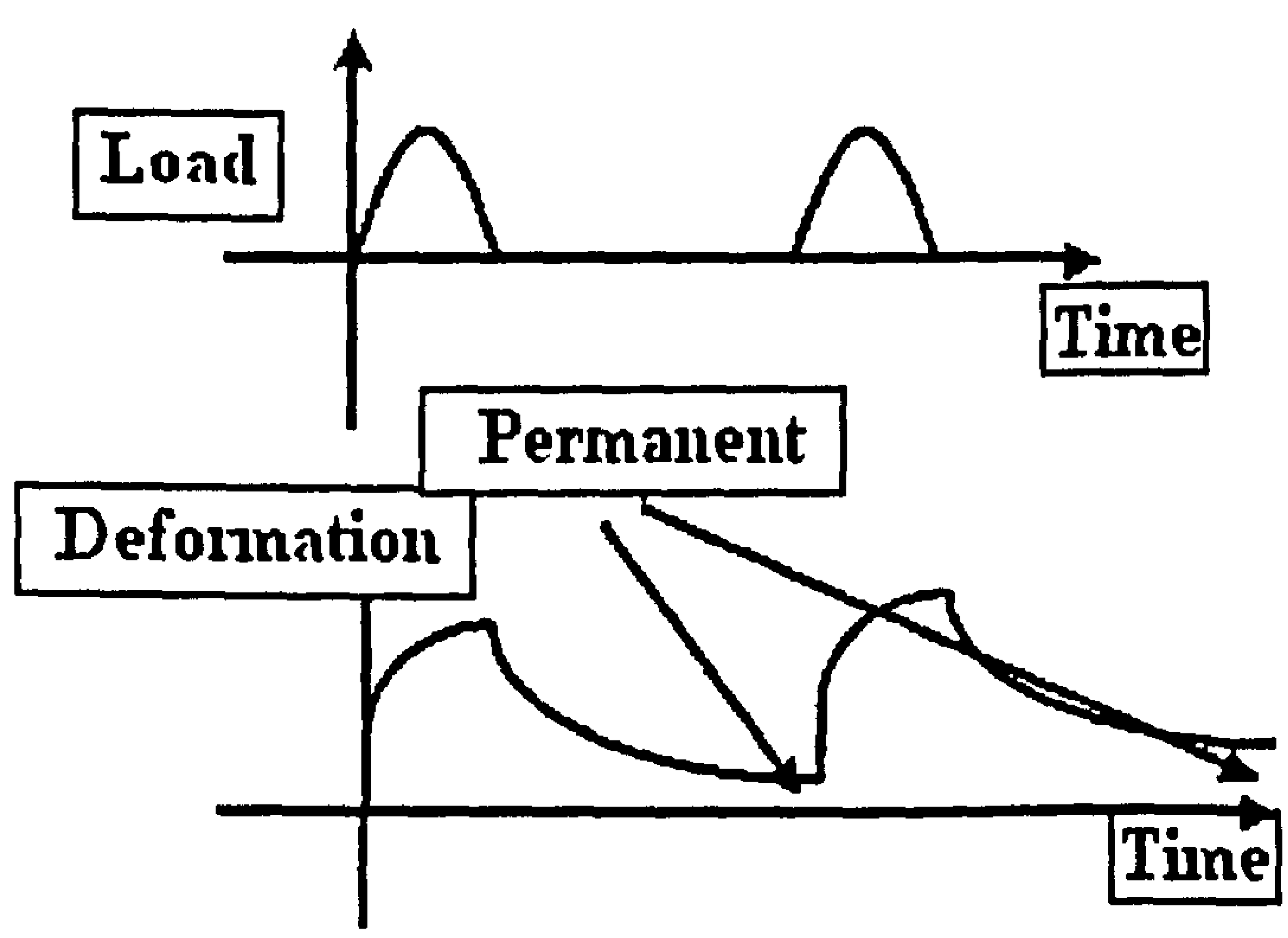


Fig. 2-8: Deformation due to cyclic application of load (Richard, 2008)

It is thus clear that rutting caused by accumulation of permanent deformation in asphalt layers is the primary source of flexible pavement wear and tear. To reduce this form of wear and tear it is necessary to pay more attention to the selection of materials and mix design. To be able to design mixtures that have adequate resistance to rutting, the properties of the constituent materials on their permanent deformation response and effect of mixtures' volumetric composition must be clearly understood. Furthermore, there should be a simple measure of resistance of mixtures to rutting that can be used at mix design stage to enable estimation and selection of rut resistant mixtures. These issues are the main areas focus of this research work.

2.7 Visco-elastic Behaviour of Porous Asphalt Mixture

The response of materials to load is defined by the stress-strain behaviour. Elastic materials show time-independent behaviour and can be characterized by its elastic modulus. Viscous materials behave time-dependently and exhibits continuous non-zero strain after the stress has been removed. Asphalt binder exhibits both elastic and viscous behaviour; thus it is considered visco-elastic material. Figure (2-9) shows the stress-strain outcome of the three materials under a simple creep test. A constant load is applied to a material at time (t_0) and removed at time (t_1) (Figure 2-9).

The elastic material instantly deforms to a continuous strain after a load is applied and recovers to zero when the loading is removed (Figure 2-9 a). Linear viscous materials deform at a constant rate when the load is applied at time (t) and continues to deform at the same rate until the load is removed, beyond which there is no further strain alteration (Figure 2-9 b). Visco-elastic materials experience an instant strain followed by a gradual time-dependent strain rise up to time (t_1). When the load is removed the material experiences a small immediate strain recovery, followed by a time-dependent strain recovery (Figure 2-9 c).

The amount of recoverable strain would be a function of the time allowed for recovery, whilst the non-recoverable strain is called permanent deformation. Most visco-elastic materials display a significant amount of delayed elastic response that is time-dependent but fully recoverable. Some types of binder; display significant plastic strains over finite recovery times, thus having a visco-plastic behaviour.

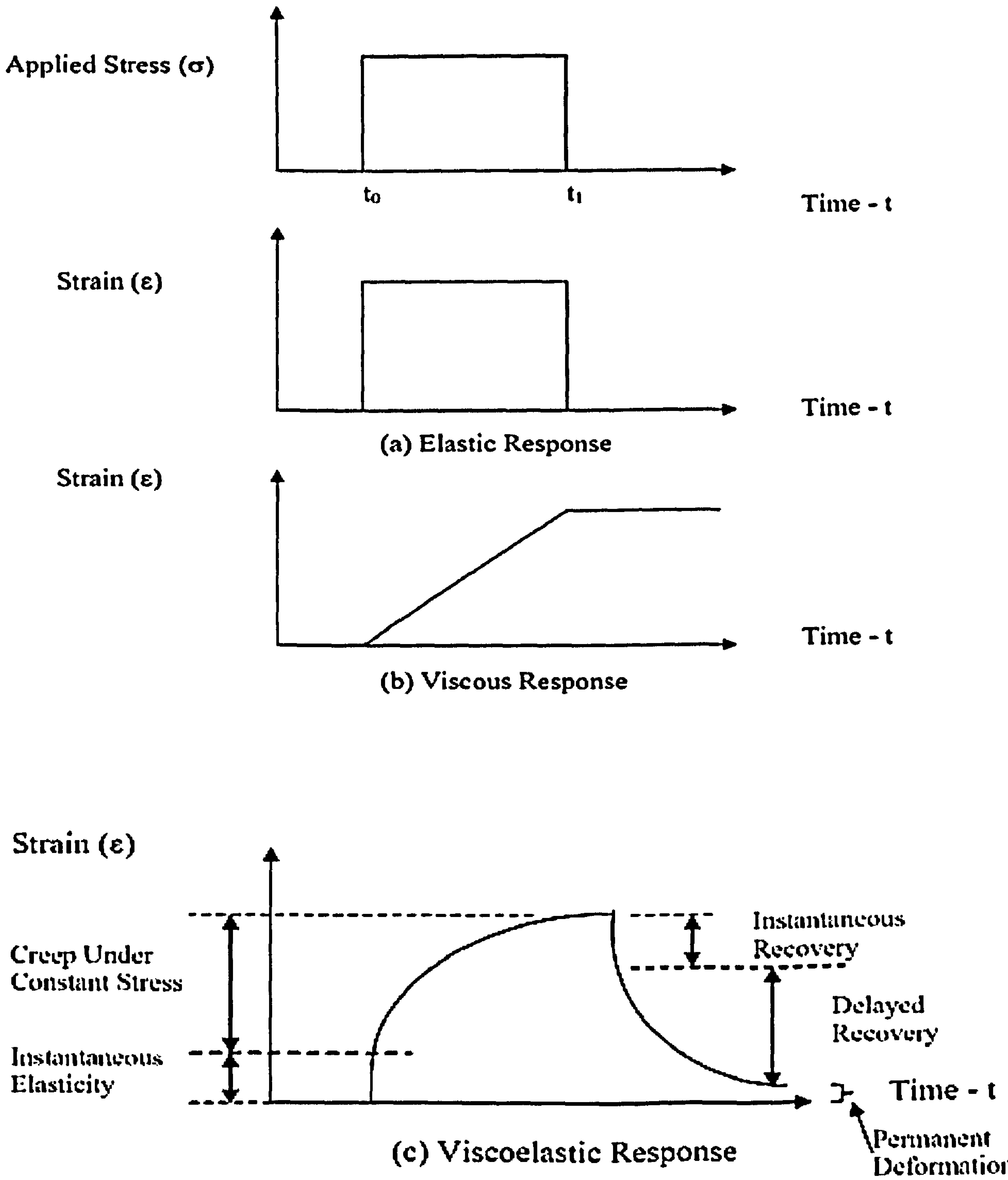


Fig. 2-9: Idealized Response of Elastic, Visco and Visco-elastic Materials under Constant Stress

Loading: a) Elastic material b) Viscous material c) Visco-elastic material

(Abbas A. et al, 2004)

2.8 Aggregate Characteristics Affecting HMA Performance

Shape, texture, and angularity are among the properties of aggregates that have a significant effect on the performance of hot-mix asphalt as most of the available information on the influence of aggregate characteristics on performance emphasizes. Most of the test methods used in the literature did not separate the influence of angularity from that of texture. Therefore, the term macrostructure is used in this study to reflect the combined effect of angularity and texture. Previous research confirms that aggregate geometric irregularity improves the resistance of HMA to rutting. Also, aggregate macrostructure influences the resistance of asphalt mixtures to fatigue cracking (Hunter, 2000).

In general, angular aggregates that increase mix stiffness are needed for thick pavements, while smooth aggregates that reduce mixture stiffness are needed for thin pavements to provide resistance to fatigue cracking (Kandhal et al, 1998a). Aggregates macrostructure also improves bonding between the aggregate surface and asphalt binder, and thus generally minimizes stripping problems.

2.9 Aggregates Quality Needed for Open Graded Asphalt

Good quality aggregate with rough macrostructure, higher abrasion value, crushing value and polish resistance value is selected, specially shaped and sieved aggregates for porous asphalt are critical in order to achieve the interconnected air voids so that high permeability can be measured (Kandhal et al, 1998b). As an example of this imbalance, the mineral production in the United Kingdom in 2006 shows the discrepancies in values between crushed aggregates and natural aggregates (BGS, 2006), as shown in Table (2-3).

Classification	Description	Examples
Rounded	Fully water worn or completely shaped by attrition	River or sea gravels, seashore and windblown sand
Irregular	Naturally irregular or partly shaped by attrition with rounded edges	Pit sand, pit gravels, land and dug flints
Angular	Possessing well-defined edge intersection of roughly planer faces	Crushed rocks of all types, talus
Flaky	Material, usually angular of which the thickness is small relative to the width and/or the length	Laminated rocks

Table 2-2: Classification of Aggregates Shapes **(Patrick, 2003)**

Construction Minerals	**Thousand tonnes**
Aggregates	237, 685
of which: Land-won sand & gravel	71, 418
of which: Marine-dredged sand & gravel	20, 689
of which: Crushed rock (stone)	145, 578
Clay & shale for construction	798 (a)
Cement raw materials (limestone & chalk, common clay & shale) (GB)	15, 172 (a)
Clay & shale and Fireclay (for bricks) (GB)	7, 000
Gypsum, natural	1, 700
Slate	865

(a): 2005 data **(BGS, 2006)**

GB: Figures for GB only.

Table 2-3: Minerals Produced in the United Kingdom (UK) in 2006

There are three main types of natural aggregates; crushed stone, gravel, and sand. Crushed stone is mechanically crushed rocks. Most crushed rock is extracted from bedrock. Gravel is the product of the erosion of bedrock and surface materials (Table 2-2). Gravel can also be crushed, to increase its angularity and roughness, and that is recommended for its use in asphalt pavements. Sand can be either from abrasion of bedrock or be mechanically crushed, it is distinguished by its size, which according to British Standards (BS EN 13043, 2002) would be particles smaller than 4.75mm but larger than 75 micron.

Some early works to evaluate the main qualities required from road aggregates to be used in pavement surfacing materials have been documented by Hosking (1970), Hartley (1974) and Lees and Kennedy (1975). According to their works, they concluded that the main qualities of road aggregates utilized in pavement surfacing can be identified into six categories, they are:

1) Toughness – is the ability of aggregate to resists the slow crushing and the rapid loading.
2) Hardness – is the ability of aggregate to resists the abrasion or attrition.
3) Resistance to stripping – is the ability to maintain adhesion to any bituminous binder with which the aggregate is used.
4) Resistance to polishing – is the ability of aggregate for wearing course materials and surface dressing to resists the polishing by vehicles tyre.
5) Resistance to weathering effects in the pavement (e.g. resistance to frost action, swelling and softening by water).
6) Ability to contribute to strength and stiffness of total mix by intrinsic aggregate strength and shape properties.

All these road aggregates qualities requirements were commonly used by road engineers as a reference to assess the aggregate suitability for producing high quality bituminous mixtures. The following sections of the thesis will provide more information on the above road aggregates properties.

2.10 Road Aggregates Properties

2.10.1 Aggregate Crushing Strength

Crushing strength is the ability of the road-stone used in pavement surfacing to resist the crushing under a gradually applied compressive load by the vehicles (Hunter, 2000). The crushing strength of aggregates can be determined by using the Aggregate Crushing Value (ACV) test specified in BS 812: Part 110 (1990). This test is currently used by the road engineers and the researchers. There are many factors affecting the crushing strength of aggregates such as the porosity of aggregates. It was noted that the higher the density, of an aggregate, the higher the crushing strength, and the main factors affecting the toughness or crushing strength of aggregates had been explained by Hartley (1974). An example of various suggested minimum crushing strength value required from aggregate proposed for different usages has been recognized by Shergold (1948) and these propositions are summarized in Appendix A.

According to Appendix A, it demonstrated that the crushing strength requirements of an aggregate utilized in asphalt mixtures are mostly depending on the locations of pavement construction layer, the type of material composition and the traffic stresses. For example; the aggregate with (ACV) not greater than 13% was suitably applied in water-bound macadam (or unbound material) and to be utilized under heavy traffic, whilst at the other extreme, aggregate for use in the body of asphalt pavement road surfacing structure with light to medium traffic conditions might be permitted to have an ACV up to 40%.

2.10.2 Resistance to Abrasion and Polishing

In highway pavements, the opposing effects of traffic tending to increase the state of polishing of road stone aggregates, whereas, weathering, frost action, wetting and drying and temperature changes all tend to remove or reduce the polished state of road stone aggregates (Lees, 1978). The Department of Transport Specification for Road and Bridge works (DOT UK, 1976) specified that the minimum polished stone value, (PSV), and the maximum aggregate abrasion value, (AAV), of aggregates used for wearing course shall be as described in the contract.

The Department of Transport Technical Memorandum H16/76 (DOT UK, 1976) asked for specific requirements for aggregate properties and macrostructure depth for bituminous surfacing for new motorway and road construction. This Technical Memorandum H16/76 is shown in Appendix B. In addition to specifying minimum PSV's for aggregates to be used in such surfacing materials, the Technical Memorandum H16/76 states maximum Aggregate Abrasion Value, (AAV), requirements for different traffic loading calculated as that using the lane at a time equal to two-thirds of the anticipated life of the surfacing. These maximum AAV requirements are shown in Appendix C.

2.10.3 Resistance to Stripping

Adhesion failure is defined as a collapse of the bonding forces between an aggregate and its bituminous binder coating. The stripping is a phenomenon associated with the presence of water or moisture content. Hallberg (1950 and 1958) who observed that although, statistically, basic rocks have exposed better adhesion quality than acid rocks, he went to suggested that: "*The comparatively poor performance of acid rocks may be not only related to the high silica content but also to the formation of hydroxides of sodium, potassium and aluminum in the water medium adjacent to the mineral surface. Because of these molecules in the water would have the effect of lowering the interfacial tension and hence also the adhesion tension*".

The above research findings were supported by Hunter (1994) as he found that: most aggregates have a greater attraction for water than for bitumen due to bitumen's lower surface tension and thus, lesser wetting power. It is therefore difficult for bitumen to displace water, which has already created a film over the aggregate macrostructure Hunter (1994).

Adhesion between the aggregate surface and bitumen may be destroyed by water forcing its way under the bitumen shelling layer and stripping it free from the aggregates surface. The softer the bitumen the greater is the stripping risk and that is more apparent with acidic or high silica aggregates. Low silica aggregates such as limestone are thought to have a greater chemical attraction for the polar carboxylic acid components of bitumen.

2.10.4 Resistance of Pavement to Weathering Effects

Resistance to weathering is not an engineering property by itself but it can help in evaluating the reaction to weathering factors, which can lead to greater or less variation in the engineering properties of strength, resistance to polishing, abrasion and adhesion. The potential to resist weathering effects not only the action of aggregates but also the binder used. This is because of the consistency of binder will changes with temperature. During the warm weather, the binder will becomes softer, whereas in cold weather, the binder will becomes harder. These changes effect caused by weathering will lower the durability of pavement.

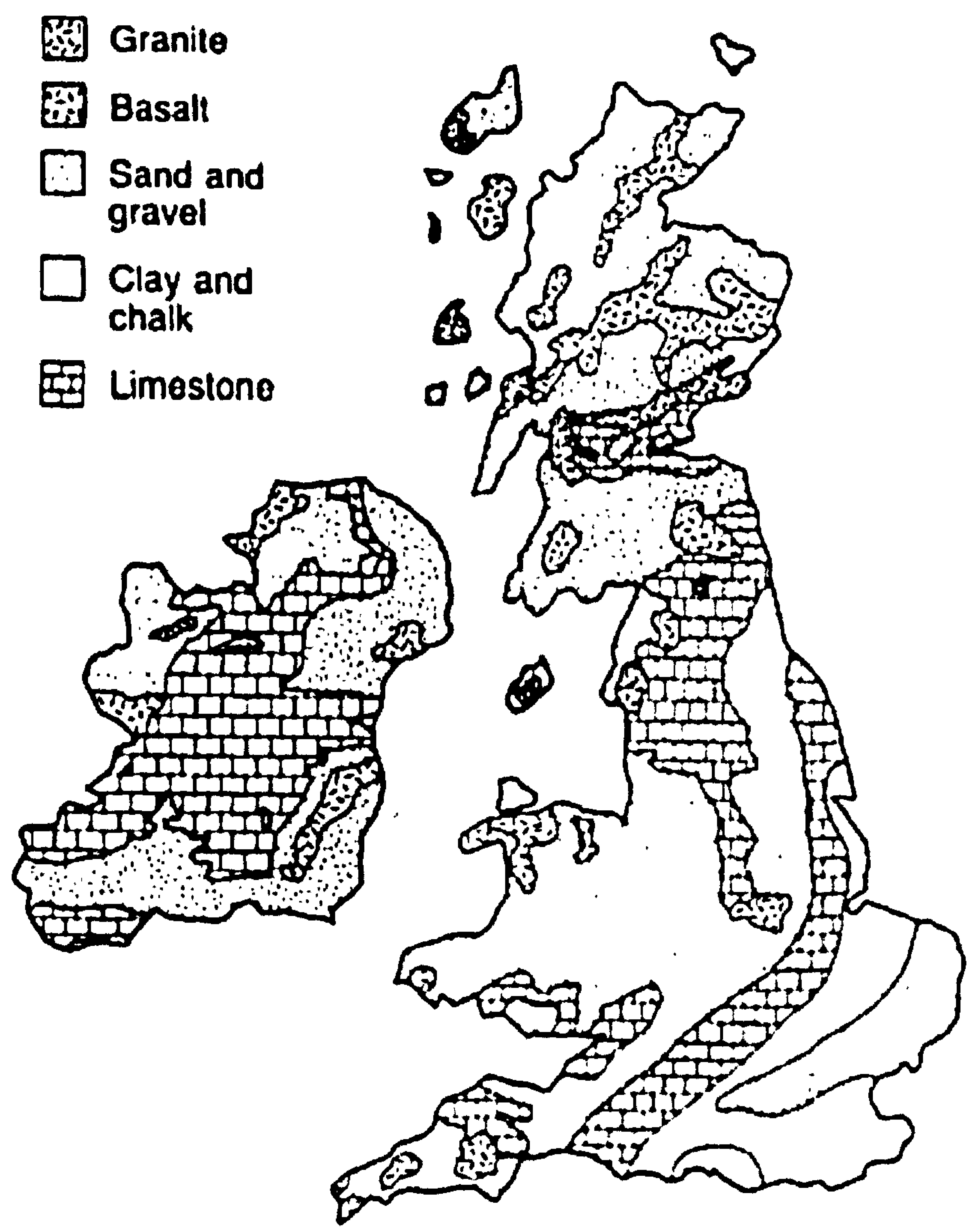


Fig. 2-10: Simplified geological map of the UK showing main distribution of Rock types used in the Pavement industry **(Hunter, 2000)**

The majority of road-stones that are used in the UK and other European countries for bituminous surfacing materials are adequately strong and long-lasting to experience little or no weathering during the life of the pavement material of which they form a part. Fig. 2-10 shows a basic geological map of the UK showing main distribution of Rock types used in the Pavement industry.

Normally, the weathering effects on pavement are such as by frost action and hot season. Some instances have been reported in the UK by Hosking and Tubey (1969). They reported that the aggregate type named (Dolerites and Teschenites) utilized in road surface pavement caused premature failure and that was expected by the result of deterioration in stockpiled during service life. As according to these results, they found that the premature failures of bituminous mixture are much less when the aggregate coatings are adequate with binder and the mix is designed to a low or zero permeability.

2.11 Aggregates gradation

Gradation refers to the particle size distribution of aggregates and generally described in terms of particle size distribution curve (also known as gradation curve or gradation chart), therefore, gradation must be a main concern in HMA mix design. As gradation of aggregate has a profound effects on all HMA properties and it is essential to perform a sieve analysis to determine the type of gradation such as dense graded, gap graded and open graded (Roberts et al, 1996).

When a washed sieve analysis is done to obtain a washed gradation, the aggregates are washed over a 75micron sieve to remove the dust or fine particles. The aggregate is then dried, weighed, and graded as a dry analysis. The amount of weight lost after washing is the passing 75micron material or dust. In designing asphalt mixtures, an accurate determination of the amount of material finer than 75micron has significance on the performance of the mixture. It is preferred to use the wet analysis method for determining the gradation of aggregates to be used for asphalt mixtures (Patrick, 2003).

(Mamlouk and Al Basyouny, 1999) evaluated the effect of aggregate gradation on the rutting potential of Superpave mixes. They prepared several mixtures with different aggregate gradations, performed creep test, and analyzed the results using the VESYS-3AM (a visco-elastic multilayer programme) to estimate the rut depth of the different mixtures. They concluded that both the aggregate gradation and aggregate nominal size affected the rut depth for specific pavement section as estimated by the VESYS-3AM software. Specifically they found out those mixtures prepared using aggregate gradation passing below the restricted zone (on the Superpave gradation chart) had better resistance to rutting as compared to those made from aggregates with gradation passing through or above the restricted zone.

However, as the comparisons were made based on rut depth predicted by model rather than measured rut depth, the study is subject to limitations of the model. (Cross et al, 1999) evaluated two mixtures made from aggregates with different gradations. The aggregate for first mixture had an S-shaped gradation that stayed below the maximum density line and passes below the restricted zone.

The second mixture was made from aggregate with finer gradation that stayed above the maximum density line and above the restricted zone. Other mixtures with aggregate gradations 5 to 20% coarser, as measured on the 4.75 mm sieve, than the fine and coarse mixtures were too prepared and tested. The mixtures were assessed for permanent deformation as well as other mechanical properties using Asphalt Pavement Analyzer (rut tester). They concluded that the finer gradation had better resistance to permanent deformation, which apparently was in contradiction with conclusions of Mamlouk and Al Basyouny (1999) in the previous paragraph. Figure 2-11 shows measured rut depth at 8000 cycles for the mixtures evaluated in the study by (Cross et al, 1999).

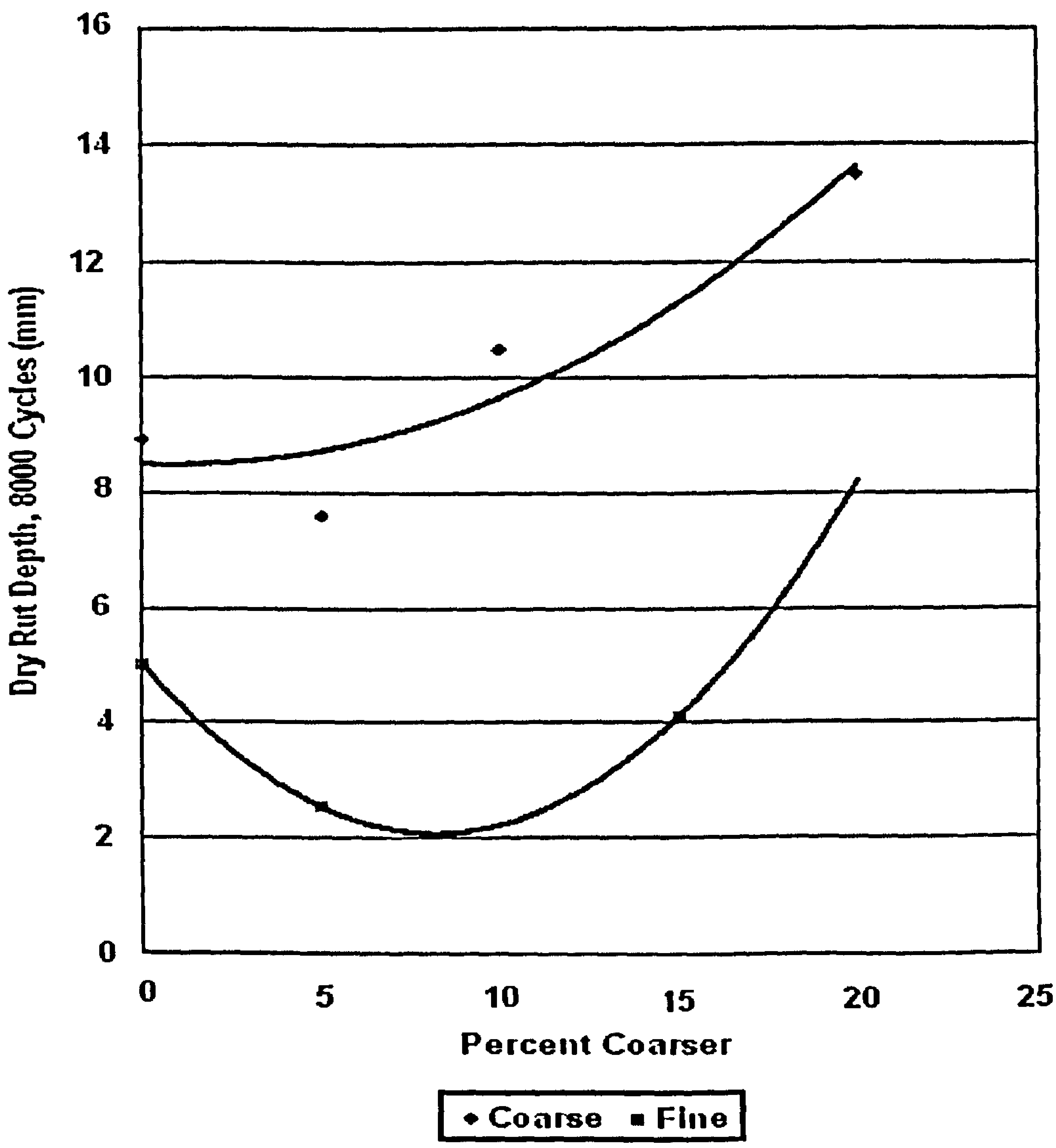


Fig. 2-11: Rut depth at 8000 cycles (Cross et al, 1999)

2.12 Asphalt

Around 6000 before Christ (B.C.), the Sumerians (IRAQ nowadays) had a thriving ship-building industry that produced and used asphalt for caulking and waterproofing. As early as 2600 B.C. asphalt was also used by the Egyptians as a waterproofing material and also to impregnate the wrappings of mummies as a preservative, and it was also used by other ancient civilizations as a mortar for building and paving blocks used in temples, reservoirs, irrigation systems and highways. The first recorded use of asphalt as a road building material was in Babylon around 625 B.C., and the asphalts used by early civilizations occurred naturally and were found in geologic strata as either soft, workable mortars or as hard, brittle black veins of rock formations.

Natural asphalts formed when crude petroleum oils worked their way up through cracks to the earth's surface. The action of the surrounding environment such as sun and wind expel the lighter oils and gases, leaving a black leftover. Natural asphalts were extensively used until the early 1900s. After the discovery of crude oil, and the increasing popularity of the automobile, served to greatly expand the asphalt industry. Modern petroleum asphalt has the same durable qualities as naturally occurring asphalt, with the added advantage of being refined to a uniform condition free from organic and mineral impurities. Most of the petroleum asphalt produced today is used for highway surfacing.

2.13 Cement Coating of Aggregates

Generally, aggregate quality has a predominant role in HMA mixes performance. The availability of good quality aggregates is of concern in many countries including the UK. Upgrading and developing the properties of the available, plentiful secondary aggregates will solve the problem of this type of aggregates of being secondary, and then it can be accepted by road engineers as a sound material. Previously upgrading of secondary aggregates had been studied but not to the extent of studying its effects on permanent deformation level. This project has concentrated on testing three types of OGFC mixes with two different asphalt binders in different laboratory testing techniques.

The cement coating of secondary aggregates addresses the macrostructure of aggregates particles and improves the bond between these particles and the asphalt binder; as it will roughen the aggregates surface and that is suppose to lead to increase the bonding between the aggregate particles and asphalt binder. The new cement coated aggregates can then be utilized in asphalt mixes by regarding it as new upgraded aggregates and usual mix procedure are applied to it. A thorough description of the cement coating method will be discussed in the next chapter, (Chapter three).

The two types of aggregates that were utilised in this research work are:

1. Arcow aggregates that were brought from "Arcow quarry" of Helwith Bridge, Horton-in-Ribblesdale in North Yorkshire. It belongs to a rock type named Silurian Greywacke (fine grained siltstone deposited about 420 to 440 million years ago).

2. Croxden aggregates that were brought from "Croxden quarry" of Croxden Common, Cheadle, Stoke-on-Trent, Staffordshire. It is comprised of major inclusions of quartz and quartzite together with lesser inclusions of sandstone, siltstone, granitic fragments and ironstone.

The chemical compositions of these two types of aggregates are shown in Table 2-4 below, as provided by Tarmac Ltd.

Substance (Chemical Symbol)	**Arcow**	**Croxden**
SiO_2	59.3%	91.70%
TiO_2	0.8%	0.15%
Al_2O_3	12.2%	2.41%
Fe_2O_3	6.7%	0.97%
MgO	2.3%	0.10%
CaO	5.1%	1.55%
Na_2O	0.8%	-
K_2O	4.4%	1.33%
SO_3	2.4%	-
LOI	5.8%	1.60%

Table 2-4: Chemical Compositions of Utilized Aggregates **(Tarmac limited 2008)**

2.14 Asphalt Absorption

Aggregate absorption is the quantity of water an aggregate will absorb when it is saturated in water. Aggregate, especially porous ones will also absorb the asphalt binder. Owing to the viscosity of the asphalt binder and the fact that nearly all aggregates are hydrophilic (water loving), the aggregate will absorb less asphalt binder than water. Absorption of asphalt by a mix can be calculated if an aggregate is very absorptive; it will continue to absorb the asphalt binder after the primary mixing at the mixing plant, hence leaving less of an asphalt binder film on its macrostructure.

The absorptive capability of an aggregate is usually accounted for in the laboratory mix design process. The amount of asphalt binder in a mix less than the asphalt binder absorption of the aggregate is identified as the effective asphalt binder content. If an aggregate absorbs too much of the asphalt binder, the mix will become less cohesive and difficult to be placed and compacted. The mix will also have a low asphalt film thickness on the aggregate surface, which can lead to mix stability problems. These aggregates will require higher asphalt binder content to provide a mix with a satisfactory performance.

Highly porous aggregates ought to contain other desirable qualities to overcome their excessive asphalt demand. Absorption is expressed as a percentage by mass of aggregate rather than as a percentage of total mixture. Absorption is determined by the equation (Patrick, 2003).

$$P_{ba} = 100 \times \frac{G_{se} - G_{sb}}{G_{sb} G_{se}} \times G_b \qquad (2.1)$$

Where:

P_{ba} is the absorbed asphalt (%),

G_{se} is the effective specific gravity of aggregates,

G_{sb} is the bulk specific gravity of aggregates and

G_b is the specific gravity of the binder.

CHAPTER 3

Materials Design

3.1 Cement Coating Design Concept

Increased aggregate angularity is well known to increase the resistance to permanent deformation in bituminous mixtures; this phenomenon is documented in references such as Brannan et al (1991). Generally road stone cement coating was not new, Schmidt (1973) showed that adding dry cement at any percentage of less than 10% of the total weight of crushed rock aggregates in the mix will produce an enhanced strength properties of the mix.

Daoud et al (1980a) had carried out experimentation using Portland cement for coating crushed local primary aggregates for the production of high-quality mixtures. Initial experiments were anticipated to improve the general mixture by enhancing the frictional component of stability and the aggregate-binder affinity and adhesion. The main aim was being to reduce excessive rutting. The aggregate was treated with about 11-15% by weight (of the total aggregate in the mix) by Portland cement with sufficient water to satisfy the requirements of aggregate absorption and cement hydration. The cement-aggregate mixture was allowed to hydrate for two days to form a coating of hydrated cement before it was crushed for separation before its use. Encouraging results led to the construction and monitoring of a 500m trial section on a major road. The performance of this trial section was judged to be very satisfactory, and the technique was subsequently introduced into the construction specification of the major road network.

A Kuwaiti government regulation document (M of PW, 1980), published at that time, compared, in the laboratory, the performance of cement coated aggregates, with the corresponding performance of two conventional mixtures, of the same gradation and type of parent aggregate. Results showed that, on the whole, cement coating of the parent aggregate had the effect of increasing the Marshall stability, the optimum binder content and the void content of the mixture. These are all desirable qualities in the production of modern mixtures, especially stone mastic and porous asphalt type mixtures. Compactability tests, (i.e. Marshall Test's results at 50 and 75 blows) also showed that cement treated mixtures contained 37% more voids after 75 blows than ordinary mixtures at 50 blows. This indicated that cement coated mixtures are capable of reducing permanent deformation, supported by Kandhal et al (1992).

Another advantage of cement coating on primary crushed aggregates highlighted in Daoud's research work (Daoud, 1980) was the ability of treated mixtures to resist the deleterious effects of water on the pavement performance.

In an immersion-compression test (AASHTO T165) which was conducted by the Ministry of Public Works in Kuwait (M of P W, 1980), where the alternative procedure in which the immersed specimens are kept in water for 24h at 60°C ± 1°C was adopted, the following statements summarised:

(i) Untreated specimens lost about 50% of their compressive strength after immersion, specimens that had 1 per cent hydrated lime and those treated with cement appeared completely resistant to water.

(ii) The large drop in compressive strength of untreated specimens occurred despite of their high fines content of 8 to 10% (mostly limestone dust), which should have improved the mechanical component of adhesion and thus the resistance to stripping by increasing the viscosity of the original binder in the mixture. This was thought to be because, unlike active fillers such as hydrated lime, limestone dust is inert and insoluble in water and does not therefore change the physicochemical mechanism at the aggregate-bitumen interface, which is known to influence greatly the adhesion and stripping of bitumen films.

(iii) The change of filler content from 8% to 10% appeared to cause no significant change in the index of retained strength in almost all cases.

(iv) The compressive strength, in the case of both dry and immersed specimens was seen to increase by an average of 20% for specimens with hydrated lime treatment and 13% for specimens with heavy cement treatment. Specimens that had light cement treatment yielded the greatest improvement in compressive strength. The improvement where found to be 52% and 34% for the dry and the immersed conditions respectively.

The above statements provide evidence to support the hypothesis that cement coating in the correct proportions can significantly increase the performance of a road material, in both wet and dry environments.

Work by Daoud et al (1980b) and Daoud (1980), at the Road Research Centre in Kuwait, indicated that the optimum coating, (in terms of uniformity of distribution of the added cement paste among different size fractions of the treated aggregate), was usually achieved at 11-15% of cement, by weight of the aggregate. This depended on a number of material and processing variables, (including aggregate type and gradation, water/cement ratio, etc.).

Later Guirguis et al (1982) studied the findings of Daoud et al (1980b) and Daoud (1980), and they concluded that using a range of 11-15% of cement in the coating process was not economically possible for many applications. They thus considered it necessary to determine the effect on bituminous mixture properties, by using lower percentage of cement added to the utilized primary expensive crushed rock aggregates. Three levels of cement coating were therefore considered and the resulting coatings were recognized as:

1. Light (5%),
2. Medium (6.5%) and
3. Heavy (8%)

They found, in summary, that the addition of cement as a coating had the effect of:

1. Increasing the stability,
2. Lowering the potential for binder bleeding,
3. Increasing the resistance to the effects of water and
4. The mixtures tended to remain stiff at higher temperatures and resist excessive brittleness at very low temperatures.

All the research work previously mentioned had applied cement coating on primary crushed rock aggregates. In this research work wet cement coating with additives will be used for upgrading waste natural aggregates, which are available in huge quantities in desert, sea and river beds for use in road pavements and to remove or ease the restriction implemented currently by road engineers on their use in bituminous mixtures.

Interestingly, no more work has been found in this research area other than the previously mentioned work conducted by Vaughan in the period of 1995-1998.

3.1.1 Description of the Cement Coating Concept

Natural aggregates represented by Croxden aggregates (Plate 3-1A & B) that has a rounded, smooth macrostructure was coated with a layer of cement paste; ordinary Portland cement, 2% of sand and 10% of lime mixed with water at a water/cement ratio of 0.4 as it was found to be the optimum. Dry cement was representing 10% of the aggregate dry weight. Aggregates sizes retained on 4mm sieve and bigger were coated with cement paste, as smaller particles were found to be difficult to coat as they were sticking together and making small agglomerations, and that seems to affect the overall mix gradation. In order to have a strong adhesion between the cement paste and the smooth surface of the aggregates sufficient hydration time should be allowed, the designated period for this was 28 days with continuous curing of the coated aggregates at room temperature of 20^{0}C.

Figure (3-1) shows a diagram of the effective asphalt content that is normally shielding aggregate particles within any asphalt mix, "Principles of Construction of Hot Mix Asphalt Pavements Manual Series No. 22" (Asphalt Institute, 1983), while Figure (3-2) shows a schematic drawing of the cement coating concept.

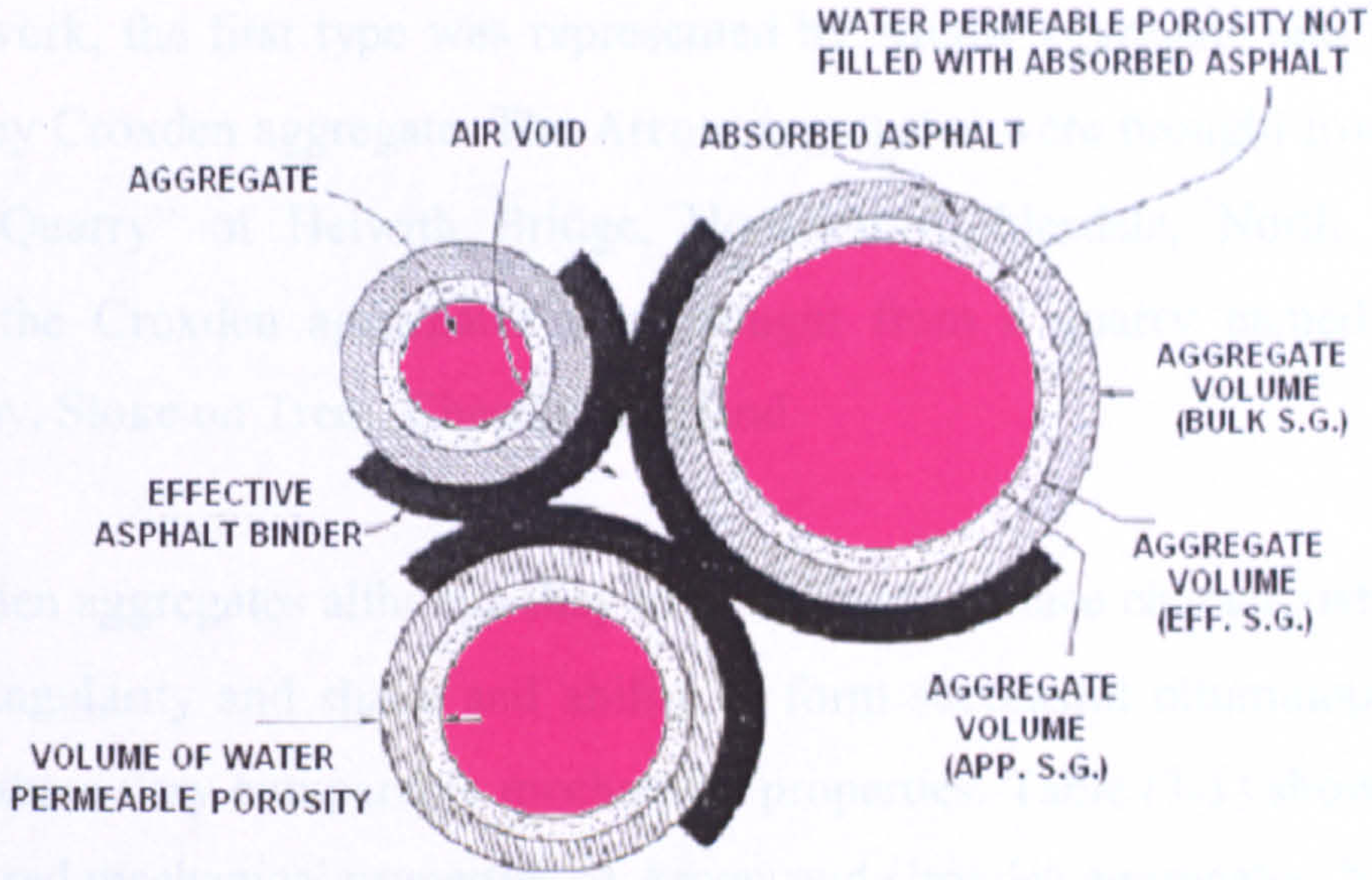


Fig. 3-1: effective Asphalt Content **(Asphalt Institute, 1983)**

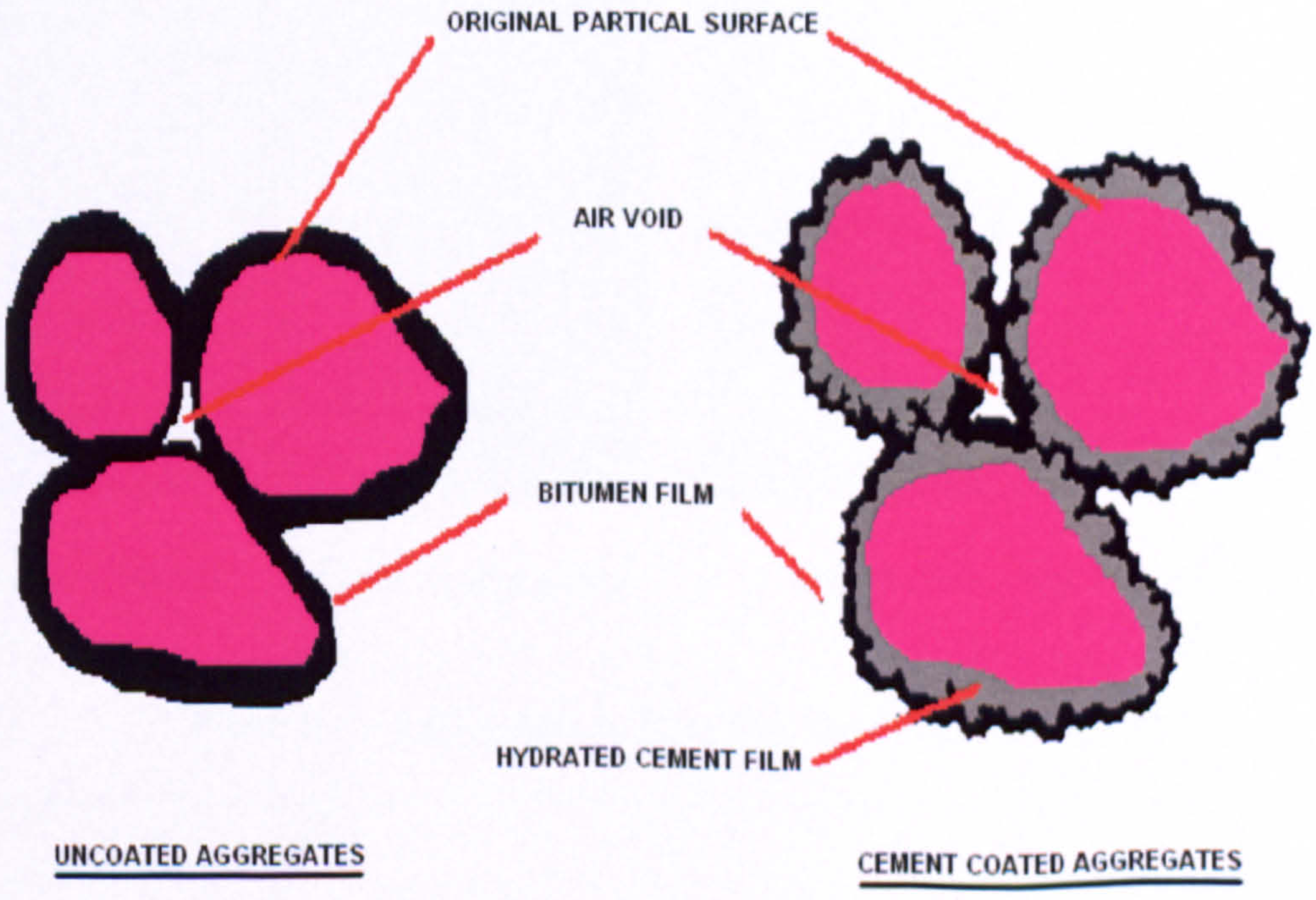


Fig. 3-2: Cement Coating Concept **(ABBAS A., 2009)**

Two types of aggregates representing prime and secondary quality aggregates were utilized in this research work, the first type was represented by Arcow aggregate and the second was represented by Croxden aggregate. The Arcow aggregates were brought from a quarry named "Arcow Quarry" of Helwith Bridge, Horton-in-Ribblesdale, North Yorkshire, England. While the Croxden aggregates was brought from a quarry named "Croxden quarry" of Freehay, Stoke on Trent, Cheadle, England.

Arcow and Croxden aggregates although they have different surface characteristics such as macrostructure, angularity and shape and ability to form successful bituminous mixtures (Plate 3-1), they share very comparable mechanical properties. Table (3-1) shows the most commonly measured mechanical properties of Arcow and Croxden aggregates, however the values of AIV soaked, TFV soaked and PSV for the Croxden type are not shown because these properties are not normally obtained for aggregates such as Croxden as it is normally used exclusively in concrete mixes, and not in bituminous mixtures.

A: Unwashed Croxden Aggregates

B: Washed Croxden Aggregates

C: Cement Coated Croxden Aggregates

D: Arcow rock crushed aggregates

Plate 3-1: Aggregate utilized in the research works

This is another main reason for choosing these two types of aggregates for this research work. After the addition of ordinary Portland cement paste, the effect of this cement coating on the asphalt mixes characteristics and the ability of the coating to produce a successful bituminous road material have been tested by performance tests such as Repeated Load Indirect Tensile test (RLIT), Repeated Load Axial test (RLAT) and index tests such as wheel tracking tests, as discussed in chapters five, six and seven of this thesis.

Property Tested	**Aggregate Type**	
	Croxden	**Arcow**
	Result	
Particle Density (OD) (kg/m^3)	2.61	2.72
Particle Density (SSD) (kg/m^3)	2.63	2.73
Water Adsorption (% by weight)	0.9	0.6
Aggregate Crushing Value (ACV)	11	10
Aggregate Impact Value (AIV) dry	15	10
Aggregate Impact Value (AIV) soaked	-	12
10% Fines Value (TFV) dry	380	390
10% Fines Value (TFV) soaked	-	330
Polished Stone Value (PSV)	-	63
Aggregate Abrasion Value (AAV)	1.2	6.1
Water Soluble Chloride (% Cl)	0	<0.01
Water Soluble Sulphate (g/l SO_3)	0.01	<0.1
Acid Soluble Sulphate (% SO_3)	0.01	0.04
Magnesium Sulphate Soundness Value	98	100

Table 3-1: Properties of Arcow and Croxden Aggregates **(Tarmac limited 2008)**

3.1.2 Choice of Cement Coating Type and Amount

There is a wide range of cement types available. These include:

- Ordinary Portland Cement (OPC),
- Rapid Hardening Portland Cement (RHPC),
- Low Heat Portland Cement (LHPC),
- Sulphate Resisting Portland Cement (SRPC), and
- High Alumina Cement (HAC).

OPC, RHPC and LHPC are all of the same composition; the variation in performance is reached by the relative fineness of grinding of each cement type, the finer the grinding the faster the cement sets. LHPC setting time is the slowest because of its course grinding, and because the chemical reaction of setting is slow, the amount of heat produced in the chemical reaction, termed heat of hydration, is also low. RHPC has the finest grinding and sets the fastest of the three cement kinds. OPC falls in the middle of these kinds. OPC appears to be the most appropriate cement type for the coating application because it is relatively cheap, available in large quantity controlled batches, and because of its previous use in bituminous mixtures, as an additive, will be in some way predictable in its behaviour. Previous work at LJMU had studied and designed the amount of cement coating required for coating the different types of aggregates under investigation. It had been determined by applying various percentages of cement paste, by weight of aggregate, to the aggregate until full coverage of the aggregate particles was achieved. The optimum w/c ratio was found to be 0.4. Table (3-2) below shows the related test findings (Al Nageim H. Robinson H., 2006, Vaughan, 1999).

Cement Paste by Weight of aggregate (%)	Coverage (%)
4	30
6	55
8	80
10	100

Table 3-2: Percentage of Coverage of Cement Paste to Aggregate Particles

Results for the Croxden gravel (Figure 3-3) clearly showed that the optimum w/c ratio was 0.4. The coating at 0.4 w/c ratio was also ranked highest by visual examination, being the most homogeneous, and consistent in cover thickness. 0.4 w/c ratio coating was then chosen for all Croxden aggregates coating.

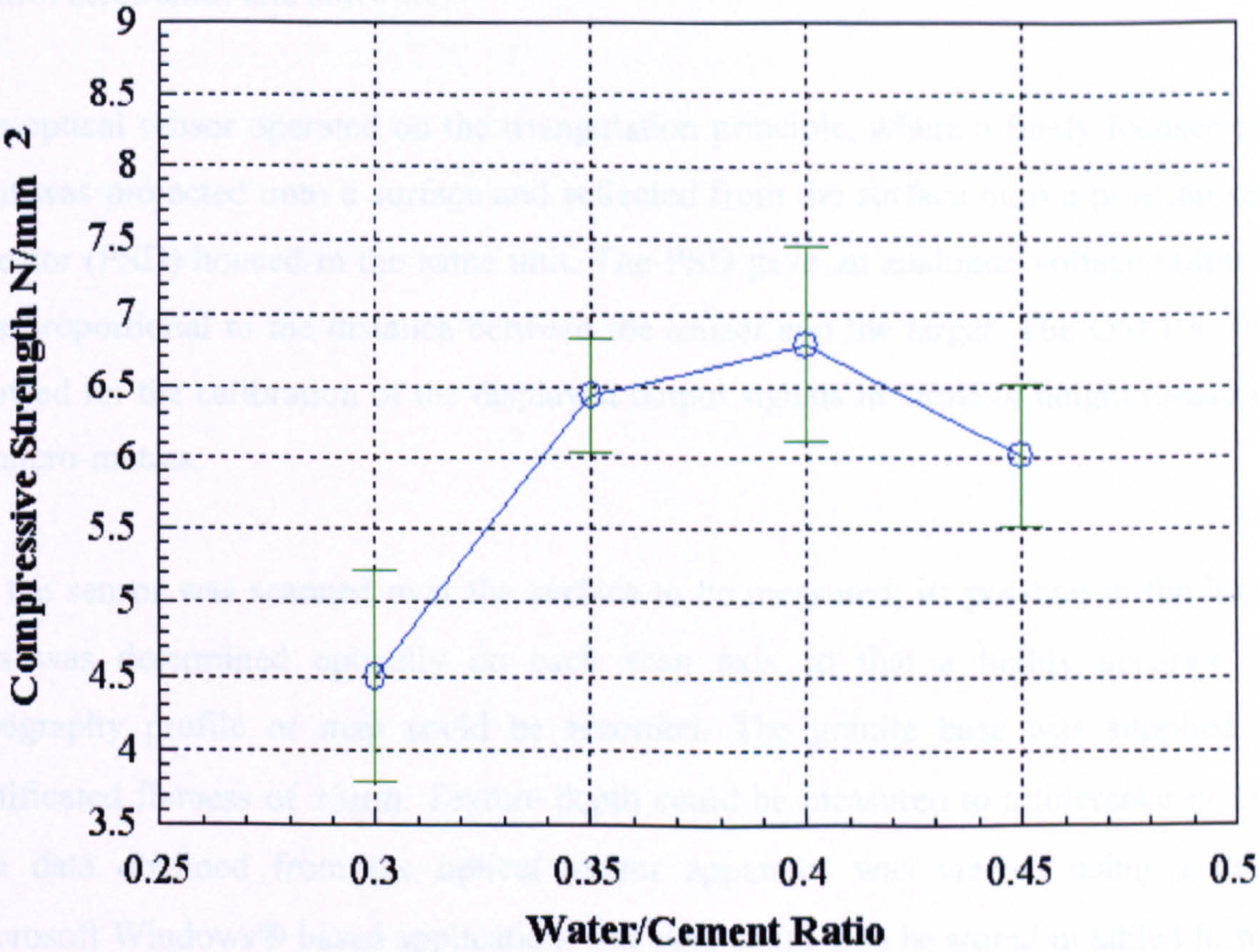


Fig. 3-3: Determination of Cement Paste W/C Ratio for Croxden aggregate

(Vaughan, 1999)

For the purpose of this research work only aggregate size fractions retained on a 4mm sieve and larger have been coated with cement paste, as the smaller sizes have proven to be difficult to be cement coated as they were agglomerating together and difficult to separate.

3.1.3 Surface Roughness Measurement of Utilized Aggregates

A surface roughness test was performed to indicate the relative texture depths of coated and uncoated aggregates. One of the mechanisms of improved internal shearing resistance of an aggregate is the surface texture improvement of an aggregate. This surface texture is often measured by a fractured face surface.

3.1.3.1 Surface Roughness Measurement Apparatus

The measurement apparatus comprised a Uni-scan Instruments® Optical Surface Profiling System, model OSP100, a high accuracy, and non-contact topography mapping system. The system consisted of a precision 3 dimensional scanner, granite bed, optical sensor, control electronics and software.

The optical sensor operated on the triangulation principle, where a finely focused point of light was projected onto a surface and reflected from the surface onto a position sensitive detector (PSD) housed in the same unit. The PSD gave an analogue voltage output which was proportional to the distance between the sensor and the target. The OSP100 software allowed for the calibration of the displayed output signals in terms of height measurements in micro-metres.

As the sensor was scanned over the surface to be measured, its position in the horizontal axis was determined optically on each scan axis so that a highly accurate surface topography profile or map could be recorded. The granite base was supplied with a certificated flatness of ±3μm. Texture depth could be measured to a tolerance of ±0.5μm. The data obtained from the optical sensor apparatus was viewed using a dedicated Microsoft Windows® based application. The data could then be stored in tabled form, or as a full colour, high resolution contour map, and could be exported in a variety of formats to other software applications.

3.1.3.2 Surface Roughness Measurement Method

Several pieces of aggregate were selected to represent each type being tested. The aggregate was obtained from the size fraction retained on a 14mm sieve. This larger size was used as this was easy to manipulate and hold in place in the scanning machine. The aggregate was positioned so that a flat face was presented to the scanning apparatus. The flat face was used so a map of the surface texture was produced without undue influence from the general curvature of the aggregate. The OSP100 always scans with an aspect ratio of three parts in width to four parts in length. The area chosen for this study was the area considered the largest possible without including the effects of aggregate curvature. The area chosen, therefore, was 7.5mm x 10mm.

Plates (3-2 to 3-10) below show some representative magnified photos of the aggregates under consideration. Comparing uncoated Croxden aggregates' Plates (3-2, 3-3 and 3-4) with coated Croxden, Plates (3-8, 3-9 and 3-10), it can be noticed that adding cement paste has increased the texture size, not just in height, where from Table (3-4) an improvement of 89.42% has been achieved but also in the horizontal scale as well. The Croxden aggregate, when coated, adopts a surface appearance similar to that of Arcow and has a texture depth better than Arcow aggregates, see Plates (3-5, 3-6 and 3-7) below.

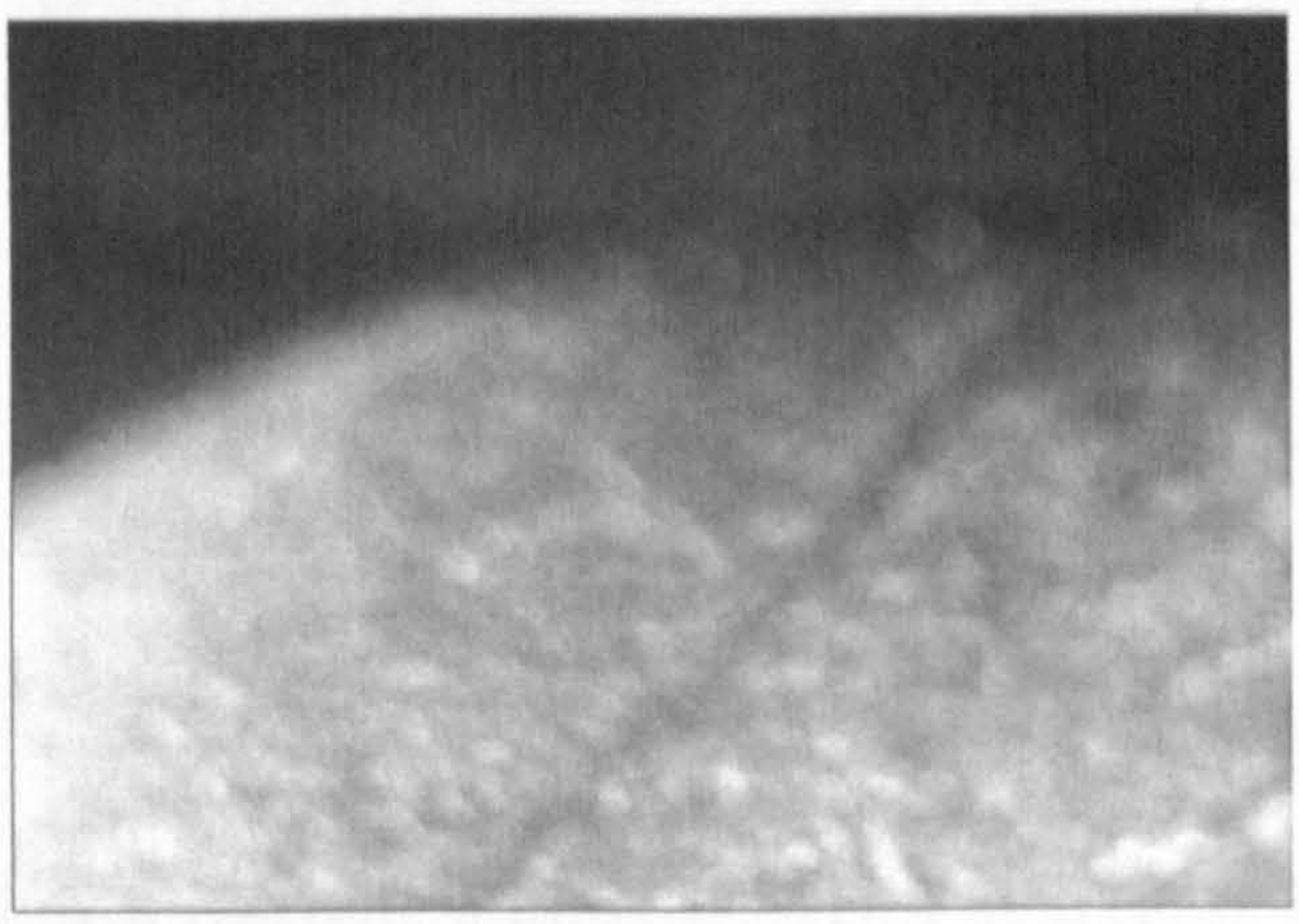


Plate 3-2: Microscopic photos of Uncoated Croxden aggregate 0.7X image

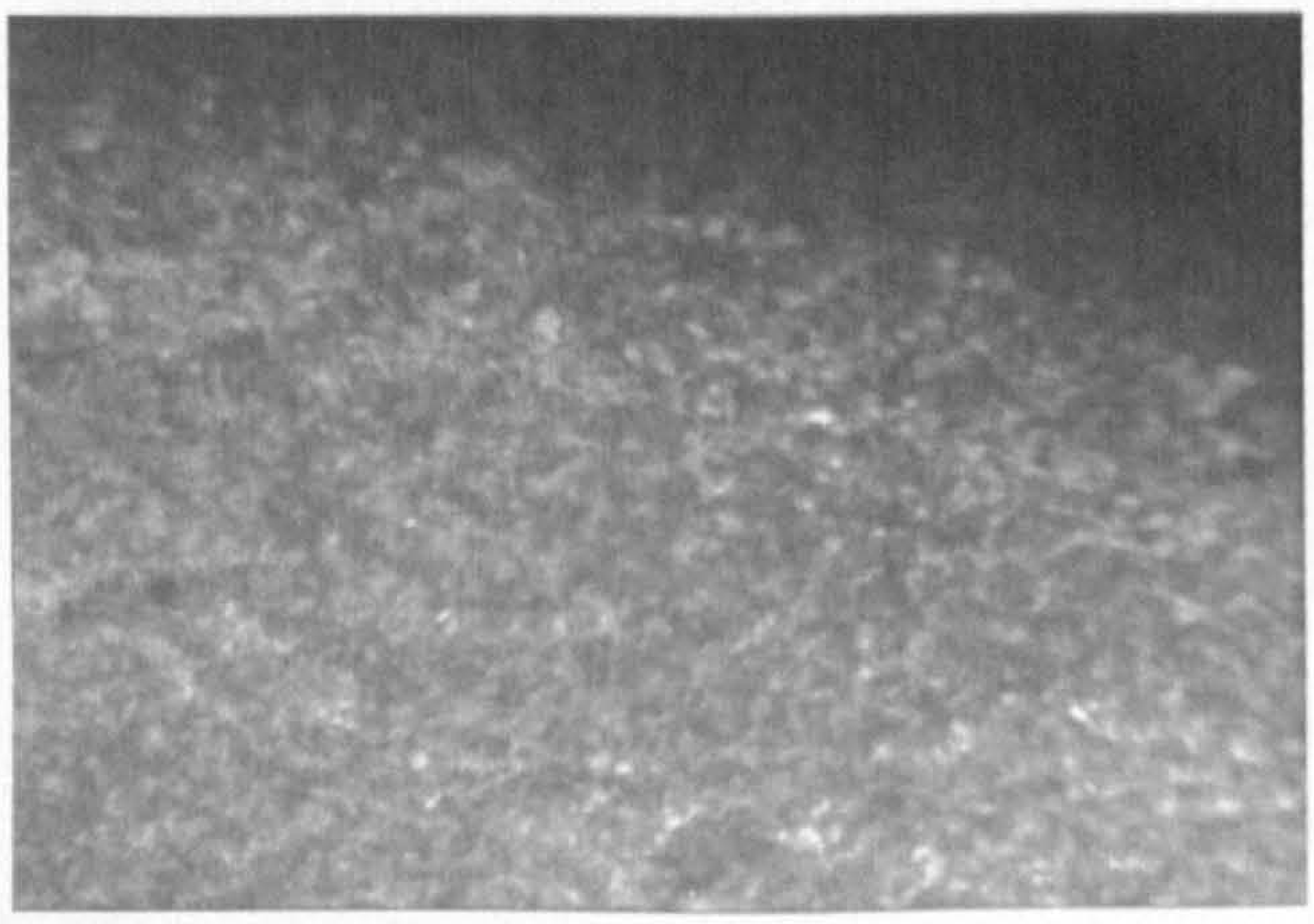


Plate 3-3: Microscopic photos of Uncoated Croxden aggregate 0.7X image

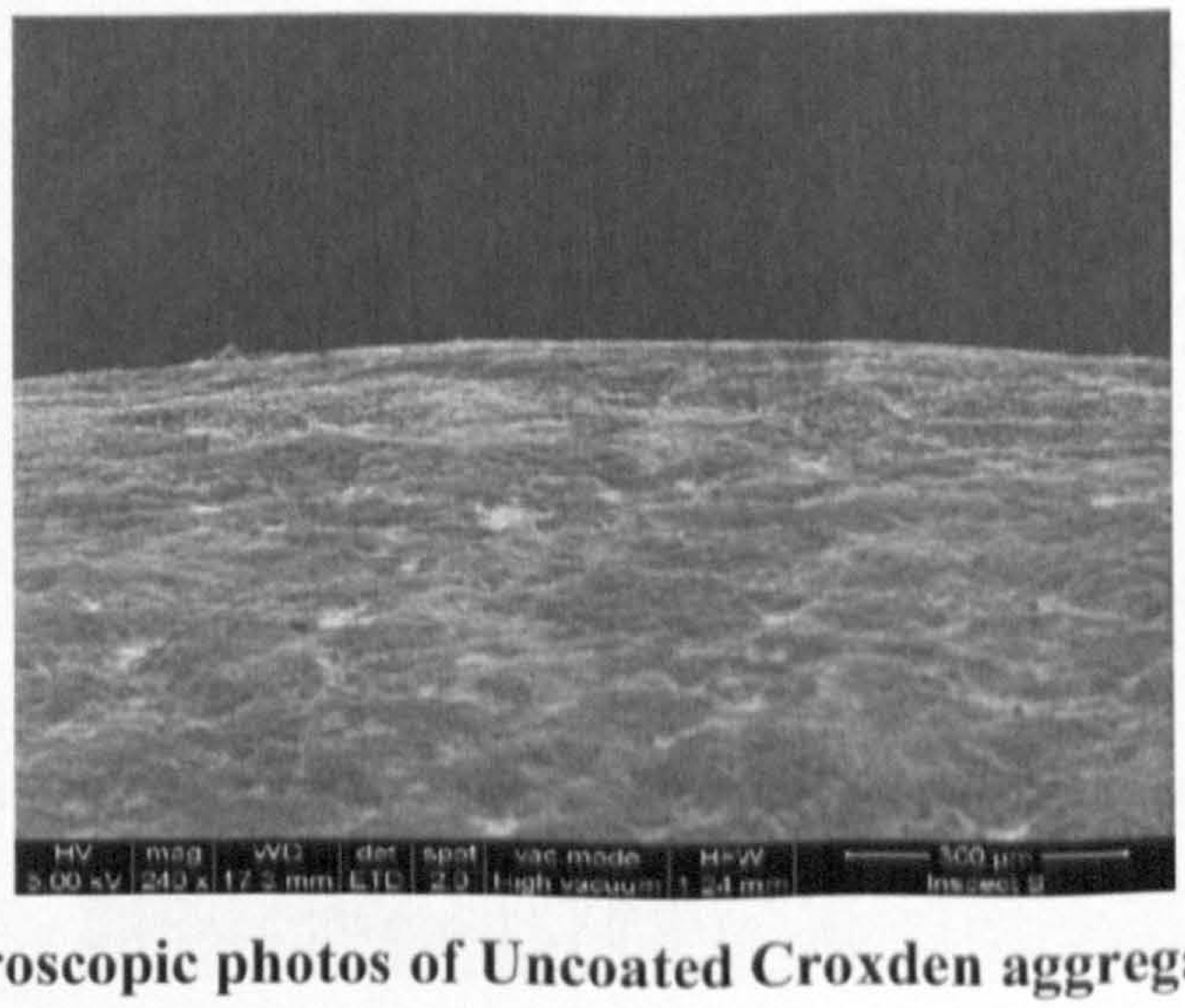


Plate 3-4: Microscopic photos of Uncoated Croxden aggregate 240X image

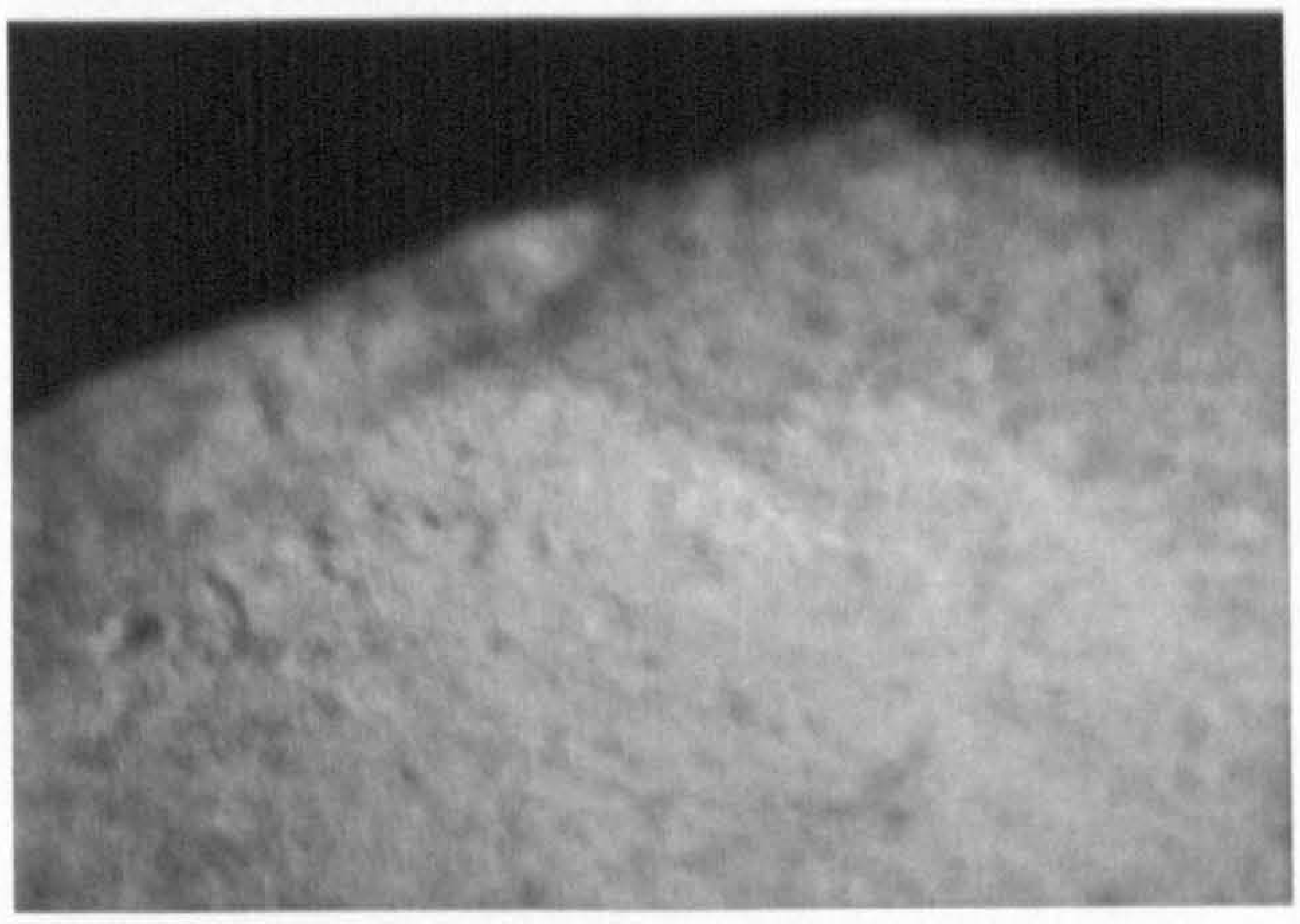


Plate 3-5: Microscopic photos of Arcow aggregate 0.7X image

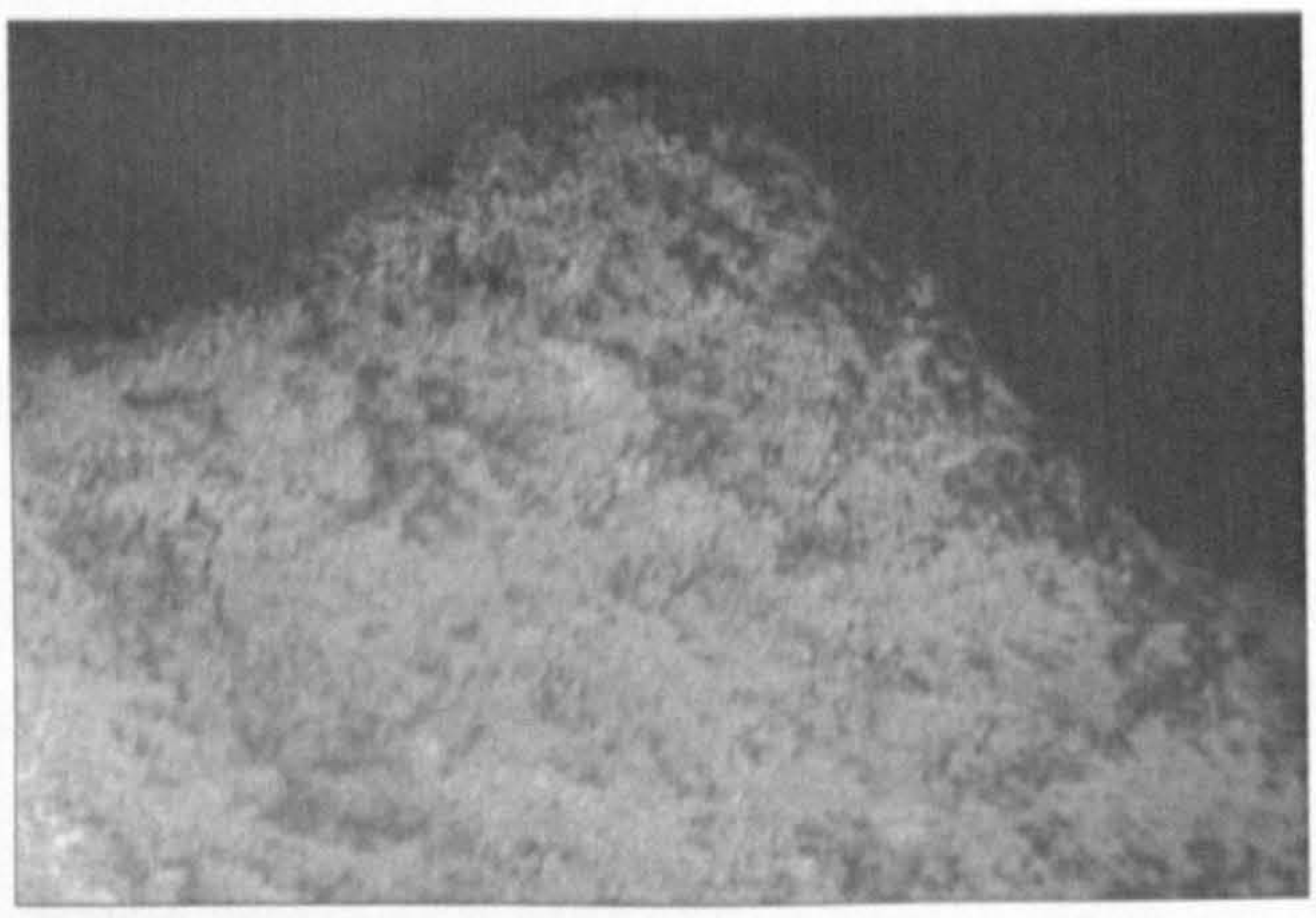


Plate 3-6: Microscopic photos of Arcow aggregate 0.7X image

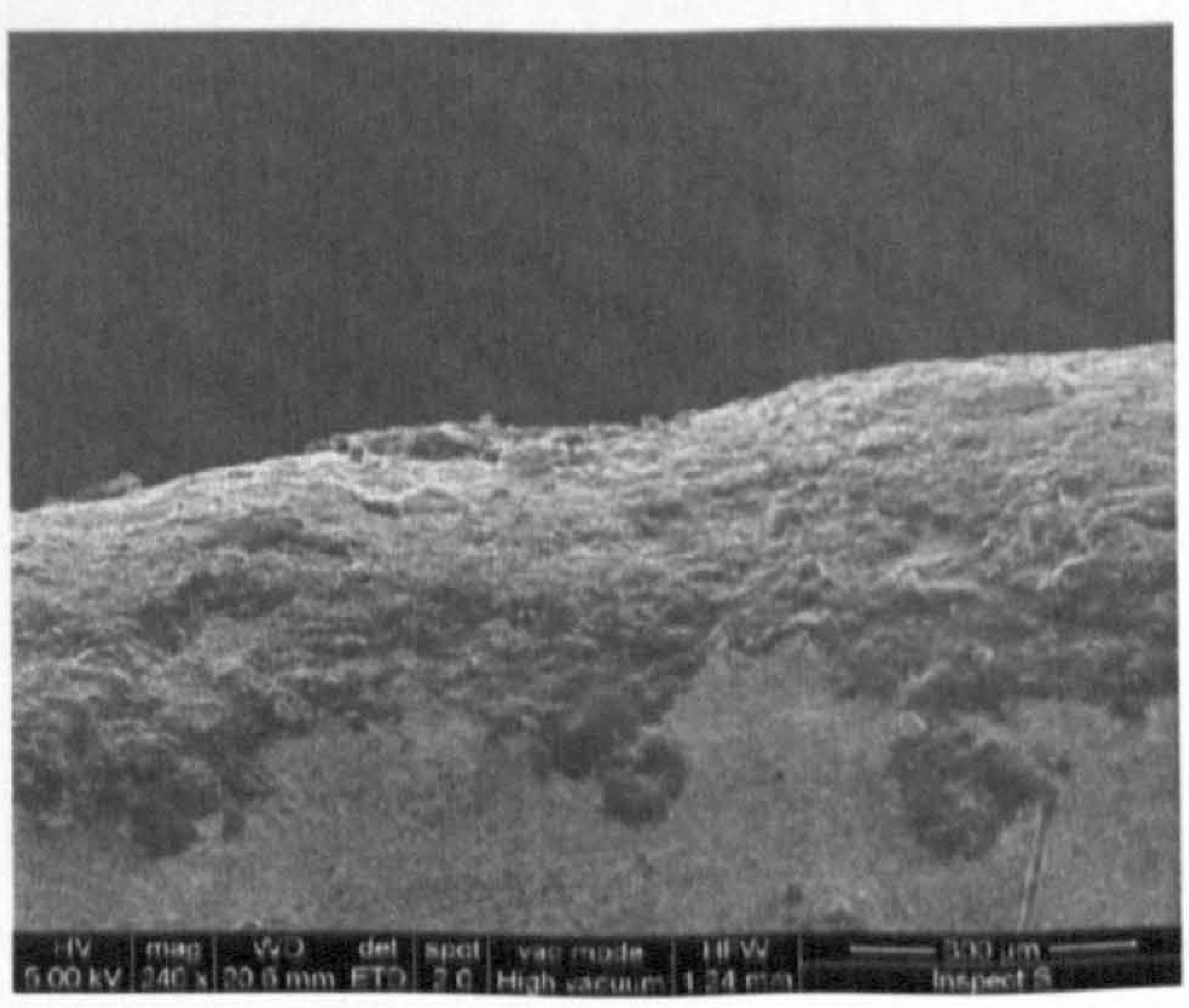


Plate 3-7: Microscopic photos of Arcow aggregate 240X image

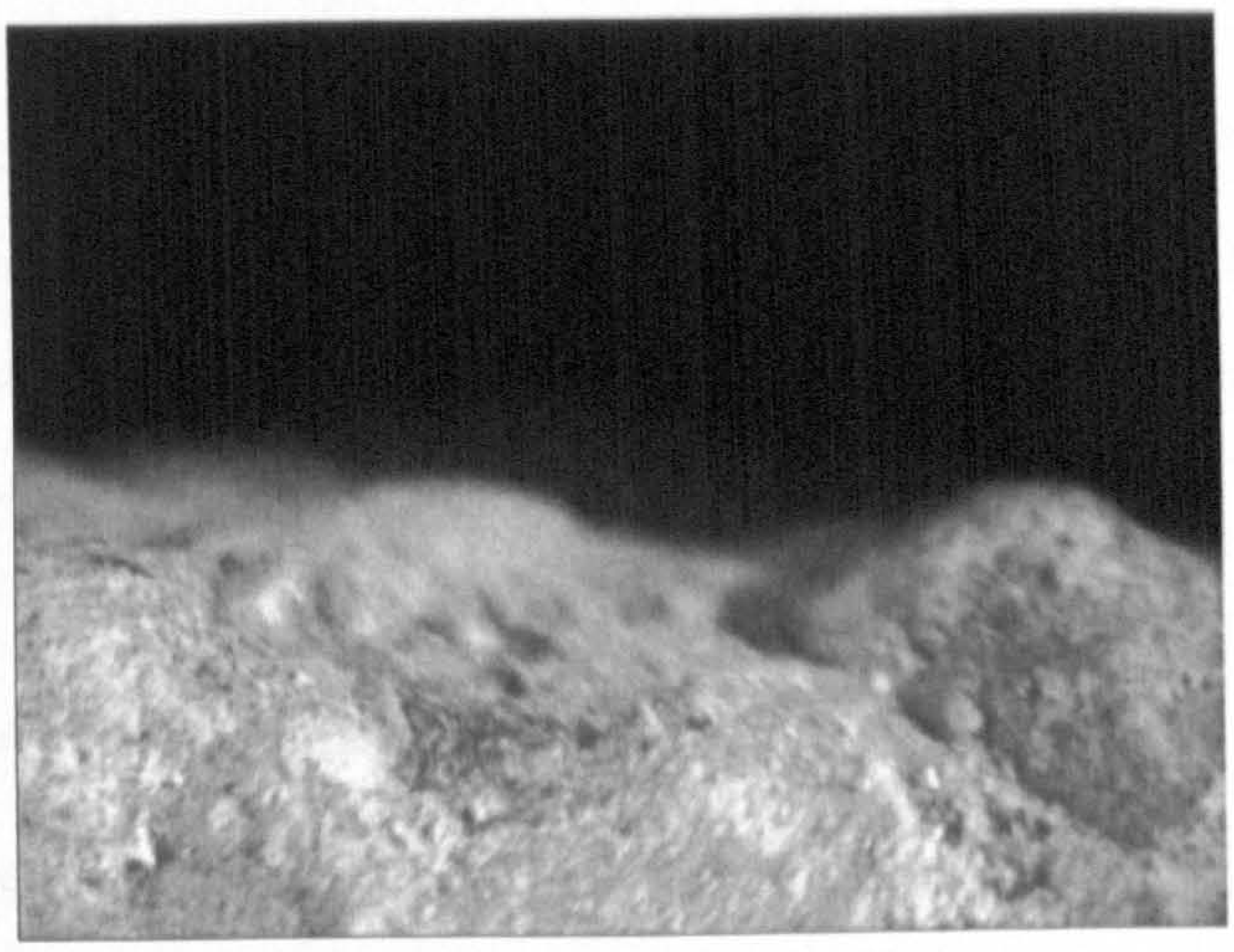


Plate 3-8: Microscopic photos of Cement Coated aggregate 0.7X image

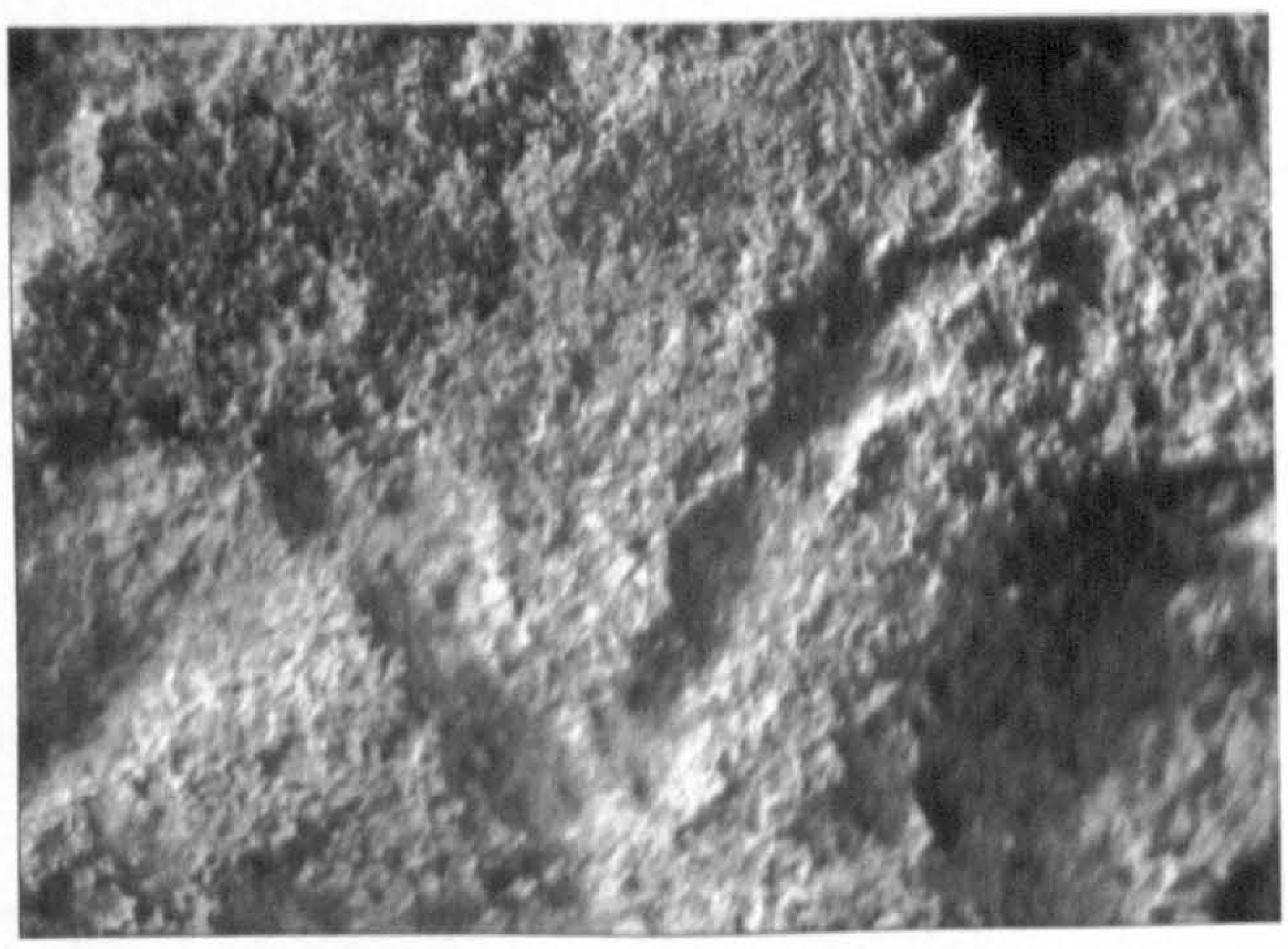


Plate 3-9: Microscopic photos of Cement Coated aggregate 0.7X image

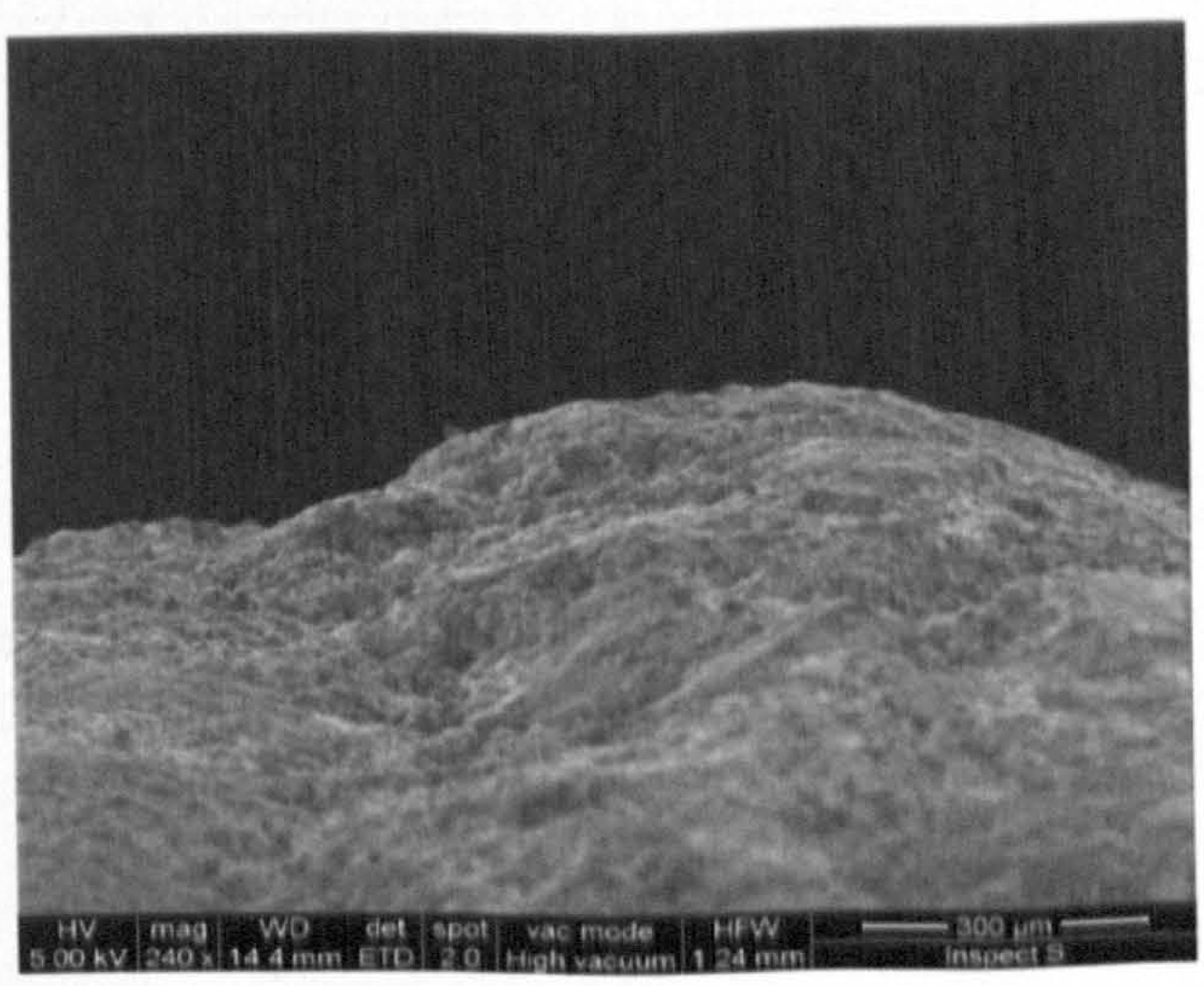


Plate 3-10: Microscopic photos of Cement Coated aggregate 240X image

This work has been repeated several times and therefore it can be concluded, based on microscopic photos of the different aggregates, that coating an aggregate with cement paste can enhance the poor surface qualities and roughness properties to provide a surface texture quality similar to that provided by an aggregate such as Arcow, which is known for its high performance and uses in road bituminous mixtures.

3.1.3.3 Surface Roughness Measurement Results

Table (3-3) shows results obtained by Vaughan (1999), he found that with the addition of cement coating, Croxden achieved an increase in average surface texture depth of 63.85%. Coated Croxden also demonstrated an improvement, in texture depth, of 26.53% over Arcow.

Aggregate Type (20mm in size)	Maximum Microtexture Depth (μm)	Average Microtexture Depth (μm)	Improvement In Surface Roughness (%)
Croxden Sample:			
1	85.74	85.95	63.85
2	82.74		
3	89.38		
Coated Croxden Sample:			
1	124.75	140.83	
2	168.78		26.53
3	128.97		
Arcow Sample:			
1	112.11	111.30	
2	117.05		
3	104.75		

Table 3-3: Microtexture Depth of 20mm Aggregates **(Vaughan, 1999)**

This work has been repeated several times and therefore it can be concluded, based on microscopic photos of the different aggregates, that coating an aggregate with cement paste can enhance the poor surface qualities and roughness properties to provide a surface texture quality similar to that provided by an aggregate such as Arcow, which is known for its high performance and uses in road bituminous mixtures.

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Croxden Sample:			
1	85.74		
2	82.74	85.95	
3	89.38		63.85
Coated Croxden Sample:			
1	124.75		
2	168.78	140.83	
3	128.97		
Arcow Sample:			26.53
1	112.11		
2	117.05	111.30	
3	104.75		

Table 3-3: Microtexture Depth of 20mm Aggregates **(Vaughan, 1999)**

This conclusion has been enforced in this research via the study conducted on aggregates samples for the three types of aggregates, using the same adopted technique, but with a nominal size of 14mm instead of 20mm, the results are shown in Table (3-4).

Aggregate Type (14mm in size)	Maximum Microtexture Depth (μm)	Average Microtexture Depth (μm)	Improvement In Surface Roughness (%)
Croxden Sample:			
1	77.01	79.97	89.42
2	84.52		
3	78.39		
Coated Croxden Sample:			
1	144.28	151.48	
2	153.22		
3	156.94		32.85
Arcow Sample:			
1	127.56	118.13	
2	115.31		
3	111.52		

Table 3-4: Microtexture Depth of tested 14mm Aggregates

Improvement between Aggregate Types	**Improvement achieved (%)**
Croxden – Cement Coated Croxden	40.05
Cement Coated Croxden - Arcow	23.82

Table 3-5: Comparison in aggregate's depth texture between Vaughan's study and current study

The improvement in the aggregate's surface roughness that were achieved in this study, more than that obtained by the previous study of Vaughan (1999), was about 40% in comparing the surface textures of the Croxden and that of the cement coated Croxden aggregates, while a percentage of about 24 was obtained by the comparison between cement coated Croxden surface texture with that of Arcow (Table 3-5). The improvement was due to the introduction of 2% (by weight of cement) of sand and 10% (by weight of cement) of dry lime to the cement coating used by Vaughan in his previous study, where he had used pure cement to add to the coarse aggregates.

3.1.3.4 Shearing Resistance

Shearing Resistance of asphalt mixes directly influence the types and severity levels of distresses detected in hot mixed asphalt pavement layers. Rutting is one type of stress that is a function of the shearing resistance of the material. The shearing resistance of a HMA is a function of the inter-particle cohesion and friction in addition to the amount of stress applied to the material. The cohesiveness of the mix depends on the amount of asphalt added to the mix, the degree of asphalt coating absorbed by the aggregate's particles, and by the properties of the asphalt binder itself. The properties of the asphalt binder affecting cohesion vary with temperature and age. The cohesiveness of the mixture can also be affected by its attraction (or lack of it) to the aggregates used in the mix and by the micro and macro structure (texture) and porosity of the aggregates. The internal friction which develops in an asphalt- aggregate mixture is a function of:

1. the applied stress,
2. the shape and texture of the aggregate,
3. the particle size distribution (gradation),
4. the shearing resistance of the aggregate itself, and
5. The properties and amount of asphalt used in the mix.

Considering the number of factors involved that can affect the shearing resistance of asphalt, it is obvious that a single shear strength test or value can not adequately charecterise an asphalt-aggregate mixture.

Aggregate Type	Angle of Internal Friction (ϕ) (Degrees)	
	Vaughan's (1999)	This research work
Uncoated Croxden	44.68	43.82
Coated Croxden	48.94	49.0 1
Arcow	48.15	47.94

Table 3-6: Angle of Internal Shearing Resistance of Coated and Uncoated Aggregates

Vaughan (1999) research work Results (Table 3-6) for angle of internal friction show that cement coating had improved the angle of internal shearing resistance of Croxden by 9.53%. Coated Croxden had achieved an angle of internal shearing resistance 1.64% better than Arcow. Similar improvement of 11.84% in the angle of internal friction has been achieved using the modified cement coating in this research work which is 2.23 % better than Arcow.

It is noticeably concluded that coating an aggregate with cement paste has improved the shearing resistance of the coated aggregate. This led to the conclusion that coated aggregate should lead to improvement in the overall strength of a bituminous mixture. In relation to this aspect; Al Nageim and Robinson (2006) has stated that: "*The internal shearing resistance of an aggregate increases, as the surface texture - increasing frictional resistance to sliding, and overall angularity - increasing interlocking, are improved, and with an increase in the angularity of the constituent aggregate, the shearing resistance of the resulting bituminous mixture increases also. This indicates that increasing the texture and angularity of an aggregate - increasing its shearing resistance, will increase the strength of a mixture produced from it*". This confirms that an increase in aggregate angularity produces a steepening in the bituminous mixture's angle of internal friction. This in turn produces a mixture with an increased magnitude of compressive strength or Stability.

3.2 Mixture Design

3.2.1 Introduction

In order to determine the material compositions for the three selected mixes, a Marshall Mix design was carried out in accordance with (BS PD 6691, 2007) specifications for the (AC 14 open surf) asphalt mixtures were implemented. The control mix was composed of crushed Arcow aggregates and the other two mixes were made with Croxden coated and uncoated aggregates respectively. Two types of binders were utilized 100/150 and 40/60 pen bitumen. The design criteria for the three mixes were purely volumetric (air voids content). The specimens density were obtained by means of the saturated surface dry technique for 100 x 63.5mm specimens and through direct measurement of specimen dimensions for 305 x 305 x 50mm and 500 x 180 x 50mm specimens.

Prior to the composition of the asphalt mixtures, the sieved aggregate fractions were first washed in water to remove the dust. The purpose of removing the dust was to have a better control of the composition of the mixture. After washing, the aggregates were dried either in the oven or by lying it down on Polythene sheets and then collected in large containers ready for specimen preparation. Although the obtained aggregates were supposedly virgin, traces of coating dust was found in the obtained aggregates. The weight of the aggregate fractions were measured and then heated in the oven together with the molds, base plate, mixing arm and the mixing pan for 8 hours. The aggregates for the three mixtures were heated at a temperature of 10^0C above the bitumen Equiviscous Temperature (EVT). The heated aggregates were then mixed together with bitumen heated at EVT temperature as well. After mixing for 1.5 minutes using the Hobart mixer, the mixture was compacted by a preheated 4.54 kg Marshall Hammer. During the compaction process 2 x 50 blows were applied on the specimen with each side receiving 50 blows. The aim of the mixture design was to obtain an (AC 14 open surf 100/150 and AC14 40/60 open surf) mixes containing an air voids content of more than 20% as specified by the BS EN standards (BS EN 12697-8, 2003).

Property tested	Procedure	Applied Specification
Binder Run-Off	Binder Drainage Test	BS DD 232: 2005
Voids Contents	Volumetric measurement	BS EN 12697-8:2003
Bulk density	before compaction	BS 812 - 2 : 1995
Bulk density	after compaction	BS EN 12697 6:2003+A1:2007
TMD	(Loose sample)	EN 12697-5: 2002+A1:2007 (E) (Procedure A)
Elastic Stiffness	RLIT	BS EN 12697-26:2004
Deformation	RLAT	BS DD 226 -1996
Deformation	Wheel Tracking	BS EN 12697-22:2003

Table 3-7: Current Utilized Mix Design Specifications

3.2.2 Mix Design Procedures

The preparation of test samples requires upmost care and patience to ensure identical samples' manufacturing process. Each step in the preparation process is crucial in determining the response of the asphalt mixture specimens. Table (3-7) shows this search laboratory experimental work utilized Standards. The British and European Standards, utilized in this research work, have laid down detailed guide lines and testing procedures in order to standardize the testing procedure and the methods of preparation. Broadly, the preparation of the sample can be classified into the following:

1 Sieving aggregates into different aggregate size fractions
2 Mix design
3 Prepare batch mixes
4 Preheat the batch mixes and asphalt
5 Batch mixing of asphalt and aggregate
6 Oven heating
7 Compaction
8 Air void determination

3.2.3 Aggregate Sieving

Sieving (Gradation) is one of the more important properties of aggregate mixture used in asphalt mixes' manufacturing. It affects both the stability and the durability of the HMA mixes. Therefore, gradation is a primary consideration in asphalt mix design. Gradation is generally determined by sieve analysis that involves passing the aggregates through a number of sieves, piled on top of each other, with bigger openings sieve is at the top and sieve with smallest opening is at the bottom, and then weighing the aggregates retained on each sieve. The gradation is normally expressed as total percent passing various sieve sizes. The aggregates used in road construction are largely obtained from local suppliers of natural rock. This aggregate obtained from the quarry will range from very fine soil to large sized rocks. The properties of the final test sample will largely be determined by the overall size of the aggregates and the relative percentages of various sizes of aggregates in the sample.

The aggregates used in this research work were Arcow and Croxden aggregate types. The main reason for this selection was due to their surface textures which results in different potentials and bond strengths between aggregates and asphalt. Arcow aggregates have a rough surface texture which results in good bonding and low stripping potential, while Croxden aggregates have a smooth and polished surface texture resulting in poor bonding and high stripping potential (Plate 3-1 above). The main chemical composition properties of the two types of aggregates are listed in Table (2-4) in Chapter two above. Interestingly, Croxden has a high content of SiO_2, 91.7%, compared with Arcow which has only 59.3 %.

Therefore it makes them oil hating and oil loving aggregates respectively (Al Nageim and Robinson, 2006; Tarmac, 2008).

The aggregates have been separated into different sized fractions through sieving. The sieves used were basic sieve set plus set 2, conforming to BS EN 13043 (2002) and Detailed in BS PD 6682-2 (2003).

The gradation for such OGFC mixes was selected in accordance with the specification limits by BS EN 13108-1 (2006) and BS PD6691 (2007) as shown in Table (3-8).

Grading	% Passing						
	20 mm	14 mm	10 mm	6.3 mm	2 mm	0.063 mm	Pan
14 mm	100	98-100	62-68	32-38	16-17	Up to 5	0

Table 3-8: Aggregates Gradation for Open Graded Friction Coarse Mixes According to British Standards PD 6691:2007

The gradation shown in Table (3-9) is that of the research work after washing the aggregates twice, to eliminate the attached dust and dirt. The sieve analysis procedure was conducted according to BS EN 1015-1 (1999) and the grading was expressed in percentages by mass of total aggregate.

Grading	% Passing						
	20 mm	14 mm	10 mm	6.3 mm	2 mm	0.063 mm	Pan
14 mm	100	98	68	36	11	2	0

Table 3-9: Aggregates Gradation Implemented by the Research for the Three Types of OGFC Mixes

Table (3-10) below shows the selected gradation but represented by Fuller's method of raising the sieve size to 0.45 (Asphalt Institute, 1983).

Sieve Size (mm)	$(Size)^{0.45}$	Specification Limits (%)		% Passing	Cumulative Retained	% Retained
		Lower	Upper			
20	**3.850**	100	100	100	0	0
14	**3.279**	98	100	98	2	2
10	**2.818**	62	68	68	32	30
6.3	**2.289**	32	38	36	64	32
2	**1.366**	16	17	11	89	25
0.063	**0.288**	0	5	2	98	9
Filler	-	-	-	-	100	2
						$\sum$=100 %

Table 3-10: Selected OGFC Gradation represented by Fuller's method

3.3 Asphalt binders utilized in the research work

Generally asphalt is extracted by the distillation of crude petroleum using different refining techniques. At an ambient temperature asphalt is a semi-solid material that should be heated in order to mix it with aggregates. Asphalt is a strong, durable binder with excellent adhesion ability and waterproofing characteristics. In this research work and in consultation with Tarmac and Nynas Bitumen, two types of asphalt binders were utilized and they were provided by NYNAS ltd.

The first type is the 100/150 penetration grade (pen) that is representing a soft binder and the second type is the 40 /60 pen that represents a hard binder. They therefore cover the two extremes of asphaltic materials used in the pavement industry, and thus using them in the testing of cement coating will provide the necessary properties for the new mix containing coated aggregates and both the two extremes of the soft and hard binders.

The crude oil for these two asphalt binders came from Venezuela (NYNAS, 2008). The specifications of these two asphalt binders according to BS EN 12591 (2000) are shown in Table (3-11) below.

PAVING GRADES	TEST METHOD	100/150 pen	40/60 pen
Penetration at 25^{0}C	EN 1426	100-150	40-60
Softening point ^{0}C	EN 1427	39-47	48-56
Kinetic viscosity at 135 ^{0}C mm^2/s	EN 12595	175	325
Resistance to hardening at 163 ^{0}C	EN 12607-1	-	-
Change in mass % (max ±)		0.8	0.5
Retained penetration % (min)		43	50
Softening point after hardening ^{0}C (min)	EN 1427	41	49
Flash point ^{0}C (min)	EN 22592	230	230
Solubility % (min)	EN 12592	99.0	99.0

Table 3-11: Asphalt Binders Specifications (BS EN 12591, 2000)

3.4 Preparation of Batch Mixes

After the mix design, the amount of aggregates required to make one sample was determined. This was obtained by making a trial sample for each type of mix. A slightly higher quantity of aggregates was used to take care of the losses which may happened when transferring and mixing, and that would be a few grams (up to 10g). Once the quantity of aggregates required to make a sample have been determined, the required quantity of the different fractions were obtained from the mix design. Then each fraction of aggregate was carefully weighed and mixed together to obtain the batch mix.

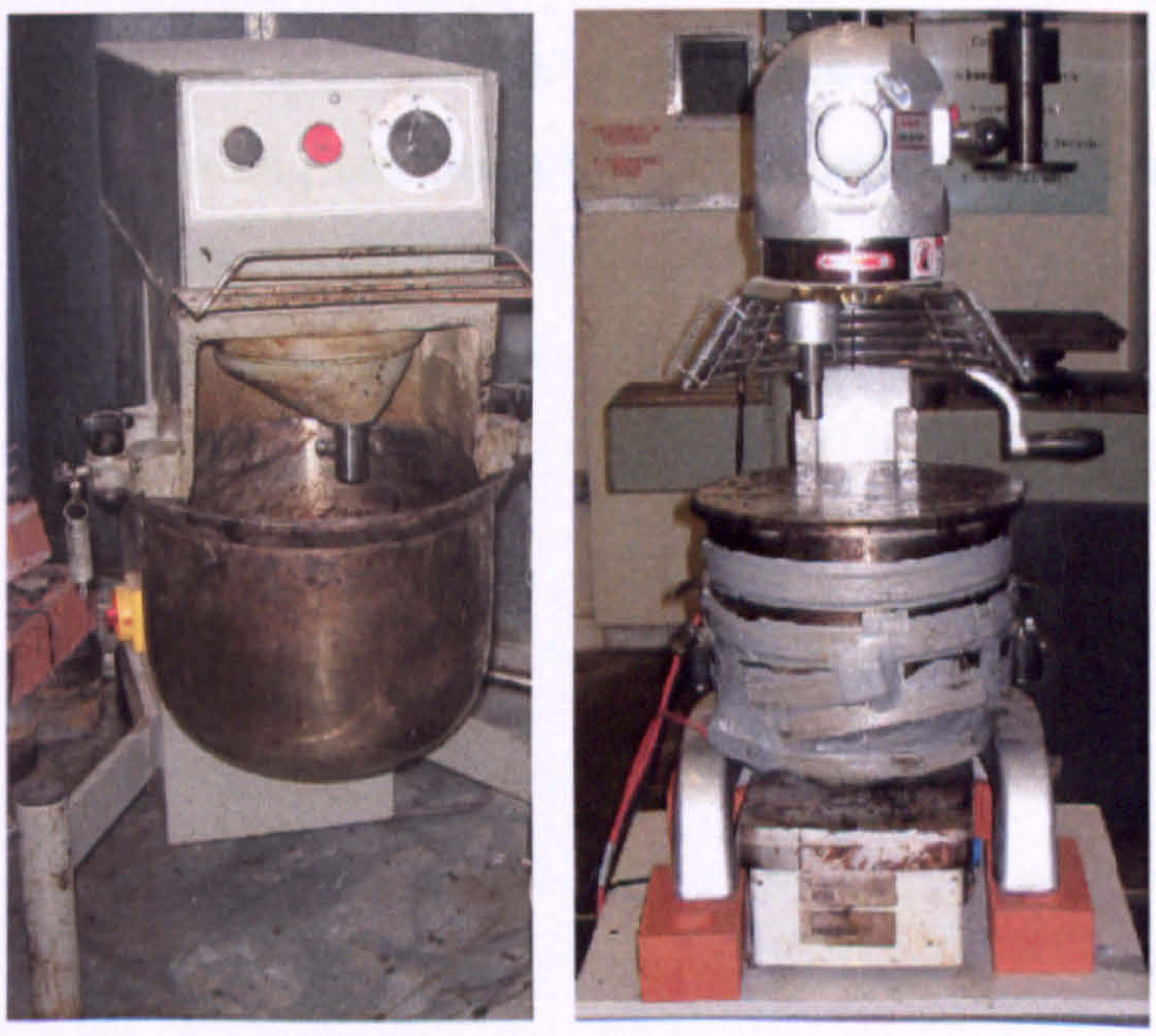
3.5 Preheat batch mixes and asphalt to mixing temperature

The batch mix and the asphalt were heated to a mixing temperature of 140^{0}C for the 100/150 pen binder and 155^{0}C for the 40/60 pen binder as specified by BS EN 12697-35

(2004) section 6.1.1 and as shown in Table (4-2) of Chapter Four. The batch mix was then kept in the oven at the mixing temperature for at least four hours (BS EN 12697-35, 2004). The asphalt was also kept at the mixing temperature for one hour and all the mixing tools and equipment were also preheated to the mixing temperature.

3.6 Mix Preparation

Once the batch mix was preheated to the mixing temperature, the aggregate was transferred to the preheated mixing bowl. The required amount of binder, as determined by the binder content test in the mix design, was carefully poured into the mixing bowl. Extreme care has been taken at this stage since even a slight increase or decrease in the binder content will significantly alter the properties of the asphalt mix. Also concern was taken to minimize the loss of binder which adheres to the walls of the mixing bowl. The batch mix was mixed with the hot binder at the mixing temperature for three minutes (BS EN 12697-35, 2004) using two types of Hobart mechanical mixers, type A mixer for mixing samples of 100 x 63.5mm in size and type B for mixing samples of 500 x 180 x 50mm and 305 x 305 x 50mm in size (Plate3-11).



A **B**

Plate 3-11: Samples Mixing Machines

Plate 3-12: Marshall Hammer Compactor

3.7 Oven Heating

Once the mixing was completed, the mix was spread out on a relatively wide pan and kept inside the oven at the compaction temperature, (135^0C) for the 100/150 pen binder and (150^0C) for the 40/60 pen binder, for two hours. Care was taken in order not to lose the fines sticking to the sides of the mixing bowl in the process. The compaction temperature is the temperature at which the kinematic viscosity of asphalt is 0.5 Pa s. Also the mold and any other equipment used in the compaction process were also kept in the oven at this time. Marshal hummer was put on top of a sand bed, which was resting on a heating plate for at least ten minutes prior to compaction.

3.8 Sample Compaction

The required amount of mix was put in a mold and then compacted by poking it with a spatula 15 times around the perimeter and 10 times over the interior. The amount of mix required was fixed by trial and error so as to obtain the required height of the final compacted sample of about 63.5mm. Once trial mixes were prepared a mechanical Marshall hammer was used to compact laboratory specimens of 101.6 mm diameter with the height mentioned above. The Marshall hammer (Plate 3-12) achieves compaction in a sample by dropping a (4.535kg) flat-faced weight onto the surface of the sample from a height of (457.2 mm). The sample received an equal number of 50 blows on each face, as the loading condition is to be representing a medium traffic effect (Table 3-12) (Asphalt Institute, 1983).

Mix Criteria	Light Traffic (< 10^4 ESALs)		Medium Traffic (10^4 - 10^6 ESALs)		Heavy Traffic (> 10^6 ESALs)	
	Min	Max	Min	Max	Min	Max
Compaction (number of blows on each end of the sample)	35		50		75	
Stability	500 lbs (1112 N)		750 lbs (1668 N)		1500 lbs (3336 N)	
Flow (in units of 0.01 inches)	8	20	8	18	8	16
Percent Air Voids	3	5	3	5	3	5

Table 3-12: Marshall Compaction Specification (Asphalt Institute, 1983)

Special care was needed for handling the Uncoated Croxden samples during the compaction process as it was noticed that during the turning over to the mold, so it can be struck on its other side with 50 blows, some samples were intending to collapse, and that was because they were of OGFC mix type. So about 1 ½ hour was needed for the samples to cool down and then can be extracted from the mold.

3.9 Determination of Volumetric Properties

For determining the air voids of the intended mixes, first it was necessary to calculate both of the theoretical maximum specific gravity (TMD or Gmm) of loose samples and also the bulk specific gravity (Gmb) of the compacted specimens. The ratio of the weight in air of a unit volume of an uncompacted asphalt mix (i.e. without air voids), at a specified temperature, to the weight of an equal volume of distilled water at the same specified temperature is called the Theoretical Maximum Density (TMD) of that loose mix. The test method for finding the TMD of asphalt mixes is described in BS EN 12697-5 (2002), and BS EN 12697- 6 (2003) was utilized for carrying out tests to determine the Gmb's of the compacted samples.

Table (3-13) shows the average TMD & Gmb for loose and compacted specimens with two types of binders respectively.

Mix Type	**Gmb** for 100 / 150 pen binder	**TMD (Gmm)** for 100 / 150 pen binder	**Gmb** for 40 / 60 pen binder	**TMD (Gmm)** for 40 / 60 pen binder
Arcow	2.137	2.660	2.126	2.652
Uncoated Croxden	2.162	2.561	2.410	2.558
Coated Croxden	2.034	2.540	2.014	2.529

Table 3-13: TMD & Gmb Tests Results for Loose & Compacted Specimens

The bulk specific gravity of a specimen is less than the TMD of the same mix as the bulk specific gravity of a specimen is the ratio of the weight of a unit volume of a compacted specimen (including permeable air voids) at 25^0C to the weight of an equal volume of distilled water at same temperature. As normally the distilled water density is (997.1) kg/m^3 at 25^0C, and as the ambient temperature in the laboratory was 20^0C, therefore, a correction factor for the distilled water density was used as 1.0012 x (997.1) kg/m^3 as in clause 10.1 of BS EN 12697-5 (2002).

Once the actual air void in a sample was determined, it was compared with the target air void. Even though the sample was compacted for the target air voids, the actual air voids in the sample can be different from the target air void. If there was too much difference between the actual air void and target air void, the sample was disregarded and a new sample was made.

Figure (3-4) shows the permeability due to air void contents where air void contents ≥ 5% to 15% provide minimum moisture susceptibility resistance. For OGFC, air voids above 10% is preferred (Tarmac personal communications in 2007).

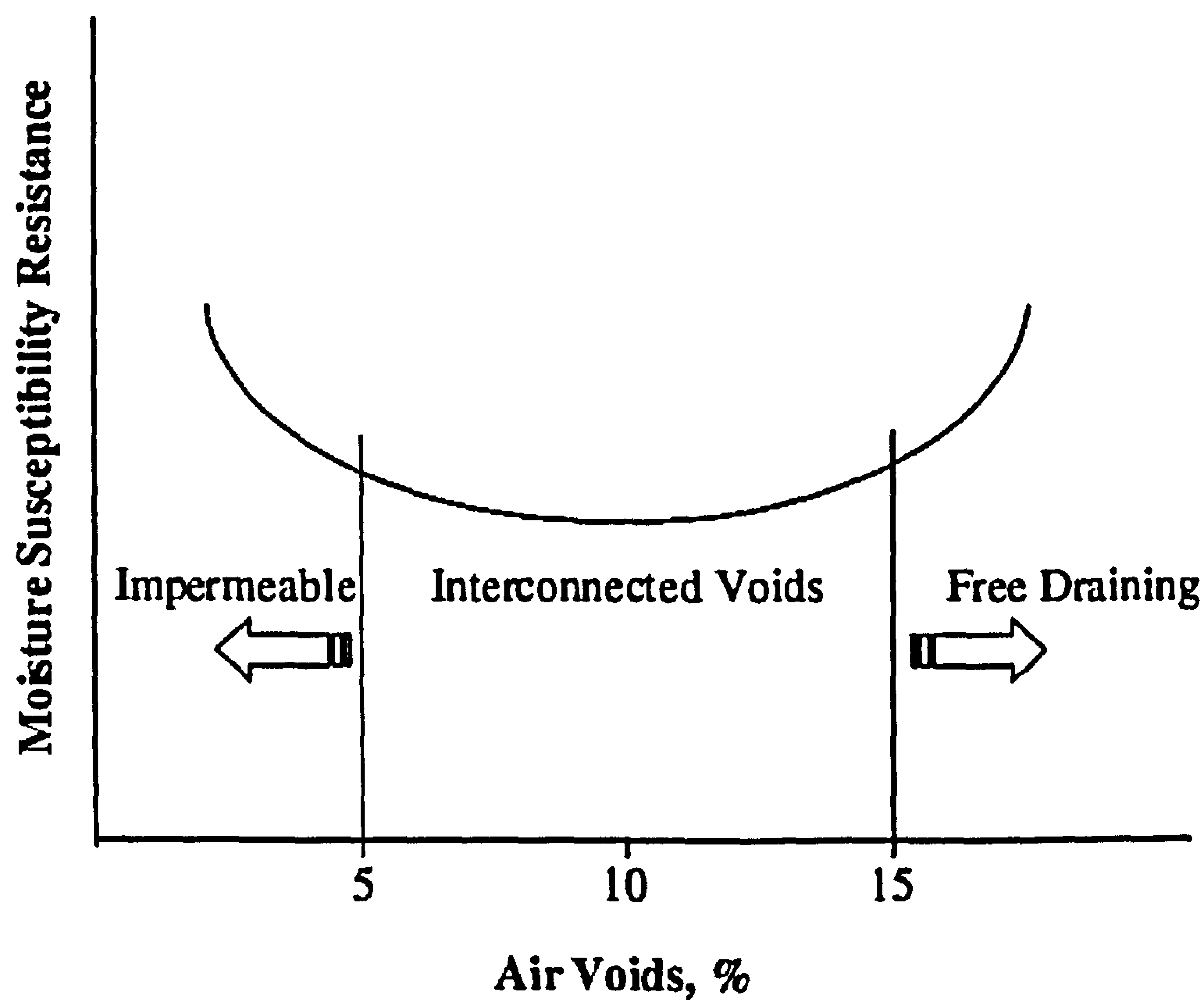


Fig. 3-4: Permeability due to Air Voids content level (Deacon et al, 2002)

3.10 Mix Designation

The BS EN 13108 Standards and the guidance in PD 6691:2007 use a defined format for describing each type of asphalt mixes. This is known as the mix designation. The revised clauses for each type of mix in Series 900 of the Specification for Highway Works (11/07) include a list of permitted mixture designations which has been implemented in full and without alteration. Hence the types of mixes that were adopted for this research work are to be having the following new Mix designated names:

$\boxed{\text{AC 14 Open Surf 40/60}}$ and $\boxed{\text{AC 14 Open Surf 100/150}}$ (BS IAN, 2007)

3.11 Aggregates Gradation Modification

As mentioned in section 3.1 above, the mix design used as a control mix is Arcow mix. The nominal maximum aggregate size is 14mm. All gradation curves of the three mixtures go below the restricted zone and then turn upward across the maximum density line at No. 4 sieve size (Figure 3-5). The main purpose of this chosen gradation shape is to assure sufficient air voids content of the asphalt mixture (Mamlouk and Al Basyouny, 1999). In order to facilitate study of the cement coating of secondary aggregates on asphalt mixtures, the coarse part (No. 4 sieve size and larger) of each mix design was modified with the fine parts of the mixes kept unchanged. The corresponding gradation chart for all mix design series (sieve size in mm raised to power 0.45) is presented in Table (3-7) above, and Table (4-3) in the next chapter of this thesis.

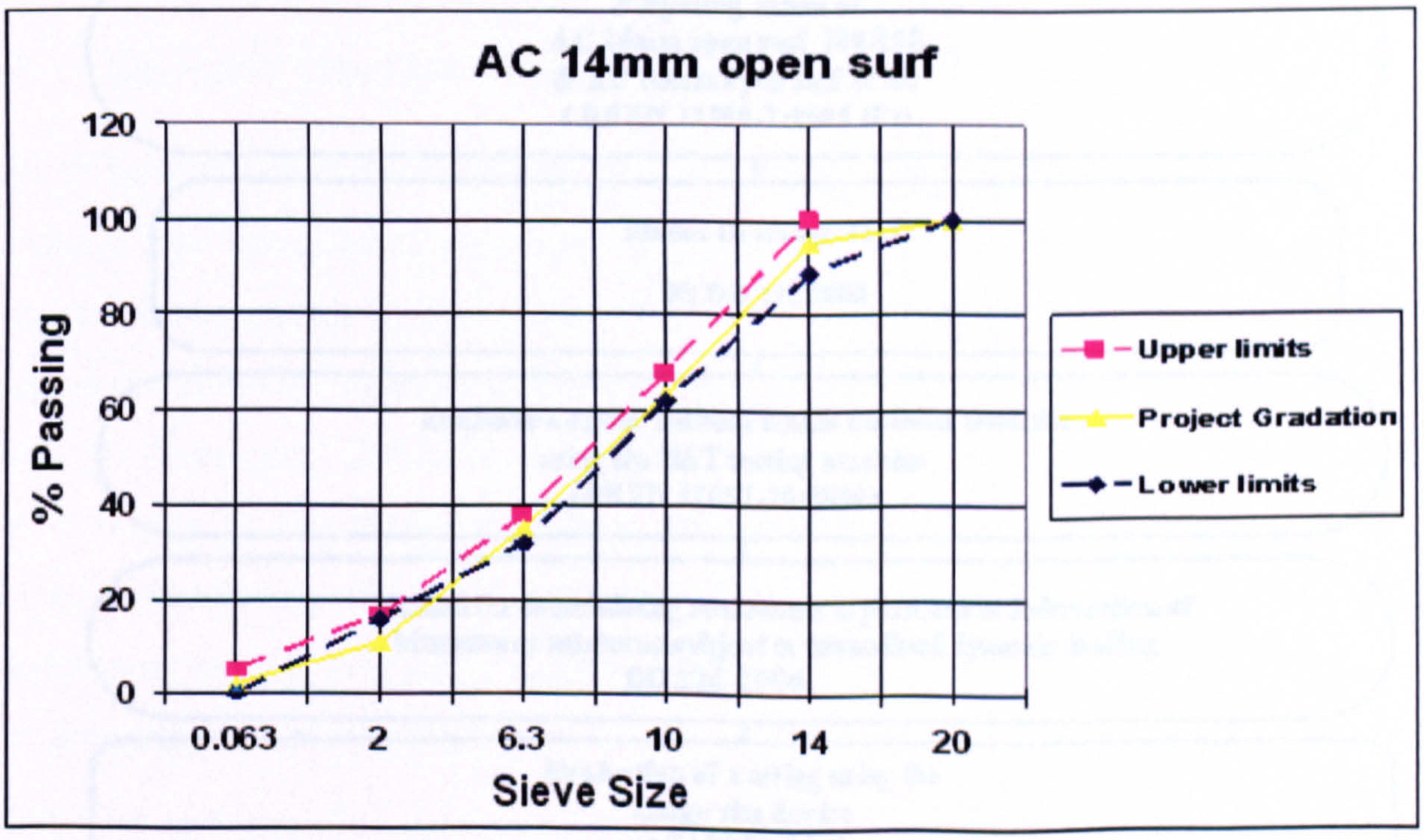
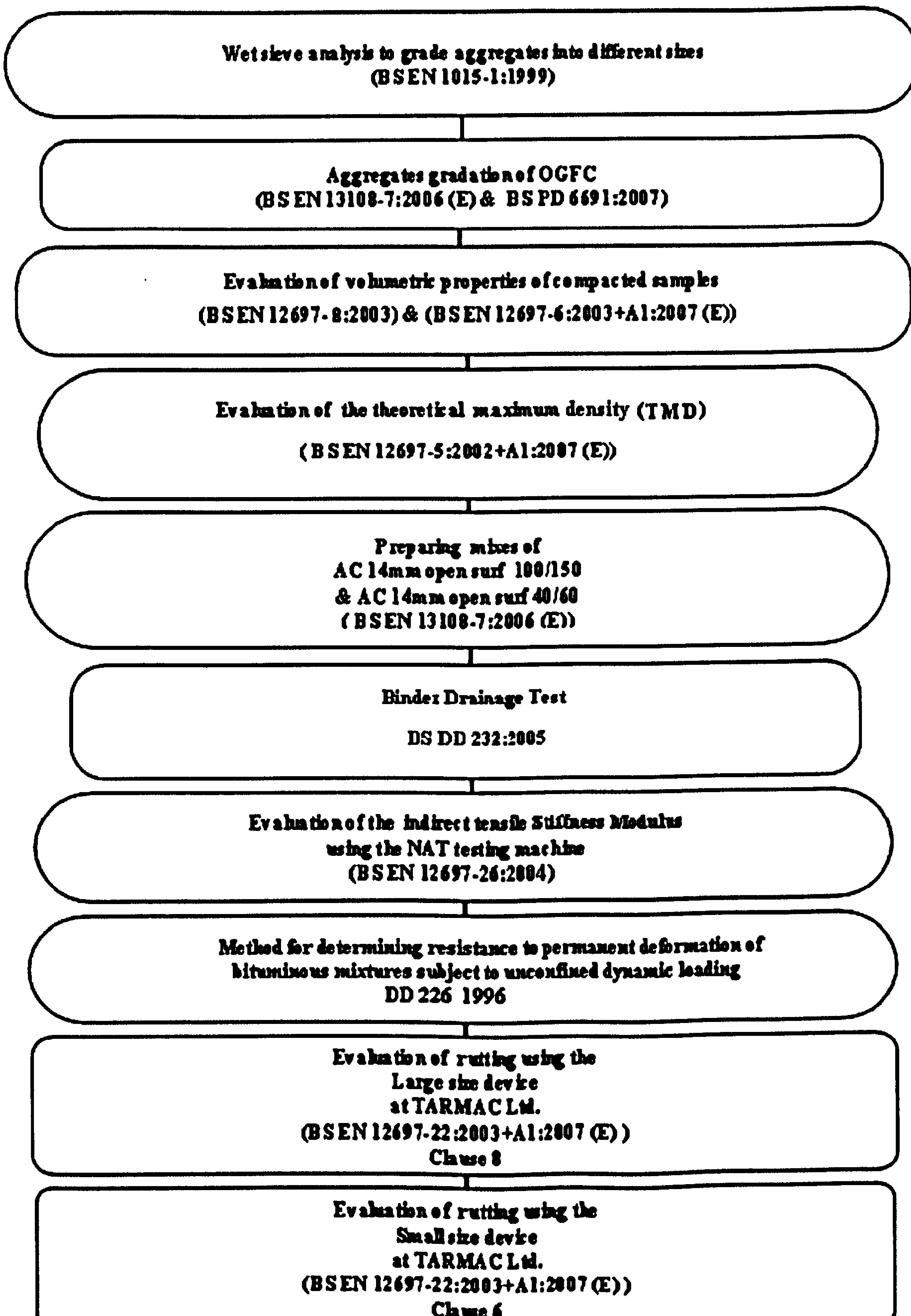


Fig. 3-5: Permissible Gradation Limits and the Chosen Gradation (BS PD6691, 2007)

Fig. 3-6: Laboratory Tests Flow Chart



CHAPTER 4

Design of OGFC for Performance Tests

(NAT and Wheel Tracking Testing)

4.1 Introduction

This chapter will be discussing the mix design procedures that were followed to compose the Open Graded Friction Course (OGFC) asphalt mixes utilized for Repeated Load Indirect Tensile Test (RLIT), Repeated Load Axial Test (RLAT) and the Wheel Tracking Test, detailed in chapters five, six, and seven of this thesis.

4.2 Aggregate Types and Bituminous Binders Utilized for Mixes Preparations

The influence of the cement coating on Porous asphalt having 20mm nominal aggregates gradation was investigated previously at LJMU and the limited initial results were very encouraging in regards of performance testing of mixes containing cement coated natural aggregates. However testing was undertaken at 20^0C only and limited to triaxial machine only (Al Nageimand Robinson, 2006, Vaughan, 1999). The previous investigation was concentrating on the adhesion between the binder and aggregates and was lacking the ability to further investigate the cement coating concept in a wider range of temperature applications as well as to apply torture tests such as wheel tracking tests, which is one of the main aims of this research work.

Open Graded Friction Course is gaining momentum nowadays in the pavement industry around the world for being environmental friendly by having noise reduction ability, and after discussions with Tarmac ltd., it was decided to investigate the influence of cement coating of natural aggregates, on the mechanical performance of laboratory prepared mixes of OGFC represented by (AC 14 open surf) gradation (BS PD 6691, 2007). Table (4-1) shows the (AC 14 open surf) gradation utilized for this research investigation.

Two types of aggregates representing prime and secondary quality aggregates were utilized in this research work, the first type was represented by Arcow aggregate, normally excepted by road engineers for use in road pavement, and the second was low quality natural aggregate represented by Croxden aggregate and not qualified for use in road surfacing mixtures due to its smooth surface and poor binding with the binder (Al Nageim and Robinson, 2006). Two types of Binders 100/150 pen graded and 40/60 pen graded have been utilized to represent soft and hard binders respectively.

Sieve Size (mm)	$(\text{Size})^{0.45}$	Specification Limits (%)		% Passing	Cumulative Retained	% Retained
		Lower	Upper			
20	**3.85**	100	100	100	0	0
14	**3.279**	98	100	98	2	2
10	**2.818**	62	68	68	32	30
6.3	**2.289**	32	38	36	64	32
2	**1.366**	16	17	11	89	25
0.063	**0.288**	0	5	2	98	9
Filler	-	-	-	-	100	2
						Σ=100 %

Table 4-1: Gradation Specification According to BS EN Specifications **(BS PD6691, 2007)**

They therefore cover the two extremes of usages of asphaltic materials and thus using them in the testing of cement coating on providing the necessary properties of the new mix containing coated aggregates will be prevailed. The crude oil for these two asphalt binders came from Venezuela (NYNAS, 2008).

4.3 Binder Drainage Test

4.3.1 Introduction

This section concentrates on describing the method adopted for determining the optimum binder contents for the three (AC 14 open surf) mixes utilized in this research work. The adopted method was following the new (BS DD 232 (2005) entitled: "Method for determination of the maximum binder content of bituminous mixtures without excessive binder drainage". Full test procedures are explained in (section 4.3.3) below.

4.3.2 Summary of Binder Drainage Test

The binder drainage test involved preparing eleven samples, for each of the three types of the (AC 14 open surf) mixes, having weight of 1100g and transferring it into a perforated basket, which in turn was placed, on a pre-weighed tray in an oven at the maximum mixing temperature for time not exceeding 3.25 hours. The test was conducted at the temperature specified in Table (4-2), which provided approximate temperatures at which various grades of binder have a viscosity of 0.5 Pa s (Pascal second) (BS EN 12697-35, 2004). At the end of the test, the mass of binder drained was determined. The test was repeated for a series of binder contents, ranging between (3.5-7.0%) of the total aggregate weight, and the amount of material drained measured each time. The data was presented as a graphical plot between drained binder content versus mixed binder content. A typical plot is shown in Figure (4-1).

Paving grade of bitumen	Reference temperature for : ^0C		Paving grade of bitumen	Reference temperature for mixtures of types other than mastic asphalt ^0C
	Mixtures of types other than Mastic asphalt	Mastic asphalt mixtures		
20/30 pen	180	250	250/330 pen	130
30/45 pen	175	240	330/430 pen	125
35/50 pen	165	230	500/650 pen	120
40/60 pen	155	220	650/900 pen	115
50/70 pen	150	-	V12000	115
70/100 pen	145	-	V6000	110
100/150 pen	140	-	V3000	100
160/220 pen	135	-	V1500	90

Table 4-2: Reference temperatures for mixes with paving bitumen (BS EN 12697-35, 2004)

The mixed binder content is assumed to coincide with 0.3% drainage. The target binder content (Tm) is equivalent to (mixed binder content - 0.3) %. The term 'target binder content' refers to the maximum binder content that can be safely accommodated without the risk of excessive binder run-off during mixing, transportation or laying (BS DD 232, 2005).

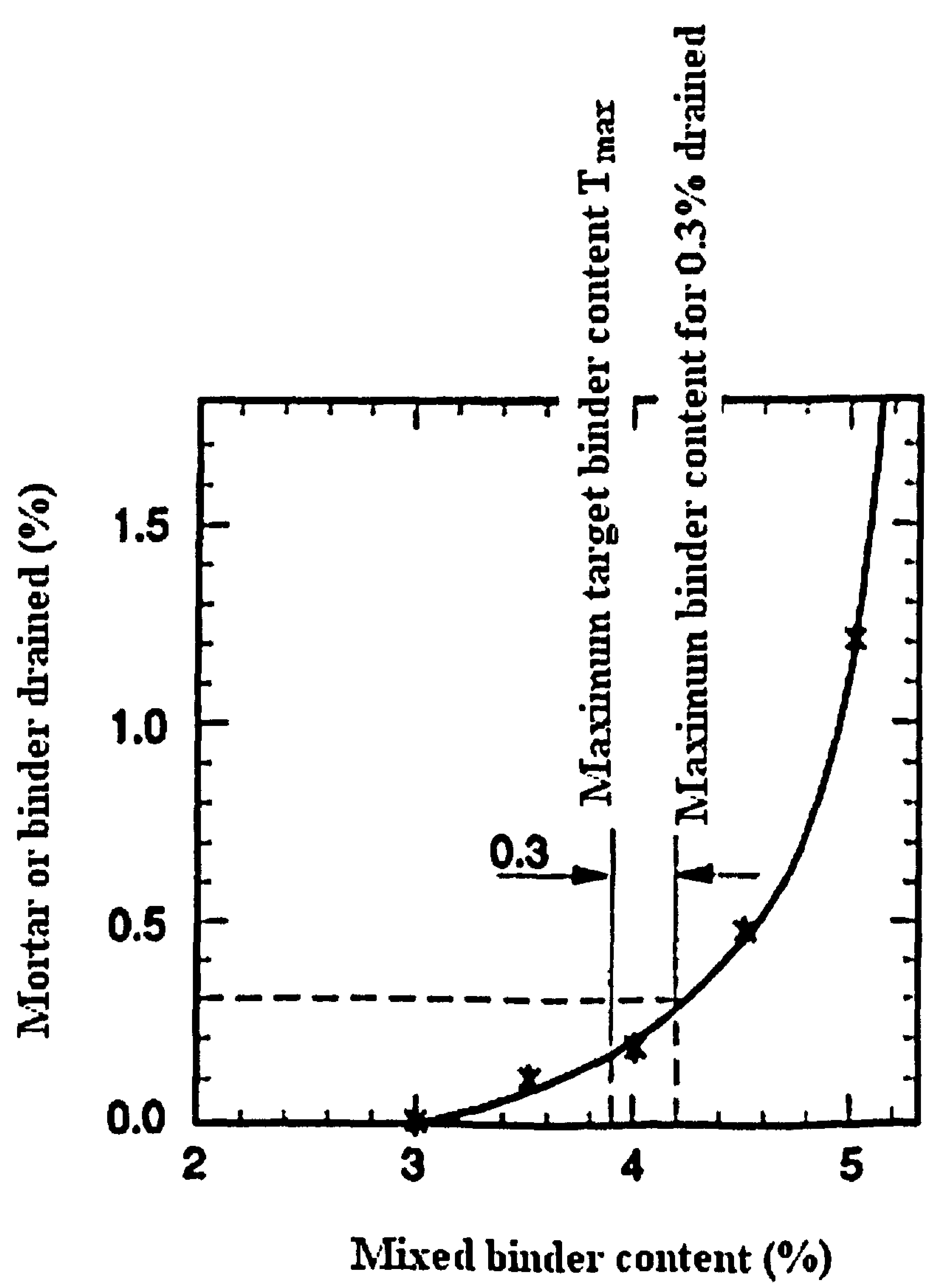


Fig. 4-1: Typical Graph of Mortar & Binder Drained (BS DD 232, 2005)

4.3.3 Binder Drainage Test Equipments

1 Digital scale, 5.5kg capacity, accurate to 0.1 g.

2 Oven, vibration free, with fan assisted air circulation, and fitted with thermostatic control to maintain the temperature to ±1 °C in the range 50 °C to 200 °C.

3 Hobart Mechanical mixing unit with a capacity of 10L capable of combining the test aggregate and binder quickly and thoroughly without loss.

4 Drainage baskets (at least 10), constructed from perforated metal with 3.1 mm ± 0.1 mm diameter holes and with 40% open area on sides and base to form 100 mm cubes with feet at each corner of the base of 3 mm diameter and 5 mm in height.

5 Metal trays (10, for each type of the three mix), approximately 150 mm square and 10 mm deep.

6 Spatula, with a blade approximately 150 mm long by 25 mm wide.

4.4 Materials Testing Procedures

Eleven batches of 1100g of aggregate were weighed in the proportions described in Table (4-3) and Figure (4-2). Each batch was then placed in a separate tin. The oven was preheated to the mixing temperature, 140°C and 155°C for the 100/150 pen and 40/60 pen respectively (Table 4-2), before adding to the tins with the aggregates. The binder and the mixing implements, which were left to heat for at least two hours prior to use, were heated and the compaction hammer was also heated, to compaction temperature, for at least 10 minutes prior to compaction.

Each disposable aluminum tray was weighed to the nearest 0.1g (W_1). Each batch of aggregate was transferred to the mixing bowl. The heated binder was stirred in its container and the required amount of binder, to the nearest 0.5g, was weighed by difference into the mixing bowl to give 3.0 % by mass of aggregate. The mixing bowl was fitted to the mixer which was run for 60 seconds. Using a spatula any material on the top and sides of the bowl, and the mixing blade was scraped into the mixture. The mixing was then started for another 60 seconds. The first mixture was used to condition the mixing bowl so it was discarded. The same procedures were repeated with fresh materials. The material was then transferred to a basket, ensuring that the mixer bowl and mixing blade were scraped thoroughly using the spatula. This operation was conducted as rapidly as possible to minimize the loss of heat. The basket was placed on the foil tray and then into the oven at 140°C, (Table 4-2), for between 3 hours and 3.25 hours for the soft binder and at 155°C for the harder binder. The basket and tray was then removed from the oven, and the basket removed from the drainage tray.

Grading	% Passing						
	20 mm	14 mm	10 mm	6.3 mm	2 mm	0.063 mm	Pan
14 mm	100	98	68	36	11	2	0

Table 4-3: Aggregates Gradation Implemented by the Research for the Three Types of Mixes

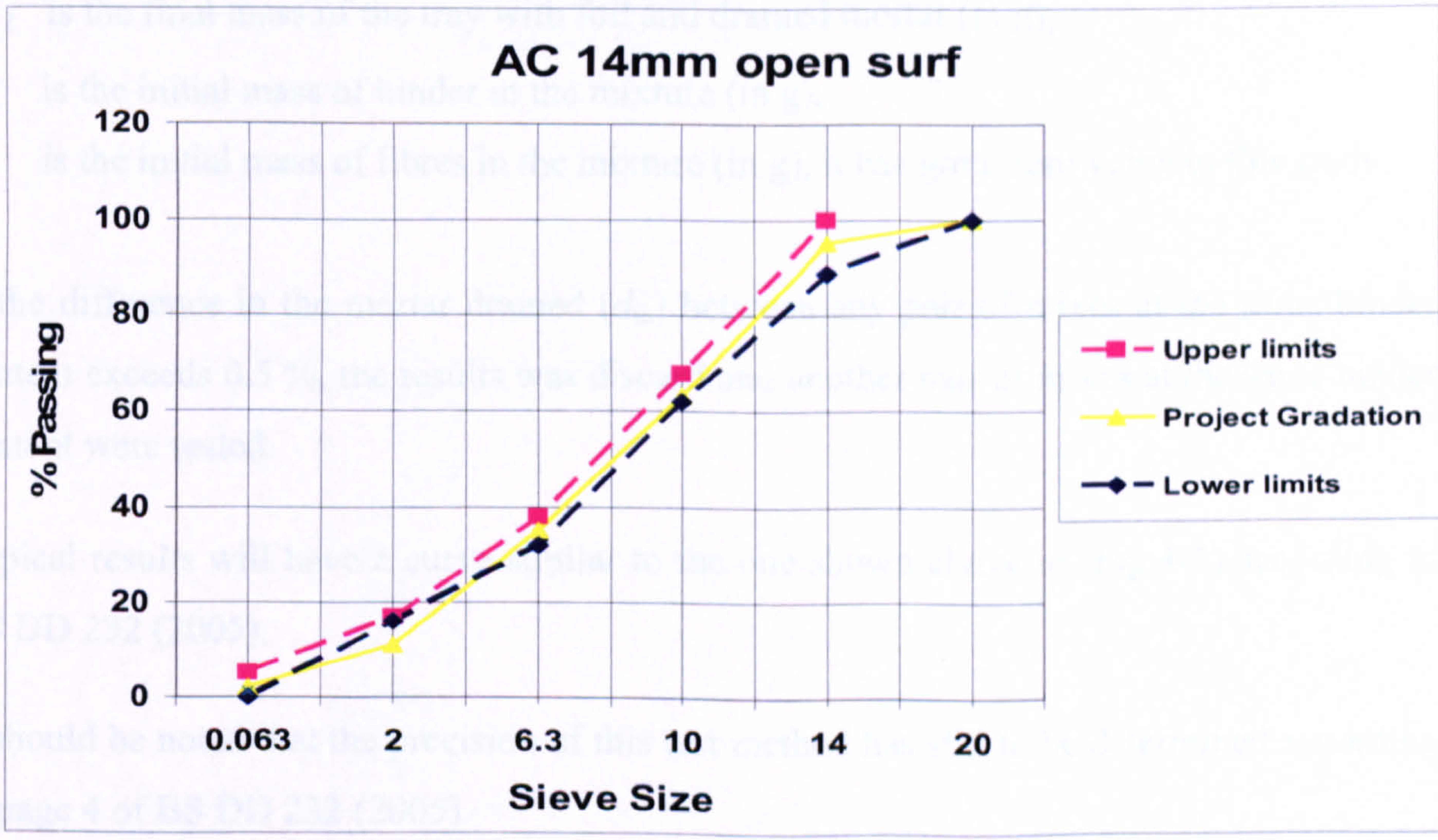


Fig. 4-2: Permissible Gradation limits and the chosen gradation **(BS PD6691, 2007)**

After the tray had cooled sufficiently it was weighed again to the nearest 0.1g (W_2). After the first mixture was placed into the oven, the same procedure was repeated for a duplicate sample at the same binder content. Then the procedure for producing these batches was repeated five times, but with increasing the binder content to 3.5, 4.0, 4.5, 5.0 and 5.5 % by total mass of aggregate, respectively. Some times it was necessary to conduct a further binder content test with increments of 0.5% up to 7 %, as the outcome of the drainage test up to 5.5% did not reveal an obvious graph curvature that represent the optimum binder content for that particular mix, as occurred with some tests (Fig.4-4 & 4-5). Figure (4-3 to 4-8) show the binder drainage test results.

4.5 Drainage test's Calculation method

The mortar drained from each sample d_m (in %) can be calculated from the equation:

$$d_m = 100 \times \frac{(W2-W1)}{(1100+B+X)} \qquad (4.1)$$

Where:

W_1 is the initial mass of the tray with foil (in g):

W_2 is the final mass of the tray with foil and drained mortar (in g),

B is the initial mass of binder in the mixture (in g),

X is the initial mass of fibres in the mixture (in g), it has got a zero value in this study.

If the difference in the mortar drained (d_m) between any pair of mixes at the same binder content exceeds 0.5 %, the results was discard and another pair of mixes at the same binder content were tested.

Typical results will have a curve similar to the one shown above in (Fig.4-1) according to BS DD 232 (2005).

It should be noted that the precision of this test method has still to be determined according to page 4 of BS DD 232 (2005).

The results of the six tests corresponding to each type of the three mixes and two types of asphalt binders are tabulated in Table (4-4) and graphs representing these values are shown in Figures (4-3 to 4-8) below, where high accuracy of results were achieved. This is evident by the smooth curves produced and coincides in shape to the typical graph shown in Figure (4-1).

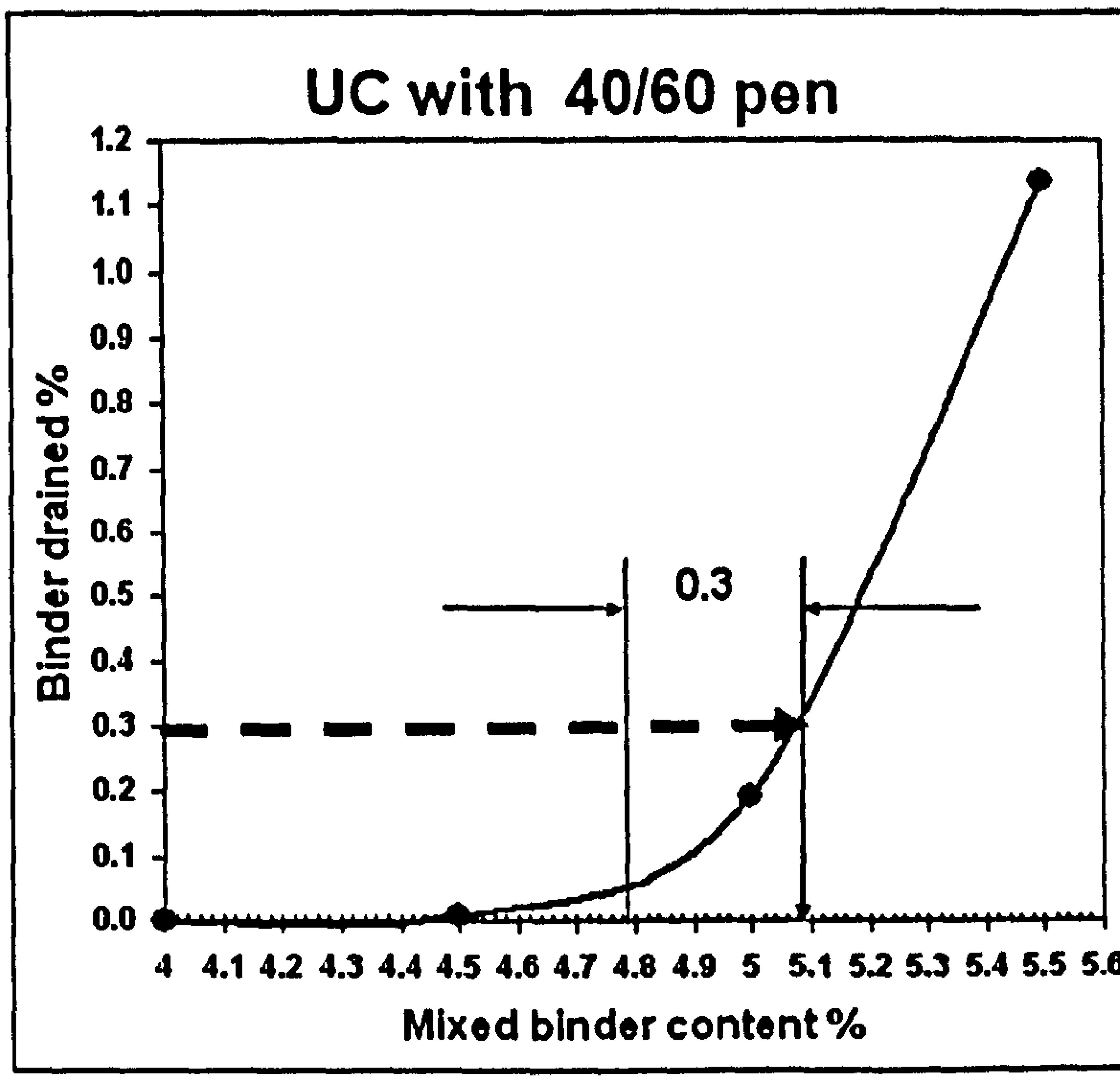


Fig. 4-3: Binder drainage test result for Uncoated Croxden Mix with 40 /60 pen

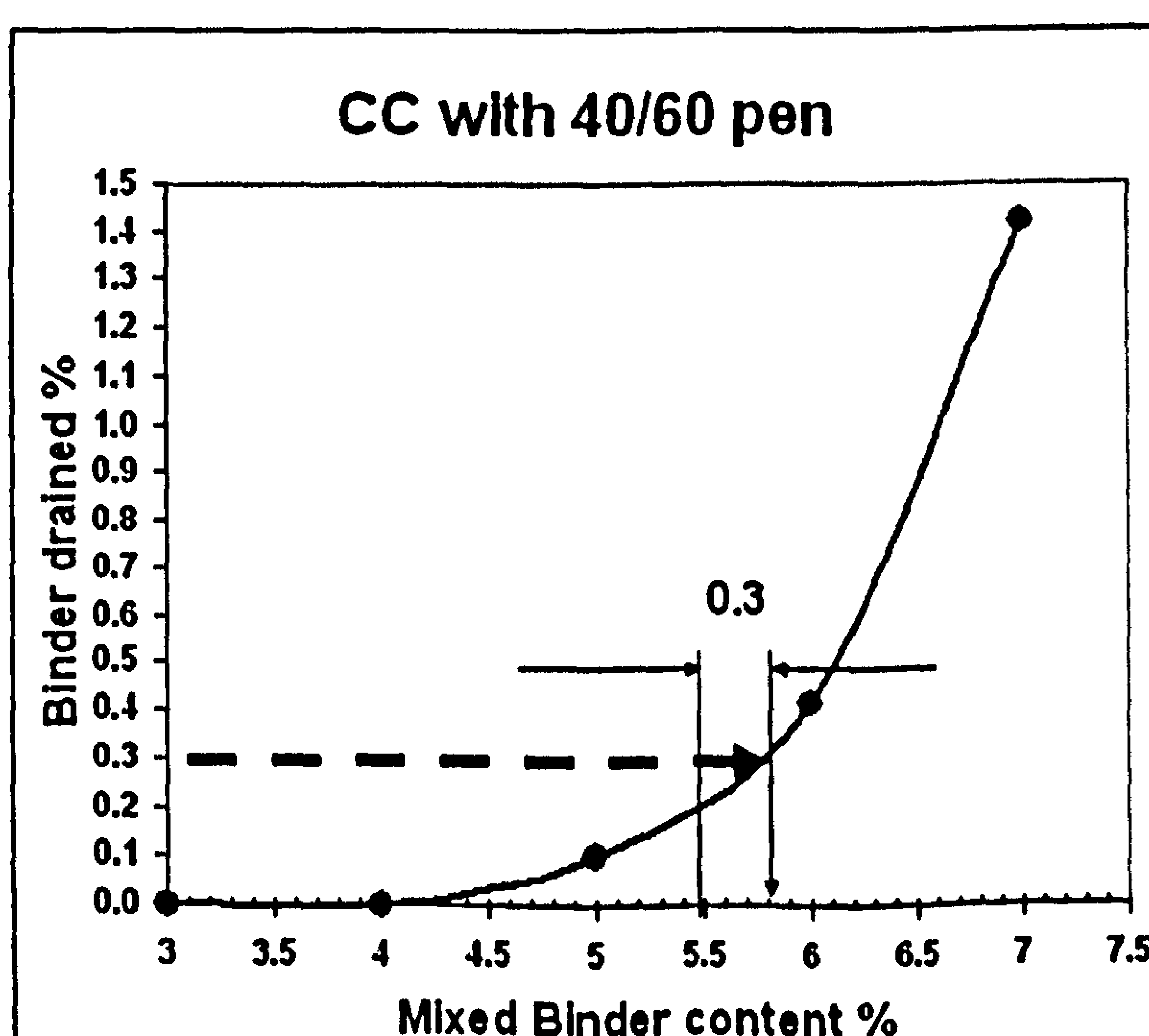


Fig. 4-4: Binder drainage test result for Coated Croxden Mix with 40 /60 pen

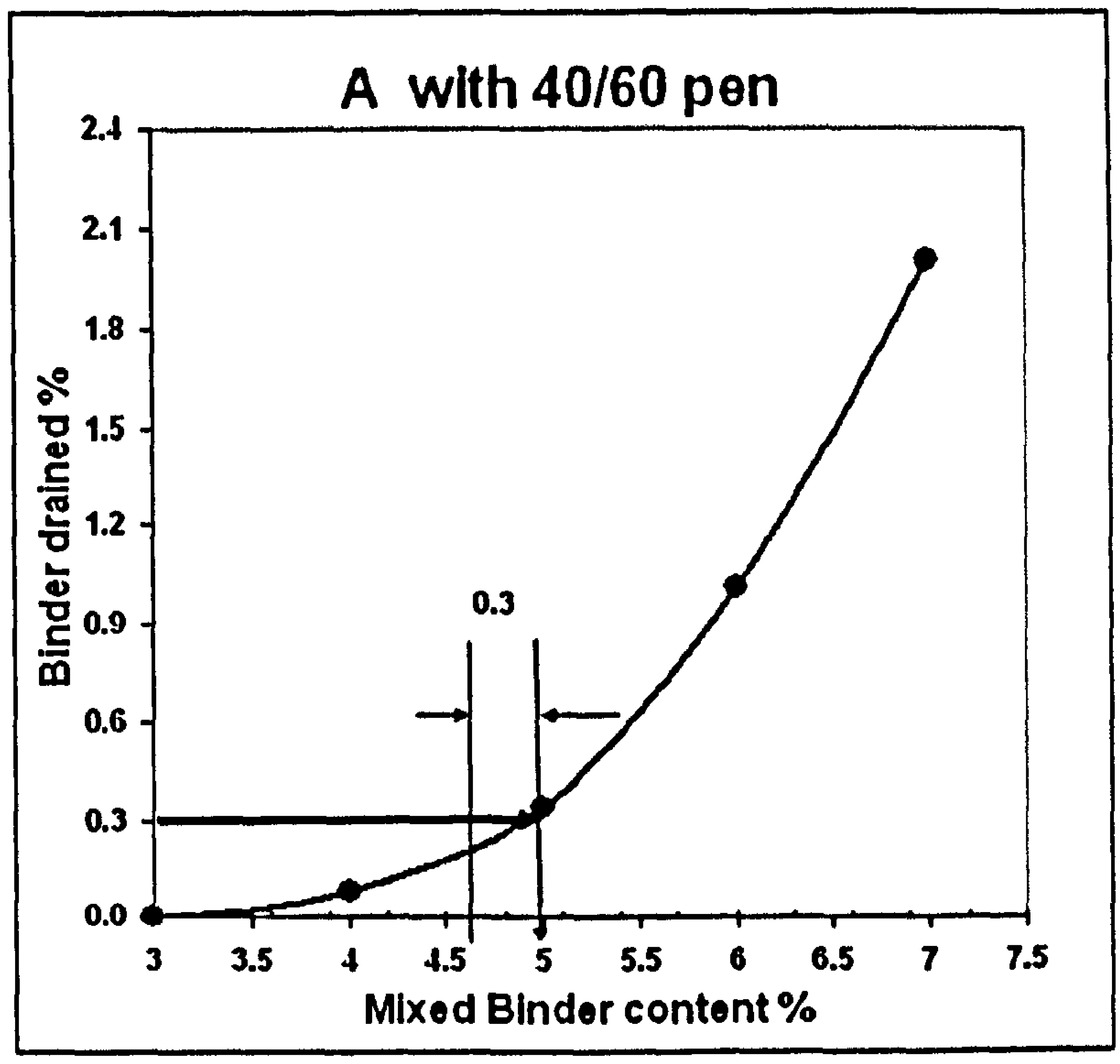


Fig. 4-5: Binder drainage test result for Arcow Mix with 40 /60 pen

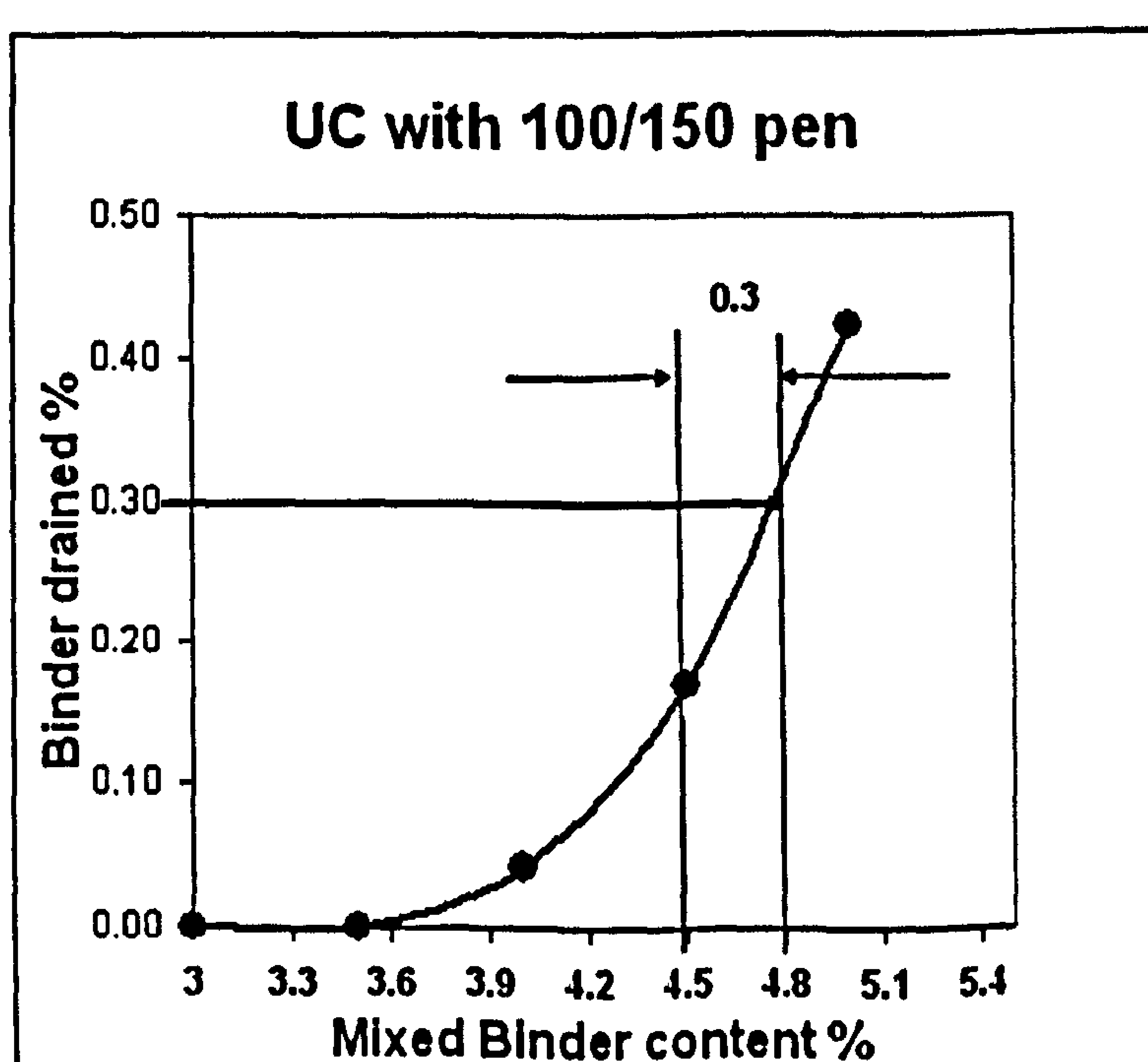


Fig. 4-6: Binder drainage test result for Uncoated Croxden Mix with 100 /150 pen

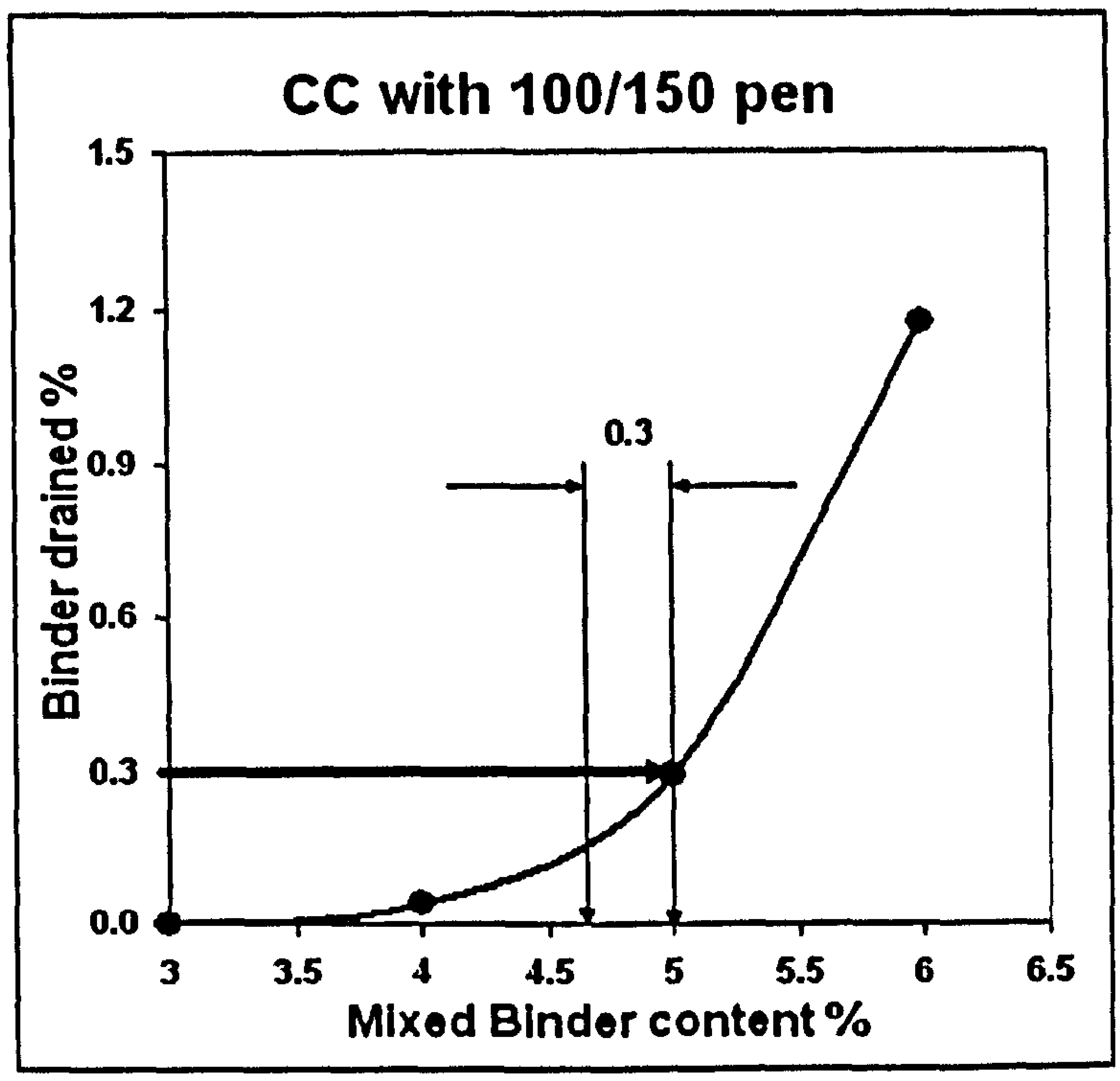


Fig. 4-7: Binder drainage test result for Coated Croxden Mix with 100 /150 pen

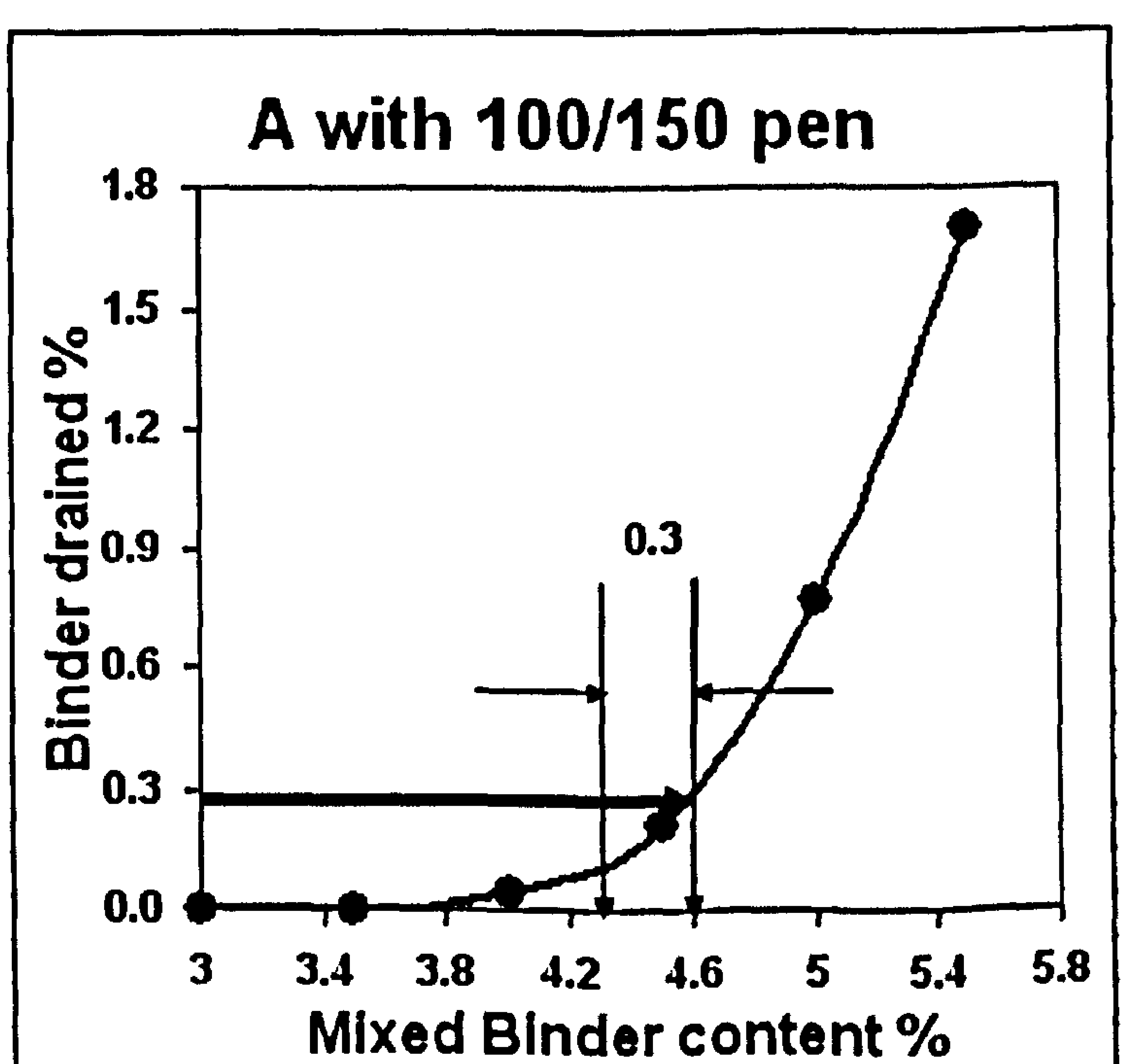


Fig. 4-8: Binder drainage test result for Arcow Mix with 100 /150 pen

Mix Type	Binder content For 100/150 pen (%)	Binder content For 40/60 pen (%)
Uncoated Croxden	4.5	4.8
Coated Croxden	4.7	5.5
Arcow	4.3	4.7

Table 4-4: Results of Binder Drainage tests

Interestingly, the drainage tests has shown that the optimum binder contents of the coated aggregates were higher than the other two types of aggregates, due to the introduction of the cement paste, indicating that the coated material has improved the adhesion between the aggregates and the binders, which will results in good mix performance as will be shown in the following sections and chapters of this thesis.

4.6 Materials and Mix Design

4.6.1 Materials

In order to determine the materials and material compositions for the three selected mixes, Marshall Mix design was carried out in accordance with BS PD 6691 (2007) specifications for the (AC 14 open surf) asphalt mixtures as follows:

- Mix one is the control mix and was composed of crushed Arcow aggregates[1],
- Mix two was containing uncoated Croxden aggregates[2], and
- Mix three was containing cement coated Croxden aggregates, as described in the previous chapter of this thesis.

[1] The chemical and physical properties of Arcow aggregate were shown in Table 3-1 of chapter three.
[2] The chemical and physical properties of Croxden aggregate were shown in Table 3-1 of chapter three.

Two types of unmodified asphalt binders were utilized in this research work and they were provided by NYNAS ltd. UK, the first type is the 100/150 penetration grade (pen) that is representing a soft binder and the second type is the 40 /60 pen that represents a hard binder. The crude oil for these two asphalt binders came from Venezuela (NYNAS, 2008). The specifications of these two asphalt binders according to BS EN 12591 (2000) are shown in Table (4-5) .

The three mixes mentioned above were made with these two types of binders: so in total six types of mixes were prepared and they are:

- Arcow (control mix) mix with 100/150 pen binder,
- Uncoated Croxden mix with 100/150 pen binder,
- Cement coated Croxden mix with 100/150 pen binder,
- Arcow (control mix) mix with 40/60 pen binder,
- Uncoated Croxden mix with 40/60 pen binder, and
- Cement coated Croxden mix with 40/60 pen binder.

PAVING GRADES	TEST METHOD	100/150 pen	40/60 pen
Penetration at 25^{0}C	EN 1426	100-150	40-60
Softening point ^{0}C	EN 1427	39-47	48-56
Kinetic viscosity at 135 ^{0}C mm^2/s	EN 12595	175	325
Resistance to hardening at 163 ^{0}C			
-change in mass % (max ±)	EN 12607-1	0.8	0.5
- retained penetration % (min)		43	50
- softening point after hardening ^{0}C (min)	EN 1427	41	49
Flash point ^{0}C (min)	EN 22592	230	230
Solubility % (min)	EN 12592	99.0	99.0

Table 4-5: Asphalt Binders Specifications **(BS EN 12591, 2000)**

4.6.2 Aggregates Gradation Modification

The six mix designs used were of (AC 14mm open surf) types. The nominal maximum aggregate size for all the mixes is 14mm, gradation curve of the mixtures goes between the restricted limits of the BS PD 6691 (2007) (see Figure 4.2 above). The curve goes below the restricted zone of the fine aggregate size. The main purpose for choosing this is to assure sufficient air voids content of the asphalt mixture. Gradations passing through the restricted zone are thought to have low resistance to permanent deformation, but this has been disapproved in some studies as will be presented in the following sections. This alteration is allowable in the current BS PD 6691 (2007) specification. The asphalt content levels for mixes with modified gradation were kept the same.

4.6.3 Mix Design of OGFC Specimens

Laboratory test specimens were prepared for the different mechanical tests used in this research work. Each test has specific requirements in terms of specimen shape and dimensions. Sieve analysis was conducted as the first step in Marshall Design process to divide aggregates into different sizes and obtain the desired aggregate gradation required for each mixture design. Croxden aggregate was washed with water before sieving to eliminate the effect of dust, attached to the aggregates surface, on the cohesion between aggregates and subsequent cement paste utilized for coating.

The purpose of removing the dust was to:

i. get rid of the amount of dust (aggregate with size smaller than 0.063mm) coated on the courser aggregates to determine the required amount of fillers need to be added for each mixture type,
ii. have a better control of the composition of the mixture and
iii. ensure better adhesion between the Croxden aggregates' surface and the cement coating paste, as mentioned in chapter three.

Samples, for the three types of mixes, have been produced to be suitable for each corresponding testing protocol:

1 100 x 63.5mm specimens for the NAT facility (RLIT and RLAT tests),

2 305 x 305 x 50mm specimens for the Wheel Tracking test (Small Scale Device), available at Tarmac ltd. Wolverhampton's Laboratories.

3 500 x 180 x 50mm specimens for the Wheel Tracking test (Large Scale Device), available at Tarmac ltd. Wolverhampton's Laboratories.

4.6.4 Specimens Preparation for the NAT facility (RLIT and RLAT tests)

After washing, the aggregates have been dried in the oven at 110^{0}C overnight to be ready for specimen preparation. The required amount of aggregates for each type of mixtures was weighed and heated for at least 8 hours at the mixing temperature. The 101mm diameter moulds, base plates and the mixing bowl together with the appropriate asphalt binder have been heated for about 3 hours at 140^{0}C for the soft binder and of 155^{0}C for the harder binder[3]. While the aggregate was in the oven at mixing temperature, the binder was taken out of the oven and whisked gently inside the holding tin; to have more homogeneous binder.

The heated aggregates were then mixed together with the required amount of the heated bitumen binder and the preheated filler was added to the mix at this stage as quickly as possible to minimize the heat loss. After mixing for 3 minutes using the Hobart mixer, the mixture was then compacted by a pre-heated 4.54 kg Marshall Hammer. During the compaction process 2 x 50 blows were applied on the specimen with each side receiving 50 blows. Mixing time was controlled to be for 3 minutes when done by the Hobart mixer, while when the machine was malfunctioning, five minutes was regarded as sufficient time for manual mixing as specified in Table (4-6) (BS EN 12697-35, 2004).

Mixture type	**Binder**	**Reclaimed asphalt**	**Mechanical mixing**	**Hand mixing**
Stone mastic asphalt	Paving grade	Not included	4 min	5 min
		Included	5 min	As appropriate
	Modified	Included or not included	5 min	As appropriate
Other than stone mastic asphalt	Paving grade	Not included	3 min	5 min
		Included	5 min	As appropriate
	Modified	Included or not included	5min	As appropriate

Table 4-6: Maximum mixing Time for mechanical and manual mixing

(BS EN 12697-35, 2004)

[3] Mixing temperature was increased by up to 10^{0}C to what has been specified by the standard to compensate for the temperature differences inside the ventilated oven {BS EN 12697-35 (2004) "Bituminous mixtures: Test methods for hot mix asphalt : Laboratory mixing"}.

4.6.5 Volumetric Properties of 100 x 63.5mm Specimens

Design criteria for all the mixes were purely volumetric (air voids content) so samples have been compacted, by Marshall Compactor as explained in (section 3.8 of chapter three), to achieve 20% ± 1% air void contents. The average height of each specimen was determined and each specimen was then weighed in air. Specimen's diameters have been assumed to be equal to the mould inner diameter (i.e. 101mm). The specimens' bulk densities were measured according to the current British Standard BS EN 12697- 6 (2003) from equation 4-2 below:

$$D = \frac{4M}{\pi d^2 h} \tag{4.2}$$

Where:

D is the bulk density of compacted specimen,

M is the mass of specimen in air,

d is the diameter of specimen, and

h is the average height of specimen.

To verify the volumetric properties of the mixtures, Theoretical Maximum Densities (TMD) for the six mixes have been calculated following the test method specified in British standard BS EN 12697-5 (2002). Trial and error tests on some extra number of samples were made to achieve the above mentioned level of air voids for each mix and were verified by measuring the bulk specific gravity of each sample and calculating the air voids after compaction and cooling. Air voids percentages were calculated using the formula (BS EN 12697-8, 2003):

$$V_v = 100 \times (1 - G_{mb}/G_{mm}) \tag{4.3}$$

Where:

V_v is the air voids content of the mixture, in 0.1 percent (v/v),

G_{mb} is the bulk density of the compacted sample and

G_{mm} is the Theoretical Maximum Density for loose sample of the same mix.

Table (4-7) and Figure (4-9) show the average volumetric properties of all the mixtures used in this part of the research project for the 100 x 63.5mm sized specimens, and total volumetric analysis are shown in Appendix E (Tables E 4-1 and E 4-2).

Mix Type / Densities & % Air Voids	Arcow	Uncoated Croxden	Coated Croxden
Gmb for mixes with (100/150 pen)	2.137	2.162	2.034
Gmm for mixes with (100/150 pen)	2.660	2.561	2.540
%Air Voids	19.7	15.6	19.9
Gmb for mixes with (40/60 pen)	2.12 6	2.140	2.014
Gmm for mixes with (40/60 pen)	2.652	2.558	2.529
%Air Voids	19.9	16.3	20.3

Table 4 -7: Volumetric Properties of 100 x 63.5mm Specimens

After compaction the specimens were not immediately extracted from the mould. They were allowed to cool down for about 1.5 hours before extruding the specimens, and that because it was noticed that these mixes collapse when they were extruded immediately after compaction, due to the high air void contents especially the uncoated Croxden mixes. Interestingly, the compaction results of the samples have confirmed that the compaction of Croxden mix is so easy due to the surface smoothness (see section 3.1.3), and this is why a lower percentage of air voids were achieved.

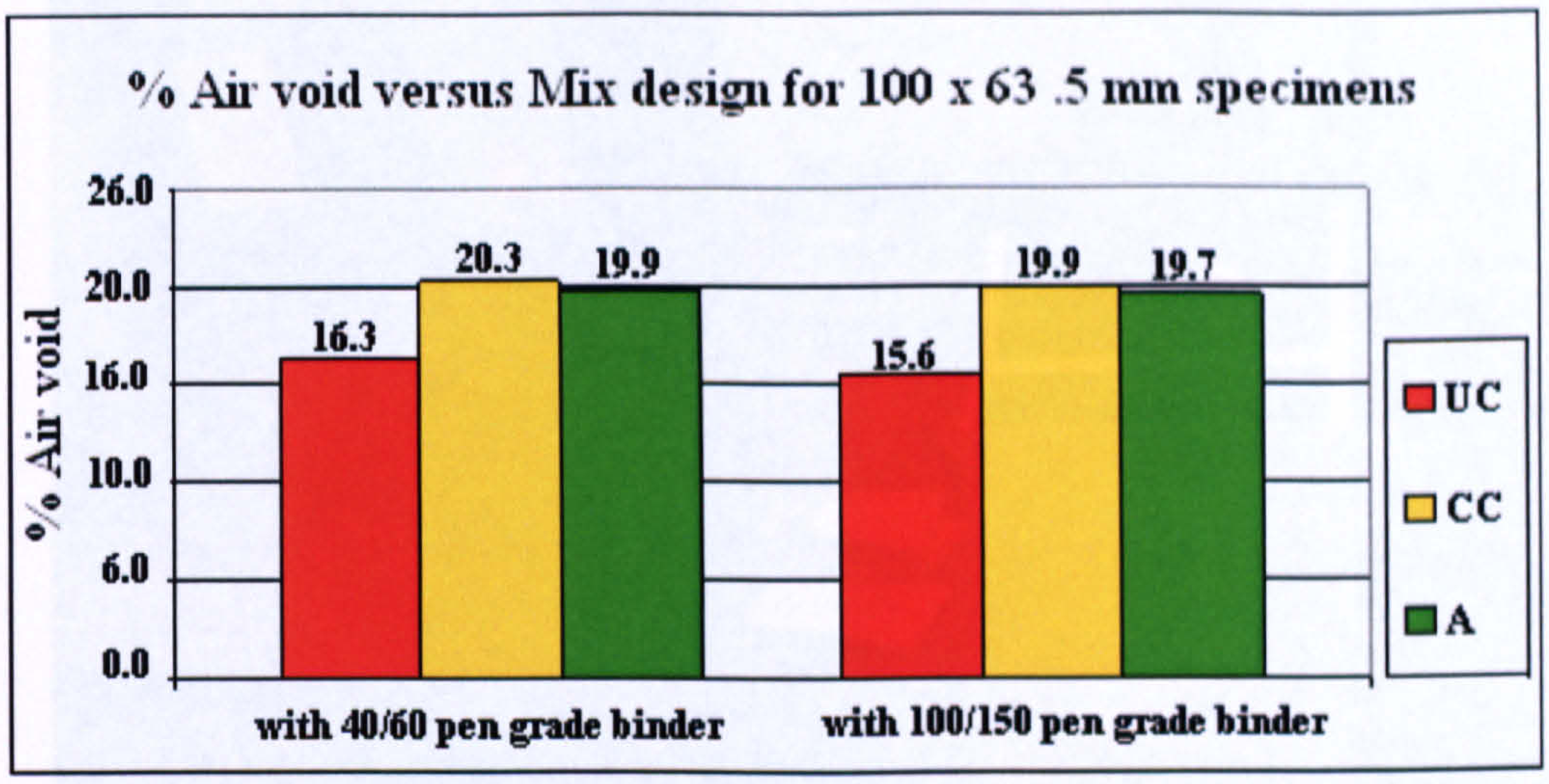


Fig. 4-9: Percentage air void versus mix design with 40/60 & 100/150 pen grade binders respectively

4.6.6 Specimens Preparation for the Wheel tracking tests

As mentioned earlier, two types of samples were produced for the wheel tracking tests:

1. 305 x 305 x 50mm specimens for the Wheel Tracking test (Small Scale Device), and
2. 500 x 180 x 50mm specimens for the Wheel Tracking test (Large Scale Device).

Square slabs of (305 x 305 x 50mm) were manufactured for the small scale Wheel Tracking Test, while rectangular slabs of (500 x 180 x 50mm) were manufactured for the large scale Wheel Tracking test (BS EN 12697-22, 2003). Compaction based on final specimen height resulted in more uniform air void values for the test specimens, so this method was used for compaction, using a vibrating compactor (Plate 4.1). This is a compactor that is capable of compressing a specimen by applying a vibratory compaction on the mix until the specimen designated thickness is reached. This was achieved after some modification to the machine. The rectangular moulds were heated to the compaction temperature before pouring the loose mix into it, to be ready for compaction.

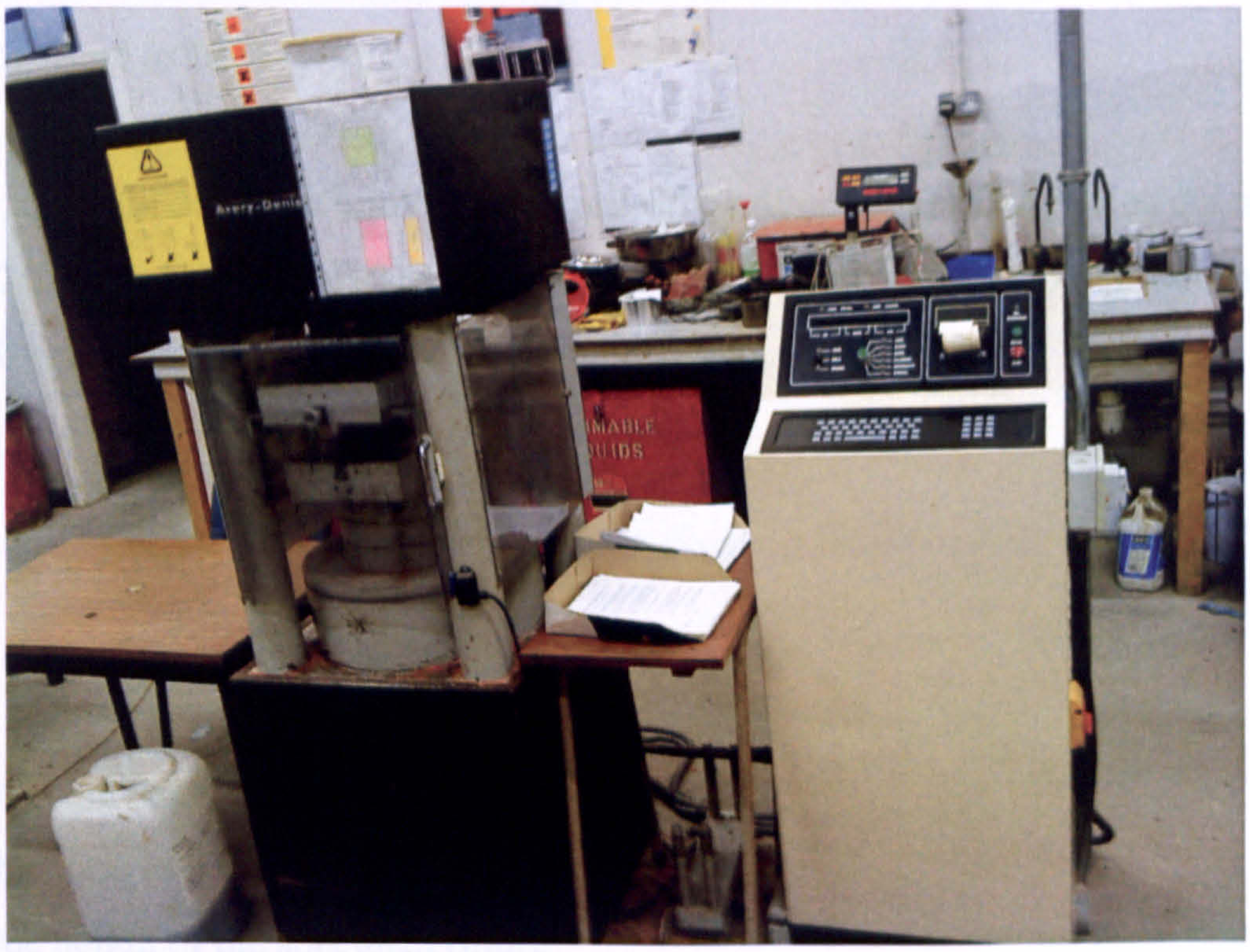


Plate 4-1: Square Shaped Samples' Compactor

Loads with a 26-60 Hz frequency was used to apply vibrating compaction to slab samples through a specially designed and modified compaction plates and moulds. The utilized compactor was modified to apply the required vibration. The use of a vibrating compactor is not new and has been shown to give satisfactory results (Al Nageim, 1989).

4.6.6.1 Volumetric Properties

4.6.6.1.1 Bulk Density of compacted specimens (Gmb)

As the mixtures are to have void contents of about 20% (v/v), the determination of bulk density of compacted specimens according to procedure A in BS EN 12697- 6 (2003) cannot be reliably conducted on the OGFC mixtures because of this high air void contents, and the determination of Saturated Surface Dry (SSD) weight of the compacted specimens, according to procedure B of the same BS, is not reliable for such mixes having such high air void contents and due to this the specimens have high water absorption levels (Cooley et al, 2002). Procedure C is suitable for measuring the bulk density of compacted specimens with air voids levels up to 10 % (v/v).

Therefore, the bulk density (Gmb) was determined by dimensional analysis, as described in BS EN 12697- 6 (2003) standard, test procedure D[4]. The heights of specimens were measured by calipers:

$$Gmb = \frac{m_1}{h\,x\,l\,x\,w}\,x\,10^6 \tag{4.4}$$

Where:

G_{mb} is the bulk density of the specimen (kg/m^3),

m_1 is the mass of the dry specimen (g),

h is the height of the specimen (mm),

l is the length of the specimen (mm), and

w is the width of the specimen (mm).

4.6.6.1.2 Theoretical Maximum Density (TMD) (Gmm)

TMD is a hypothetical value representing a compacted specimen containing no air voids. The theoretical maximum density of three duplicate specimens having the same gradation and optimum binder contents for each type of mix were tested in accordance with BS EN 12697-5 (2002) standard , and the following equation was implemented:

$$\text{Gmm} = \frac{m_2 - m_1}{1000 x \left(V_P - \dfrac{m_3 - m_2}{\rho_W} \right)} \tag{4.5}$$

Where:

Gmm is the maximum density of the bituminous mixture, to the nearest 0.1,

m_1 is the mass of the pyknometer plus head piece and spring (g),

m_2 is the mass of the pyknometer plus head piece, spring and test sample (g),

[4] Procedure D described in this Standard is suitable for void contents greater than 15 % (v/v).

m_3 is the mass of the pyknometer plus head piece, spring, test sample and water (g),

V_P is the volume of the pyknometer, when filled up to the reference mark (m^3), and

ρ_w is the density of water at test temperature, to the nearest 0.1 kg/m^3.

4.6.6.1.3 Voids in Total Mixture (VTM, Vv)

VTM is the total volume of air voids between the asphalt coated aggregate particles throughout a compacted specimen. It is expressed as the percentage of the bulk volume of the specimen. The voids in compacted specimens were obtained by using the following equation BS EN 12697-8 (2003):

$$Vv\ (\%) = 100 \times (1 - G_{mb}/G_{mm}) \qquad (4.6)$$

Where:

V_v is the air voids content of the mixture, in 0.1 percent (v/v),

G_{mb} is bulk specific gravity of the compacted specimen, and

G_{mm} is maximum theoretical specific gravity of the mixture.

4.6.7 Results of Volumetric Properties

Table (4-9) and Figures (4-10 & 4-11) present the average volumetric properties determined for each specimen, of 500 x 180 x 50mm and 305 x 305 x50mm dimensions, of all the mixes using equations (4.4 - 4.6). It shows together with Figures (4-10) and (4-11) that the addition of cement coating has influenced the bulk density of Croxden aggregates by increasing its surface roughness, and that has resulted in increasing the average porosity of the compacted specimens by 30.9 % compared with Croxden mix.

This is understandable as it is related to the aggregates bulk density before adding the asphalt binder to the mix. The change in the bulk density and subsequent change in the specimens' porosity is credited to the increase in the aggregates angularity. The complete volumetric property data is given in Appendix E (Tables E 4-3 to E 4-6).

Densities & % Air voids \ Mix Type	Arcow	Uncoated Croxden	Coated Croxden
Gmb for mixes with (100/150 pen)	1.995	2.014	1.780
Gmm for mixes with (100/150 pen)	2.524	2.321	2.241
% Air Voids	21	13.2	20.6
Gmb for mixes with (40/60 pen)	1.997	2.022	1.780
Gmm for mixes with (40/60 pen)	2.522	2.319	2.238
% Air Voids	20.8	12.8	20.5

Table 4 -9: Volumetric Properties for (500 x 180 x 50) mm & (305 x 305 x 50) mm Samples

This observation was noticed by Daoud et al (1982), who stated that: *"The higher void content obviously indicates a reduction in compactability of these mixes and is expected consequently to result in lower stabilities. However, it seems that the gain in mix stability due to enhancement of aggregate texture, (by pre-coating with cement), is more than the loss due to the lower density and higher voids of the resulting less-compactable mix"* . Daoud work had been based on mixing cement with all aggregate fractions as can be seen in Table (4-10).

Type and Degree of Treatment	Aggregate Proportions (%)					Properties of Compacted Mixtures			
	Aggregate Nominal size (mm)		Sand		Limestone Dust	Optimum Binder (%)	Marshall Stability Ratio (%)	Bulk Density (g/cm³)	Void Content
	19	9.5	Crashed	Natural					
Untreated	28	20	30	14.5	7.5	4	100	2.37	3.1
Hydrated Lime, percent	28	20	30	14.5	7.5	4.25	109	2.36	3.1
Cement									
Light	28	20	30	14.5	7.5	4.5	104	2.4	3.4
Medium	28	20	30	14.5	7.5	4.5	109	2.36	4.2
Heavy	28	20	30	14.5	7.5	4.5	124	2.35	4.9
Light	30	22.5	27	14	6.5	4.5	92	2.33	5
Medium	30	20	27.5	15	7.5	4.25	111	2.35	4.7
Heavy	28	20	30	14.5	7.5	4.5	124	2.35	4.9

Table 4-10: Daoud's Test Results **(Daoud et al, 1982)**

From Table (4-10) that shows part of Daoud's results, it can be noticed that increasing the amount of cement coating for the cement treated mixes had reduced the bulk density and increased porosity of the compacted specimens. The untreated mix had shown lower bulk density because of its reduced bitumen content. This observation by Daoud coincides with these research findings.

This change in the air void values for each type of mixtures will subsequently influence the rutting potential of a mixture, as it is a function of its air voids content i.e. air voids increase, rut depths also increase. Likewise, as test temperature increases, rut depths increase. However, two mixtures with the same average percent voids may have a different distribution of air voids. Intuitively, these two mixtures would be expected to respond differently under loading and yield distinct mechanical properties in laboratory testing. Furthermore, voids are not spherical within the same sample; therefore, a cross-section in one direction will show a different air void distribution than in another direction (Oda, 1972).

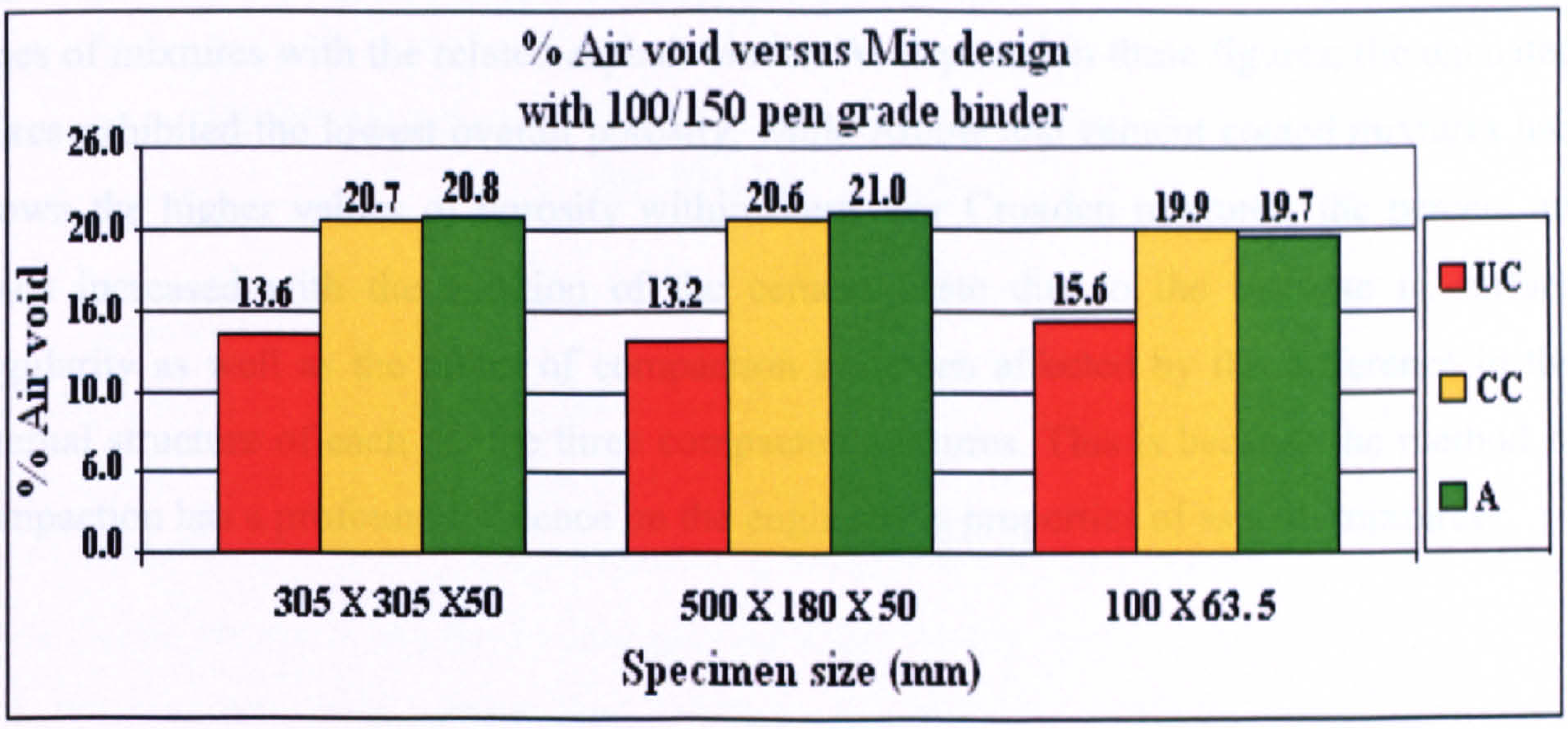


Fig. 4-10: Overall average volumetric properties for mixes with 100/150 pen

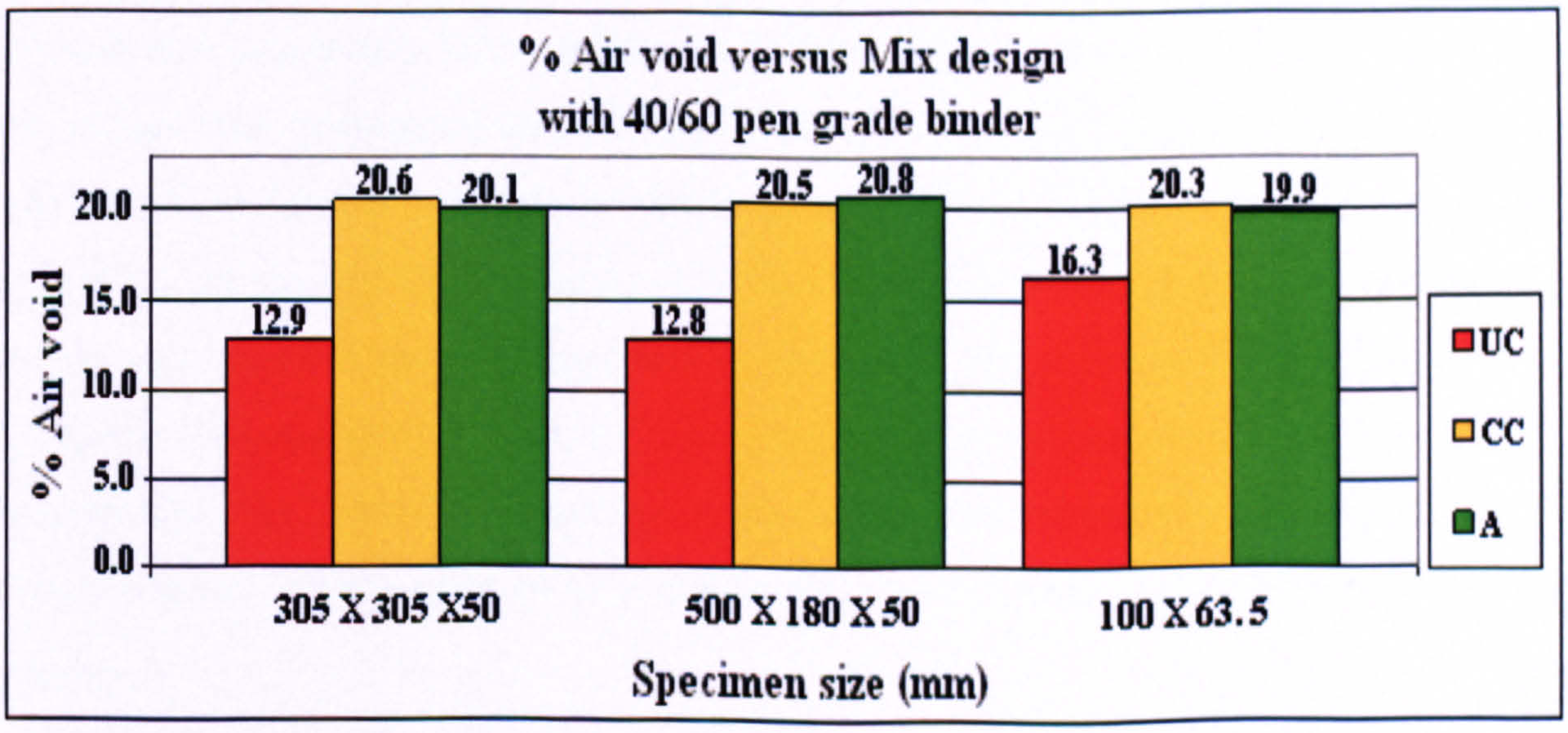


Fig. 4-11: Overall average volumetric properties for mixes with 40/60 pen

Figures (4-10) and Figure (4-11) summarize the air void percent vs. mix design for all three types of mixtures with the related asphalt binder. As depicted in these figures, the uncoated mixes exhibited the lowest overall porosity, while Arcow and cement coated mixtures had shown the higher values of porosity within them. For Croxden mixtures, the percent air voids increased with the addition of the cement paste due to the increase in surface angularity as well as the effect of compaction had been affected by the difference in the internal structure of each of the three compacted mixtures. This is because the method of compaction has a profound influence on the engineering properties of asphalt mixtures.

CHAPTER 5

Determination of Indirect Tensile Stiffness Modulus for OGFC mixes

5.1 Introduction

Asphalt material's resilient modulus is corresponding to Young's modulus of elasticity for linear elastic materials. By their nature, paving materials are not elastic, which means that they definitely experience some permanent deformation after each load cycle. The strain in visco-elastic materials can be divided into the elastic strain, also named the resilient strain, and the viscous strain. After load removal the resilient strain is recoverable. The determination of mixture stiffness is essential to the assessment and design of asphalt pavements, and mixture stiffness is necessary to evaluate the distribution of stresses and strains in asphalt pavements as well as to evaluate pavement damage. Additionally, mixture stiffness is increasingly being used in pavement design as an indicator of mixture quality (Kandhal et al, 2000).

5.2 The Resilient Modulus

As mentioned previously, HMA is known to be a visco-elastic material and, therefore it may experiences permanent deformation (rutting) after each application of the load. However, if the load is small, in comparison to the strength of the material, and after a relatively large number of repetitions (100 to 150 load repetitions), the deformation after the load application is almost completely recoverable. The deformation is proportional to the applied load and since it is nearly completely recovered it can be considered as elastic (Al-Qadi and Yoo, 2006). The resilient modulus is based on the recoverable strain under repeated loading of the pavement or a sample and is determined as follows (Ghaffarpour et al, 2009):

$$M_r = \frac{\sigma_d}{\varepsilon_r} \tag{5.1}$$

Where:

σ_d is the deviator stress and

ε_r is the recoverable resilient strain.

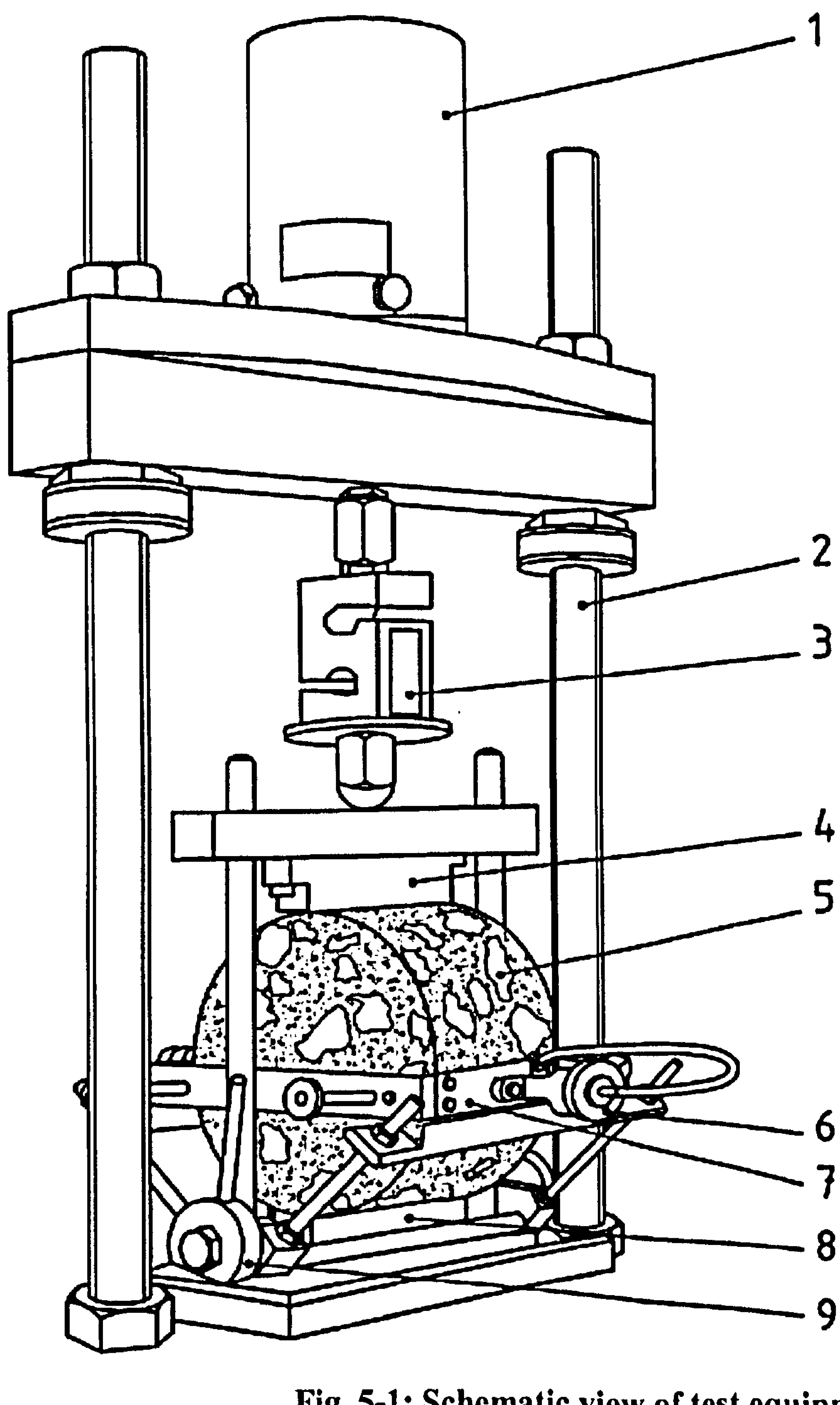
The stiffness calculated for samples prepared in a laboratory is commonly used as an input to structural analysis models to forecast pavement reaction under simulated traffic loads. Measurements of stiffness moduli are considered to be properties of a linear visco-elastic material. However, under field loading conditions, asphalt mixes may experience considerable plastic deformation and displays a non-linear elastic-visco-plastic response.

The laboratory-determined resilient modulus of the HMA depends on the following parameters:

- Resilient modulus test setup used,
- Method of compaction,
- Level of compaction,
- Temperature,
- Load level, duration, and rest period,
- Specimen size and geometry,
- Data analysis procedure, and
- Volumetric properties of the mix.

5.3 The Repeat Load Indirect Tensile test (RLIT)

The Cooper Research Technology testing device (type HYD 25) (Plate 5-1) was used to evaluate the stiffness modulus for the three types of mixes. After the samples were prepared as mentioned in chapter four, they were stored at 5^0C until testing time, then the samples needed to be tested were placed in the Cooper's device, which is a controlled temperature cabinet and brought to the specified test temperature. The samples were placed into the loading apparatus; and the loading strips were positioned in a parallel format and centered on the vertical diametric plane (Fig. 5-1).



Key
1 Pneumatic load actuator
2 Steel load frame
3 Load cell
4 Upper loading platen
5 Test specimen
6 LVDT adjuster
7 LVDT mounting frame
8 Lower loading platen
9 LVDT alignment jig

Fig. 5-1: Schematic view of test equipment (BS EN 12697-26, 2004)

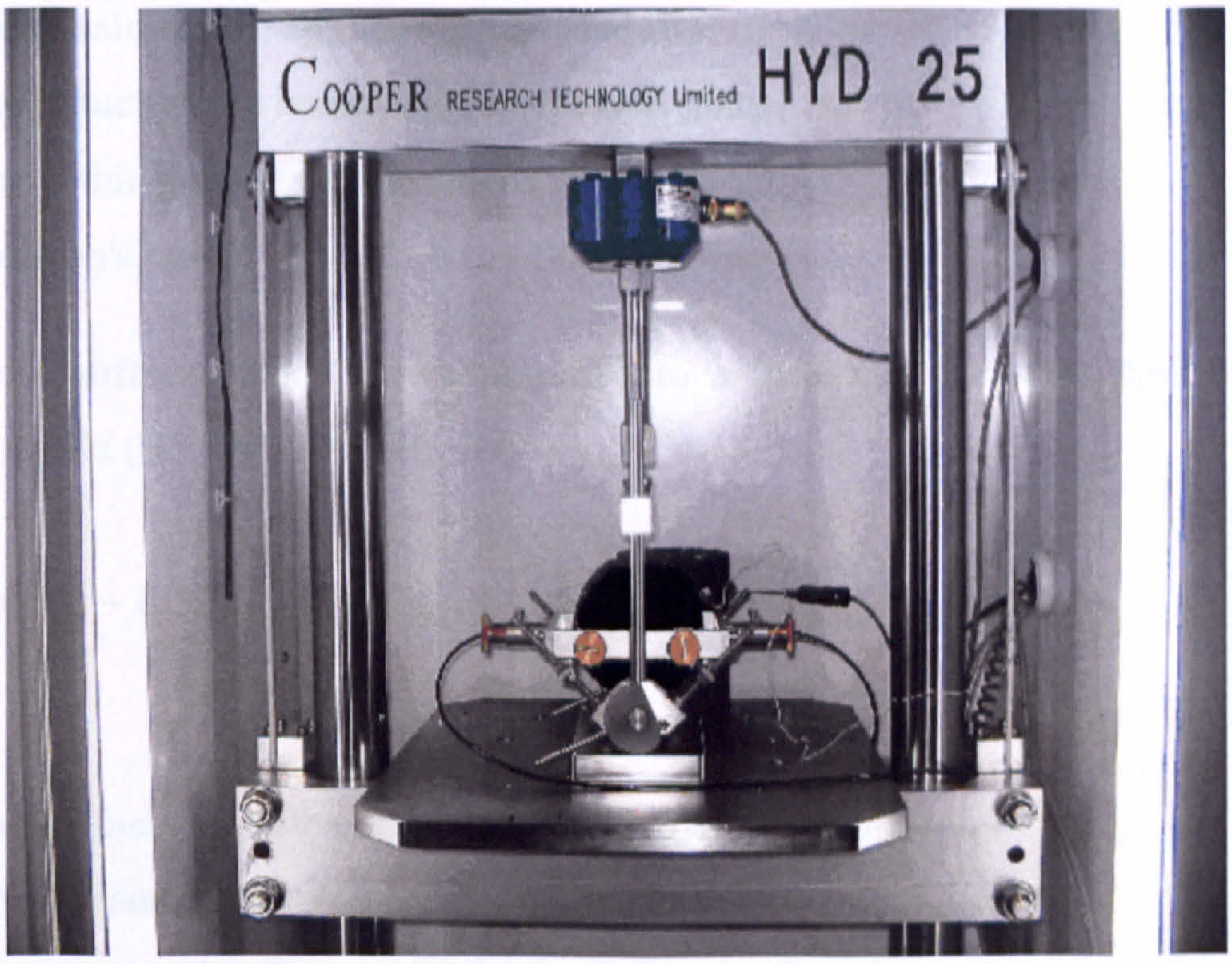


Plate 5-1: The HYD 25 Cooper's machine utilized for ITSM and RLAT tests

Tests were performed at temperatures of 10, 20, and 30°C at 1.0 Hz frequency. Testing began with the lowest temperature and continued to the highest temperature for each set of samples respectively.

For each test temperature, each sample was tested twice; following the first stiffness modulus determination, the sample was left for 20 minutes inside the temperature controlled cabinet, to allow the sample's residual strain to gradually fade away, then it was tested again on a diametric plane 90° to the first testing position. The measured resilient modulus was calculated automatically via computer software following the formula (BS EN 12697-26, 2004):

$$S_m = \frac{F \times (v + 0.27)}{z \times h} \qquad (5.2)$$

Where:

S_m is the measured stiffness modulus (MPa)

F is the peak value of the applied vertical load (N)

z is the amplitude of the horizontal deformation (mm)

h is the mean thickness of the specimen (mm), and

ν is the Poisson's ratio.

The measured stiffness modulus was adjusted to a load area factor of 0.60 using the following formula (BS EN 12697-26, 2004):

$$S'_m = S_m \times (1 - 0.322 \times (\log(S_m) - 1.82) \times (0.6 - k)) \tag{5.3}$$

Where:

S'_m is the stiffness modulus, (MPa), adjusted to a load area factor of 0.60,

k is the measured load area factor,

S_m is the measured stiffness modulus in (MPa) at a load factor k, expressed in degrees Celsius (°C).

Typical load and deformation outputs that form a resilient modulus test are shown in (Fig. 5-2).

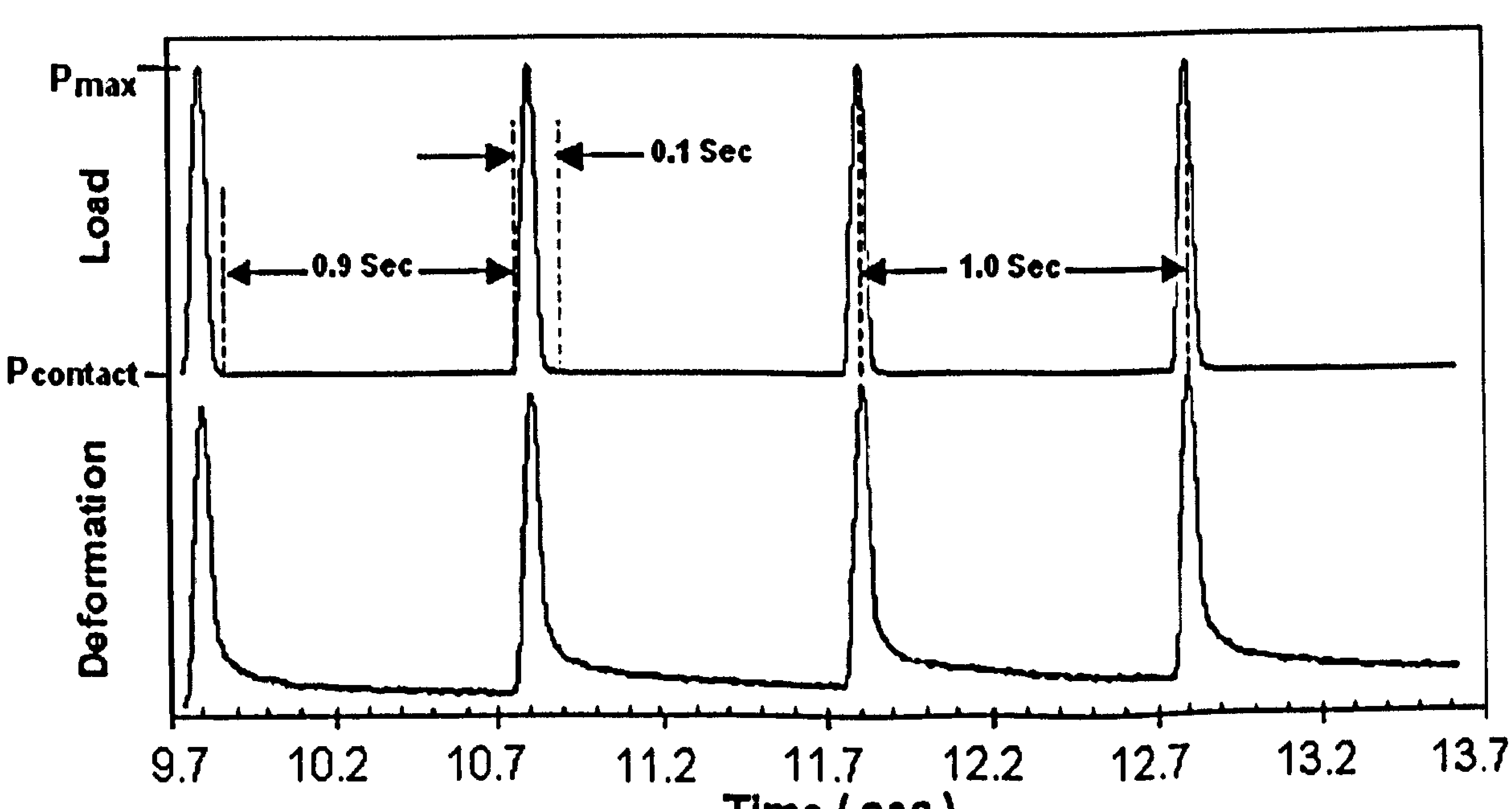
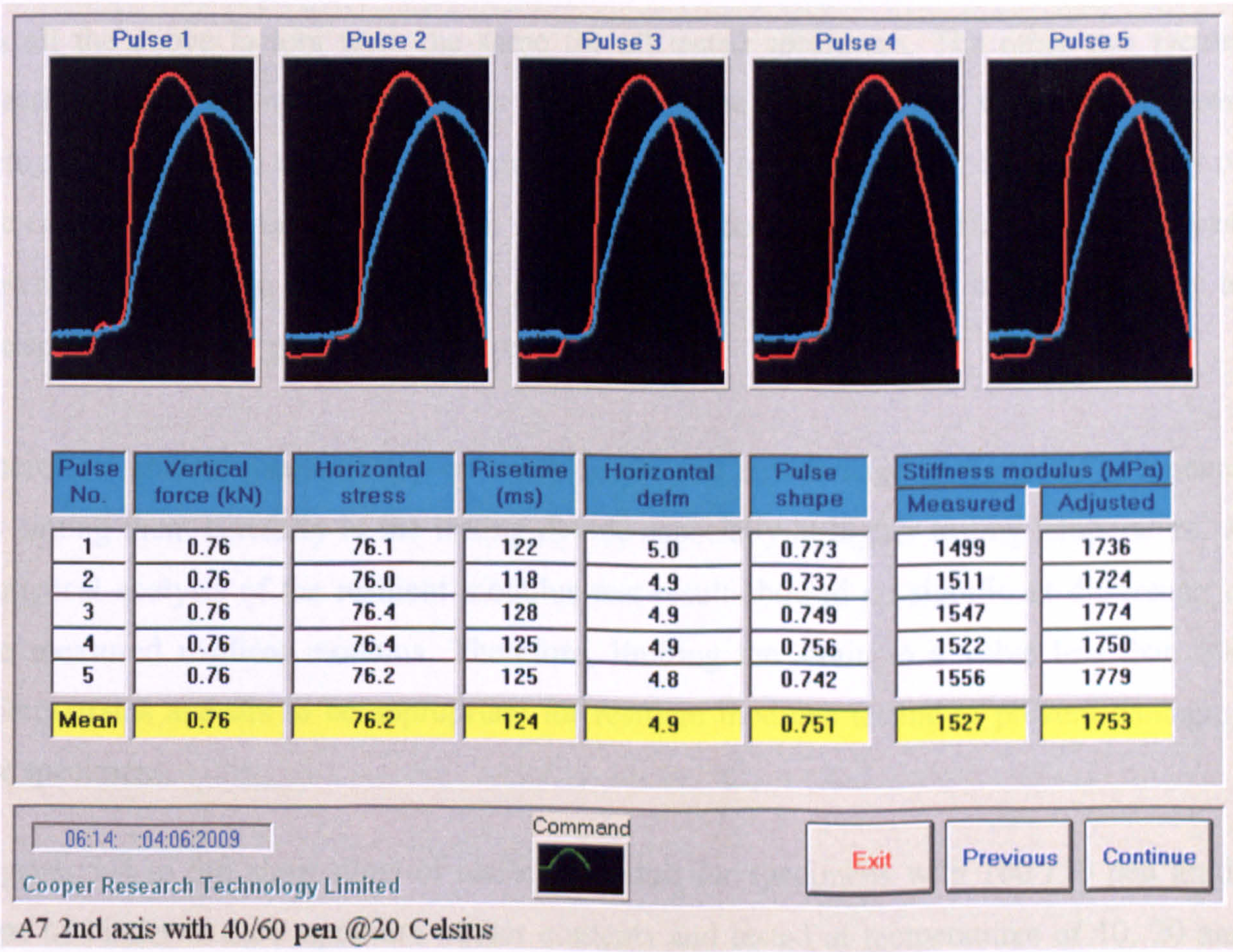


Fig. 5-2: Load and Deformation Graph of a Typical Resilient Modulus Test

Testing started by adjusting the two linear variable displacement transformers (LVDT) that are just touching the sides of the tested sample to zero reading, and a minimal contact load was applied to the sample. Each stress cycle was made up of a 0.1 second haversine pulse followed by a 0.9 second hold cycle to simulate a moving wheel load. After the entire cycle of testing was completed at 10ºC, the environmental chamber was then set to the next temperature. After two hours of conditioning, as it was found to be adequate for this type of OGFC mix to reach the new required testing temperature, the above steps were repeated until the entire sequence of temperatures was completed. The test was conducted based on the BS EN 12697-26 (2004) testing procedures. Figure (5-3) shows a proto type of data out put for the RLIT test generated by the utilized computer software.



Pulse No.	Vertical force (kN)	Horizontal stress	Risetime (ms)	Horizontal defm	Pulse shape	Stiffness modulus (MPa) Measured	Adjusted
1	0.76	76.1	122	5.0	0.773	1499	1736
2	0.76	76.0	118	4.9	0.737	1511	1724
3	0.76	76.4	128	4.9	0.749	1547	1774
4	0.76	76.4	125	4.9	0.756	1522	1750
5	0.76	76.2	125	4.8	0.742	1556	1779
Mean	0.76	76.2	124	4.9	0.751	1527	1753

Fig. 5-3: Computer software's Prototype of Data Output for RLIT Test

5.4 RLIT Test Results and Discussion

As stated above, factors affecting the resilient modulus are:

- Resilient modulus test setup used,
- Method of compaction,
- Level of compaction,
- Load level, duration, and rest period,
- Specimen size and geometry,
- Data analysis procedure,
- Temperature and
- Volumetric properties of a mix.

As all the above factors were the same for all tested specimens. The other two factors greatly affected the resilient modulus values were the temperature at which the test was being conducted and the volumetric properties of the mixes especially the bulk density of the compacted specimens. Since each specimen was tested at three different temperatures, one major concern in determining the stiffness moduli was to prevent inducing damage to the specimens at any given temperature.

Therefore, upmost precautions were made to prevent any damage to the tested specimens by setting them carefully in the testing device especially at higher testing temperature. A statistical analysis of the resilient modulus test result showed no significant difference in the measured resilient modulus. Therefore, limiting the strain to a value less than 500 micro-strains appears to be appropriate for resilient modulus testing to prevent damaging the specimens.

Figures 5-4 to 5-6 show plots of resilient moduli for specimens with 100/150 pen grade type of binder at their optimum binder contents and tested at temperatures of 10, 20 and 30^0C respectively. As can be seen from these figures the lower the temperature the higher the moduli.

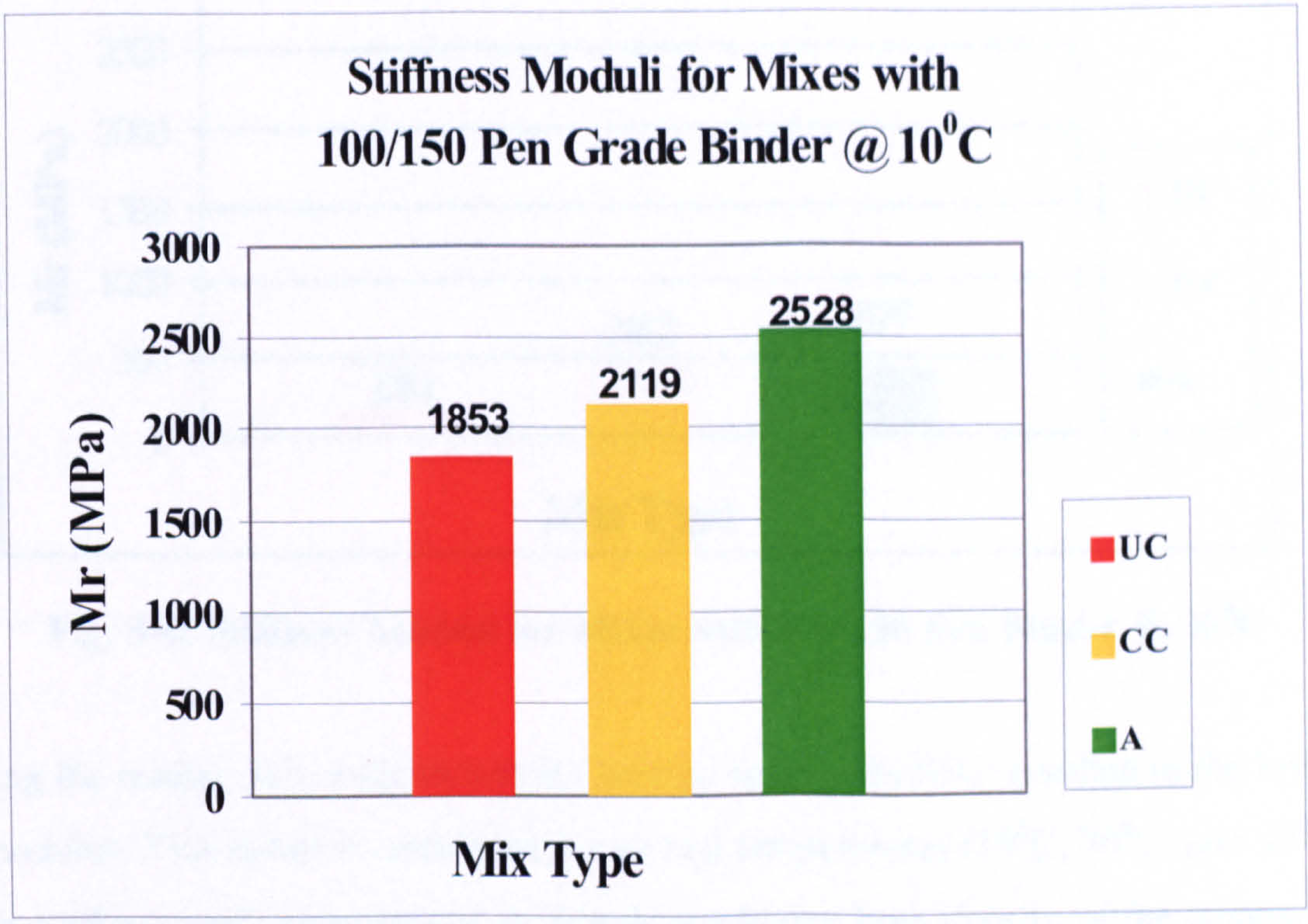


Fig. 5-4: Stiffness Moduli for Mixes with 100/150 Pen Binder @ 10^{0}C

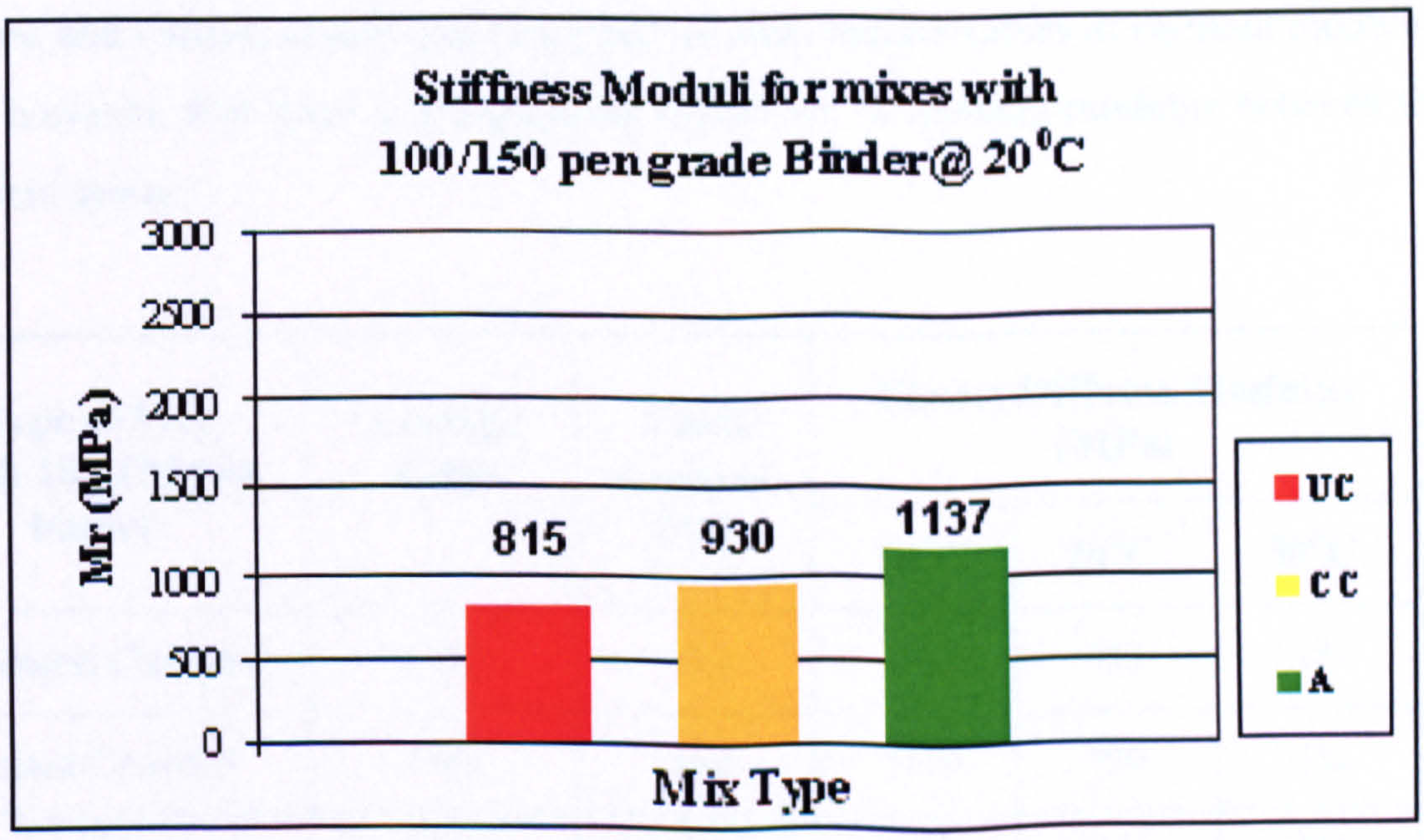


Fig. 5-5: Stiffness Moduli for Mixes with 100/150 Pen Binder @ 20^{0}C

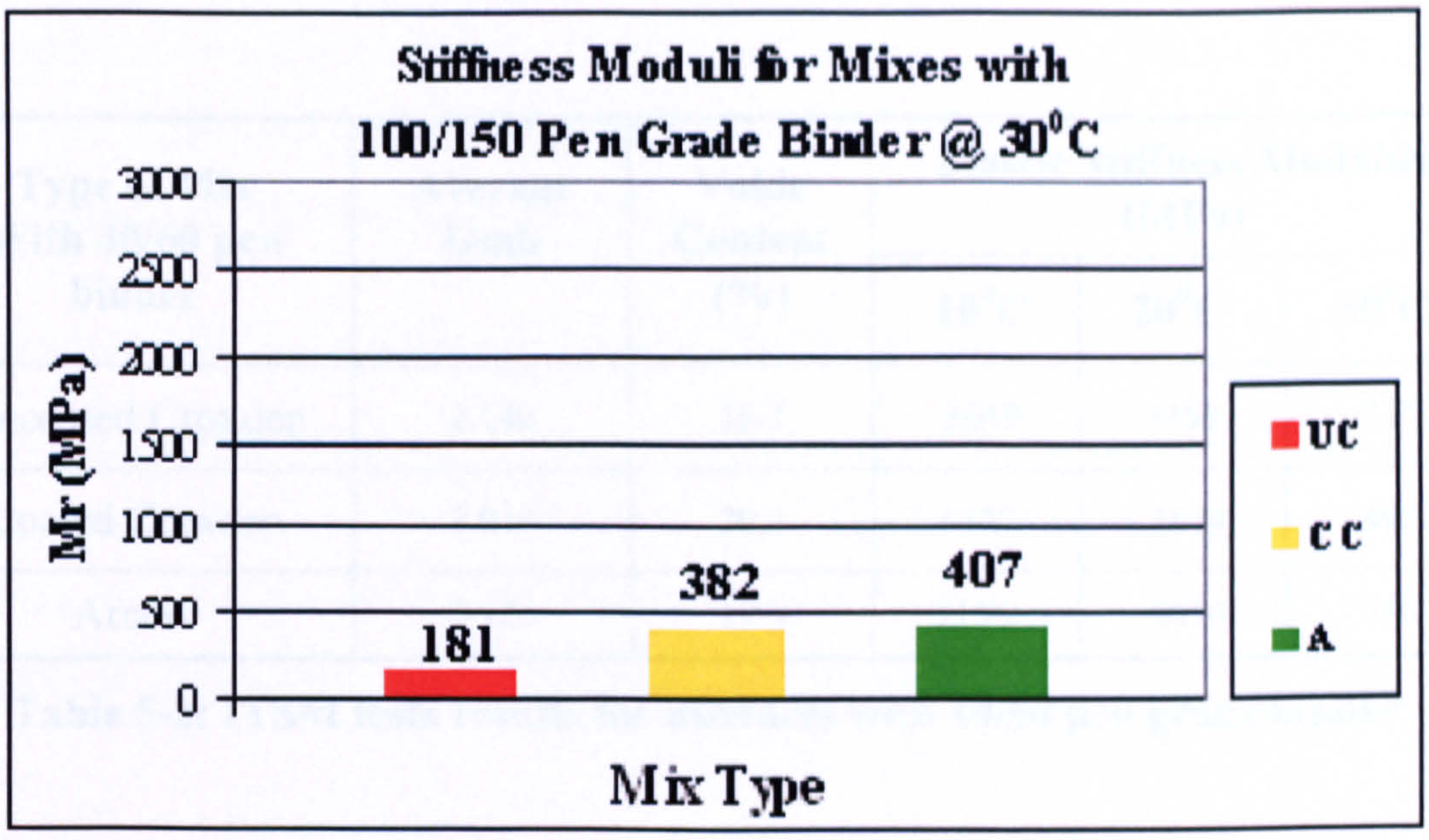


Fig. 5-6: Stiffness Moduli for Mixes with 100/150 Pen Binder @ 30^{0}C

In reviewing the results, Mix with uncoated Croxden aggregates (UC) resulted in the lowest resilient modulus. This result is consistent for all test temperatures (10^{0}C, 20^{0}C, and 30^{0}C), and is in accordance with expectations as despite the higher bulk density of the compacted specimens were having higher values than the other two mixes (see Table 5-1), the aggregate smooth surface facilitates larger strain i.e. deformation of the samples tested. On the other hand, Arcow mix (A) had the heights resilient modulus at all three test temperatures, and Cement coated mix (CC) had an intermediate values of resilient modulus. It is clear, however, that there is a significant difference in resilient modulus between the three different mixes.

Type of Mix With 100/150 pen binder	Average Gmb	Voids Content (%)	Elastic Stiffness Modulus (MPa)		
			10^{0}C	20^{0}C	30^{0}C
Uncoated Croxden	2.162	15.6	1853	815	181
Coated Croxden	2.034	19.9	2119	930	382
Arcow	2.137	19.7	2528	1137	407

Table 5-1: ITSM tests results for mixtures with 100/150 pen grade binder

Type of Mix With 40/60 pen binder	Average Gmb	Voids Content (%)	Elastic Stiffness Modulus (MPa)		
			10°C	20°C	30°C
Uncoated Croxden	2.140	16.3	3649	1199	335
Coated Croxden	2.014	20.3	4475	1674	495
Arcow	2.126	19.9	5156	2074	516

Table 5-2: ITSM tests results for mixtures with 40/60 pen grade binder

The difference in measured resilient modulus among (A), (CC) and (UC) specimens is believed to be due to the difference in volumetric properties , aggregate surface roughness as well as to the presence of the cement coating (Tables 5-1 & 5-2) which enhance binder aggregate adhesion (Al Nageim and Robinson, 2006). These findings go along with the finding of Baladi et al (1988) as they performed regression analyses to evaluate the relationship between the measured resilient modulus and mix parameters such as air voids, aggregate angularity, binder viscosity, and gradation. In their study, they reported that the resilient modulus would be affected by aggregate angularity, air voids, and binder viscosity with air voids exerting the greatest influence. It was also observed that increasing aggregate angularity and higher binder viscosities increased the amounts of the resilient moduli, and this has coincided with the current research findings as can be seen in the above table (Table 5-2).

(Mamlouk and Gemayel, 1988) had also presented a study that investigated some engineering properties of open-graded friction course (OGFC). They found that the tensile strength and resilient modulus of open-graded mixes were about 50% of the value of normal dense-graded asphalt mixes.

Moreover, the mixes studied (open-graded material) was found to be extremely sensitive to temperature (Figures 5-7). At high temperatures, these materials exhibited considerable decrease in resilient modulus (Figures 5-8 to 5-10). This behaviour in the opinion of the author is micro-mechanical in nature and is related to mix gradation under investigation, aggregate type, aggregate contact behaviour and to the viscosity of the binding asphalt material, which agree with the findings of Mamlouk and Gemayel (1988).

5.5 Influence of Temperature on Resilient Modulus

As stated in section 5.4 above, the resilient modulus was found to be highly dependent on test temperature. Correlation between the resilient modulus and the test temperature, with the resilient modulus decreasing as temperature increases, is found to be highest and best represented in an exponential form as follows (also see Figures 5-7 and 5-10):

$$M_r = \alpha e^{-\beta T} \tag{5.4}$$

Where α and β are parameters that are functions of mix properties and asphalt binder. Using Equation (5.4) the fitted exponential relationship between the resilient modulus and temperature incorporated two different coefficients, α and β, which varied with the mix type. Although temperature was found to be a major parameter affecting the HMA resilient modulus, other parameters could also play a role affecting the resilient modulus of OGFC mixes. These identified parameters can be mix properties, mainly bulk density, air voids, asphalt binder content and percentage of air voids in total mix of compacted specimens.

Figures (5-7 and 5-10) show the resilient modulus variation with temperature for 100/150 and 40/60 pen graded binders respectively for the tested mixtures. For 100/150 pen graded binder the following findings were recorded:

1. At 20^0C, UC mixes have lost 56% of its original average stiffness value, and at 30^0C it lost 90.2% of its original average value, compared with their results at 10^0C.
2. For CC mixes, average loss of 56.1% was recorded as temperature changes from 10^0C to 20^0C, and 82% were recorded when temperature changes from 10^0C to 30^0C respectively, and
3. For Arcow mixes, lost 55% as temperature changes from 10^0C to 20^0C, and 83.9% were recorded when testing temperature was raised from 10^0C to 30^0C.

On the other hand, for 40/60 pen graded binder the following findings were recorded:

1. At 20^0C, UC mixes have lost 67.1% of its original average stiffness value, and at 30^0C it lost 90.8% of its original average value, compared with their results at 10^0C.
2. For CC mixes, average loss of 62.5% was recorded as temperature changes from 10^0C to 20^0C, and 88.9% was recorded when temperature changes from 10^0C to 30^0C respectively, and
3. For Arcow mixes losses of 59.8%, as temperature changes from 10^0C to 20^0C, and 90% were recorded when testing temperature was raised from 10^0C to 30^0C.

These results show that Coating Croxden with Cement paste yielded improvements in stiffness average value of 111%, 14.1% and 14.4% at 30^0C, 20^0C and 10^0C respectively when using 100/150 pen graded binder, but with a reduction in bulk density of 5.9% (128 kg/m^3). When comparing these results, it can be seen that the anticipated improvement in stiffness with density occurs as expected, as the stiffness has increased with decreasing bulk density of the mixture.

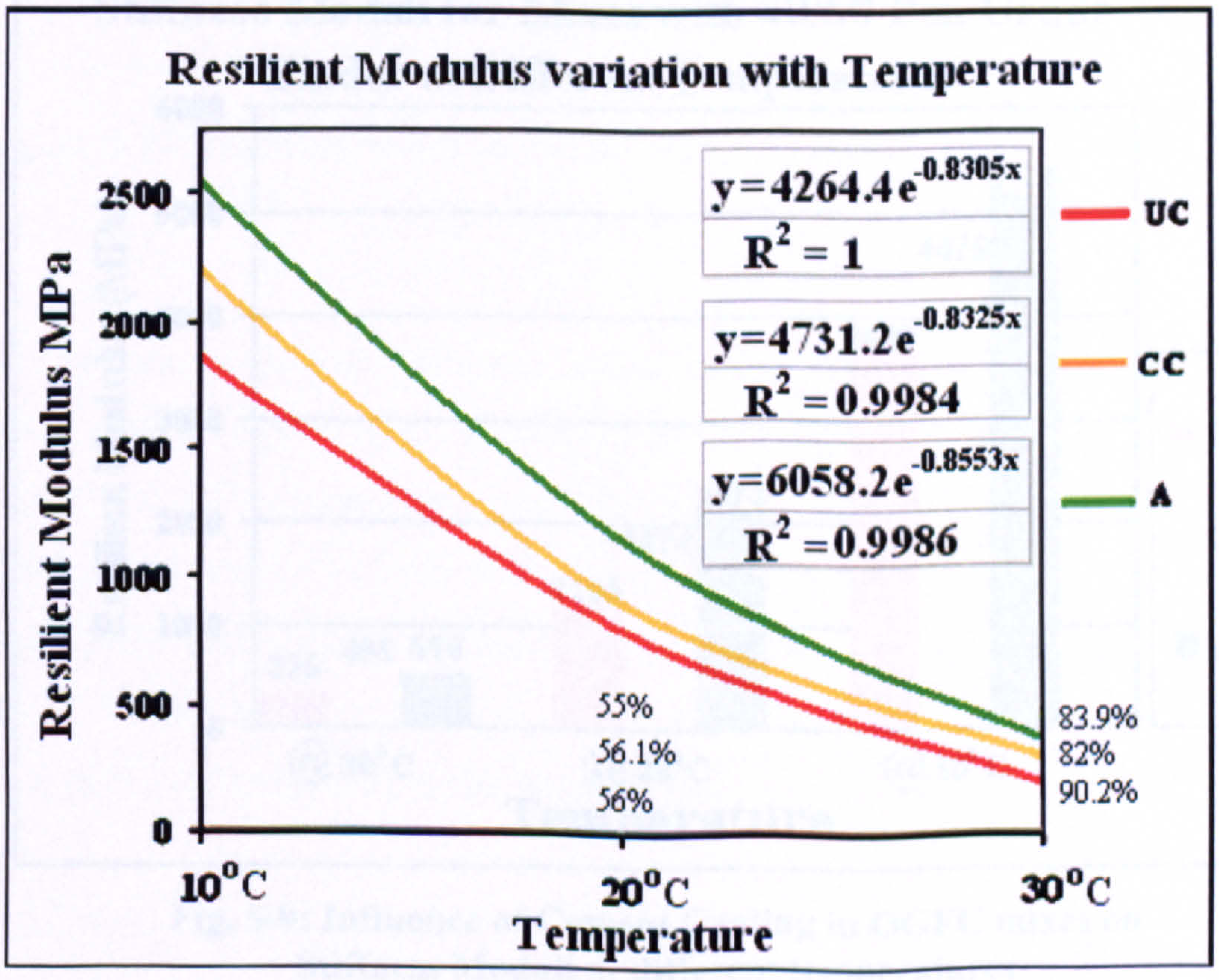


Figure 5-7: Resilient Modulus Variation with Temperature for 100/150

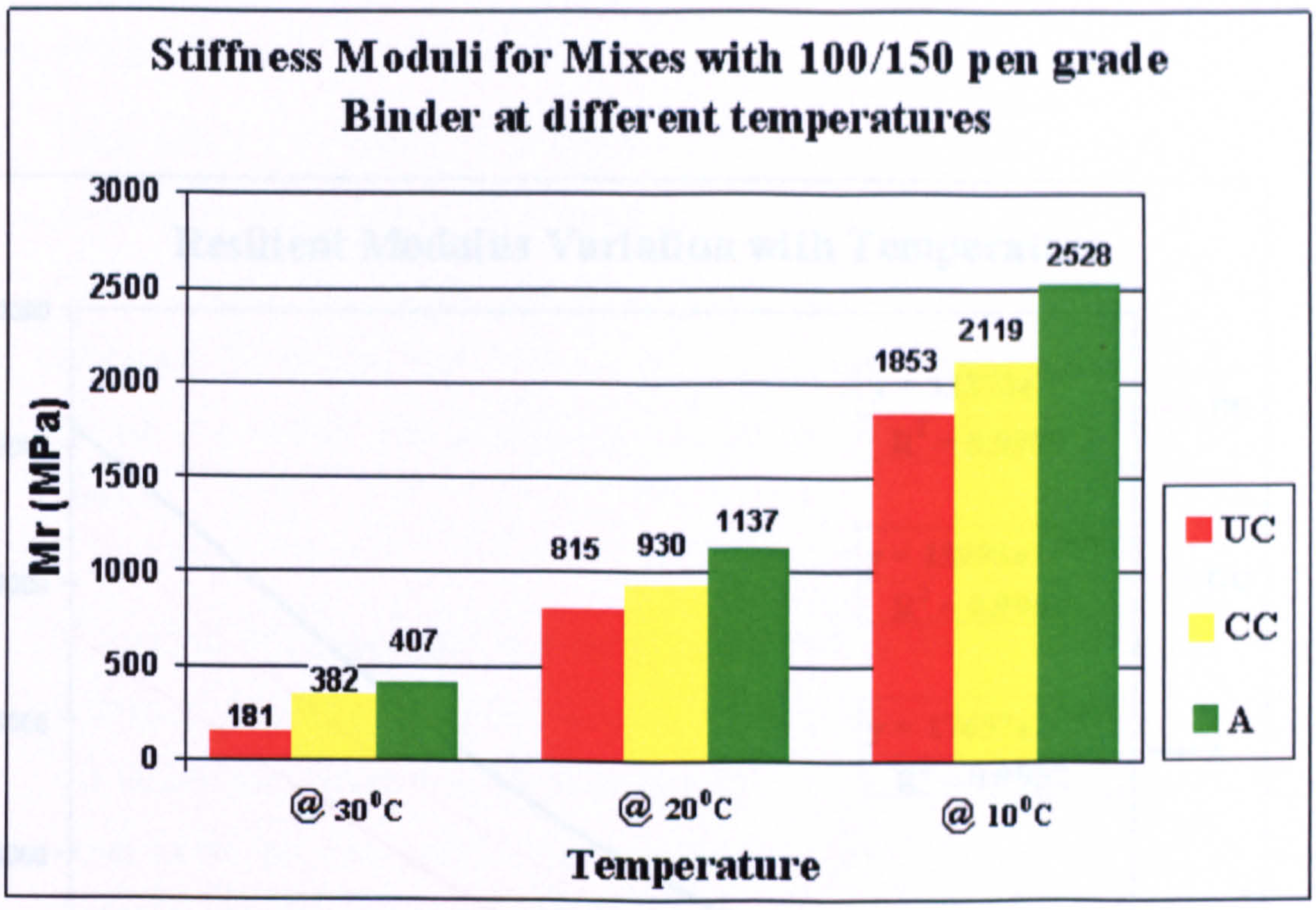


Fig. 5-8: Influence of Cement Coating in OGFC mixes on Stiffness Moduli at different temperatures

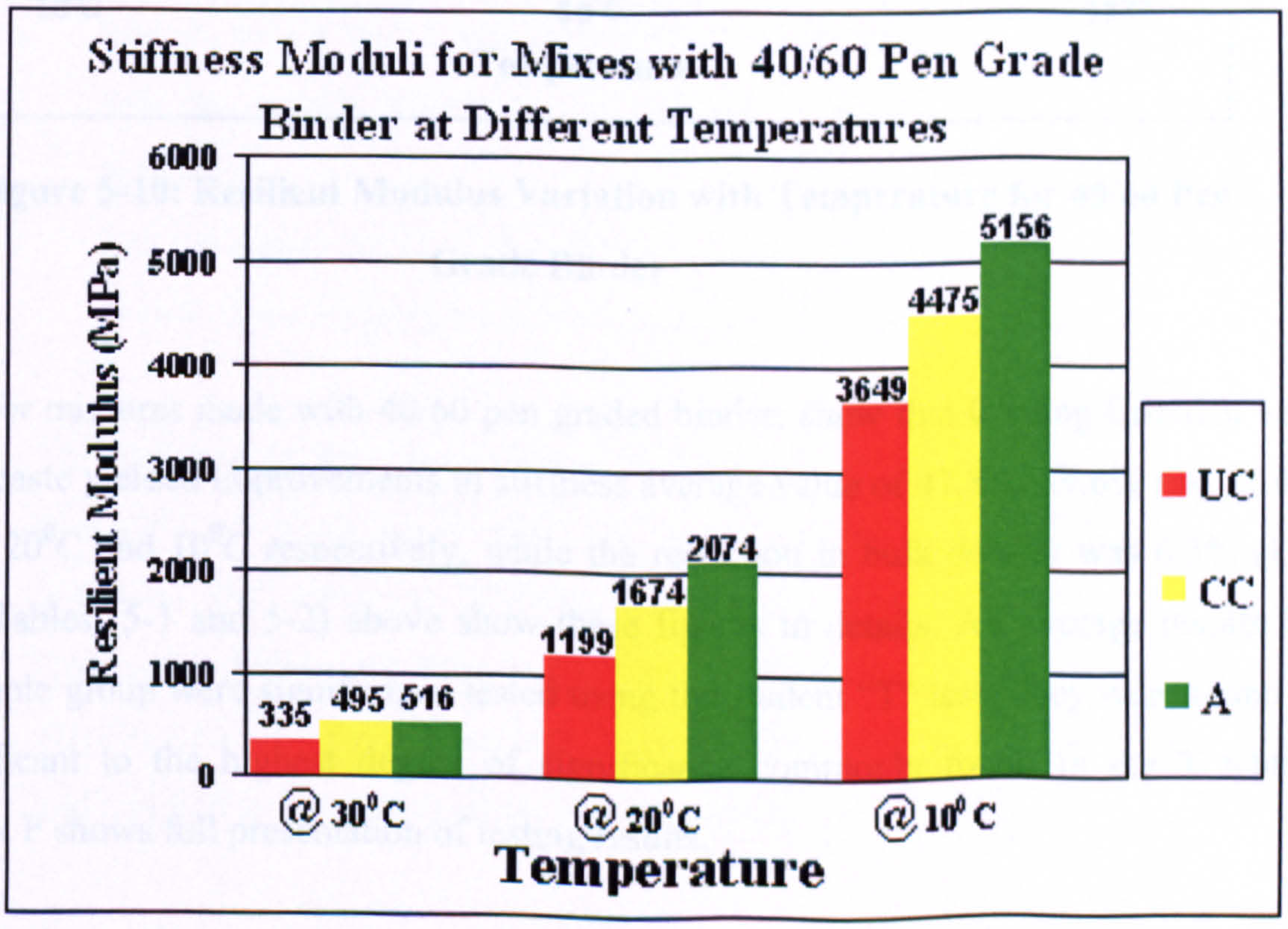


Fig. 5-9: Influence of Cement Coating in OGFC mixes on Stiffness Moduli at different temperatures

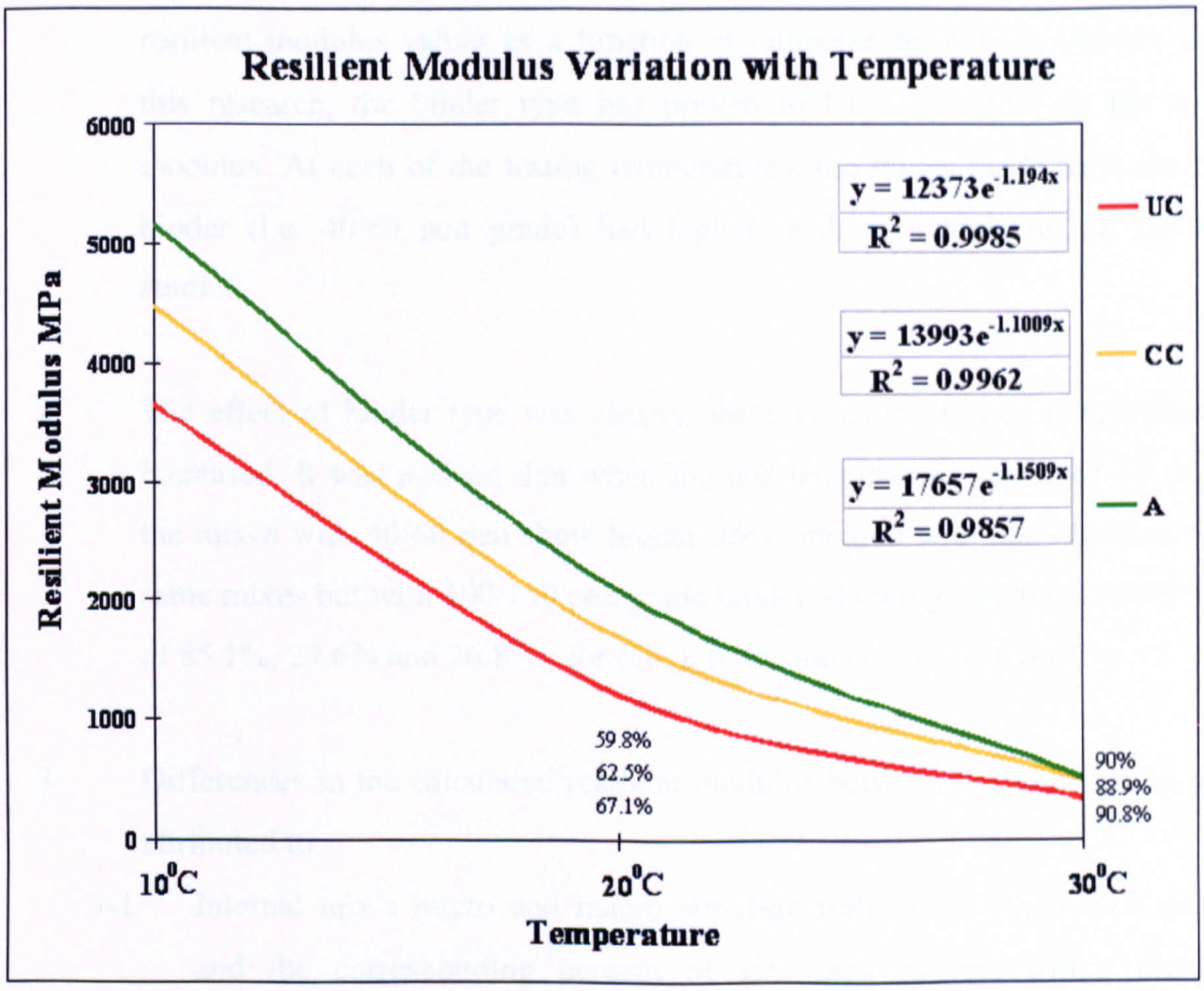


Figure 5-10: Resilient Modulus Variation with Temperature for 40/60 Pen Grade Binder

Results for mixtures made with 40/60 pen graded binder; show that Coating Croxden with Cement paste yielded improvements in stiffness average value of 47.8%, 39.6% and 22.6% at 30^{0}C, 20^{0}C and 10^{0}C respectively, while the reduction in bulk density was 6.3% (126 kg/m^3), Tables (5-1 and 5-2) above show these figures in details. All average results for each sample group were significance tested using the student "T" test. They were found to be significant to the highest degree of significance commonly found in the T tables. Appendix F shows full presentation of testing results.

Summary of Findings

1- Each mix's susceptibility to temperature was reflected by the change in its resilient modulus values as a function of temperature. For the binders used in this research, the binder type has proven to have an effect on the resilient modulus. At each of the testing temperatures, the mixes made from the harder binder (i.e. 40/60 pen grade) had higher resilient moduli for all the mixes studied.

2- The effect of binder type was clearly observed as the testing temperature was increased. It was noticed that when the test temperature increased up to 30^0C, the mixes with 40/60 pen show higher *Mr* compared with that obtained for the same mixes but with 100/150 pen grade binder, showing difference in *Mr* values of 85.1%, 29.6% and 26.8% , for (UC), (CC) and (A) respectively.

3- Differences in the calculated resilient modulus between the three mixes can be attributed to:

3-1 Internal mix's micro and macro structure reflected by their bulk densities and the corresponding percent of air voids content within the tested specimens.

3-2 The adhesion between the aggregate and the binders and in the case of cement coated aggregate between the aggregate and the binder.

4- The addition of the cement paste to Croxden aggregates had influenced the angularity of the aggregate particles, as was shown in chapter Three, and had improved the adhesion between binder and aggregates (Al Nageim and Robinson, 2006) and thus subsequently affecting the bulk density of the corresponding mix gradation and the percent air voids of compacted specimens and resulted in significant increase in the mix stiffness moduli.

5- The stiffness modulus value of a mix is highly related to the change in mix temperature, aggregates surface roughness and the compatibility of the aggregates surface to the binder (oil loving as the case with cement coated aggregates, or oil hating, as the case with uncoated aggregates) (Al Nageim and Robinson, 2006). The film of asphalt binder surrounding the aggregate particles of a mix serves as a bonding agent. As the temperature increases the asphalt binder changes from a solid state to a viscous liquid state, due to its viscosity characteristic that was mentioned earlier in chapter two (section 2-6). As the nature of the film changes with increasing temperature, the aggregates become less strongly bounded and the mix less rigid. This is evident in all the results at 30^0C as the stiffness modulus vales of the cement coated mix has improved from that of the untreated Croxden, it can be concluded that the addition of the cement paste has influenced the aggregates physical properties towards better stiffness moduli.

6- For cement coated aggregate mixes made from 100/150 pen grade binder the overall improvement in the secondary aggregates performance, in terms of *Mr* values compared with uncoated aggregate mixes, was in the range of 14.4 %, 14.1% and111% at 10^0C, 20^0C and 30^0C temperatures respectively, which is showing a recognizable improvement in the performance of natural aggregates. Whereas 47.8%, 39.6% and 22.6% improvements were recorded for the mixes made from the 40/60 pen grade at 10^0C, 20^0C and 30^0C temperatures respectively. Therefore the addition of the coating paste to the natural aggregate made a significant contribution to the material stability and turned this material from unsuitable i.e. rejected material for use in road by road engineers, to suitable and economic material for use in medium traffic roads, as can be seen from the results of the next chapter.

CHAPTER 6

Evaluation of Deformation by Repeat Load Axial Tests, RLAT

6.1 Introduction

Various types of testing have been utilised to measure resistance to permanent deformation of bituminous mixtures, such as uniaxial creep test, repeat load axial test (RLAT) and wheel tracking (torture test) (TRB Transportation Research Board, 2004). Uniaxial creep test is now being superseded by repeated load test. Repeated load test is considered more representatives of field conditions and is considered better able to distinguish between mixtures with different aggregate characteristics. For bituminous mixtures, such as porous asphalt type that is being utilised in this research study where deformation resistance depends crucially on the aggregate interlock component, therefore a degree of confinement is preferable to fully mobilise this component, (Brown et al, 1994). This chapter will investigate the RLAT test procedure and the outcome of the material performance at the three temperatures, namely 10^0C, 20^0C and 30^0C. Wheel tracking apparatus which provides more degree of confinement in measuring the material resistance to deformation under conditions similar to real traffic-road interaction will be considered in the next chapter.

6.2 RLAT for Permanent Deformation's Predictions

The main objective of this test was to determine the resistance of the three mixtures to permanent deformation. It was considered fundamental that any technique used should reflect the actual improvement imparted to the mixture by the cement coated aggregates over the conventional materials. The test was assessed by comparing the test results for three mixtures with different aggregate types and two binder contents, whose deformation properties were expected to be radically different. These mixtures were (AC 14 open surf 100/150 & AC14 open surf 40/60) mixes with corresponding percentage of binder contents as prescribed in Table (4-4) of chapter Four.

The RLAT test does not directly simulate *in-situ* loading conditions. nevertheless, for the ranking of different mixtures, in terms of resistance to permanent deformation, it has been shown that other test, namely, wheel tracking is more suitable (Gibb, 1996). On the other hand, other researchers demonstrated experimentally that mixtures with different aggregate gradings, the RLAT test results have shown good correlation with simulated wheel tracking tests results (Nunn et al, 1999).

6.3 Test Procedures

1- RLAT tests were performed in accordance with BS DD 226 (1996) and the test was run after completing ITSM test (described in chapter 5), as the ITSM test is a non-distractive test. Therefore the same samples have been utilised in both tests (i.e. ITSM & RLAT tests).

2- The test sample, had both faces ground and polished before testing, and was coated with silicon grease to provide a low friction interface between the samples under the load platens to prevent variations in the apparent sample due to end – restraint.

3- The test sample was then placed in the testing machine having a constant temperature enclosure to perform the test at its corresponding temperature.

4- After bringing the test sample to the specified test temperature, the test sample was placed centrally in position on the lower platen.

5- The upper load platen was then placed centrally on the top of the test sample and the whole assembly was positioned centrally below the load actuator.

6- The two electrical displacement transformers (LVDT's) were set up and the test sample was preloaded with a conditioning load equivalent to a stress of 10 kPa for 600s ±6s. Any axial deformation of the test sample after the application of the conditioning of the load, have been recorded by computer software.

7- The conditioning stress of 10kPa ±1kPa was applied for each sample for 600s. After that, the test sample was subjected to repeat application of the axial stress to produce a temporary axial stress of 100kPa ±2kPa. The total duration of the axial load pulse was 1s ±10ms, having a total test duration of 3600s.

8- The axial deformation of the test sample was then measured after every 10^{th} load application, up to a 100 load applications and thereafter every 100 load applications until the end of the specified test period or until failure had been detected.

9- It should be noted that, during the period of loading if the surfaces of the platens became more than 5 degrees out of parallel, the test was abandoned. This was recorded by the same computer software, and it is known as the angle of tilt of the test sample.

10- Test samples were tested at constant temperatures of 10^0C, 20^0C and 30^0C respectively for both 100/150 and 40/60 pen mixtures.

6.4 Axial Deformation Calculation Method

The mean displacement of the two transformers was placed in the following formula to give percentage axial deformation:

$$\text{axial deformation (\%)} = (\Delta h/h_o) \times 100 \qquad (6\text{-}1)$$

Where:

h_o is the original thickness of the specimen (mm), and

Δh is the axial deformation (mm).

The percentage axial deformation was calculated once each second and plotted against time to produce a graph, a specimen of which is shown in figures (6-1 & 6-2). The percentage axial deformation at 3600s was recorded as the result for that particular specimen. Forty five samples representing three types of mixtures were tested and fifteen specimens completed one mix type group.

Figures (6-3) to (6-10) represent general outcome results of RLAT tests.

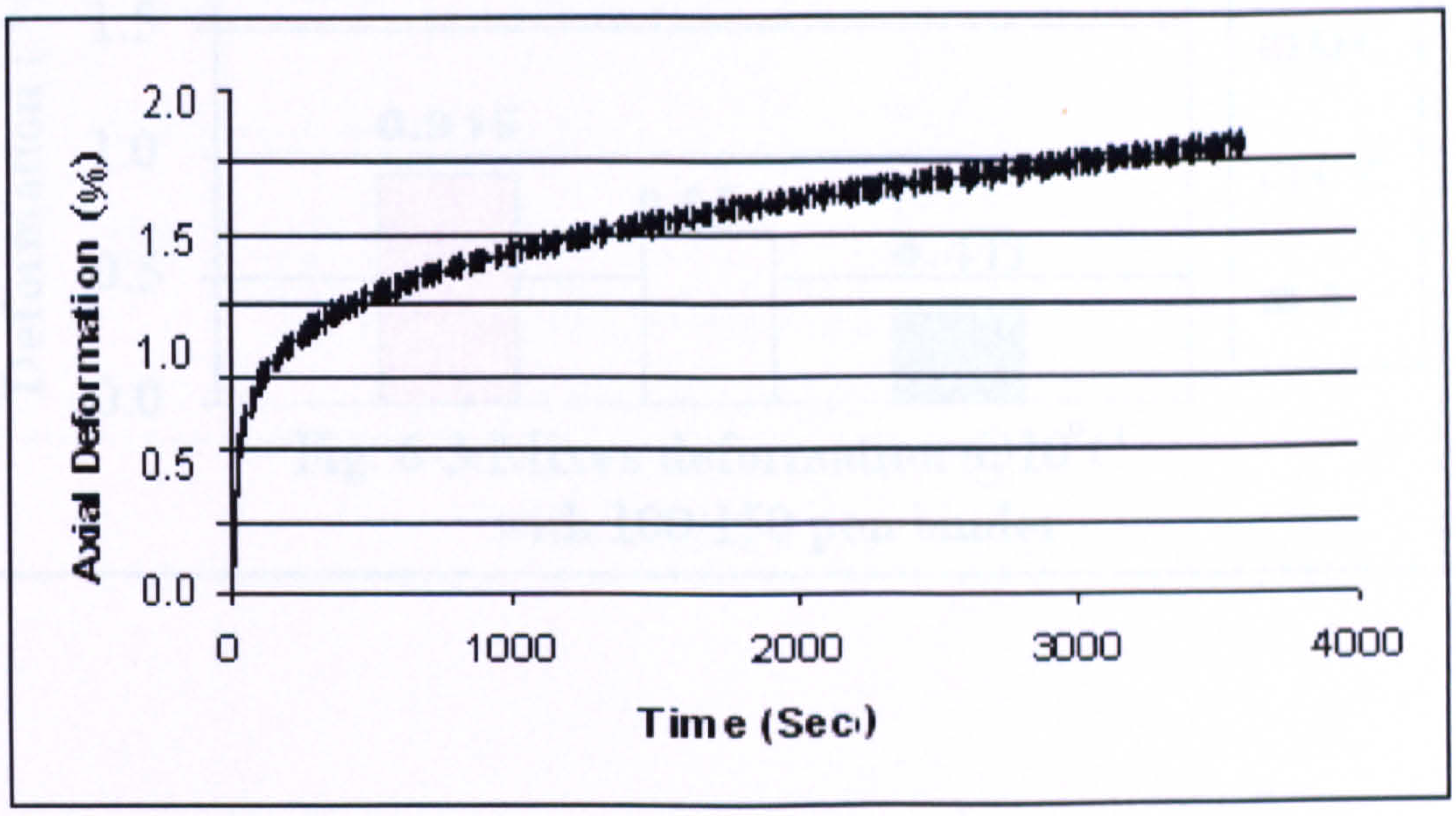


Fig. 6-1: A typical graph result plotted for a RLAT test

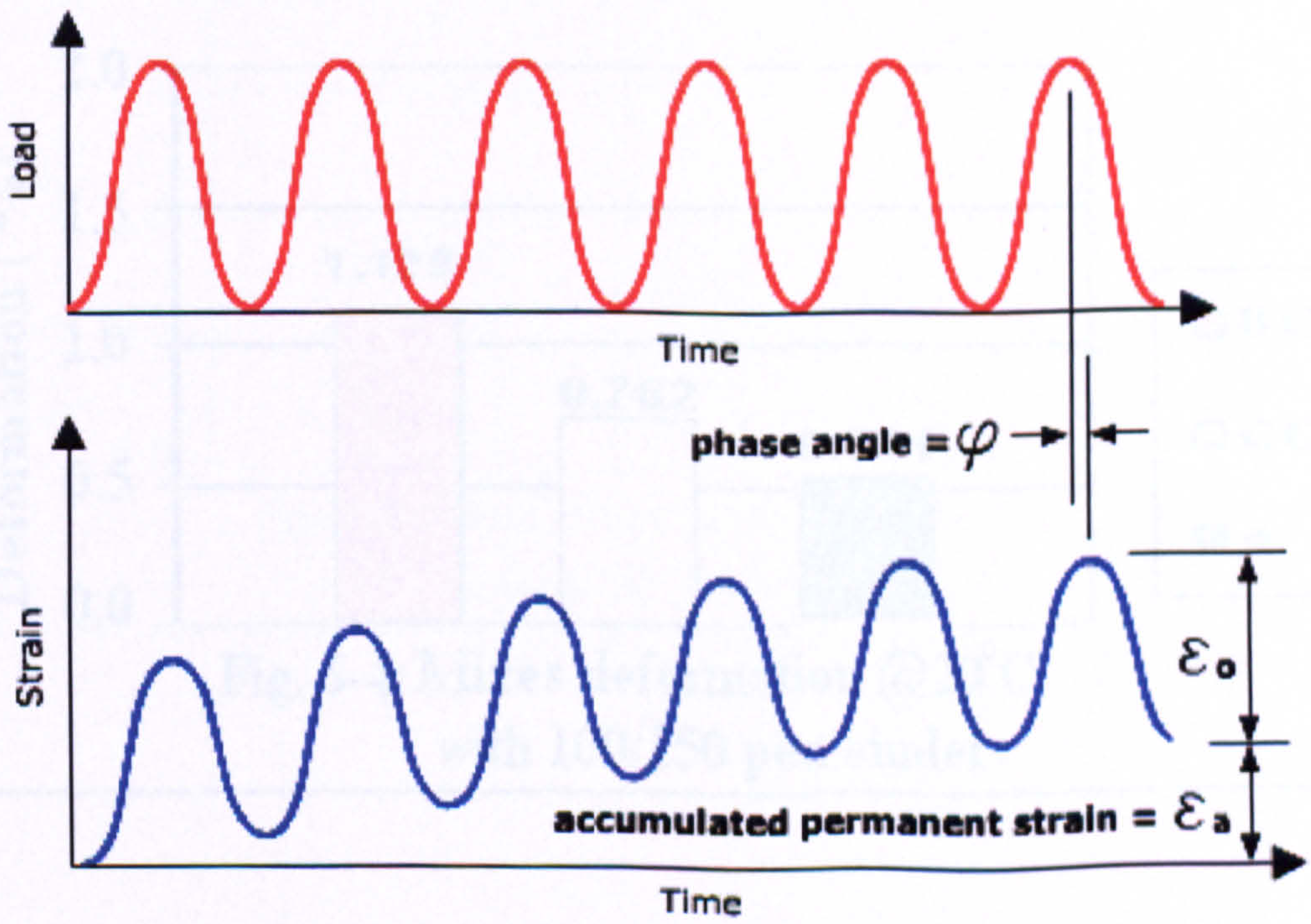


Fig 6-2: General graph of repeated load axial test (RLAT)

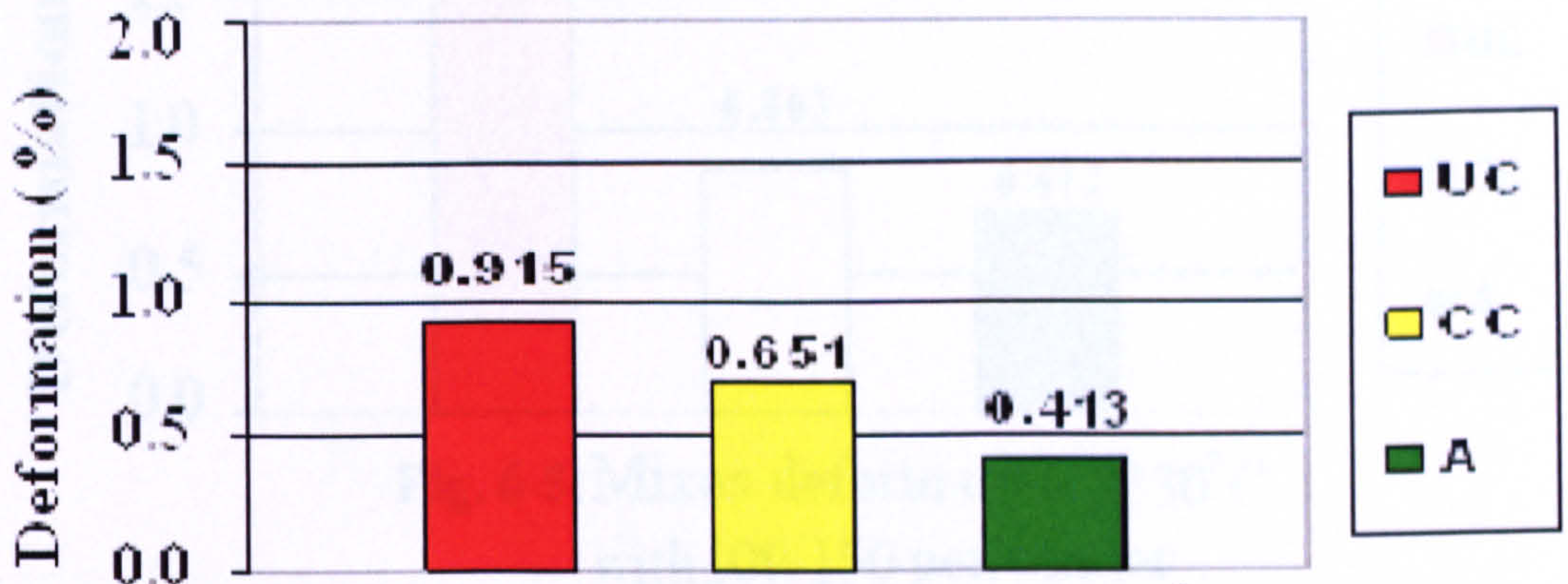


Fig. 6-3:Mixes deformation @10°C
with 100/150 pen binder

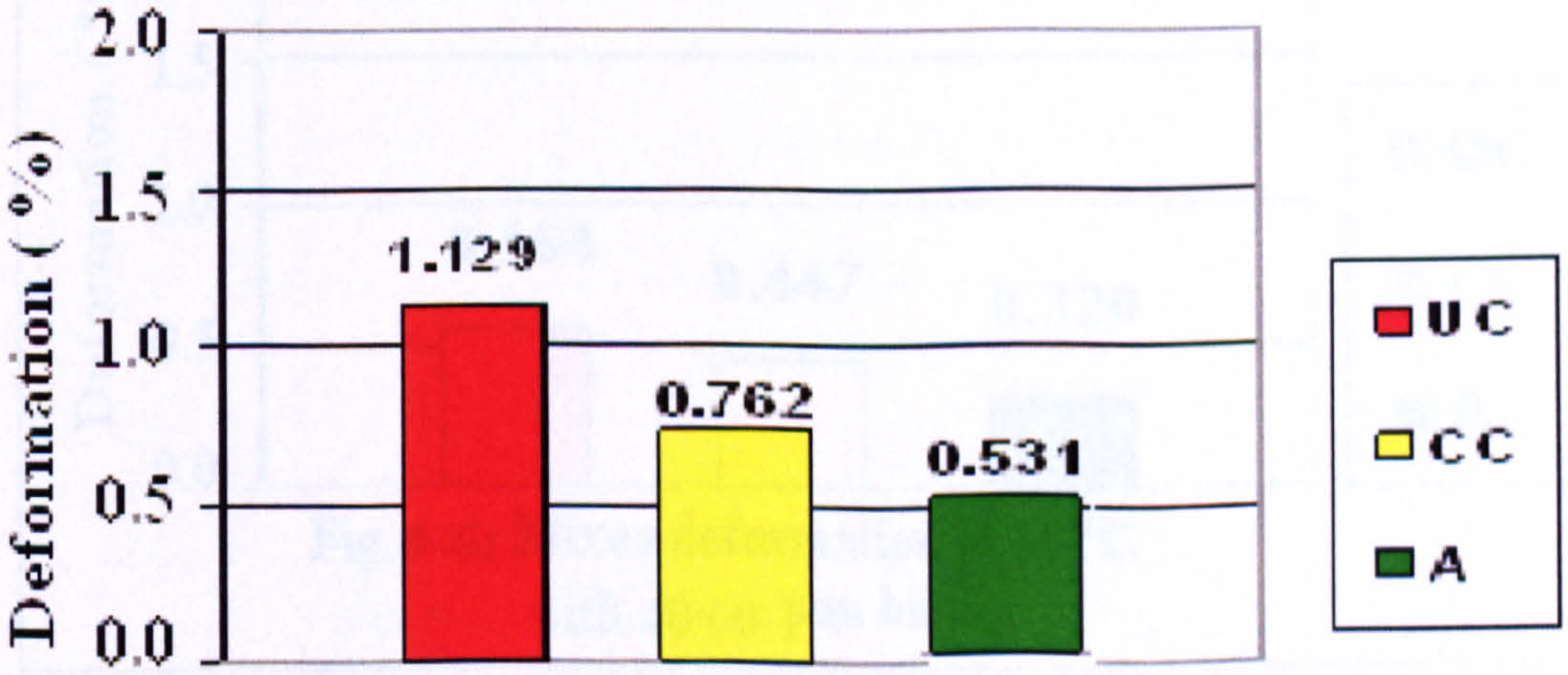


Fig. 6-4: Mixes deformation @20°C
with 100/150 pen binder

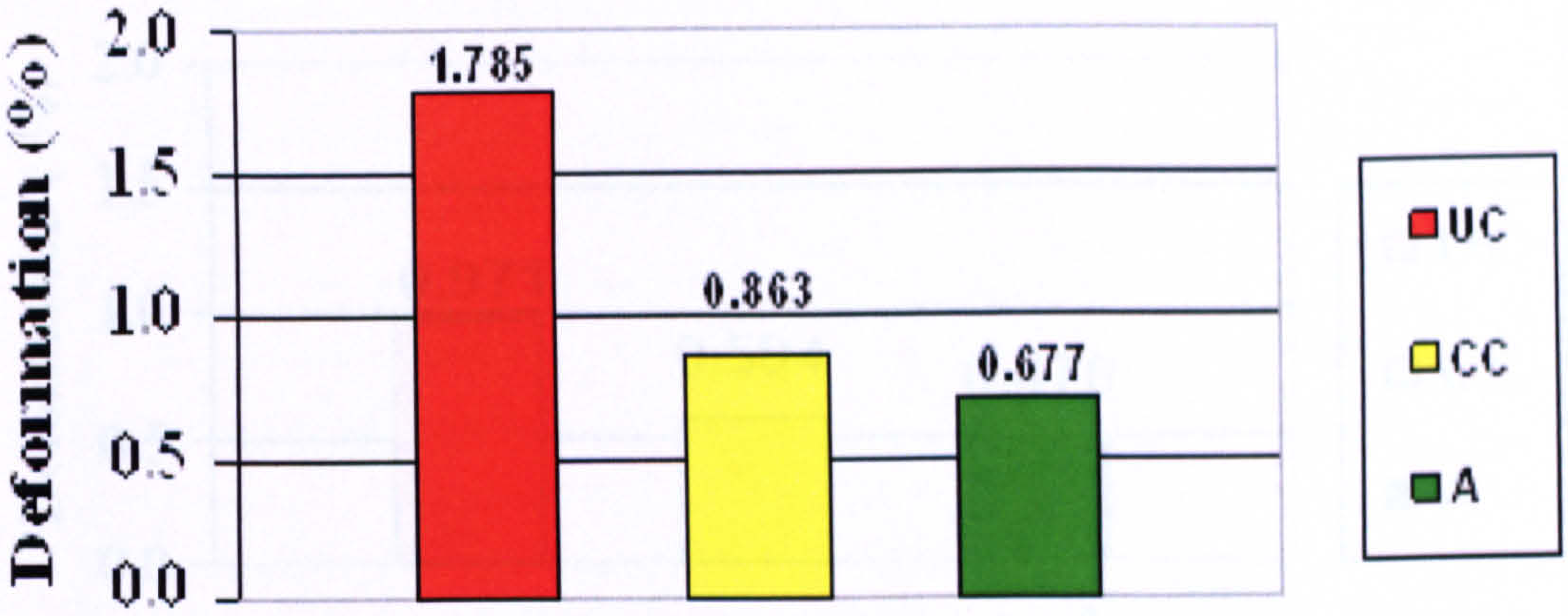


Fig. 6-5: Mixes deformation @ $30^{o}C$ with 100/150 pen binder

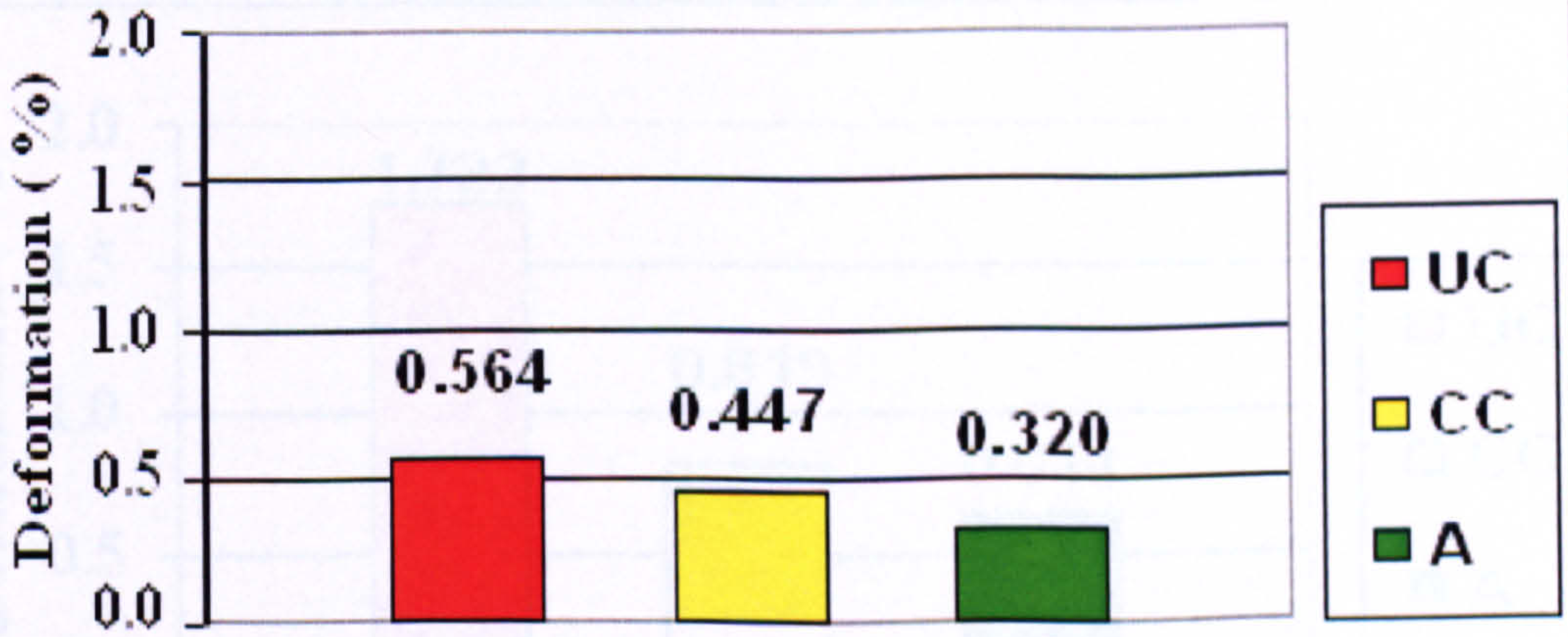


Fig. 6-6: Mixes deformation @ $10^{0}C$ with 40/60 pen binder

Fig. 6-7: Mixes deformation @20°C with 40/60 pen binder

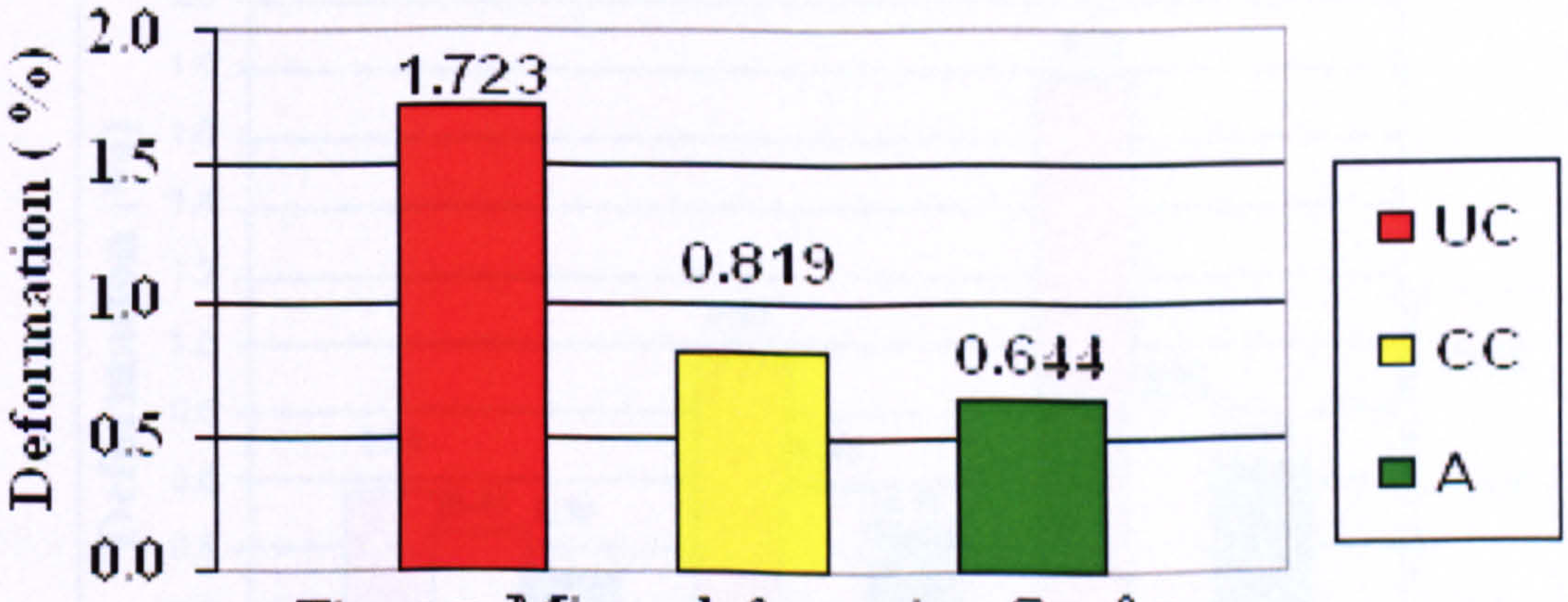


Fig. 6-8: Mixes deformation @30°C with 40/60 pen binder

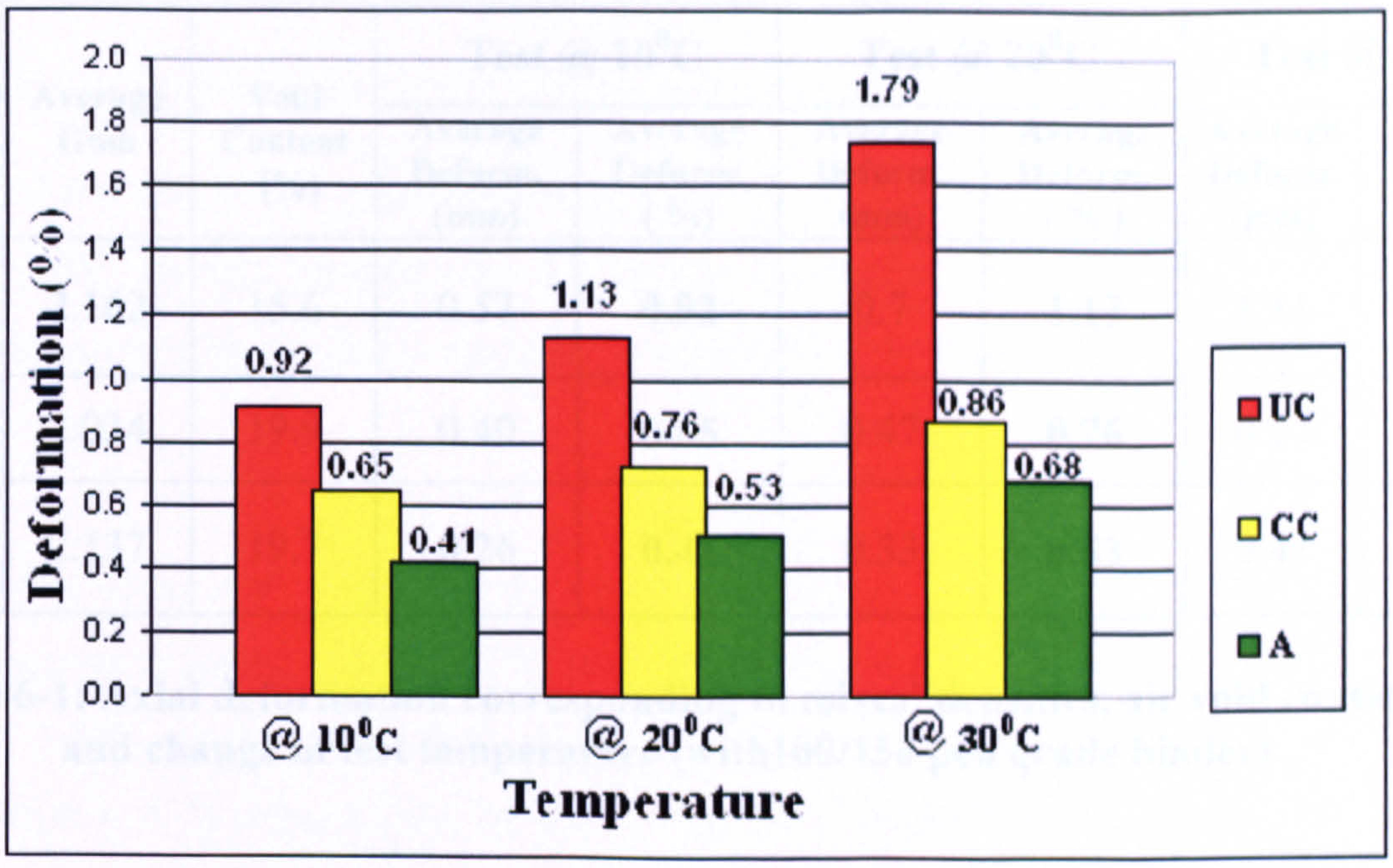


Fig. 6-9: The three types of mixtures tested @ 10^0C, 20^0C & 30^0C respectively With 100/150 pen grade binder

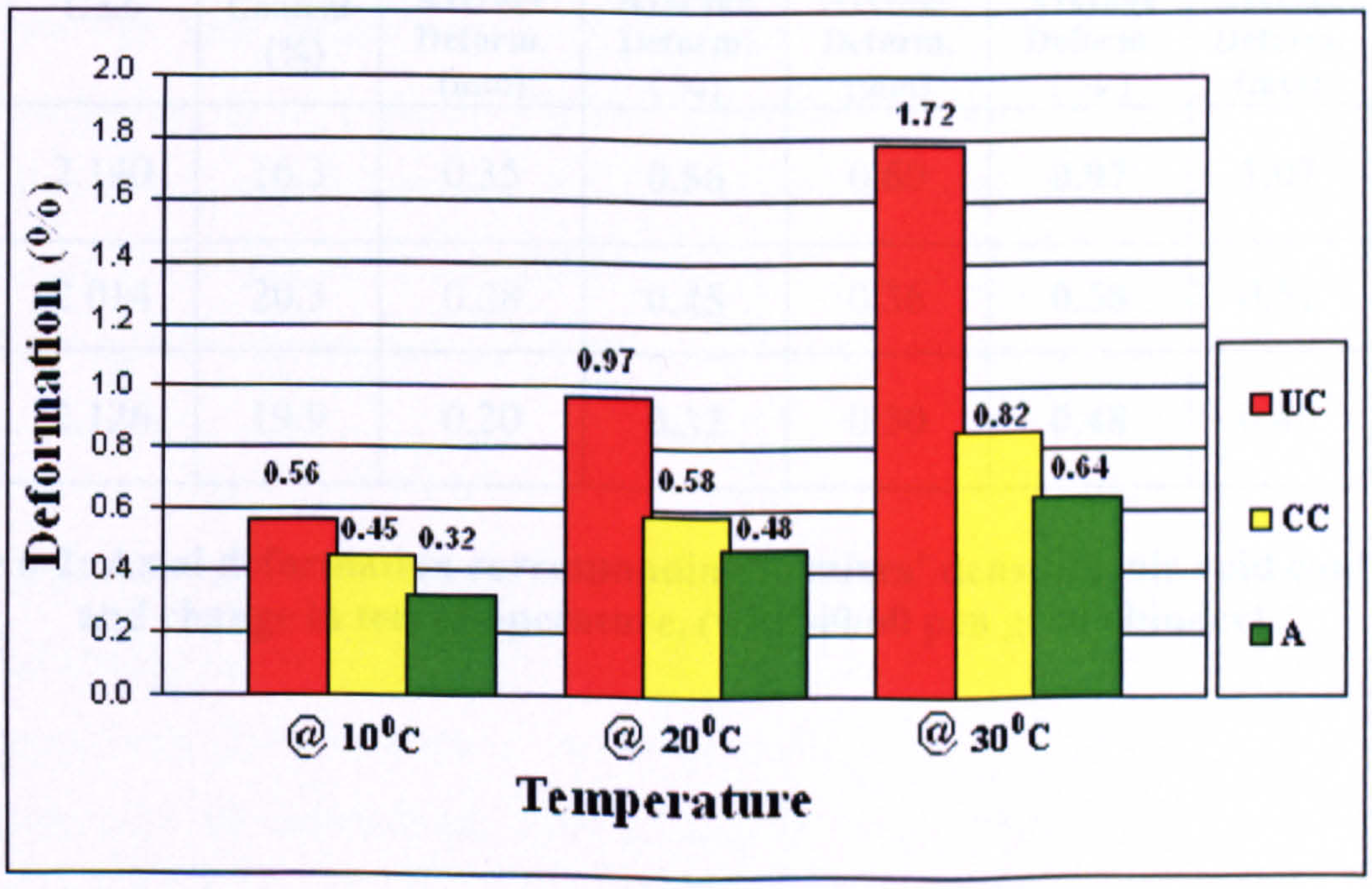


Fig. 6-10: The three types of mixtures tested @ 10^0C, 20^0C & 30^0C respectively With 40/60 pen grade binder

Type of Mix With 100/150 pen Binder	Average Gmb	Void Content (%)	Test @ 10^0C		Test @ 20^0C		Test @ 30^0C	
			Average Deform. (mm)	Average Deform. (%)	Average Deform. (mm)	Average Deform. (%)	Average Deform. (mm)	Average Deform. (%)
Uncoated Croxden	2.162	15.6	0.57	**0.92**	0.7	**1.13**	1.11	**1.79**
Coated Croxden	2.034	19.9	0.40	**0.65**	0.47	**0.76**	0.53	**0.86**
Arcow	2.137	19.7	0.26	**0.41**	0.33	**0.53**	0.43	**0.68**

Table 6-1: Axial deformation corresponding to mixes' densities, air void contents and change in test temperature (with100/150 pen grade binder)

Type of Mix With 40/60 pen Binder	Average Gmb	Void Content (%)	Test @ 10^0C		Test @ 20^0C		Test @ 30^0C	
			Average Deform. (mm)	Average Deform. (%)	Average Deform. (mm)	Average Deform. (%)	Average Deform. (mm)	Average Deform. (%)
Uncoated Croxden	2.140	16.3	0.35	**0.56**	0.60	**0.97**	1.07	**1.72**
Coated Croxden	2.014	20.3	0.28	**0.45**	0.36	**0.58**	0.51	**0.82**
Arcow	2.126	19.9	0.20	**0.32**	0.30	**0.48**	0.40	**0.64**

Table 6-2: Axial deformation corresponding to mixes' densities, air void contents and change in test temperature, (with 40/60 pen grade binder)

6.5 RLAT Tests Results Discussion and Summary of Findings

The following findings and discussion worth mentioning as results of the RLAT tests:

i. The test was found to be capable of showing the significant improvement induced by the cement coating on the mixes resistance to permanent deformation compared with the mix containing uncoated Croxden aggregates and conventional mix.

ii. Table (6-1) and Figure (6-9) also showed that samples containing Croxden aggregates tested at 10^0C, 20^0C and 30^0C displayed an average axial deformation of 0.92%, 1.13% and 1.79% of the average height, (namely 62mm, Appendix E Table E 4-1) respectively. In comparing these results with that obtained for the mixtures containing cement coated aggregate, it can be noticed that these mixes had displaced axial deformation improvements of 41.5% at 10^0C, 48.7% at 20^0C and 108.1% at 30^0C.

iii. Table (6-2) and Figure (6-10) showed the results of the same tested samples but with the hard grade binder (40/60 pen grade binder). An average axial deformation of 0.56 %, 0.97% and 1.72% of the average height (Appendix E Table E 4-2) was measured at 10^0C, 20^0C and 30^0C respectively. Under the same testing conditions, mixtures made with cement coated aggregates had shown axial deformation improvements of 24.4%, 67.2% and 109.8% at 10^0C, 20^0C and 30^0C respectively.

iv. Table (6-1) and Figure (6-9) showed that the deformation percentage of Arcow and cement coated aggregates mixtures increased by approximately 28% & 13% respectively for an increase in temperature from 20°C to 30°C, while the percentage of deformation was as high as 58% for the mixture containing uncoated Croxden aggregates with the same increase in temperature.

v. Table (6-2) and Figure (6-10) showed that the deformation percentage of Arcow and cement coated aggregates mixtures increased by approximately 33.3% &

41.4% respectively for an increase in temperature from 20°C to 30°C, while the percentage of deformation was as high as 77.4% for the mixture containing uncoated Croxden aggregates with the same increase in temperature.

The average value of each group was tested for statistical significance using a student "T" test. It was found that all the groups' averages demonstrated significant difference when compared with each other. From Figures (6-9 and 6-10), it can be concluded that the amount of axial deformation had decreased significantly by the introduction of cement coated aggregates within the bituminous mixtures, and that had led to the subsequent increase in their resistance to permanent deformation, (rutting). On the other hand, the reduction in bulk density of the coated mixtures was also thought to be closely related to this increase in the rutting resistance, which is in turn related to the increase in the coated aggregates' angularity and surface roughness mentioned in chapter three and supported by the work of Brannan et al (1991). Tables (6-1 & 6-2) noticeably show this relationship by comparing the changes in which is inter-related to testing temperature, bulk density, air voids and axial deformation due to the addition of cement paste to the aggregates within the tested mixtures.

CHAPTER 7

Evaluation of Deformation by wheel tracking tests

7.1 Introduction

Asphalt pavement deformation (rutting) can be due to insufficient pavement structural support allowing excessive stress to be transferred to the base course (structural rutting); however, the most frequent type of rutting is 'stability' rutting caused by the plastic movement of asphalt mix under heavy, often slow moving traffic, Plate 7-1. (Uzarowski and Michel, 2004)

Plate 7-1: Rutting profile (Uzarowski et al, 2004)

Rutting is a flexible pavement distress caused by the accumulation of permanent deformation in the pavement layers from the repeated application of traffic. High level of rutting in asphalt pavements is a main concern among the highway engineers, which mainly occurs during high temperature seasons. It is affected by external factors (Haas and Papagianakis, 1986) such as:

i) Pavement geometry,

ii) axle loads,

iii) contact pressure,

iv) surface shear stresses, and

v) the bonding between the pavement layers.

The effect of factors relevant to this study such as, aggregate gradation, aggregate shape, binder type and asphalt mix properties on rutting have been discussed previously in this thesis (chapters two and four).

The cost of pavement rutting repairs can be extremely high and disruptive to everyday traffic (as mentioned in chapter three). A reliable, accelerated laboratory performance test to evaluate rutting resistance of asphalt mixes is considered necessary. It would be beneficial for use in verifying mix designs, for pavement failure investigations and for new materials evaluation such as the cement coated aggregates that is under investigation in this research work.

A Novel Pavement Testing Facility has been designed in this research work, (chapter 8); to facilitate the researcher to optimize the design of the HMA containing new upgraded natural secondary aggregates. This step of the research was disrupted due to the lack of financial funding necessary to fabricate the machine. Accordingly this aim was not met.

Therefore the research team decided that with the help of Tarmac Ltd., their two indoor rutting testers, situated at Wolverhampton's laboratories, were to be used to complete this PhD project and namely to be used for the investigation of rutting of three mixes; first mix containing upgraded natural aggregates (i.e. Cement Coated Croxden aggregates), second mix is a control mix (with Arcow aggregates) and the third is containing uncoated Croxden aggregates; the first testing machine is "The Cooper Research CRT-WTEN1 wheel tracker" (Plate 7-2), and the second is "The Cooper Research CRT-WTENL – Wheel tracker" (Plate 7-3) :

The Cooper Research CRT-WTEN1 wheel tracker: Small Device according to BS EN 12697 pt 22 (2003 amended 2007), (Plate 7-2) has the specifications shown in Table (7-1), and the schematic layout shown in Figure (7-1).

SPECIFICATIONS	VALUE
Speed	53 passes per minute
Wheel load	700N or $[(700\frac{w}{50}) \pm 10]N$ where w is the tyre width
Temperature	(30 to 60 ±1)°C
Slab Dimensions	(305x305x50) mm
Slab thickness	50mm to 100mm
Rut depth transducer	40mm
Tyre Dimensions	(200 ± 5) mm diameter and (50±1) mm width
Travel Distance	(230 ± 10) mm
Tyre Thickness	(20 ± 2) mm

Table 7-1: The Cooper Research CRT-WTEN1– Wheel tracker (Small Device) Specifications

Plate 7-2: The Cooper Research CRT-WTEN1 wheel tracker (Small device)

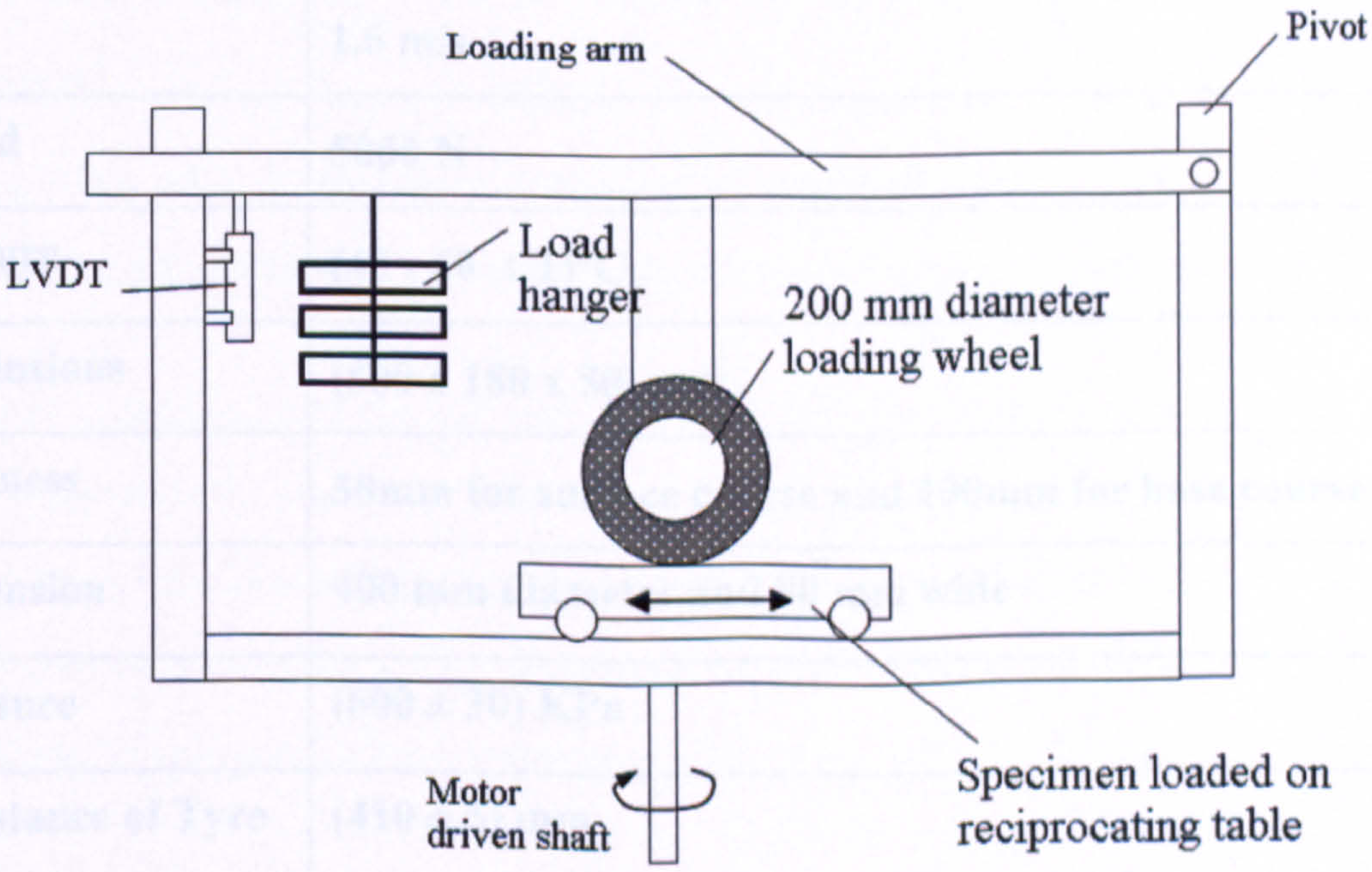


Fig. 7-1: Schematic of a small-scale wheel-tracking test

The CRT-WTEN can be used for (Cooper Research Technology, 2007):

- The identification of rut-susceptible mixtures so that expensive remedial works are avoided.
- The evaluation of new materials and formulations.
- Studying the effect of level of compaction (void content and mixture density) on resistance to rutting.
- Validating mix designs.

European Standard applied to this test is BS EN 12697 pt 22 (2003 amended 2007) (Small Device), Section 6.3 pp 11.

The Cooper Research CRT-WTENL – Wheel tracker: Large Device according to BS EN 12697 pt 22 (2003 amended 2007), Plate (7-3) has the following tabulated specifications:

SPECIFICATIONS	VALUE
Speed	1.6 m/s
Wheel load	5000 N
Temperature	(40 - 60 ± 1)°C
Slab Dimensions	(500 x 180 x 50) mm
Slab thickness	50mm for surface course and 100mm for base course
Tyre dimension	400 mm diameter and 80 mm wide
Tyre Pressure	(600 ± 30) KPa
Travel Distance of Tyre	(410 ± 5) mm
Rut depth transducer	40 mm

Table 7-2: The Cooper Research CRT-WTENL – Wheel tracker (Large Device) Specifications

7.2 System Elements of the CRT-WTEN1 – Wheel tracker (Small Device)

The CRT-WTEN1 (Small Device) comprises a solid extruded aluminum frame supporting a controlled temperature cabinet made from stainless steel and fitted with double glazed doors for full access. The test specimen is mounted on a table which is reciprocating a distance of 230mm on linear bearings at the specified speed. A loaded rubber tyred wheel runs on top of the specimen and the resultant rut is monitored as the test proceeds using a calibrated displacement transducer.

The equipment is controlled and the test data acquired, via an interface unit, using the test software supplied. The software automatically starts the wheel tracker, maintains the speed at the required level, measures rut depth and sample temperature, provides a continuously updated on-screen graph of rut depth versus time and stops the wheel tracker when the test is finished. On completion of a test the rate of rutting is calculated and a test report can be printed using the supplied software. If required, the stored test data can be analysed and compared with other test data using a spreadsheet.

7.3 System Elements of the CRT-WTENL – Wheel tracker (Large Device)

The CRT-WTENL – Wheel tracker" (Large Device) comprises a rigid steel frame supporting a stainless steel temperature controlled temperature cabinet, (40 to 60 ±1)°C around the slab, with automatic opening double glazed doors for good access.

The equipment is controlled and the test data acquired, via the enclosed data control unit. The software automatically starts the wheel moving and pushes the specimens up into contact with them. It provides a continuously updated on-screen graph of rut-depth versus time and stops the wheel tracker when the test is finished. On completion of a test the stored test data can be analysed using a spreadsheet (see plate 7-3).

Plate 7-3: The Cooper Research CRT-WTENL – Wheel tracker (Large Device)

European Standards applied to this test are:

- BS EN 12697-22 :(2003 AMENDED 2007): Bituminous mixtures, Test methods for hot mix asphalt. Wheel tracking (Large device). Section 6.1 pp 9.
- NF P98-253-1 French Standard: Déformation permanente des mélanges hydrocarbons Partie 1: essai d'orniérage (Permanent deformation of hydrocarbon mixtures. Part 1: Test of rutting).

These two devices have been used to evaluate asphalt mixes containing cement coated aggregates, in terms of rutting resistance. The principle is simple and accurately represents traffic and temperature conditions. The hot mix asphalt slab samples were prepared at LJMU's Laboratories. The mixes were prepared by compacting the samples at a specified temperature of 135^0C for the samples containing a binder of 100/150 penetration grade and a temperature of 145^0C for the samples containing a binder of 40/60 penetration grade, by compressing the mix inside the mould that is 500 mm long, 180 mm wide and 50 mm high.

The samples have then been transferred to the CRT-WTENL – Wheel tracker" (Large Device) for rut testing at Wolverhampton. Tests have been carried out in accordance with the BS EN 12697 pt 22 (2003 amended 2007) section 6.1 pp9 (Procedure B).

7.4 Rutting Susceptibility by Small Scale Wheel Tracker

A wheel tracking machine (Small device) was used to perform permanent deformation testing on square samples of (305 x 305 x 50mm) dimensions to assess the resistance to rutting of the three types of mixes under conditions which simulate the effect of medium traffic (BS EN 12697 pt 22 , 2003). A loaded wheel tracks a single sample under specified conditions of load (700N), speed (53 passes / min). Two different temperatures (30^0C & 45^0C) were used respectively for testing and the development of the rut profile was monitored continuously during the test.

The outcomes of the wheel tracking machines have also shown improvement in the cement coated aggregates performance in regards to the permanent deformation in comparison to the control mix and the uncoated aggregate mixture. This will be explained in the following sections.

Plate 7-4: Samples Undergoing Rut test by the Wheel Tracker, Small Device

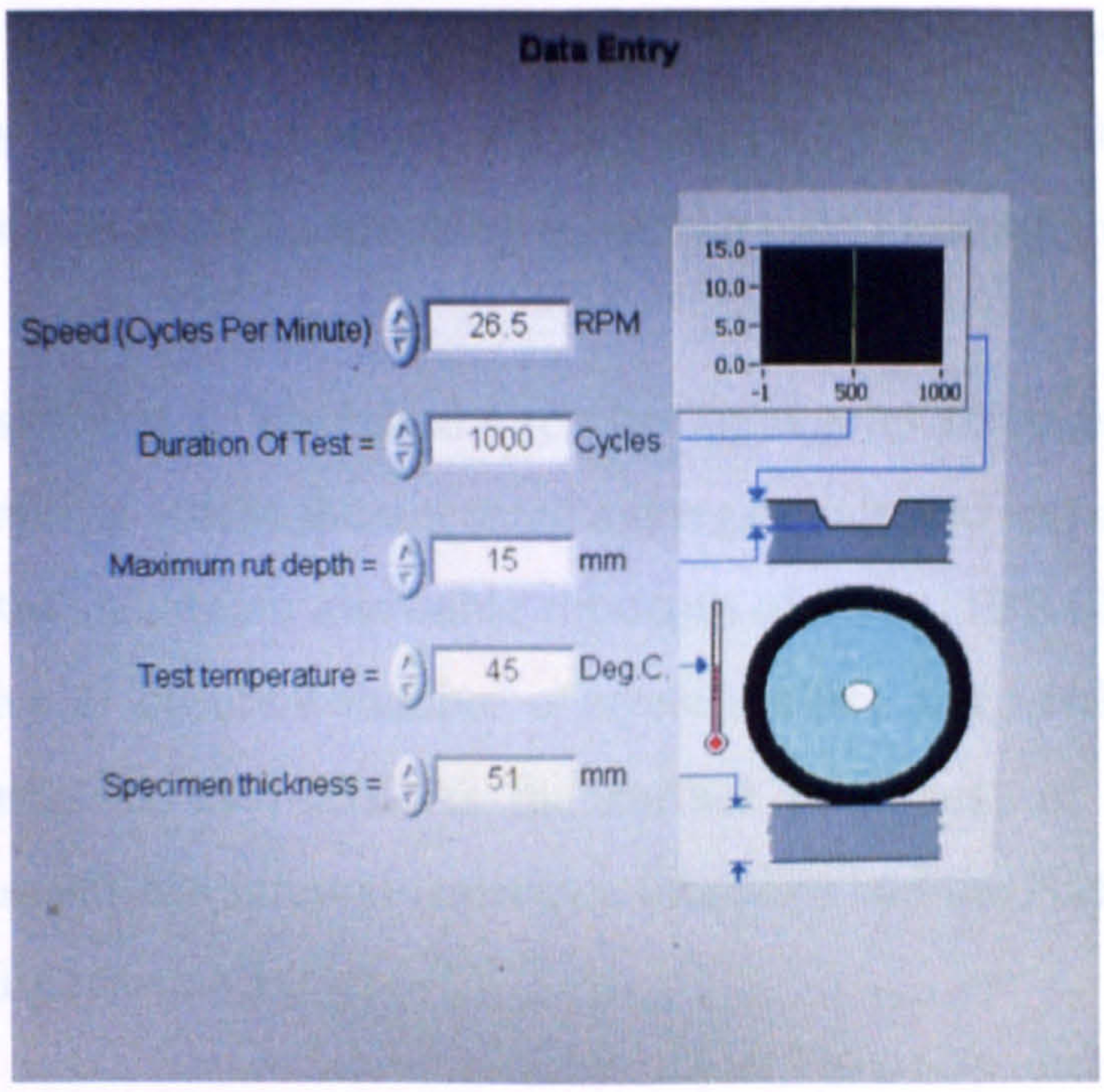


Plate 7-5: Typical Screen Display of the Small Scale Wheel Tracker

In the small scale wheel tracking, tests were performed by moving the specimen back and forth under the testing wheel (Plate 7-4). The test is carried out at the specified temperatures by placing the apparatus in temperature controlled cabinet. The specimen is loaded at predetermined passes and the rut depth at the centre of the loaded region profile is monitored against time or the number of load passes. In addition to the assessment of mixtures, it is also used for validation of the rutting prediction methods, which will be described in the next chapter. The wheel tracking test is also used as a reference for the evaluation of element testing methods (Brown et al, 1996). The LCPC mixture design method in France uses the LCPC wheel tracker test (French Standard, 1998).

Tables (7-3 and 7-4) and Figures (7-2 to 7-23) show the tests results as measured by the CRT-WTEN1 – Wheel tracker (Small Device), for the three types of mixtures with the two previously specified binder types which can be summarized as follows:

1. Mixtures with Arcow aggregate clearly show the best performance in deformation resistance than do the other two mixes,

2. Mixtures containing Cement Coated aggregates have shown a well recognised rutting resistance as the aggregates shape become rougher, the internal resistance appears to initiate at an early stage after loading, and then the graph shows stability in the level of rutting rate as stone-on-stone interlocking inside the mixture starts to take place.

3. Conversely, the skeleton of untreated Croxden aggregates remains unstable, and this had led to more rutting within the uncoated aggregate mixture and all tested specimens had experienced the maximum allowable rut depth of 15mm (BS EN 12697-22, 2003). This mixture failed at different number of cycles, which triggered the termination of their tests (Figures 7-2 to 7-23). As the surface roughness of the cement coated aggregate has changed, the internal resistance increases and the HMA mix improves its capability of rutting resistance.

4. The Arcow mixes possess the highest rutting resistance, followed by mixes containing cement coated aggregates, and mixes containing untreated aggregates came last.

5. It appears that a HMA mix can be made more stable and resistant to rutting by improving the shape and the aggregate angularity and this can be achieved by cement coating of the utilized aggregates within Hot Asphalt Mixtures.

6. From Table (7-3), Figure (7-14) and Table (7-4) and Figure (7-23), it can be seen that:

- Table (7-3) and Figure (7-14) show the results of average deformation for the three types of mixes tested with the soft grade binder (100/150 pen grade binder) at 30^0C. Percentages of average deformation were calculated for each type and found to be 10.5% for (UC), 3.82% for (CC) and 2.91% for (A), and these values are corresponding to the average heights, corresponding to each type of mix (Appendix E Table E4-3). Under the same testing conditions, but at temperature of 45^0C, tested mixes samples had shown percentage of average deformations of 29.96%, 20.25% and 20.31% for (UC), (CC) and (A) respectively.

- Table (7-4) and Figure (7-23) show the results of the same type of mixes but with hard grade binder (40/60 pen grade binder), and average deformations of 7.57%, 2.67% and

2.42%, corresponding to average heights (Appendix E, Table E 4-4), were measured at 30^{0}C for (UC), (CC) and (A) respectively. Under the same testing conditions but with testing temperature of 45^{0}C, tested samples had shown percentage of average deformations of 29.98%, 7.98% and 5.2% for (UC), (CC) and (A) respectively.

- For mixes with soft binder, the average deformation percentage of uncoated aggregate mixtures had decreased from 10.5% to 3.82% when coated with cement paste and tested at 30°C temperature (i.e. improvement of approximately 174.9%), while the percentage of average deformation of mixture containing uncoated Croxden aggregates was as low as 10.5% when tested at the same temperature. At testing temperature of 45°C, (CC) mixes had shown deformation improvement of 48% when (UC) had experienced percentage of average deformation of 29.96%.

- On the other hand, for mixes containing hard binder the average deformation percentage of uncoated aggregate mixtures had decreased from 7.57% to 2.67% when coated with cement paste and tested at 30°C temperature (i.e. improvement of approximately 183.5%), while the percentage of average deformation of mixture containing uncoated Croxden aggregates was as low as 7.57% when tested at the same temperature. (CC) samples had shown improvement of 275.7% when tested at 45°C, while (UC) was experiencing only 29.98% of average deformation.

It can be concluded that the amount of deformation had decreased significantly by the introduction of cement coated aggregates within the bituminous mixtures, and that had led to the subsequent increase in their resistance to permanent deformation, (rutting). On the other hand, the reduction in bulk density of the coated mixtures was also thought to be closely related to this increase in the rutting resistance, which is in turn related to the increase in the coated aggregates' angularity and surface roughness mentioned in chapter three and supported by the work of Brannan et al (1991). Tables (7-3 & 7-4) noticeably show this relationship by comparing the changes in which is inter-related to testing temperature, bulk density, air voids and axial deformation due to the addition of cement paste to the aggregates within the tested mixtures.

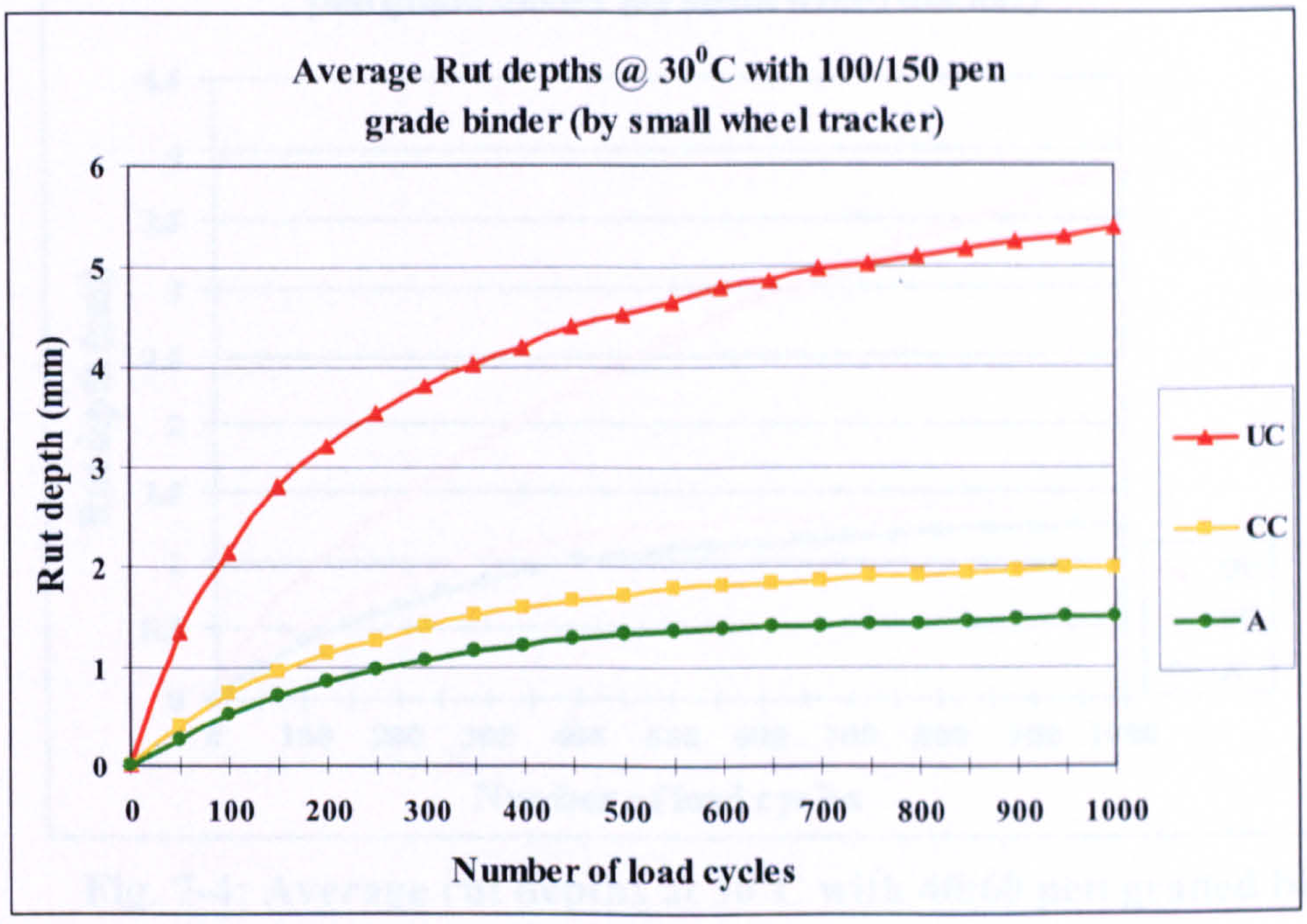


Fig. 7-2: Average rut depths at 30⁰C with 100/150 pen graded binder by small wheel tracker

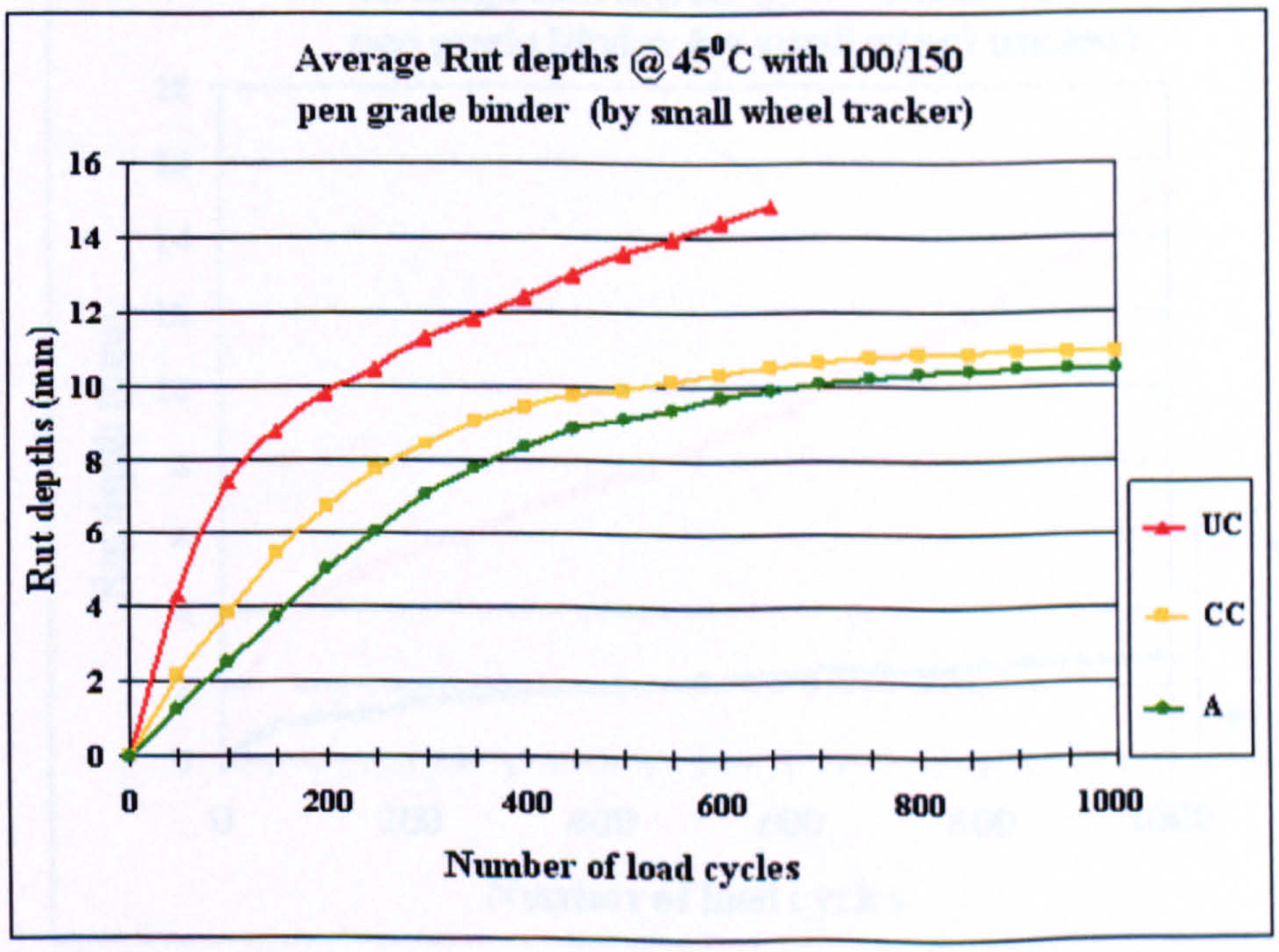


Fig. 7-3: Average rut depths at 45⁰C with 100/150 pen graded binder by small wheel tracker

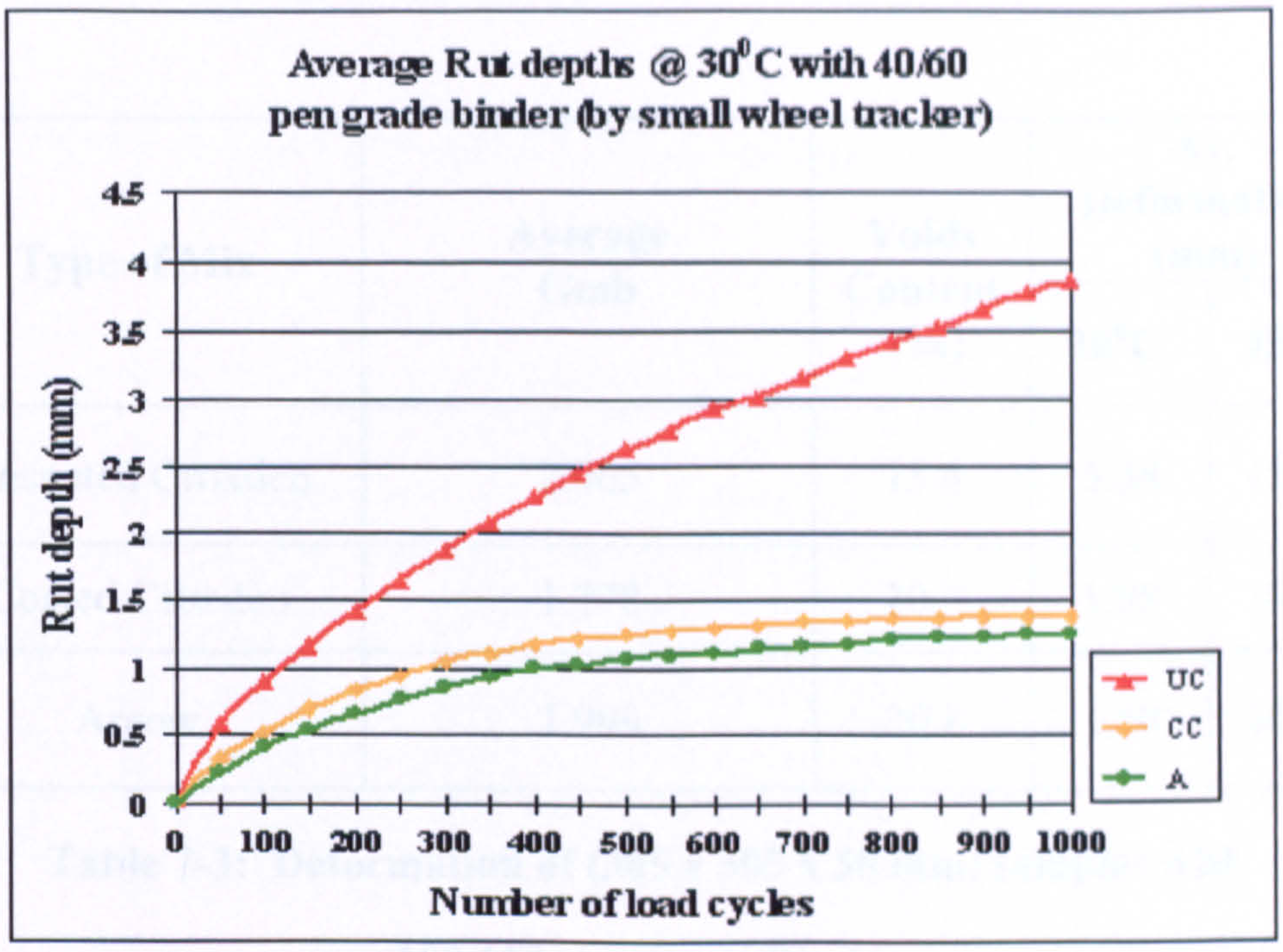


Fig. 7-4: Average rut depths at 30^0C with 40/60 pen graded binder by small wheel tracker

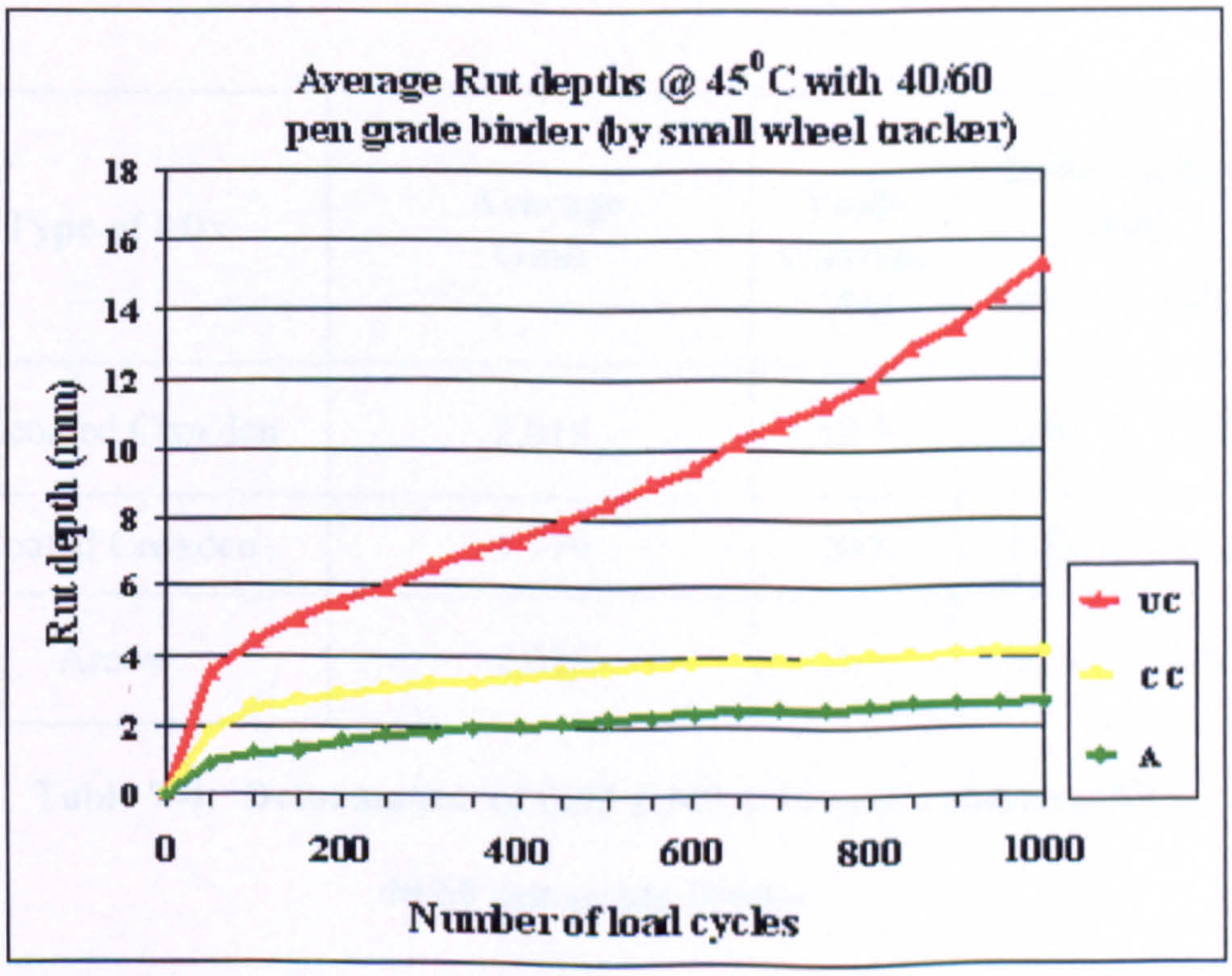


Fig. 7-5: Average rut depths at 45^0C with 40/60 pen graded binder by small wheel tracker

Type of Mix	Average Gmb	Voids Content (%)	Av. Deformation (mm)	
			30⁰C	45⁰C
Uncoated Croxden	2.005	13.6	5.38	15.34
Coated Croxden	1.778	20.7	1.98	10.49
Arcow	1.999	20.8	1.50	10.48

Table 7-3: Deformation of (305 x 305 x 50 mm) samples with 100/150 pen grade Binder

Type of Mix	Average Gmb	Voids Content (%)	Av. Deformation (mm)	
			30⁰C	45⁰C
Uncoated Croxden	2.019	12.9	3.86	15.29
Coated Croxden	1.779	20.6	1.38	4.12
Arcow	2.016	20.1	1.24	2.66

Table 7-4: Deformation of (305 x 305 x 50 mm) samples with 40/60 pen grade Binder

Test results of mixes with <u>100/150 penetration binder</u> at 45^0C & 30^0C:

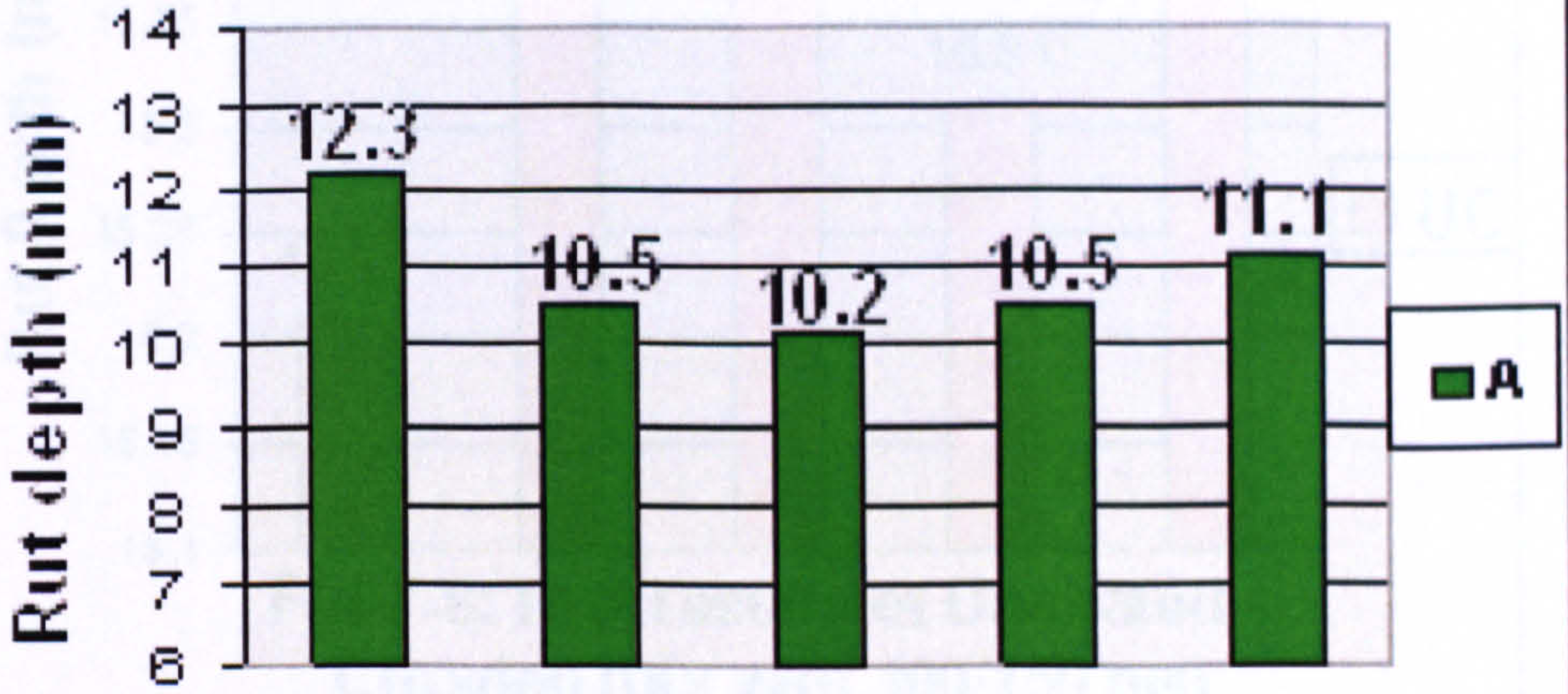


Fig. 7-6 : Test results of Arcow mix with 100/150 pen binder @ 45^0 C

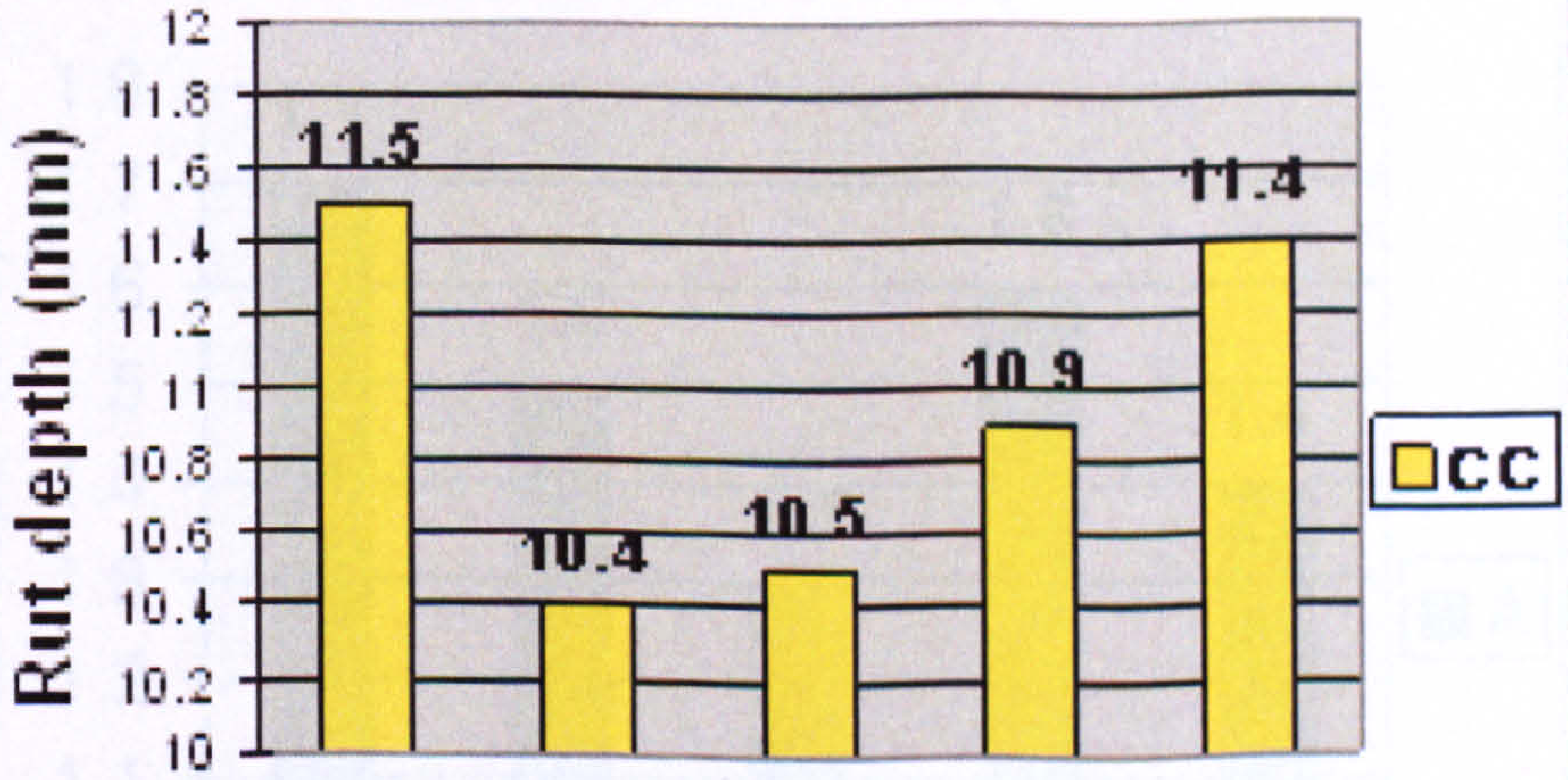


Fig. 7-7: Test results of Coated Croxden mix with 100/150 pen binder @ 45^0C

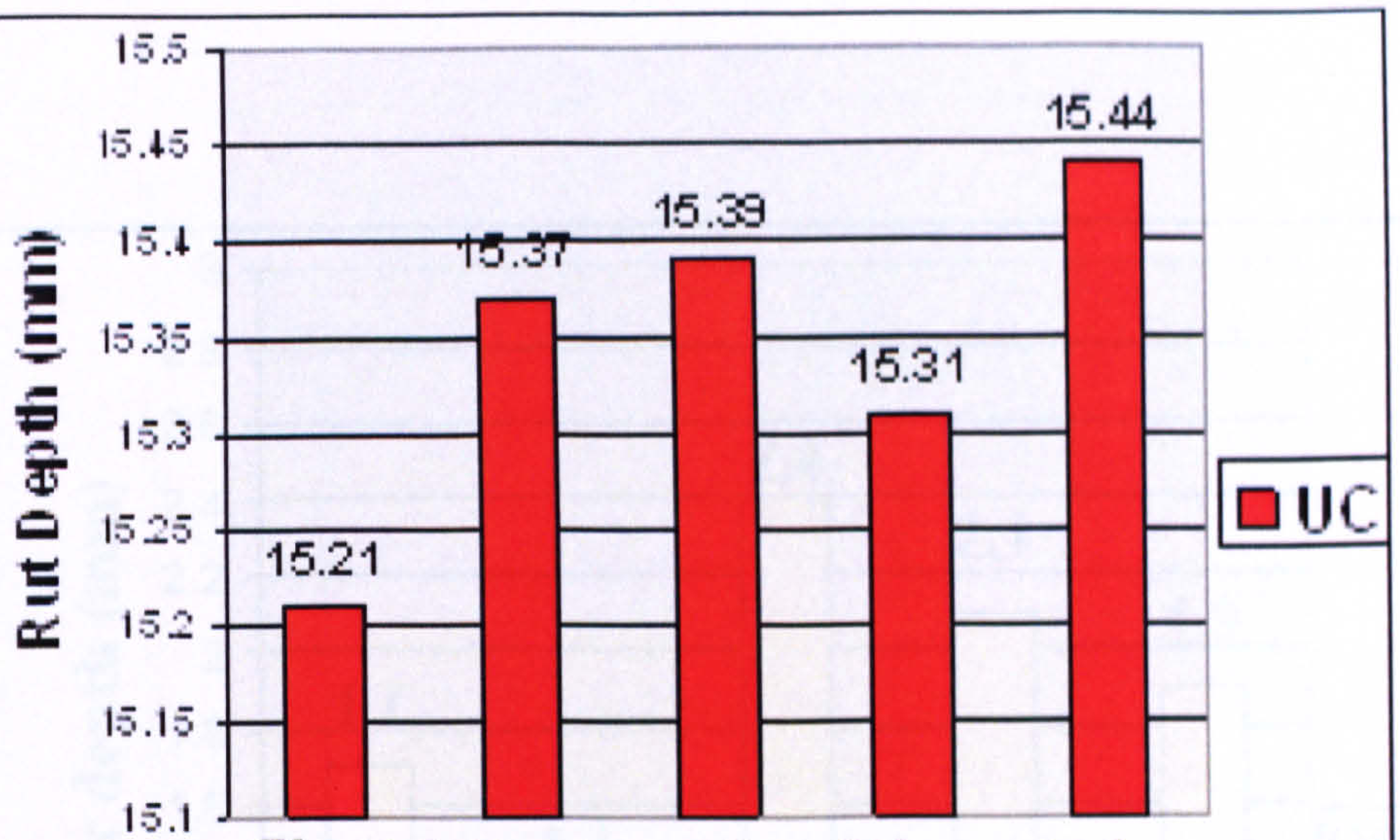


Fig.7-8: Test results of Uncoated Croxden mix with 100/150 pen binder @ 45^{0}C

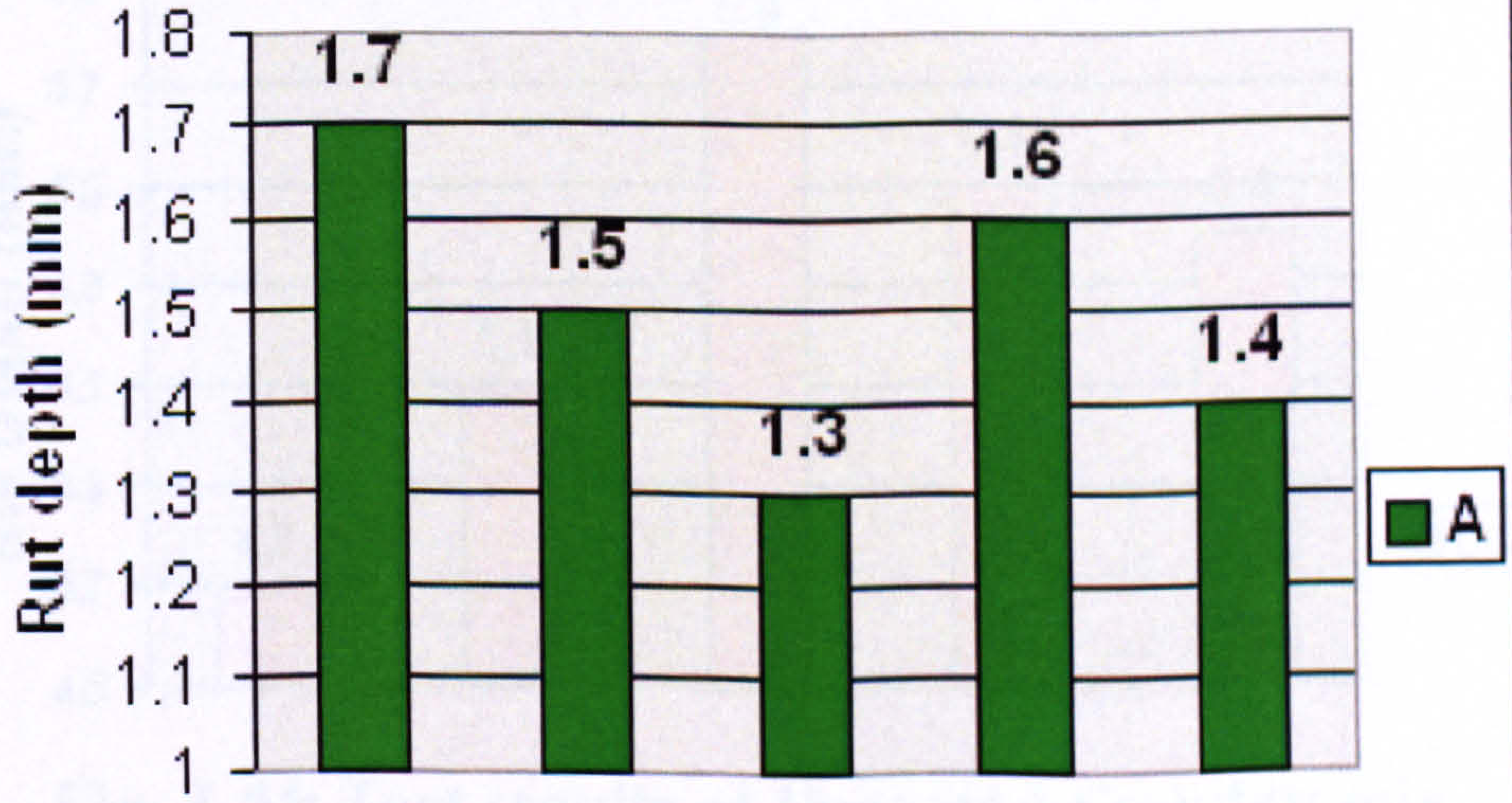


Fig.7-9 : Test results of Arcow mix with 100/150 pen binder@30^{0}C

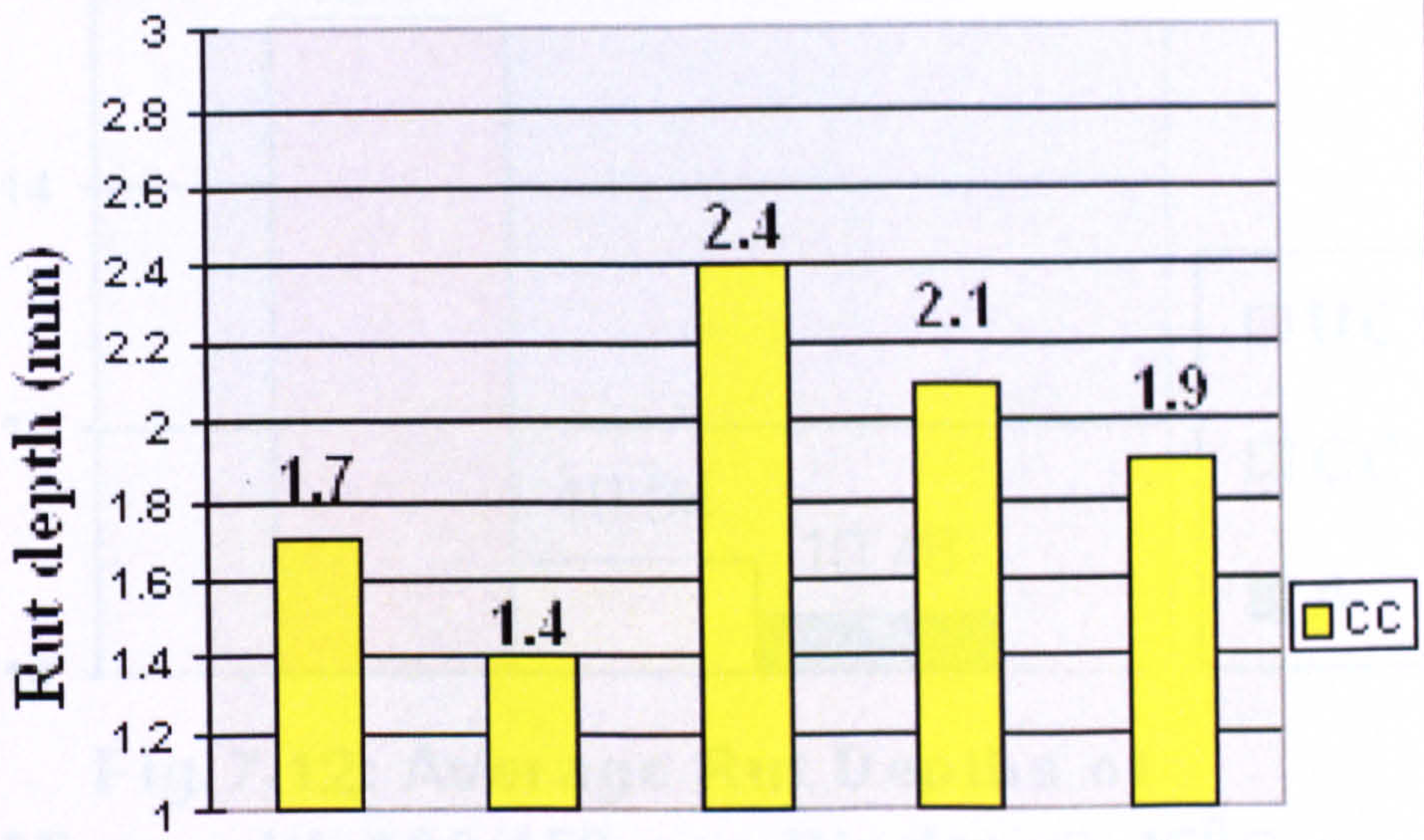


Fig. 7-10: Test results of Coated Croxden mix with 100/150 pen binder @ 30^{0}C

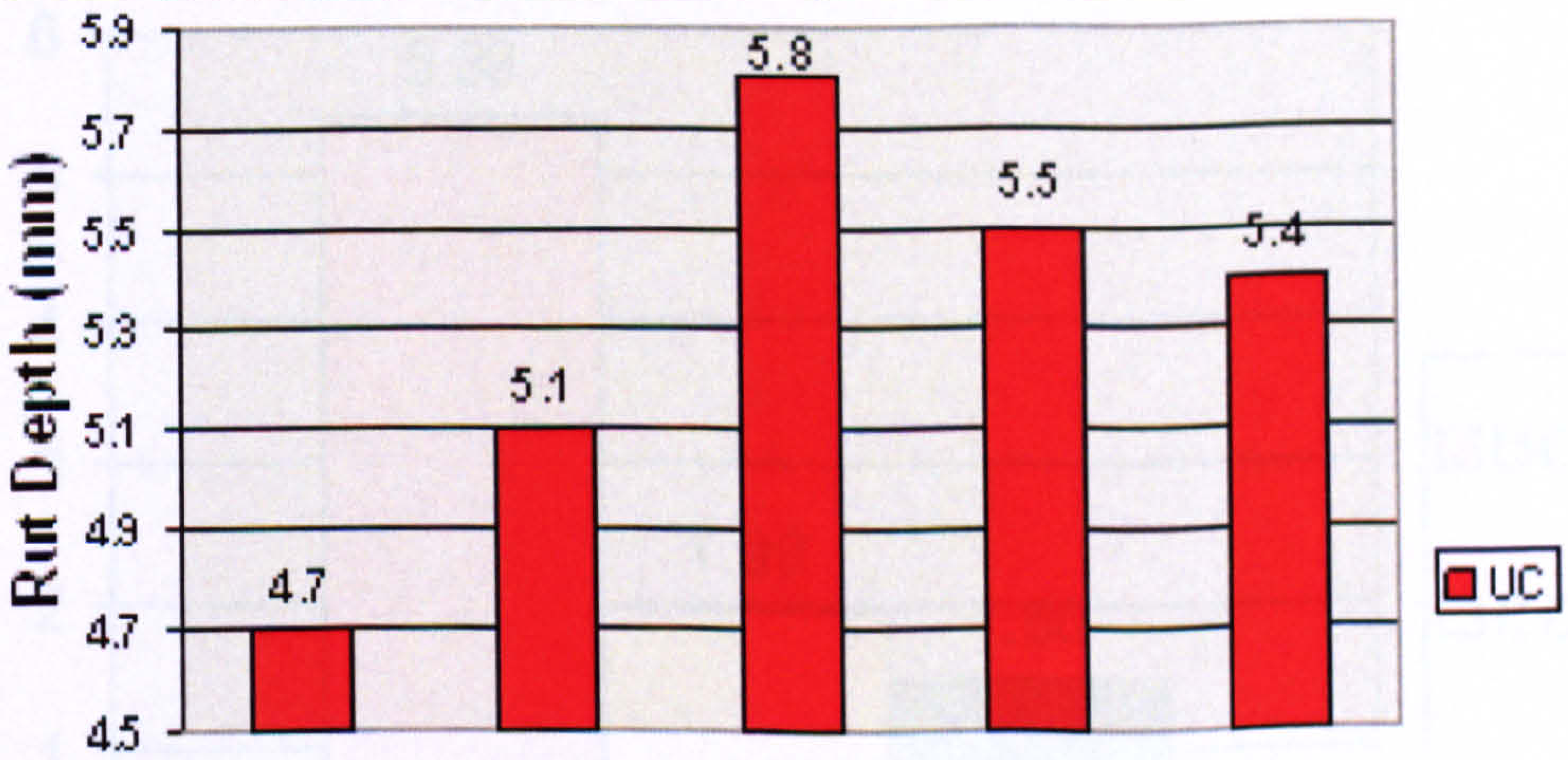


Fig. 7-11: Test results of Uncoated Croxden mix with 100/150 pen binder @ 30^{0}C

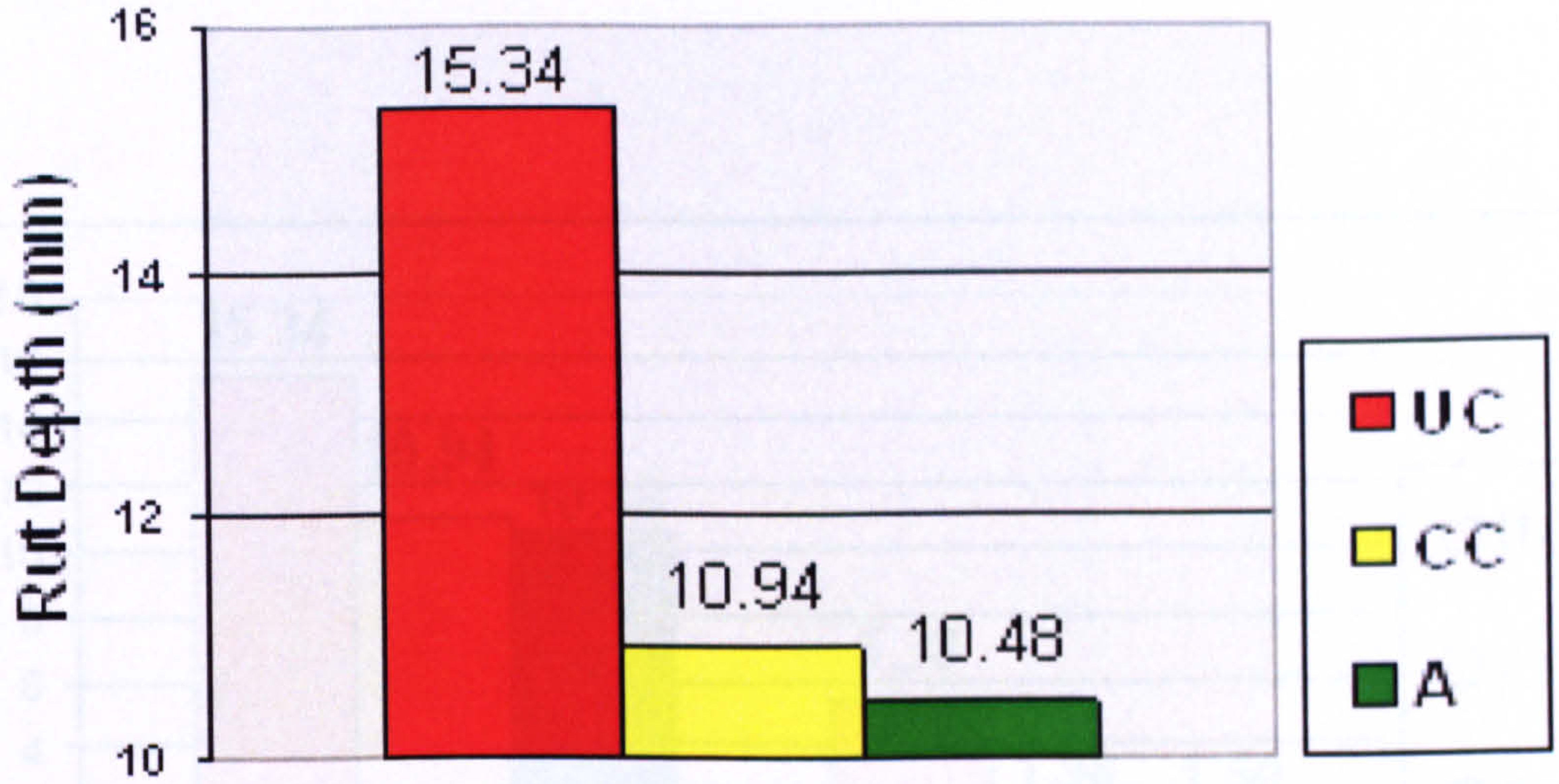


Fig. 7-12: Average Rut Depths of Mixes with 100/150 pen Binder @ 45^{0}C

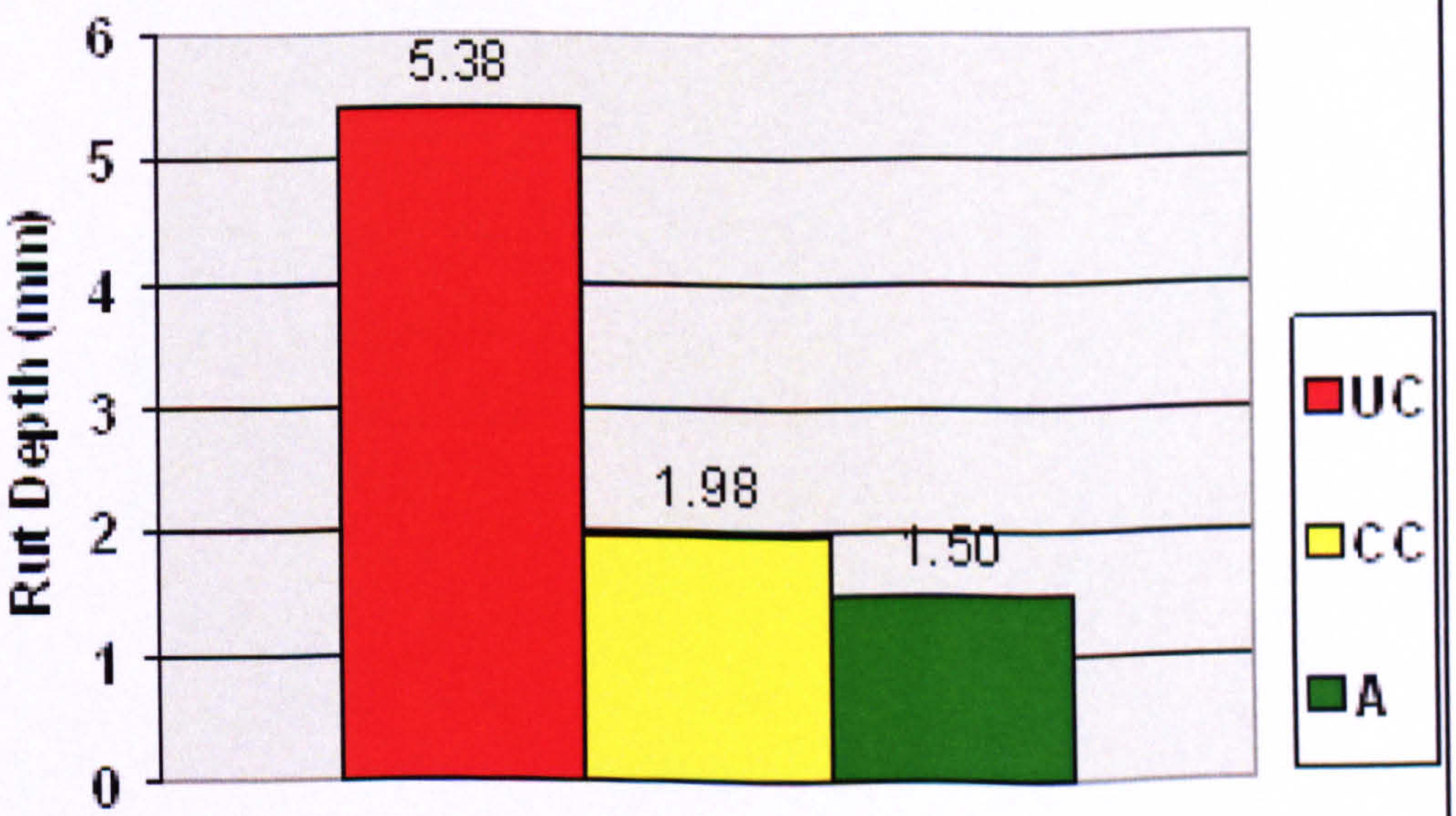


Fig. 7-13: Rut Depths of Mixes with 100/150 pen binder @ 30^{0}C

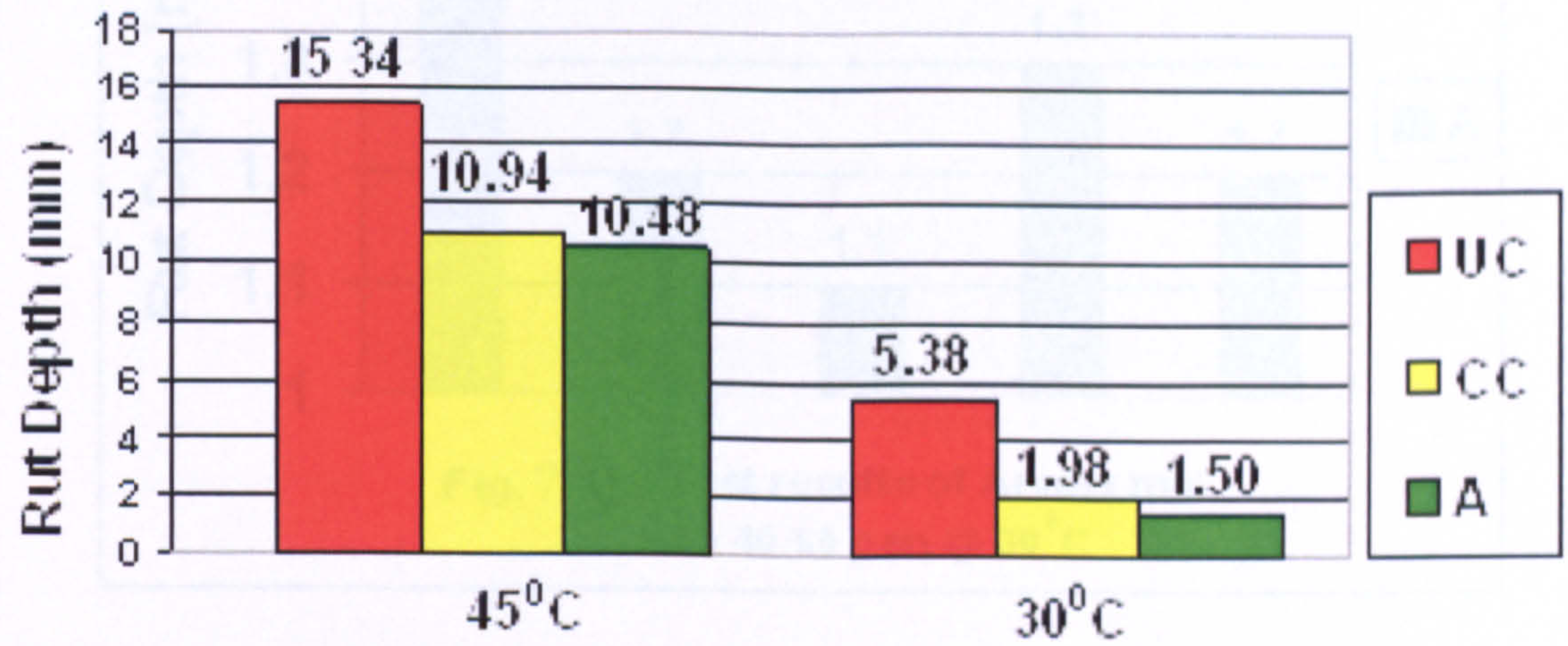


Fig. 7-14: Rut Depths of mixes with 100/150 pen binder @ 45^0C & 30^0C respectively

Test results of mixes with 40/60 penetration binder @ 30^{0}C & 45^{0}C:

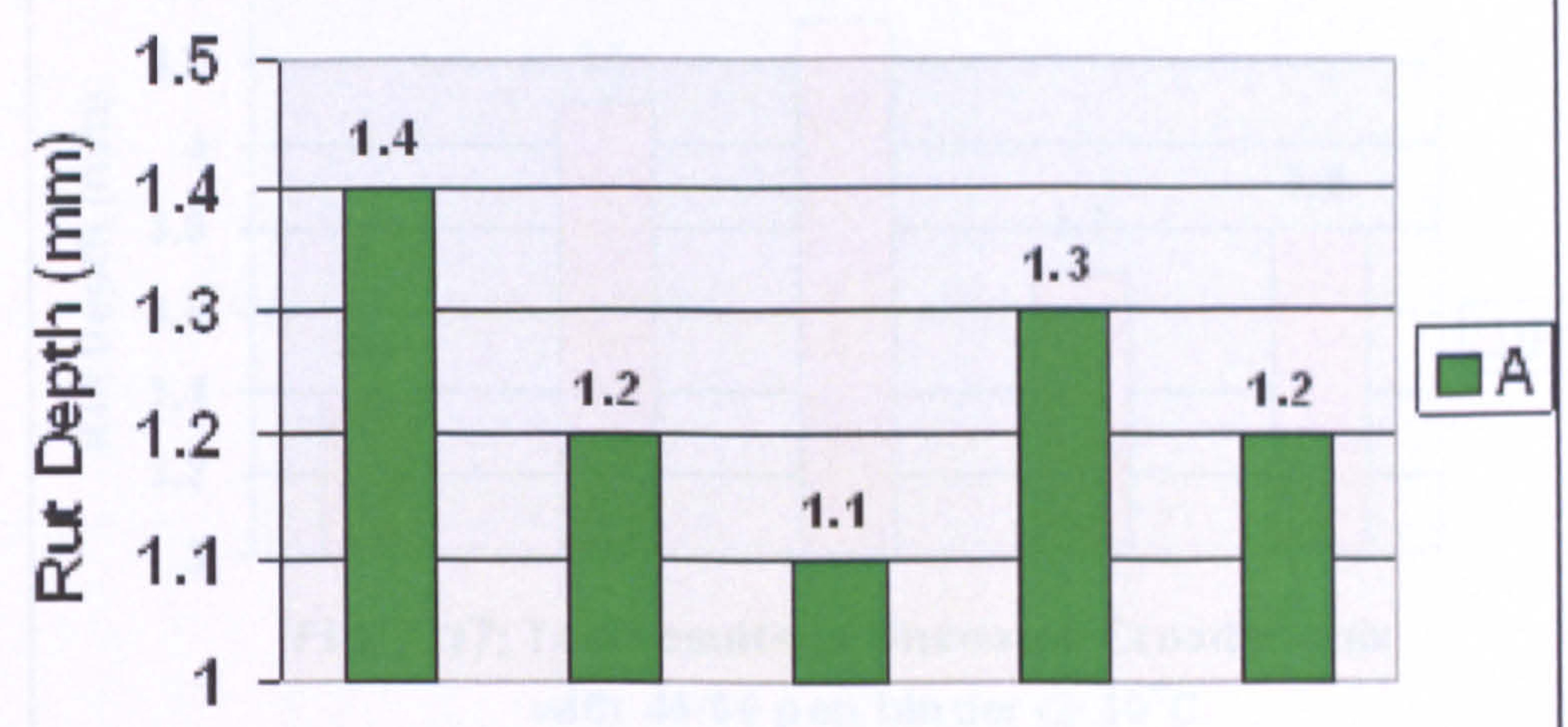


Fig. 7-15: Test results of Arcow mix with 40/60 pen @ 30^{0}C

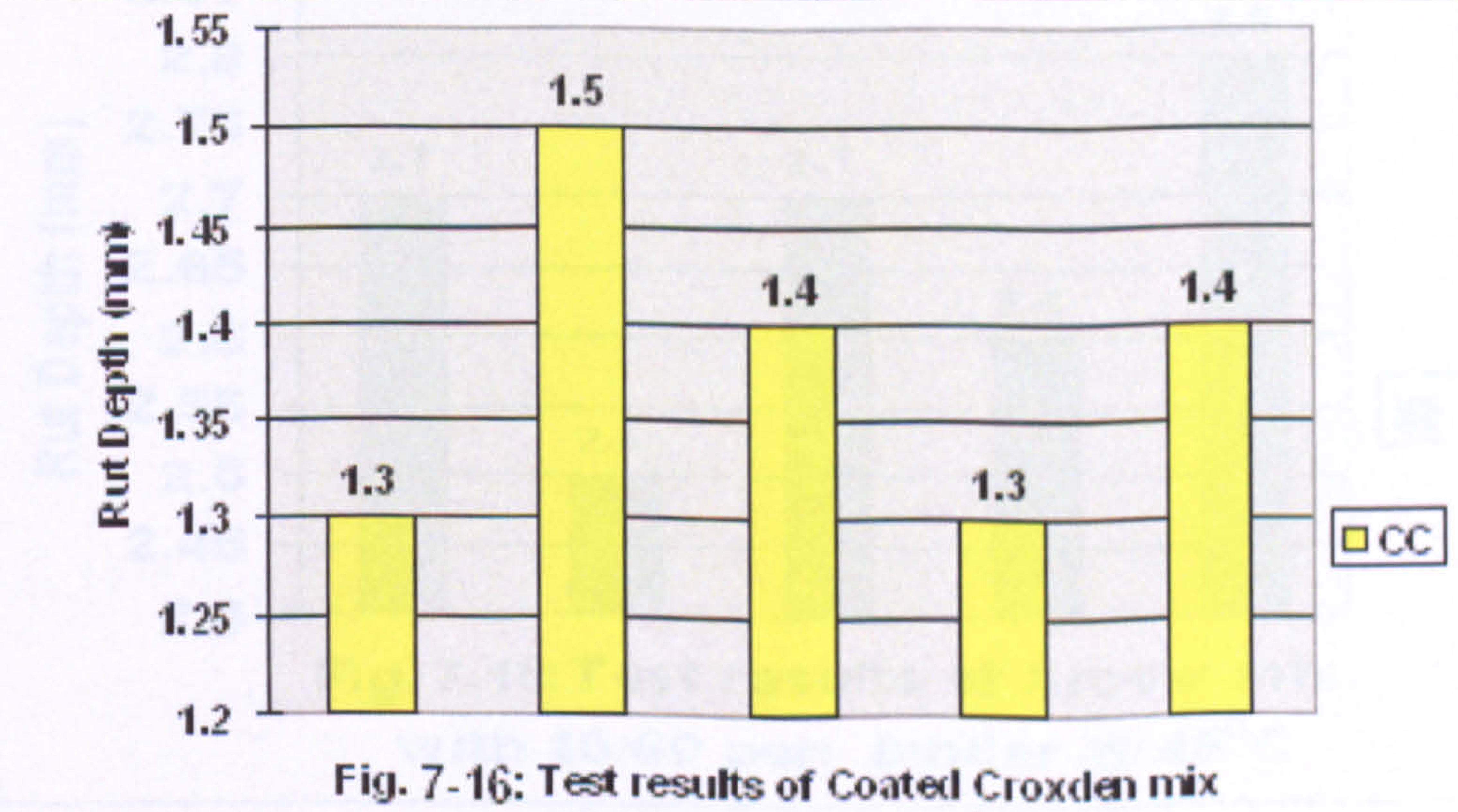


Fig. 7-16: Test results of Coated Croxden mix with 40/60 pen @ 30^{0}C

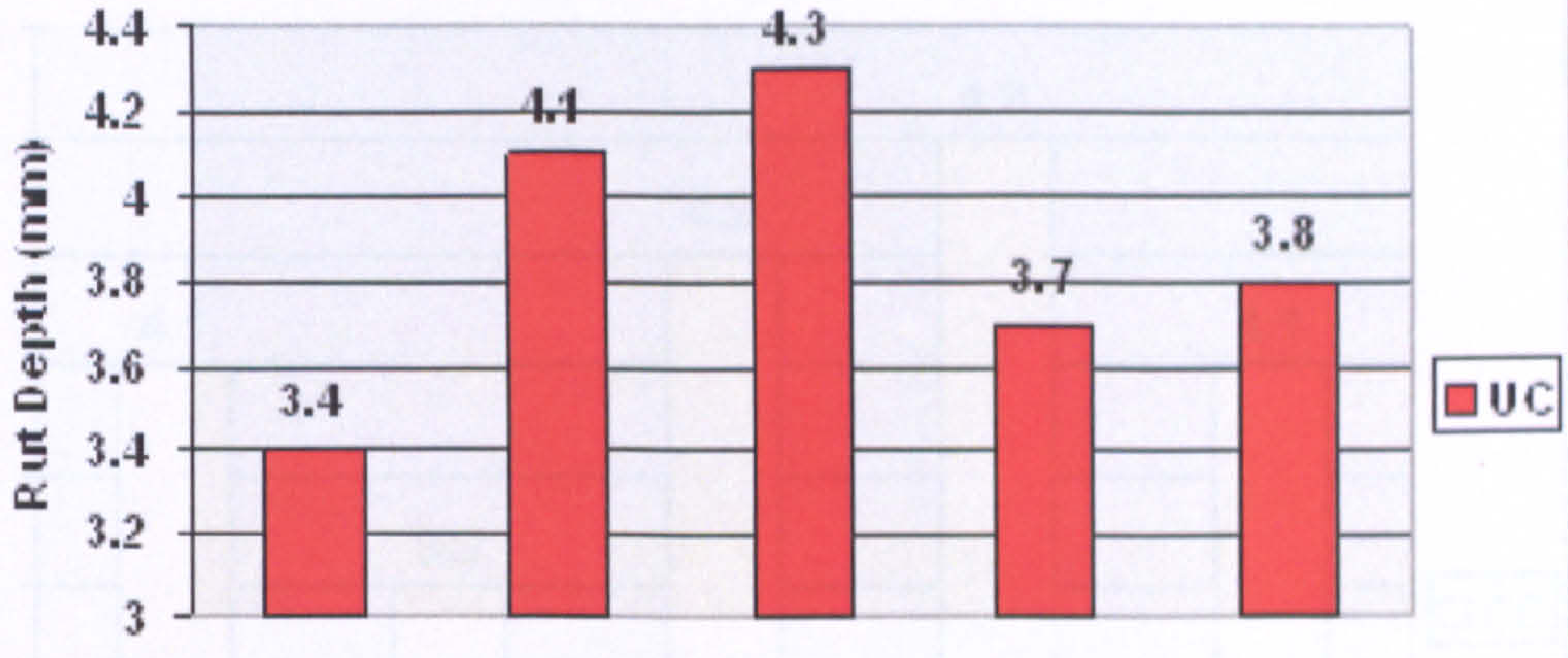


Fig. 7-17: Test results of Uncoated Croxden mix with 40/60 pen binder @ 30^{0}C

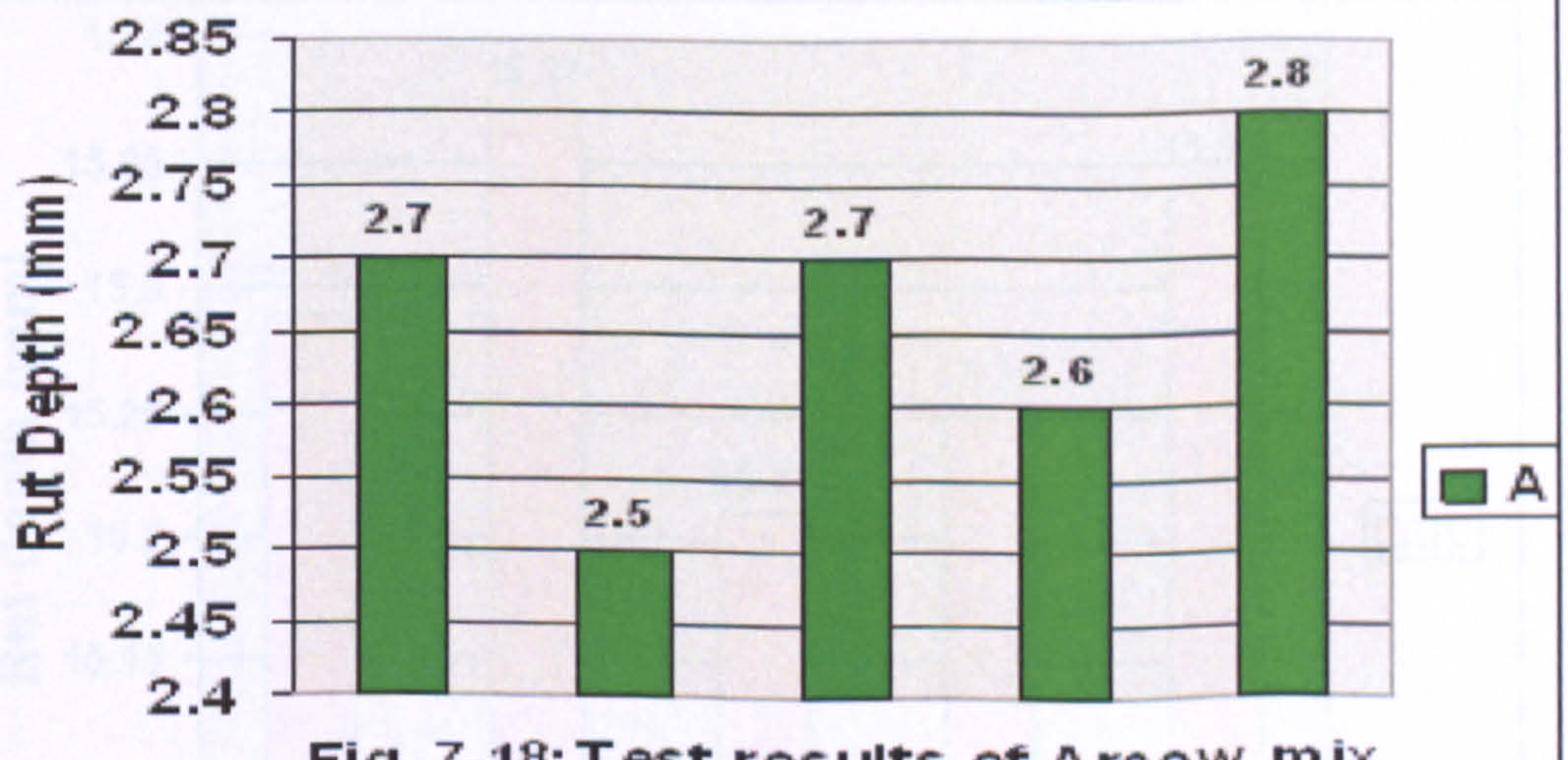


Fig. 7-18: Test results of Arcow mix with 40/60 pen binder @ 45^{0}C

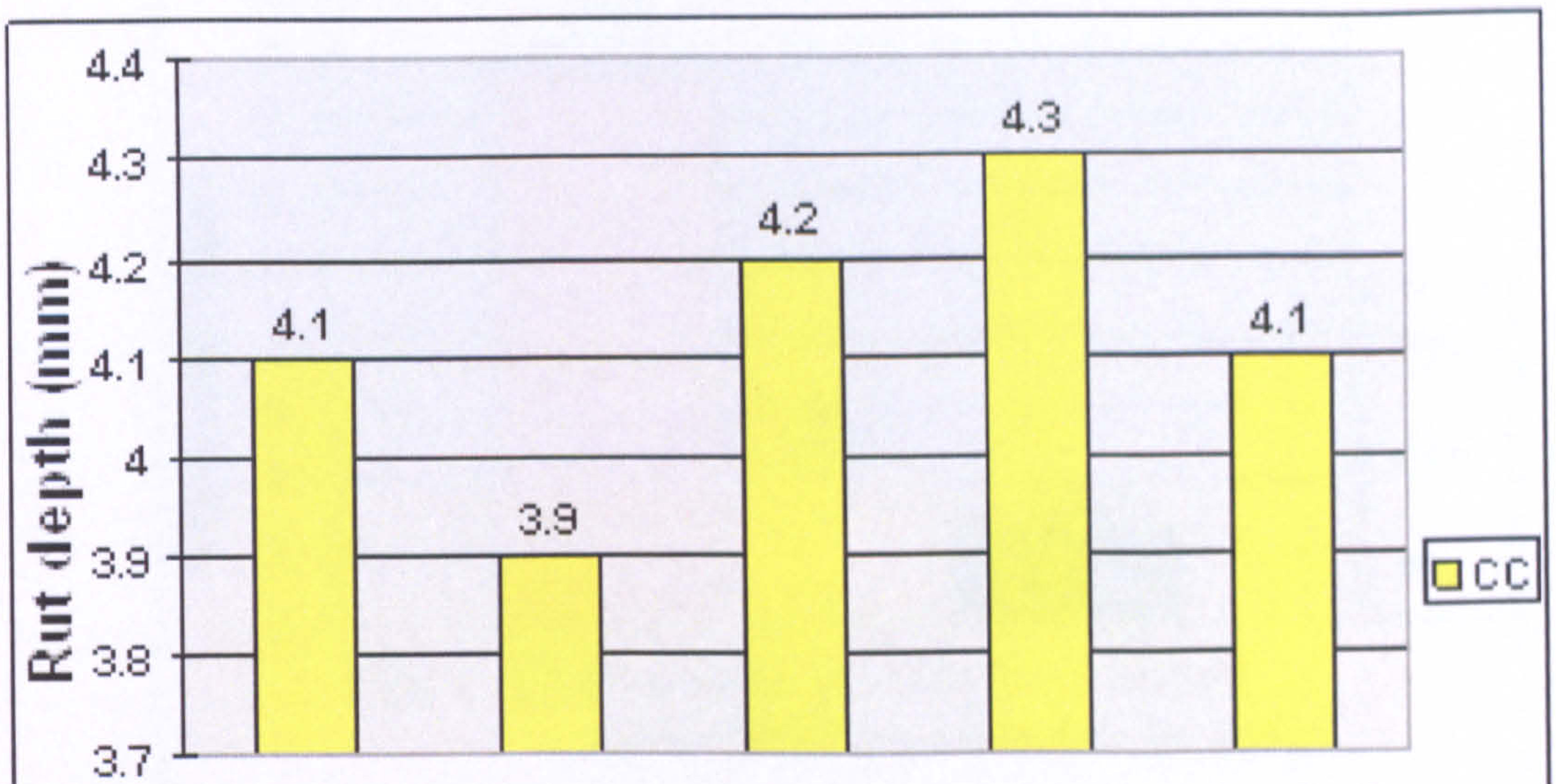


Fig. 7-19: Test results of Coated Croxden mix with 40/60 pen binder @ 45^{0}C

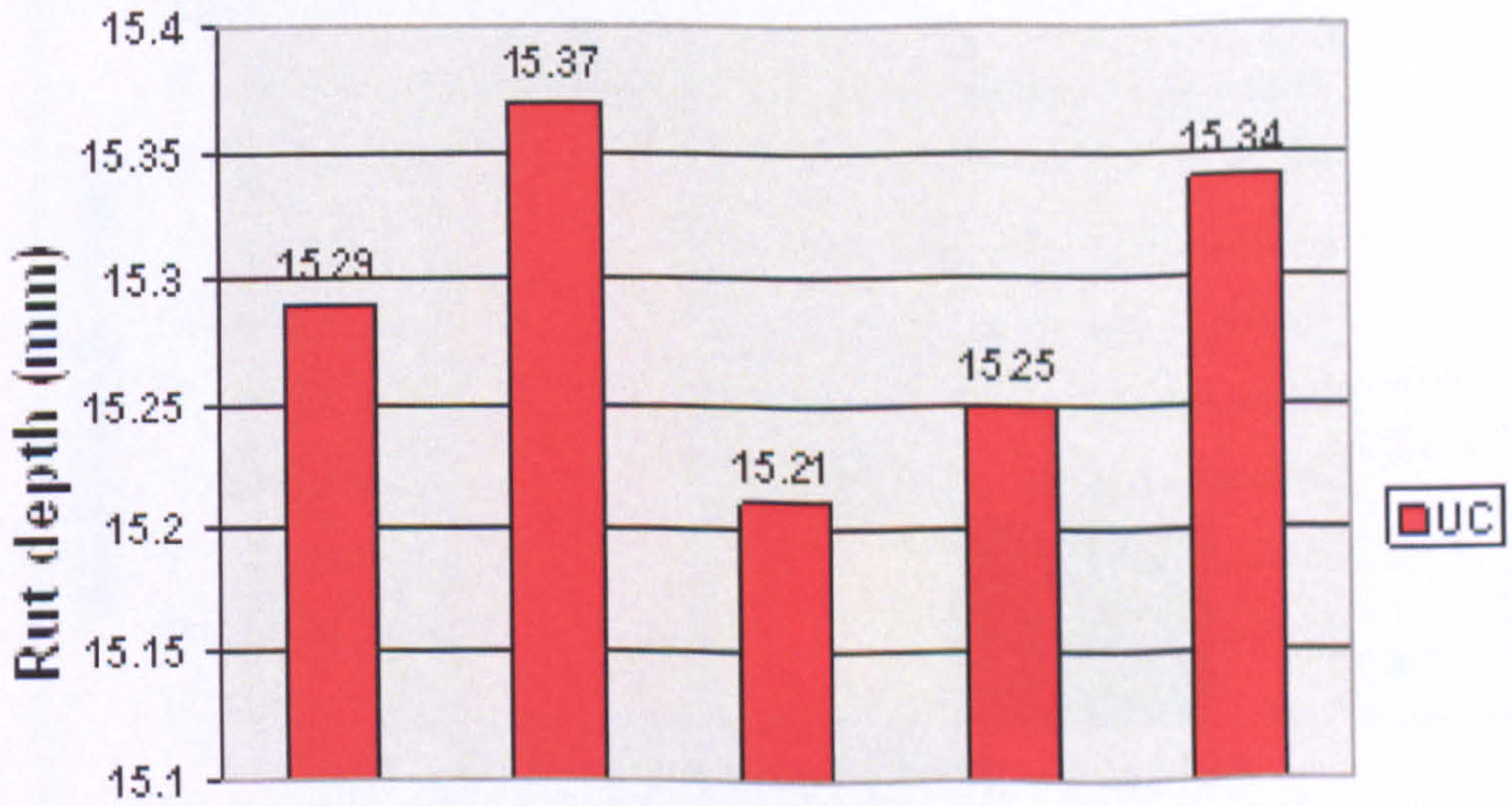


Fig. 7-20: Test results of Uncoated Croxden mix with 40/60 pen binder @ 45^{0}C

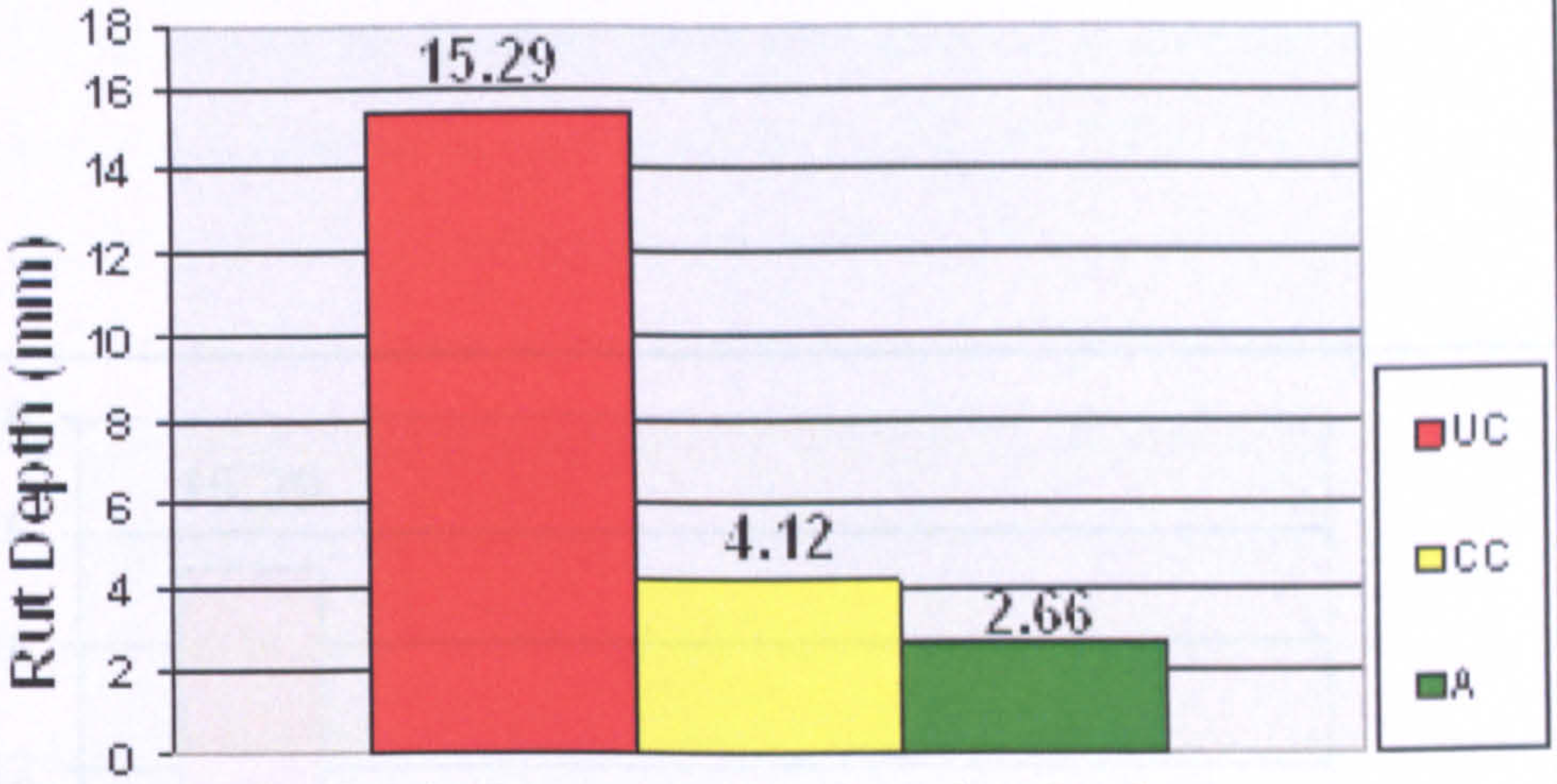


Fig. 7-21: Average rut depth of mixes with 40/60 pen binder @ 45^0C

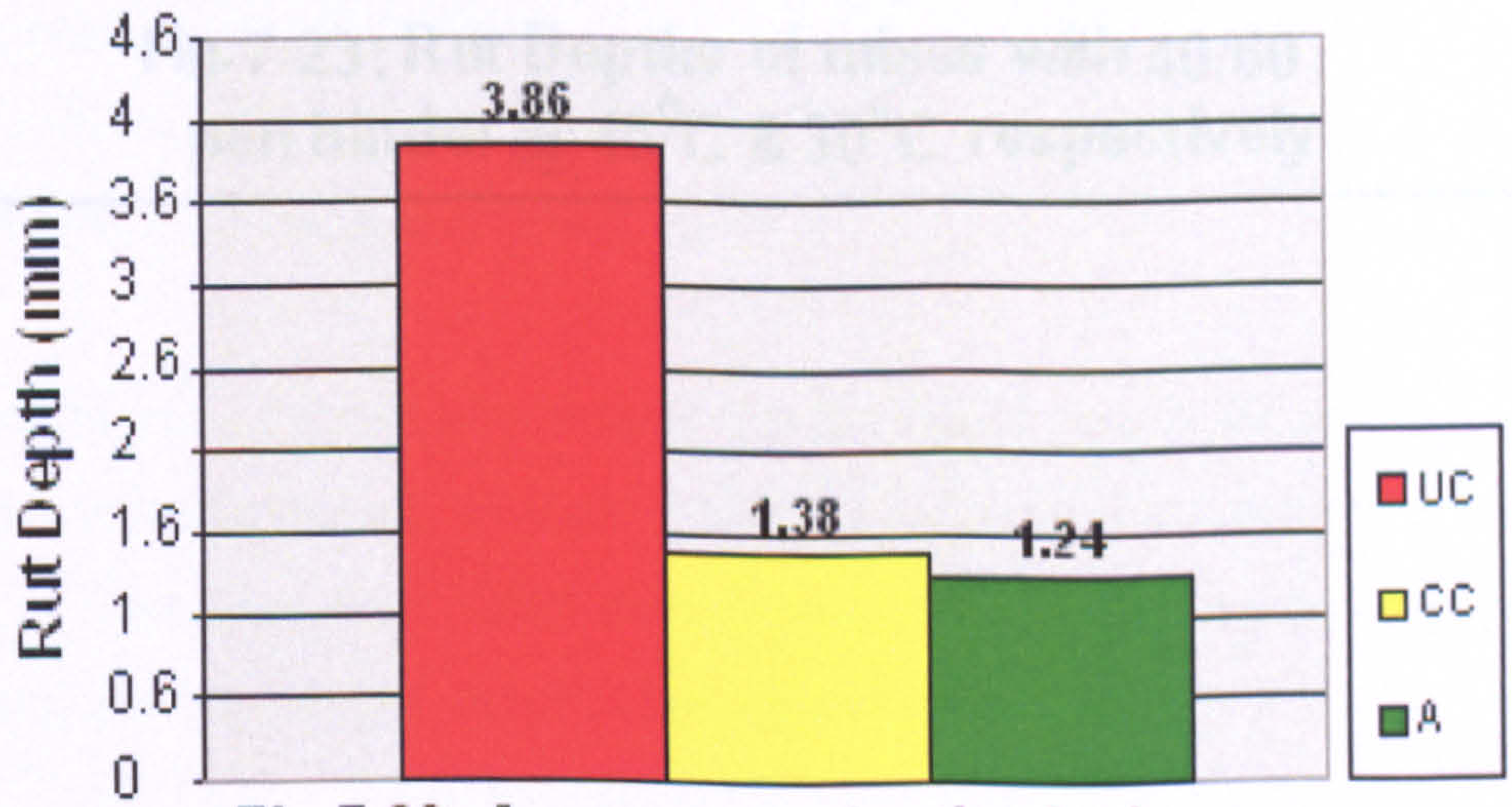


Fig.7-22: Average rut depth of mixes with 40/60 pen binder @ 30^0C

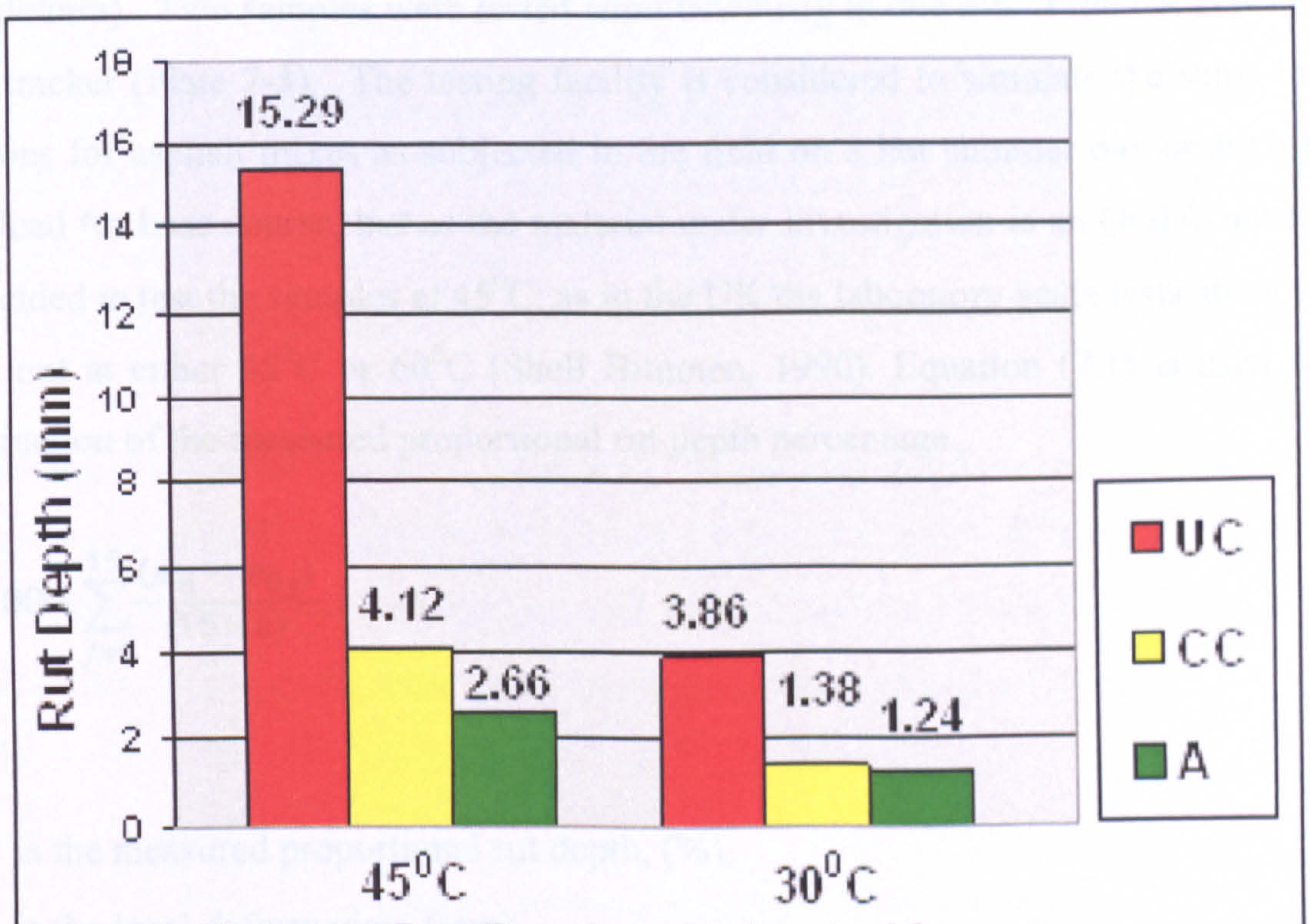


Fig.7-23: Rut Depths of mixes with 40/60 pen binder @ 45^{0}C & 30^{0}C respectively

7.5 Rutting Susceptibility by Large Scale Wheel Tracker

The rutting susceptibility of the three mixtures was assessed as a repetitive load of 5000N ± 50 N was applied via a pneumatic tyre (namely Trelleborg T522 BV pneumatic tyre, 400 mm diameter and 80 mm wide) passing on the surface of the sample at a frequency of 1 Hz (67 cycle/min). Two samples were tested simultaneously in one run of the CRT-WTENL – Wheel tracker (Plate 7-3). The testing facility is considered to simulate the same loading conditions for asphalt mixes as subjected in the field on a hot summer day under a heavy traffic load for base course, but as the material under investigation is an OGFC material, it was decided to test the samples at 45^0C, as in the UK the laboratory-scale tests are normally carried out at either 45^0C or 60^0C (Shell Bitumen, 1990). Equation (7.1) is used for the determination of the measured proportional rut depth percentage.

$$P_i = 100 \times \sum_{j=1}^{15} \frac{(m_{ij} - m_{0j})}{(15 \times h)} \tag{7.1}$$

Where:

P_i is the measured proportional rut depth, (%),

m_{ij} is the local deformation, (mm),

m_{0j} is the initial measurement at the j location and

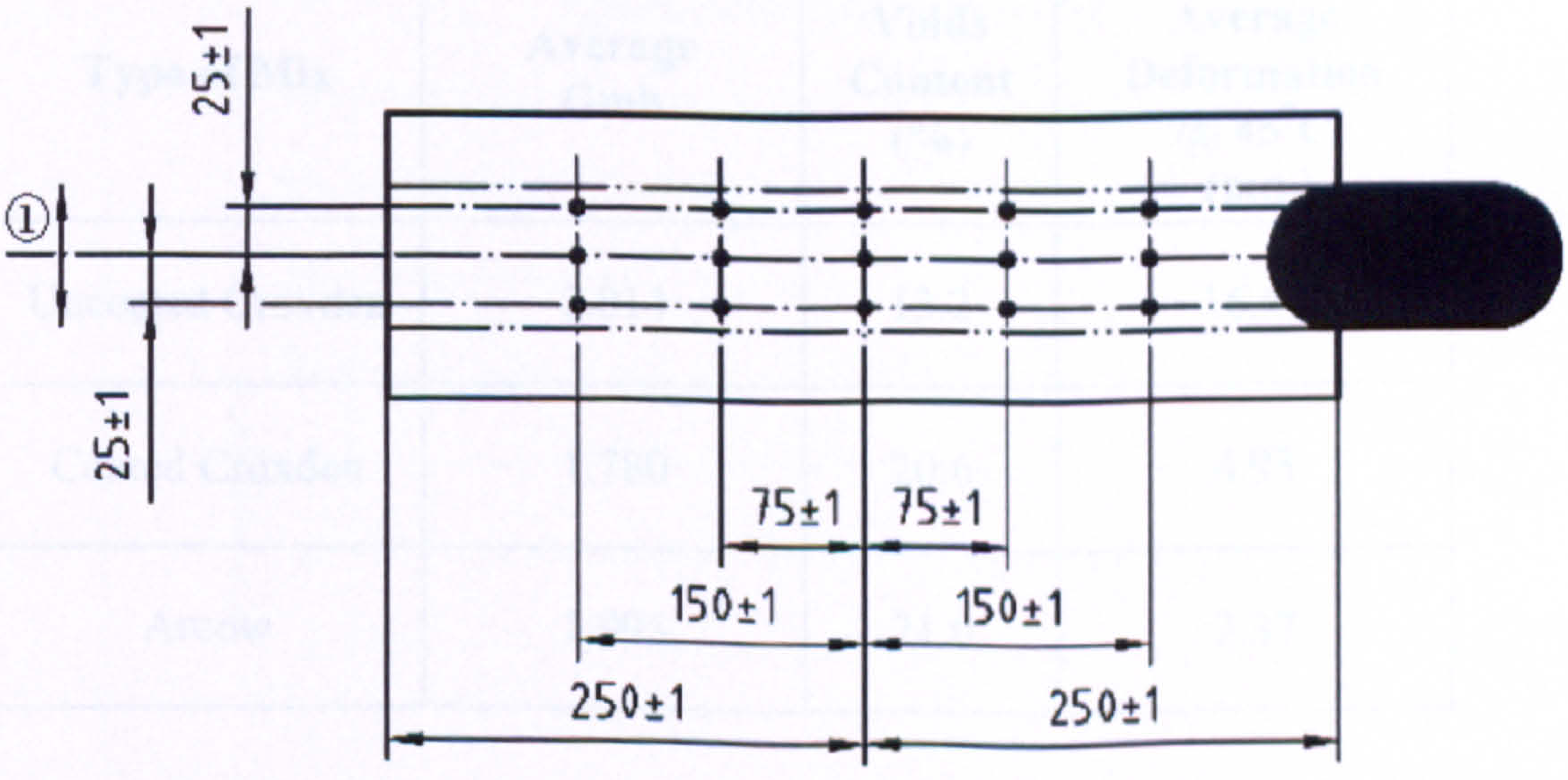
h is the specimen thickness, (mm).

Due to the health and safety requirements it was decided to reduce the number of the (500 x 180 x 50) mm samples by half, as there was some difficulty in manufacturing them. Only 12 samples of this type have been manufactured in the laboratory, using soft binder (100/150 pen grade binder) and tested at temperature of 45°C.

Before starting the test, 1000 cycles were applied at ambient temperature of 20°C, to densify the mixture and to provide a smoother surface. The thickness of each sample was then calculated by averaging 15 thickness measurements taken at 15 standard positions (Fig.7-24) using a digital gauge with a minimum accuracy of 0.1 mm. This thickness was considered as the preliminary thickness of each sample. Two Arcow mix samples were set to be tested at 60°C but resulted in excessive deformation to both samples then it was decided to decease the test temperature to 45°C (Shell Bitumen, 1990).

Another set of two samples were then heated to the new testing temperature of 45 ±2°C (Shell Bitumen, 1990) for 12 hours. The pressure of the tyres was set at 600 ± 30 kPa at the beginning of each test; The rut depth and the number of cycles have been recorded at 100, 300, 1000, 3000 and 10000 cycles for the open graded surface course mixes (BS EN 12697-22, 2003); even though, the test was curried out up to 30 000 cycles to asses the rutting potential of the new upgraded material as well as the control mix. Test results for Arcow (control), uncoated and Cement coated mixes are plotted in Figures (7-25) below.

Arcow mix showed the highest resistance to the permanent deformation followed by the Cement coated mix. Uncoated mix had shown a high rutting susceptibility as it suffered a severe plastic deformation and the test had to be terminated when the number of load cycles reached 13364 cycles and rutting reading was 16.51mm (Table 7-6).



Key:

① Transisting of wheel

Dimensions in millimetres

Fig. 7-24: Schematic View Showing Points of Rut Measurement **(BS EN 12697-22, 2003)**

When testing 50mm slabs, a mixture is acceptable according to the French specification (French Standard, 1998) if the average percent rut depths at 1,000 and 3,000 cycles are less than or equal to 10% and 20%, respectively. When testing 100mm slabs, a mixture is acceptable if the average percent rut depth at 30,000 cycles is less than or equal to 10% of the sample average depth.

Figure 7-25 shows that specimens deform gradually at the beginning of the test and the rate of deformation levels out for mixes that are stable. The amount of permanent deformation per cycle decreases and becomes quite steady after a certain number of load cycles. For unstable mixes, the rate of deformation actually increases again as more cycles are added. This can be seen occurring with the three types of mixes but more obvious with the uncoated Croxden mix, as it started to deform more rapidly than the other two mixes. The cement coated Croxden mix had shown an outstanding result when compared to the uncoated mixture.

Type of Mix	**Average Gmb**	**Voids Content (%)**	**Average Deformation @ 45^{0}C (mm)**
Uncoated Croxden	2.014	13.2	16.51
Coated Croxden	1.780	20.6	4.93
Arcow	1.995	21.0	2.37

Table 7-5: Deformation of (500 x 180 x 50 mm) samples with 100/150 pen grade Binder

Type of Mix	Rut Depth (mm) @ 1000 cycles	Rut Depth (mm) @ 3000 cycles	Rut Depth (mm) @ 10 000 cycles	Rut Depth (mm) @ 30 000 cycles
Uncoated Croxden	2.97	5.68	16.51	-
Coated Croxden	0.81	1.37	2.41	4.93
Arcow	0.6	0.96	1.14	2.37

Table 7-6: Average Rut Depths of mixes at 1000, 3000, 10 000 & 30 000 load cycles @ 45^0 C

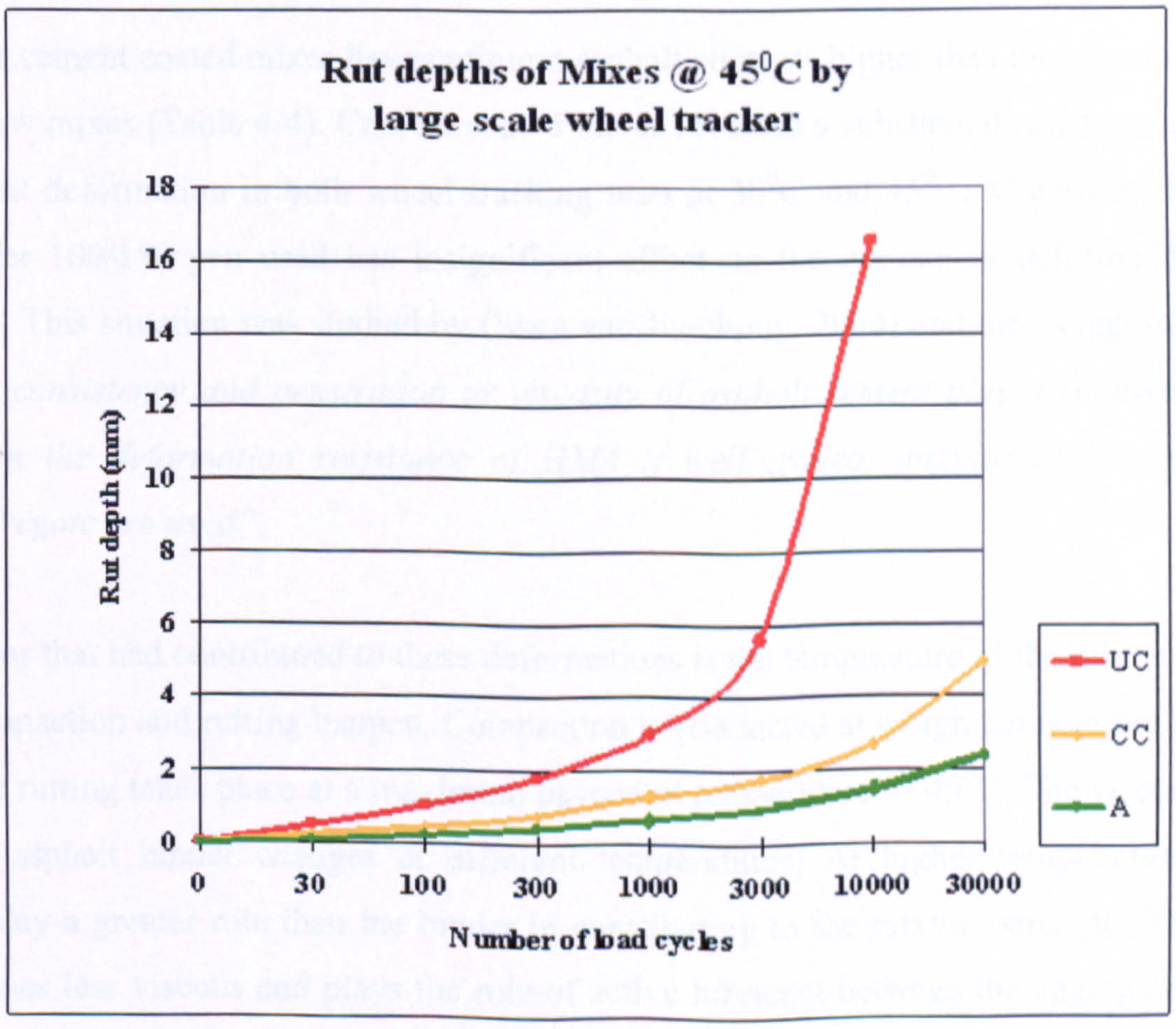


Fig. 7-25: Rut depth @ 45^0C with 100/150 pen graded binder (by Large Device Wheel tracker)

7.6 Results Discussion and Summary of Findings

In general, it is understood, through research works that high plastic flow susceptibility is due to a mix having high binder content. High plastic susceptibility may lead to high permanent deformation, as the high level of binder content in the mix can cause the loss of internal friction between aggregate particles, and that makes the applied loads to be carried out by the asphalt binder rather than the aggregate structure (Roberts et al, 1996). Although Cement Coated mixes have higher asphalt contents than the other two mixes this is not the case of deformation following the above comment by Roberts et al (1996). This can be explained by the fact that the high ratio of internal friction has a fundamental roll in deformation resistance. However, the loss of internal friction between aggregate particles at a high temperature of 45^{0}C was observed in uncoated mixes as they experienced the highest deformation as it had gained a steeper gradient after several hundreds of load cycles on its way to failure (Figure 7-25).

Although the cement coated mixes have optimum asphalt contents higher than the uncoated and the Arcow mixes (Table 4-4). Cement coated mixes revealed a substantial resistance to the permanent deformation in both wheel tracking tests at 30^{0}C and 45^{0}C. Moreover, the asphalt binder 100/150 pen used has insignificant effect on the permanent deformation performance. This situation was studied by (Shen and Jia-chong, 2004) and they suggested that: "*The consistency and penetration or viscosity of asphalt cement play a relatively small role in the deformation resistance of HMA if well-graded, angular and rough textured aggregate are used*".

Another factor that had contributed to these deformations is the temperature of the mixtures at which compaction and rutting happen. Compaction is conducted at a high temperature of 135^{0}C where rutting takes place at a maximum pavement temperature of 45^{0}C. The viscous property of asphalt binder changes at different temperatures. At higher temperatures, aggregates play a greater role than the binder in contributing to the mixture strength. The binder becomes less viscous and plays the role of active lubricant between the aggregates, and thus influences the compaction process. The results of rutting analysis clearly showed that the rut susceptibility for Uncoated Croxden mix was much higher than that of Cement Coated Croxden mix and Arcow mix (Tables 7-3 to 7-5). In other words, it is evident that specimens containing Cement Coated Croxden aggregates provided much lower rut depths compared to specimens made with Uncoated Croxden aggregates, i.e. the reduction in rutting susceptibility was mainly related to the contribution of the cement paste added to the mixture containing modified aggregates.

Interestingly, the results of the RLAT and the wheel-tracking tests are very similar in relative terms. The results from the RLAT and the wheel tracking tests are highly related, at the corresponding testing temperatures, and these have led to the conclusions that the amount of axial deformation had decreased significantly by the introduction of cement coated aggregates within the bituminous mixtures, and that had led to the subsequent increase in their resistance to permanent deformation, (rutting). On the other hand, the reduction in bulk density of the coated mixtures was also thought to be closely related to this increase in the rutting resistance, which is in turn related to the increase in the coated aggregates' angularity and surface roughness.

CHAPTER 8

The Concept led to the creation of the Low Cost Asphalt Testing Equipment (LoCATE)

8.1 Impact of Traffic Loading & Environmental Conditions

The structural configuration of the pavement structural system, including the materials that are to be used, is normally fixed by the design or the policy of the beneficial party. The structural system is then subjected to the impact of traffic loading under the existing environmental conditions that affect its performance. The response and performance of the pavement system is therefore subject to the following leading factors that have variable levels of control and are time dependent to a greater or lesser degree:

a) Pavement materials
b) Traffic conditions comprising:
    - Wheel load(s) that can be single or multiple axle
    - Wheel load(s) that can be static or dynamic
    - Wheel load(s) that wanders laterally
    - Suspension systems
    - Tyre pressure/contact stress
    - Tyre type
    - Speed
c) Environmental impact such as:
    - Wind and radiation
    - Temperature
    - Water in a variety of forms

The design of a road is primarily dependent on the number of loaded axles that will pass over it in a given life span. The UK's standard axle has traditionally been 10.5 tonnes based on a minimum of 5 axles and a gross weight of 38 tonnes. In January 1999 an EU directive, which allows an 11.5 tonnes axle and a gross weight of 40 tonnes, came into force. As road wear is proportional to the weight of an axle rose to its 4^{th} power, this means that an 11.5

tonnes axle causes about 45% more wear than a 10.5 tonnes axle (Assoc. of British Drivers, 2001). The combined effects of different axle types in the pavement design traffic then must be considered when designing it. There are two common approaches by which the combined effects are evaluated: one based on the hypothesis of accumulative damage where, the allowable number of repetitions by each load type can be established for a given form of pavement damage. A damage ratio (D_i) for load type (i) is defined as:

$$D_i = (n_i / N_i) \tag{8.1}$$

Where:

n_i is the design repetitions of load type (i) and

N_i is the allowable repetitions.

The total amount of damage caused by the mixed traffic is computed as the sum of damaged ratios of all load types. The method of equivalent load expressed the effects of mixed loads of different configurations and magnitudes in terms of an equivalent number of a preselected standard load. However, it must be noted that there exists different basis of damage upon which a load can be converted into the pre selected standard load, and the computed answers of equivalent number vary with the basis adopted. The most common form of the equivalent load used in pavement design is the "equivalent single axel load of 80 kN" abbreviated as ESAL (Fwa, 2006).

The subject of vehicle–pavement–environment interaction is complex and probably one of the most controversial aspects of Asphalt Pavement Testing (APT). Croney and Croney (1997) noted that age strengthening seems to be a key factor in the performance of well-designed pavements, Nunn et al (1997) also reported on this phenomenon. These effects were found to be the result of changes in the asphalt mixes over time, which gave support to the statement by Croney that these environmental limitations placed APT in jeopardy. Differences between loading used during APT studies and conventional traffic have also led to questioning the applicability of APT findings to the performance of conventional in-service highways. Fwa (2006) also concurred.

8.1.1 Effect of Tyres on Pavement

Radial-ply tyres on heavy goods trucks have largely replaced cross-ply tyres (Fig. 8-1), and average inflation pressures have increased from 550kPa to 690kPa. Wide-base single tyres (super singles) have been replacing dual tyres in Europe for some time, and causes worry in the pavement engineering community as these changes in operating patterns may increase pavement damage, particularly permanent deformation (Al-Qadi and Yoo, 2006).

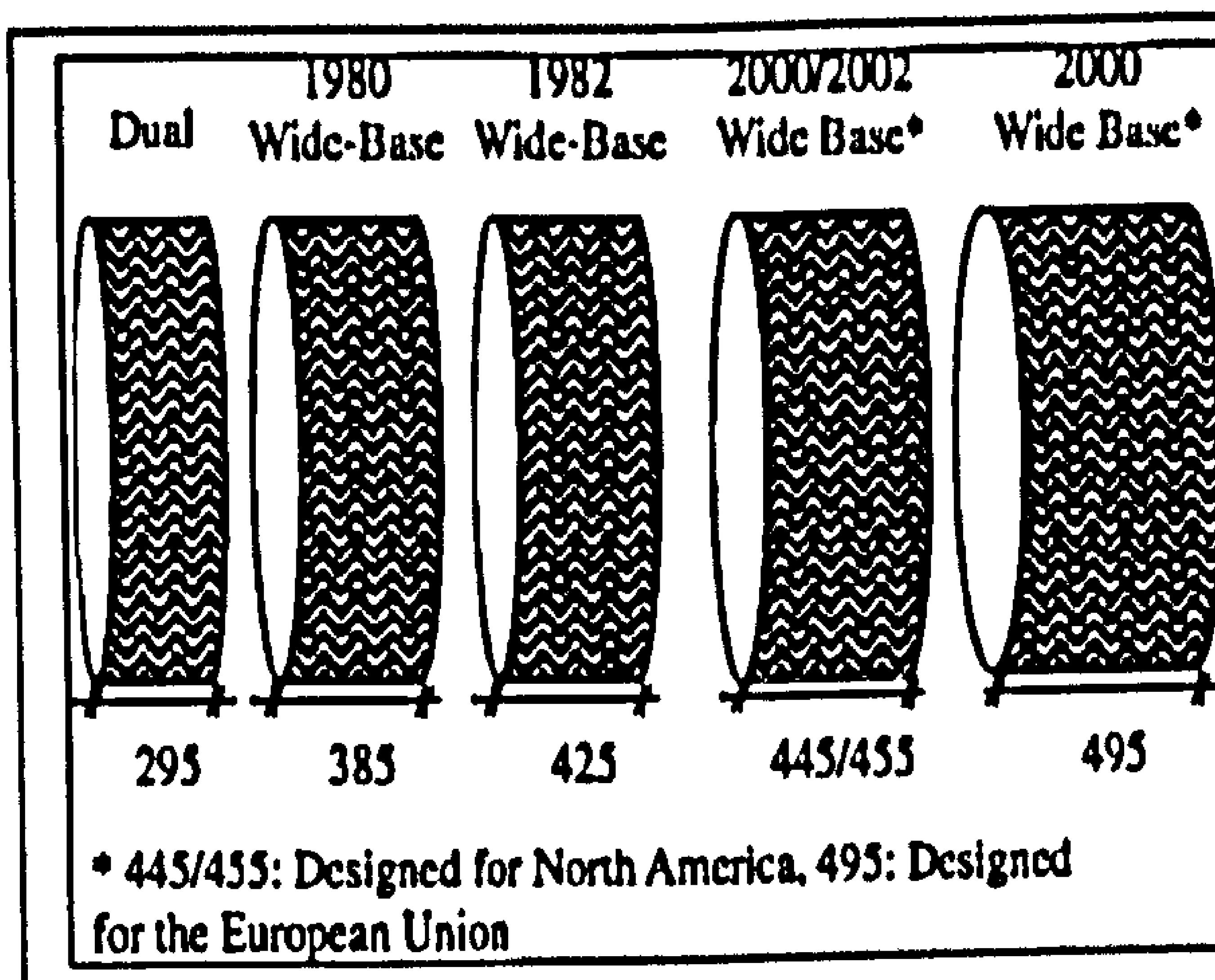


Fig. 8-1: Different Tyre Sizes
(Al -Qadi and Yoo, 2006)

Tyre types to be selected for testing facilities, based on nowadays European sales and on facts from the tyre industry on the development of new types of tyres; the following tyre types for heavy vehicles are being used (COST 347, 2001):

Current ones:

385/65R22.5, 295/80R22.5, 315/80R22.5, 12R22.5, 215/75R17.5, 12.00R20, 11R22.5 and 10R22.5

Possible future ones:

315/70R22.5, 295/60R22.5, 385/55R19.5, 385/55R22.5, 445/45R19.5 and 495/45R22.5

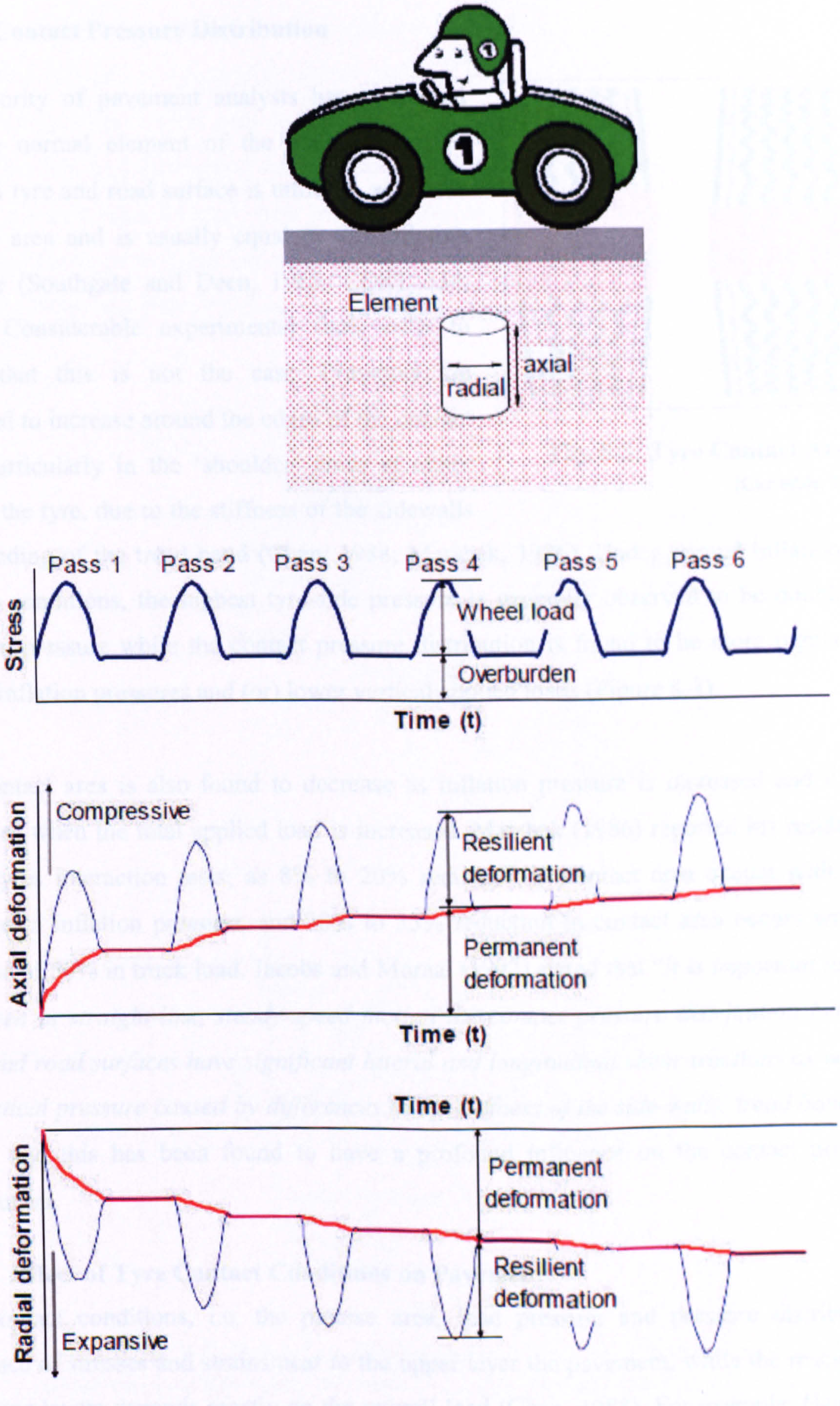


Fig. 8-2: Pavement Under traffic Loading **(Brown and Gibb, 1996)**

8.1.2 Contact Pressure Distribution

the majority of pavement analysts have assumed that the normal element of the contact pressure between tyre and road surface is uniform, acts over an oval area and is usually equal to the inflation pressure (Southgate and Deen, 1985; COST 334, 2001). Considerable experimental facts exist to imply that this is not the case. Pressures are observed to increase around the edges of the contact area, particularly in the 'shoulder' areas at either side of the tyre, due to the stiffness of the sidewalls and bending of the tread band (Chan, 1988; Marshek, 1986). Under normal inflation and loading conditions, the highest tyre-side pressure is generally observed to be double the inflation pressure while the contact pressure distribution is found to be more regular for higher inflation pressures and (or) lower vertical applied loads (Figure 8-3).

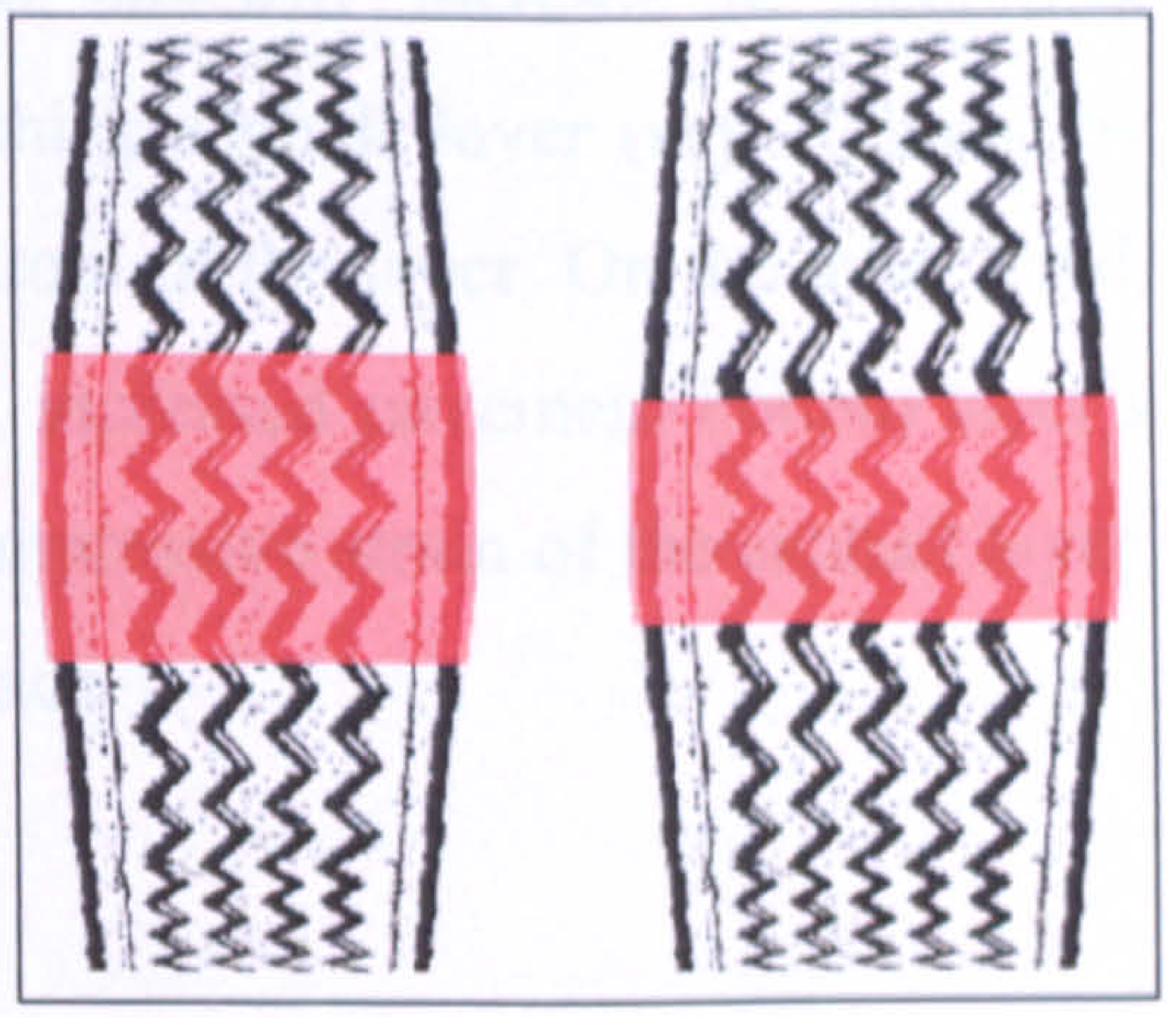


Fig. 8-3: Tyre Contact Area

(Car bible, 2006)

The contact area is also found to decrease as inflation pressure is increased and it also increases when the total applied load is increased. Marshek (1986) reported his results for truck tyres interaction tests; as 8% to 20% reduction in contact area occurs with 50% increase in inflation pressure, and 30% to 35% reduction in contact area occurs with an increase of 50% in truck load. Jacobs and Moraal (1992) stated that "*It is important to note that even in straight-line, steady-speed motion the contact pressure distribution between tyres and road surfaces have significant lateral and longitudinal shear tractions as well as the vertical pressure caused by differences in the stiffness of the side-walls, tread band and tread*" and this has been found to have a profound influence on the contact pressure distribution.

8.1.3 Effect of Tyre Contact Conditions on Pavement

The contact conditions, i.e. the precise area, load pressure and pressure distribution, influence of stresses and strains near to the upper layer the pavement, while the reaction in the lower layers depends mostly on the overall load (Chan, 1988). For example, Haas and

Papagianakis (1986) study had discovered that if tyre inflation (contact pressure) is doubled from about 400 kPa to about 800 kPa at constant load, this will increase the theoretical vertical compressive strain near to the surface of a thick asphalt layer (say 200mm) by eight times, but had little effect on the strain at the bottom of the layer. On the other hand, making the axle load doubled, with constant pressure, increased pavement bottom layer's strain by a factor of two, but had little effect on the compressive strain of the asphalt layer. Marshek (1986) and Fwa (2006) corroborated these trends.

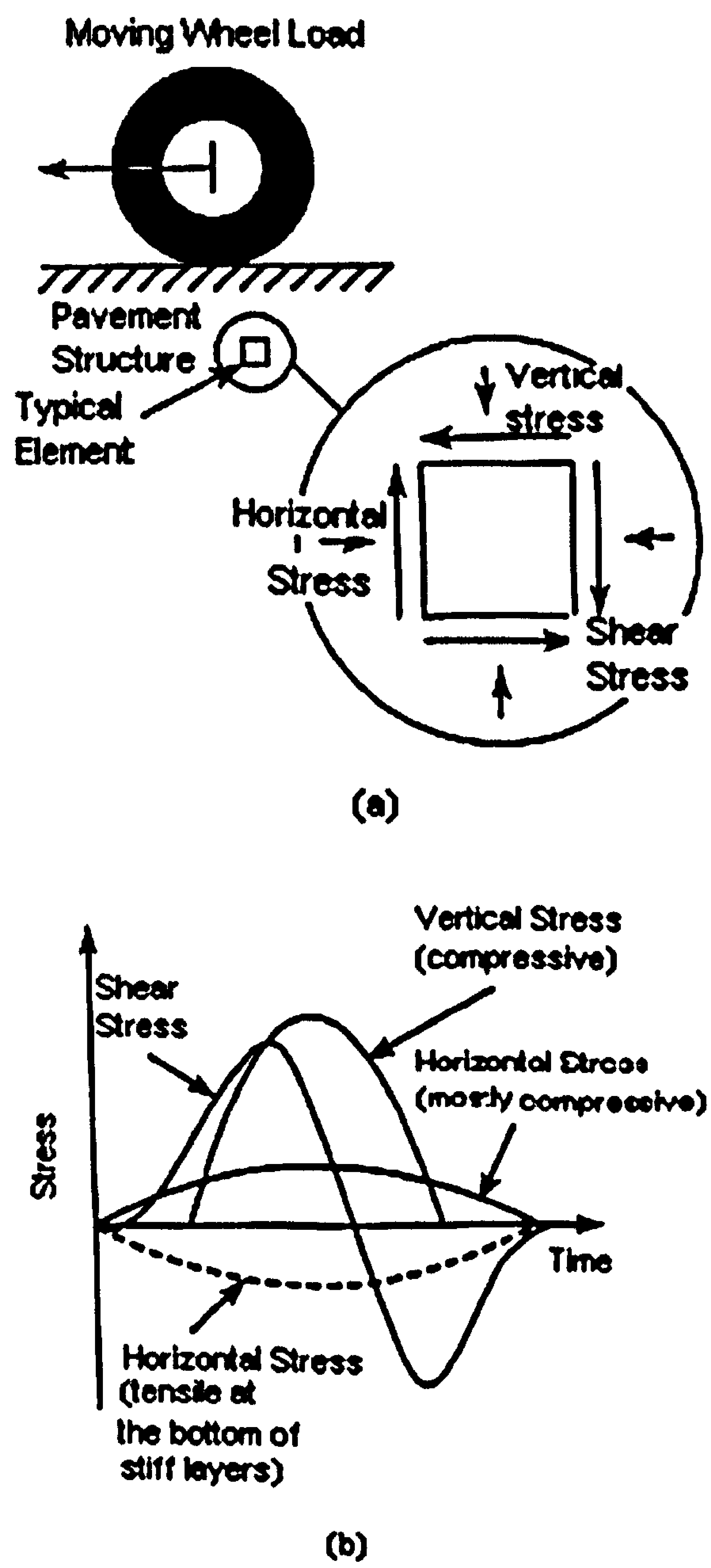


Fig. 8-4: Stress Conditions under a Moving Wheel Load:
(a) Stresses on pavement element. (b) Variation of stresses with time

(Brown and Gibb, 1996)

8.1.4 Effect of Tyre Configuration on Pavement

Dynamic tyre forces are originated by vibration of a moving vehicle agitated by pavement surface roughness and can cause axial and radial deformation which may have resilient and or permanent deformation, Figures (8-2 & 8-4).

Different axle configurations are available with a different number of wheels at the end of each axle. Axles can be single, tandem, tridem, or multiple, while wheels can be either single or dual. Small cars have single axles and single wheels. on the other hand, trucks can take different combinations of axle and wheel and load imbalance configurations as shown in Figures (8-5 & 8-6) (Fwa, 2006).

The maximum total weight for an articulated vehicle and truck-trailer combination of five or more axles has been coordinated at 40 tonnes. Yet, some articulated vehicles operating in joint transport modes, and having five or six axles, can work at 44 tonnes. (It should be noted that in two northern countries of Europe, Finland and Sweden, domestic rules allow total Vehicle Weights up to 60 tonnes) and these trucks might enter into the UK road network.

The maximum total weights for some vehicles are also dependent on the type of tyre configuration and type of suspension fitted. For example, the maximum total vehicle weight of a three axle rigid vehicle is one tonne higher (26 tonnes as opposed to 25 tonnes) if twin tyres and road friendly suspensions were fitted or no drive axle exceeds 9.5 tonnes.

In a similar approach a four axle articulated vehicle is limited to 36 tonnes, as opposed to 38 tonnes, unless dual tyres and road friendly suspensions are fitted to the drive axle. Maximum axle weights have been harmonized at 10 tonnes for a single non-driven axle. Although some single trailer axles would be used at this weight, steering axles are usually operated at 6 tonnes to 7 tonnes. The maximum for a single drive axle is 11.5 tonnes.

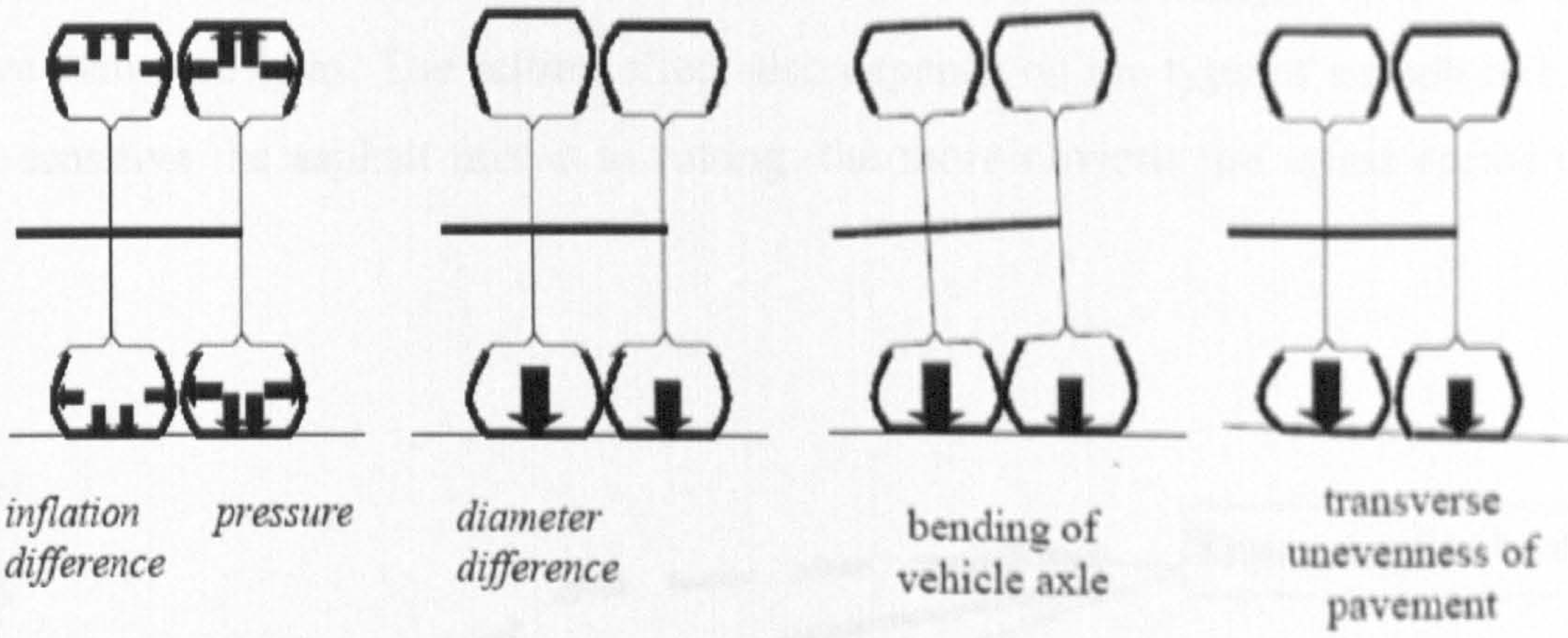


Fig. 8-5: Causes for unequal sharing between tyres in a dual tyre assembly (Load Imbalance)

(Brown and Gibb, 1996)

On the basis of asphalt strain measurements, Huhtala (1988) had studied the influence of a wide-base single tyres on road pavement and he noted that they may cause more damage than dual tyres by 3.5 to 7 times, and also noted that the severest damage is inflicted on surface layers. He also noted that the contact pressures under each tyre, in a pair, can be fairly different due to a number of reasons; including differential inflation pressures or temperatures tread wear, axle bending or transverse road roughness.

Huhtala had noted that; a wide-base single tyre is able to cause damage of about 1.5 times more than a dual pair of tyres, unequally inflated with inflation of 500kPa in one tyre and 1000kPa in the other. Gillespie (1992) noted that heavily loaded ordinary tyres on a steering axle carrying 53kN can be more harmful to flexible pavements than ordinary dual tyres carrying 89kN. It can be seen, for example, that on a 165mm thick flexible pavement, a 15R/22.5 single tyre creates 29% more strain per unit load than a pair of 11R/22.5 dual tyres.

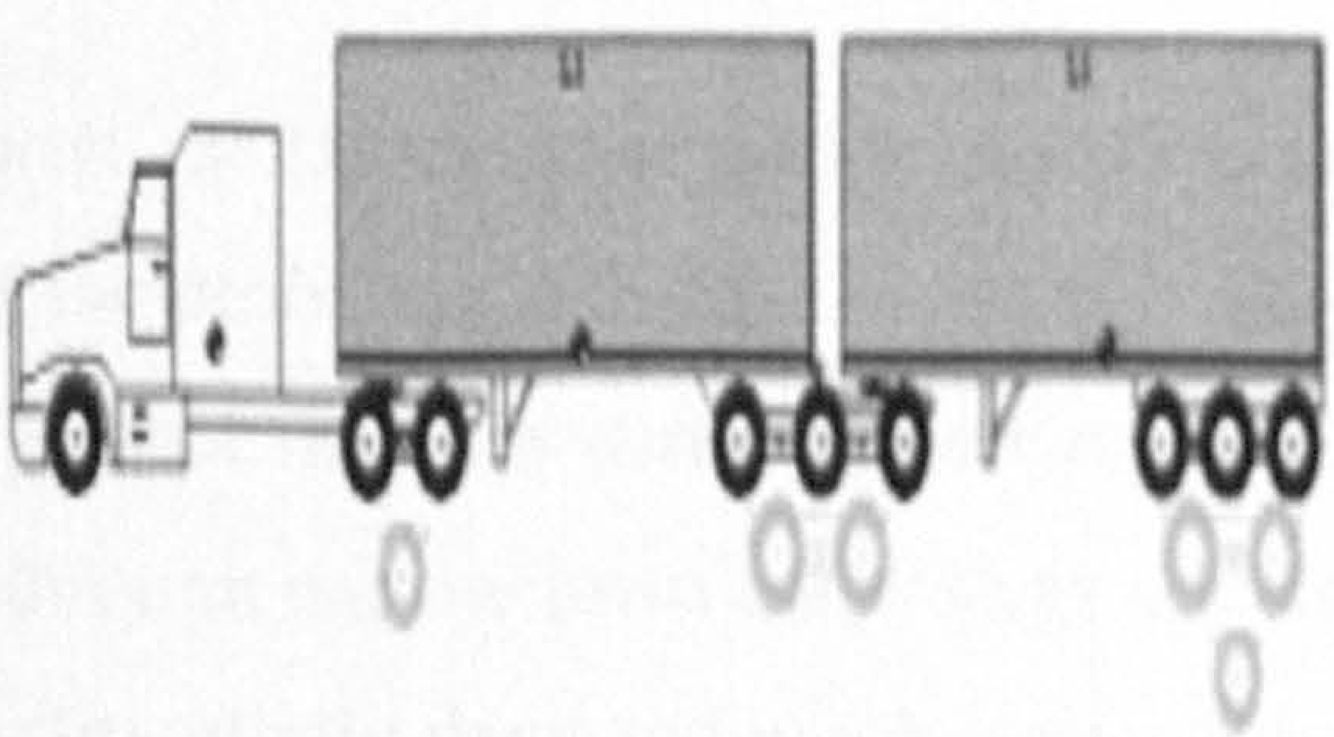


Fig. 8-6: Axles configurations

Corte' et al (1994) had studied the influence of dual and super single tyres on asphalt pavements deformation; they concluded that axles with wide single tyres are more destructive than dual tyres. The rutting effect also depends on the type of asphalt mix, and the more sensitive the asphalt mix is to rutting; the more obvious the effect seems to be (Fig.8-7).

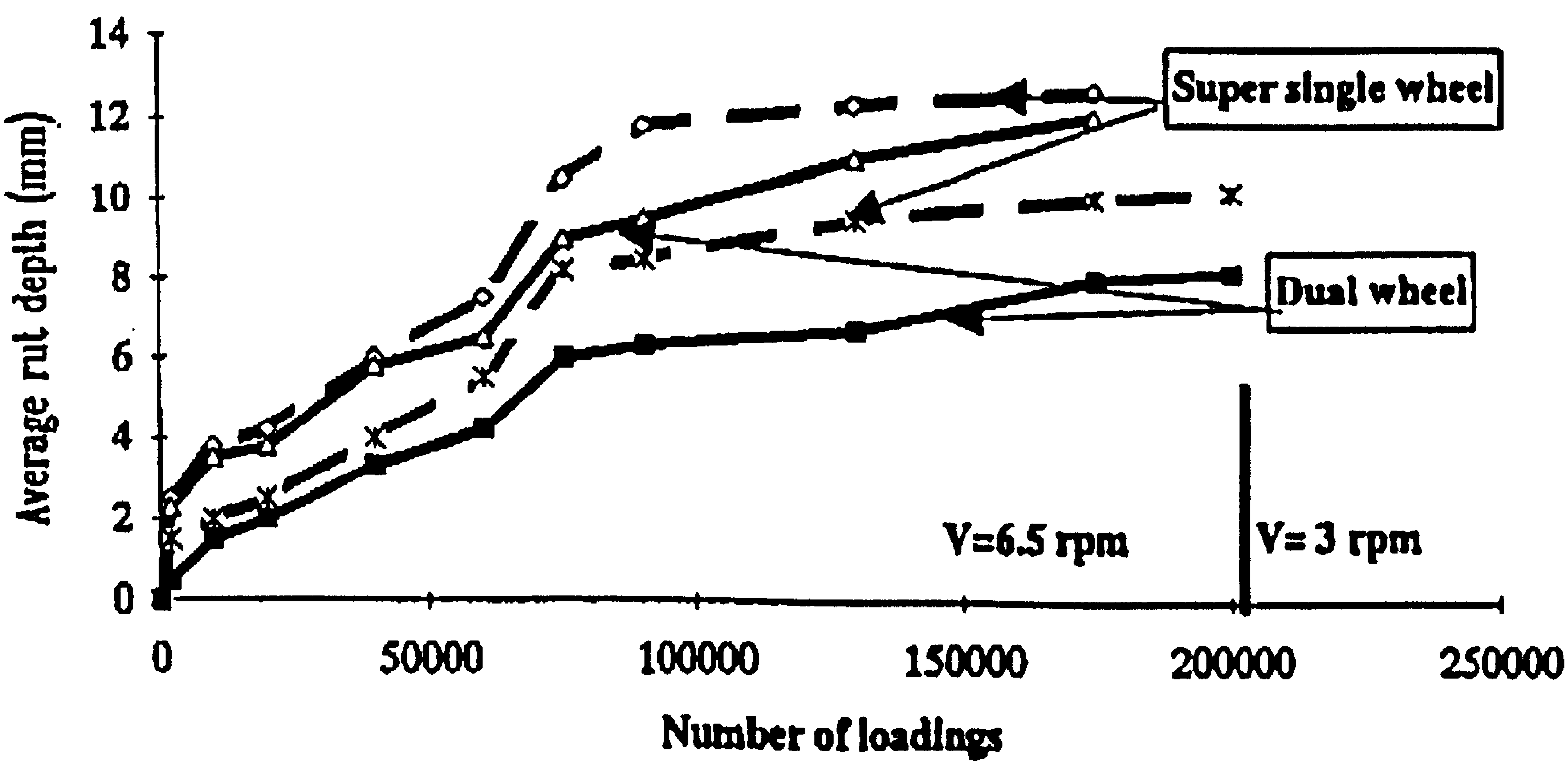


Fig. 8-7: Rutting Effects of dual and super single tyres
(CORTE et al, 1994)

8.2 Road Maintenance Cost

The total length of paved roads in the UK is approx. 372,000km, of which 3,300km is motorway (Assoc. of British Drivers, 2001). One of the problems with road systems is that many of them are not designed for the sort of traffic that they are currently subjected to. Many Heavy Goods Vehicles (HGV's) are driving through narrow town and village streets every day, and this means that these roads become structurally damaged much earlier than what their original design life were set at for normal light traffic (Assoc. of British Drivers, 2001). For instance, UK & USA government expenditures on roads and road maintenance in 1990 were as shown in (Table8-1) (FHWA, 1990).

Item	UK (£)	USA($)
Capital, new construction	4,140	35,481
Maintenance *	4,680	19,742
Policing, safety	720	6,922
Other	540	12,740
Total	10,080	74,885

* Includes all maintenance on roads, bridges, crash barriers, lighting, signs, etc.

Table 8-1: Expenditure on roads in the UK and USA in 1990

(FHWA, 1990)

Overall, the UK spent 1.17% of its $860,000M Gross National Product on roads in 1990, and the USA spent 1.44% of its GNP of $5,200,800M on roads in the same year. The huge difference between the two expenditure amounts is due to the much larger road network in the USA than the UK.

It is interesting to note that 47% of the total US expenditure on roads in 1990 was on capital outlay and new construction, compared with 41% in the UK, but that the UK spent 46% of its total on maintenance, compared with 26% in the USA. Of the UK's total $4,680M road maintenance costs, $2,160M (46%) was spent on resurfacing and patching roads, and the remainder was spent on other types of maintenance (FHWA, 1990). Based on the fourth power law [*], virtually all of this $2,160M can be attributed to the road damage caused by heavy goods vehicles.

Brown (2004) has stated that "*part of the justification for APT is the chance it provides to cut the costs of highway construction and maintenance through the use of improved engineering. £6 billion per year is spent in the UK on maintenance and development for the whole road network, and this is set to increase over the next few years, so investment in practical research that could show significant savings is worthy of serious consideration. An investment of, say, £1 million per year would represent only 0.02% of the current spend*

[*] The power of four originated from full-scale tests carried out in the late 1950s by (AASHO) Officials. Using a regression analysis it was found that the decrease in 'pavement serviceability' caused by a heavy vehicle axle could be related to the fourth power of its static load. It states that if an axle is twice as heavy as another, its relative effect on pavement performance is in the ratio of 2^4 to 1. (Collop and Cebon, 1996)

and appears justified in order to effect savings of 0.2% or about £12 million based on a benefit to cost ratio of 10. Recent work conducted by the author and colleagues for the Highways Agency suggests that the 1:10 cost: benefit ratio quoted from other countries could also be achieved the UK and that any new facility would have to be mobile to allow testing at various locations around the country. The most attractive type of facility would combine the flexibility of an HVS with the modern control and operating system of ATL'S together with its lower running costs".

In the UK, the National Audit Office (NAO) stated: "*In 2001-2002 the Highway Agency spent £502 million on road maintenance for the 8900 kilometres that it manages directly, over £56,000 per kilometre*" (NAO, 2003). However, an increase in terms of planed spending per kilometre is likely to be over 5% per kilometre, as the motorway and dual carriageway required to be maintained with higher standards that single carriageway as they carry more traffic, (NAO, 2003). In 2005-2006 the cost raised up to £534,500M (NAO, 2007).

In Europe, it is estimated that each year, around 15% of the incomes of European citizens (app. 500 billion Euro) are spent on mobility, and the bulk mobility is supported by road infrastructure. Investment in road construction and maintenance in Europe is therefore at a very high level and any improvements will have a significant effect on the overall cost benefit analysis. An important way of analyzing road pavements or maintenance options before commencing the expensive process of road construction is Pavement Testing Facilities (PTF's), and this has usually been carried out at especially designed facilities in most European countries (COST 347, 2001).

Many of the PTF's in the EU are under the management of organizations involved in the Forum of European National Highway Research Laboratories (FEHRL), which enables better coordination between the approaches taken in different countries. Although the benefits always significantly overshadow the costs, the facilities are expensive to install and operate. Accordingly, there is a need to ensure that they are used with maximum efficiency (COST 347, 2001).

8.3 ACCELERATED PAVEMENT TESTING FACILITIES (APTF's)

8.3.1 Examples of Currently Employed Laboratory (Indoor) and External (Outdoor) Testing Facilities

Generally, Accelerated Pavement Testing (APT) is defined as the controlled application of a loaded wheel to a pavement in order to simulate the effects of long term actual traffic loading conditions in a comparatively short period of time (NCHRP, 2003). The actual traffic loading condition on pavement derives from the passage of traffic wheel loads crossing the pavement surface. Even though the majority of performance tests have been developed to simulate this condition through many hypotheses, only some tests have closely simulated this condition. The common factor between these tests is the application of a loaded wheel over the surface of a sample. This process requires an understanding of pavement structure behaviour both at and after loading, and the effects of different kinds of loading and environmental conditions on the pavement. Many institutions and Government agencies as well as Universities are getting more and more interested in developing a facility to carry out more testing on road pavements, and what have been created during the last century some of them were for indoor testing and others for outdoor testing. Below are some of them, listed in an ascending order according to their cost.

8.3.1.1 Indoor Testing Facilities

a) Pulse Loading Devices

Pulse Loading Devices are used for the evaluation of thick flexible pavements where rutting is the failure mode and for the evaluation of joint or crack behaviour in rigid pavements, but present the problem of the lack of realistic loading, since rolling wheels are substituted for pulse loading, which is why it is not popular. As an example of this type of testing is the German PLF. The device has three hydraulic plate loading rigs. The pulse load

Plate 8-1: Pulse Loading Device

(COST 347, 2002)

plate moves along the test section to simulate traffic at approximately 20 km/h (12.5 mph) and applies loads to 200 kN. The facility was commissioned in 1963 (COST 347, 2002) (Plate 8-1).

b) Hamburg Wheel Tracking Device (HWTD)

This is a laboratory device utilized for the assessment of the combined effects of rutting and moisture damage. A slab of hot-mix asphalt is submerged in hot water and a steel wheel is rolled across its surface. Two samples can be tested simultaneously in one run. The wheels can be either steel (47mm wide) or solid rubber (50mm wide).

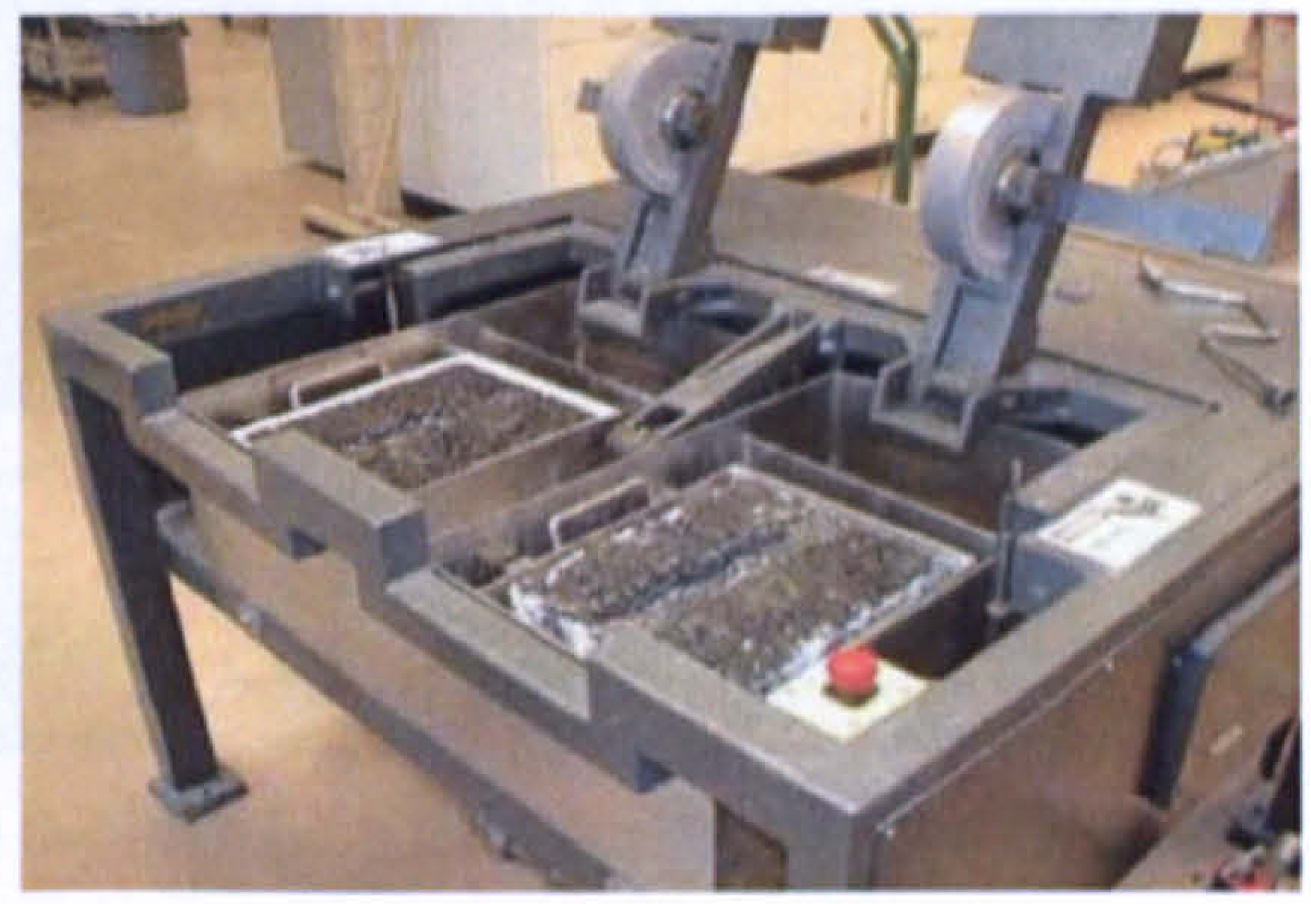


Plate 8-2: Hamburg Wheel Tracking Device

(FHWA a. 2006)

The load applied to the wheels is 710±1N. The normal temperature for the test is 50 ºC. Esso A.G. of Hamburg, Germany developed the device in the 1970's at a cost of 60,000 US$, based on a comparable British device that had a rubber tyre. The device was originally called the Esso Wheel-Tracking Device (FHWA, 2006a), and it is used, by researchers, to relate laboratory mix design of HMA to actual field performance even with the fundamental differences between the test conditions and real road section is subjected to (Plate 8-2).

c) French Laboratory Rutting Tester (FRT)

This facility measures the rutting susceptibility of asphalt paving mixtures using a reciprocating, pneumatic rubber tyre 400mm in diameter and 80mm in width. Slabs of asphalt of (500x180) mm, with depths varying between (20-100) mm, can be tested at temperatures ranging from 30°C to 60°C. Rutting susceptibility is based on pass/fail criterion. This machine is used in

Plate 8-3: French Rutting Tester

(FHWA b, 2006)

France and Canada to evaluate mixtures that either have no performance record or will be subjected to heavy traffic. This tester had a cost of 85,000 US$ in 1985 (FHWA, 2006b) (Plate 8-3).

d) The TRACKER by the University of Ulster

The TRACKER has been designed and constructed at the University of Ulster to meet the need for a multi-purpose laboratory-based research facility that can predict performance of highway materials under rolling conditions. The testing tyre has a back and forth movement during testing, and the test bed allows for full depth highway construction if required. However, various types of test sample insert may be assessed. The apparatus may also accommodate a range of other tyre sizes including super-single tyre. Its speed ranges is estimated to be up to 3km/h (The TRACKER, 1999) (Plate 8-4).

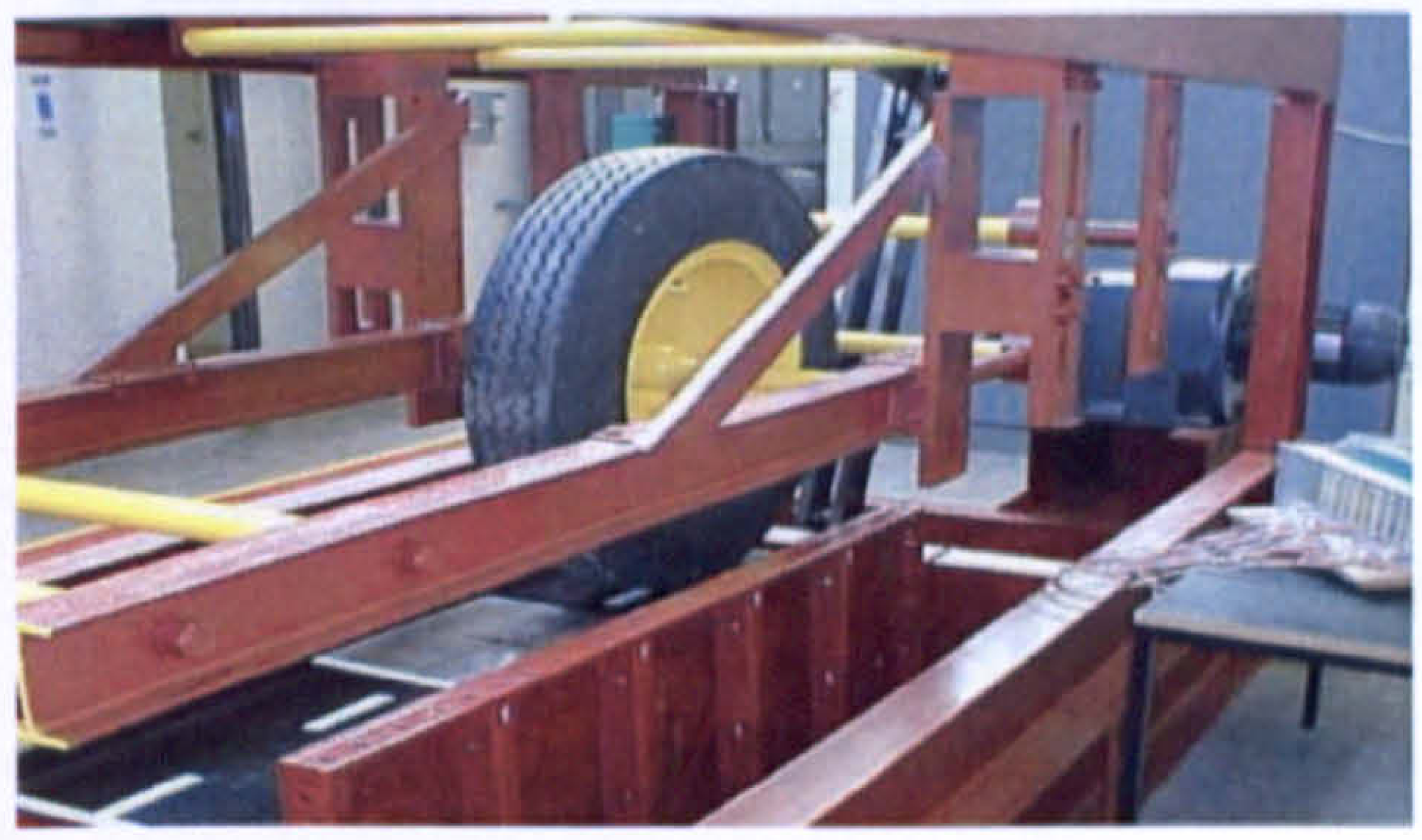


Plate 8-4: THE TRACKER, Ulster University
(The TRACKER, 1999)

e) Nottingham Pavement Testing Facility (PTF)

This linear track facility was built at Nottingham University and it is able to accommodate test sections of 4.8x2.4 m subjected to moving wheel, which moves back and forth, and loads up to 15kN at contact pressures of 570 kPa and a speed up to 16km/h can be accommodated (Nottingham University, 2005) (Plate8-5).

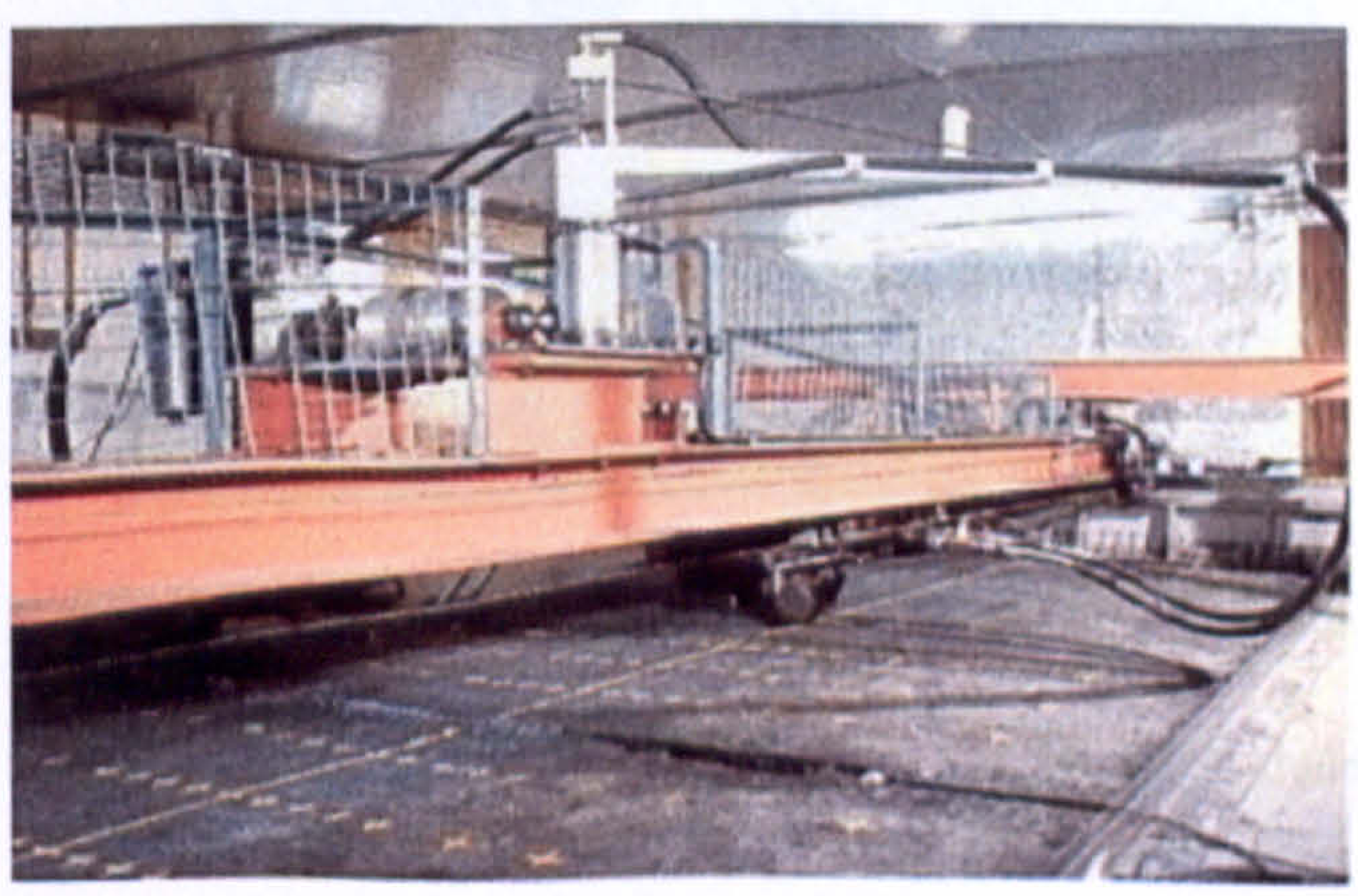


Plate 8-5: Nottingham University PTF
(Nottingham University, 2005)

f) **Transport Research Laboratory (TRL) Facility**

This Pavement Test Facility is a linear motion machine that can traffic individual test sections within an experimental road pavement. It basically consist of a portal frame mounted on rails each end, carries a dual-carriage system that allows the wheel assembly both to traffic the pavement and also to index sideways at the end of a pass to mimic the effects of traffic. The machine can also load sections in either one or both directions of motion. Testing at elevated temperatures can be conducted with the aid of infrared heaters and loads up to 102kN. The operating speed is 20km/h over a 6.7m long test section. Its cost was about US$ 1.7 million in 1984 (NCHRP, 1996) (Plate 8-6).

Plate 8-6: TRL PTF

(NCHRP, 1996)

8.3.1.2 Outdoor Testing Facilities

a) **Heavy Vehicle Simulator (HVS)**

The Heavy Vehicle Simulator (HVS) is a mobile facility that subjects road pavements to accelerated traffic. It is a mobile laboratory that can be driven to a specified location to evaluate the pavement structural response to dynamic loading and to measure deferent properties of the road under full-scale conditions. It is able to simulate the equivalent of 20 years of traffic in as little as three to six months, thus assisting engineers to understand the mechanisms of traffic-associated road failures. The HVS is hydraulically operated by load assembly carrying a single (or dual tyre) test wheel, and able of loading from 20 to 100kN at a speed of 14km/h.

Plate 8-7: Heavy Vehicle Simulator (HVS)

(NCHRP, 2004)

Depending on the requirements of the client, its cost ranged between 2-5 million US$ in 1997, and it has a fleet of new versions generated from the original model (NCHRP, 2004) (Plate 8-7).

b) Texas Mobile Load Simulator (TxMLS)

This is also a mobile linear facility capable of applying six single (or dual tyre) bogies with tandem axles loaded up to a nominal 190kN (i.e. single wheel: 22.2-111.2kN, dual wheels: 35.6-191.3kN) at test speeds up to 41.5km/h. The test pavement length is 11m with varying transverse tracking over 0.6m. It is powered with a 120kW electric motor. Its cost was 2.5 million US$ in 1995 (NCHRP, 1996) (See Plate 8-8).

Plate 8-8: Texas Mobile Simulator (TxMLS)

(NCHRP, 1996)

c) Australian Accelerated Loading Facility (ALF/AUSTR)

The ALF is a mobile machine used to simulate traffic loading in order to test rutting and cracking of pavements. It is a 29m long structural frame containing a moving wheel assembly. The wheel assembly models one half of a single axle and can apply loads ranging from 44 - 100kN. Its speed is 18.5km/h over a 9.8m test pavement section. To simulate highway traffic, the test section is trafficked in only one direction (i.e. unidirectional), and the loads are laterally distributed to simulate the lateral wander of trucks. The ALF is computer

Plate 8-9: ALF/AUSTR

(NCHRP, 1996)

controlled, permitting operation 24 hours per day, seven days a week. Its cost was 1 million US$ in 1984 (NCHRP, 1996) (See Plate 8-9).

d) French Central Laboratory of the Highways Departments Facility (LCPC)

This is the largest circular test track in operation and is situated at the Laboratoire Central des Ponts and Chaussees (LCPC) near Nantes, France. It consists of a rotating loading system having four-arms, two wheel assemblies on an inner track of 30m diameter, and an outer track of 40m in diameter. It can apply Loads of up to 75kN on a dual-wheel that can speeds up to 105km/h .With this facility, it is possible to simulate tandem-axles of 280kN at lower speeds and to simulate single-wheel loading. This highly successful APT programme, with extensive research studies and strong interaction through industry partnerships is one of the most active in the world today. It takes only 3 months of testing to destroy a thoroughfare like a national highway, which normally lasts 15 years (NCHRP, 2004) (See Plate 8-10).

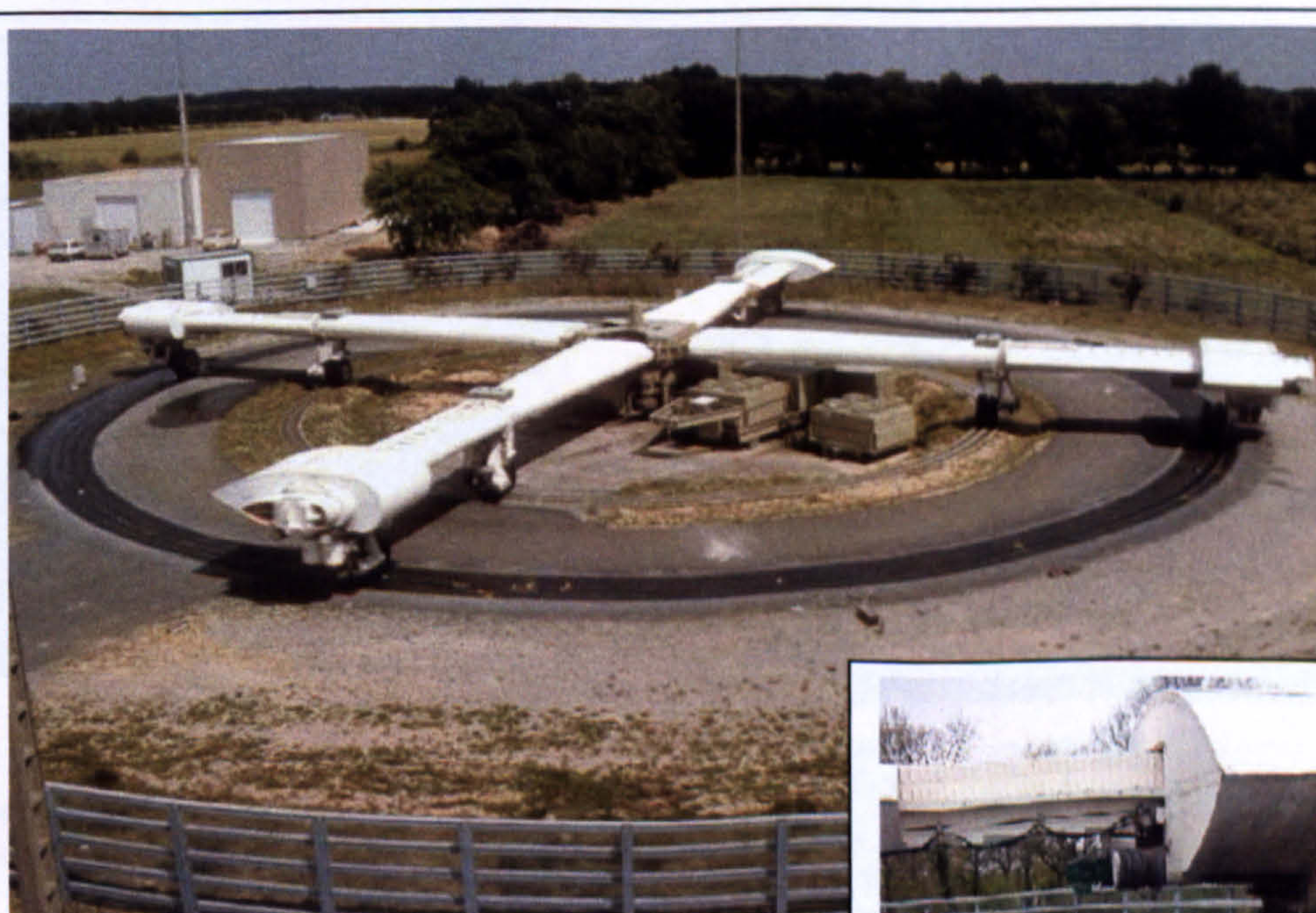


Plate 8-10: French (LCPC)

(NCHRP, 2004)

Spanish Road Track Test Facility (CEDEX)

Spain has a major facility at the Road Research Centre (Centro de Estudios de Carreteras (CEC)) near Madrid. The centre is part of the Centre for Public Works Studies and Experimentations (Centro de Estudios y Experimentacion de Obras Publicas (CEDEX)), managed by the Ministry of Public Works Transport and Environment. The facility has two parallel straights each comprising of three 25m sections joined by circular arcs. A U-shaped concrete box 8m wide by 2.6m deep test sections are built with normal construction equipment, to be able to control moisture content in the sub-grade soil. Two bogies are utilized to apply the loading, each guided by an internal wall along side the tested pavement section, and are capable of mounting up to three wheels, single or dual-tyre, half axles. The load is by gravity at speeds up to 60km/h and axle loads between 110kN and 150kN. Pavement instrumentation is monitored from a control centre that also operates the two vehicles (CEDEX, 2000) (See Plate 8-11).

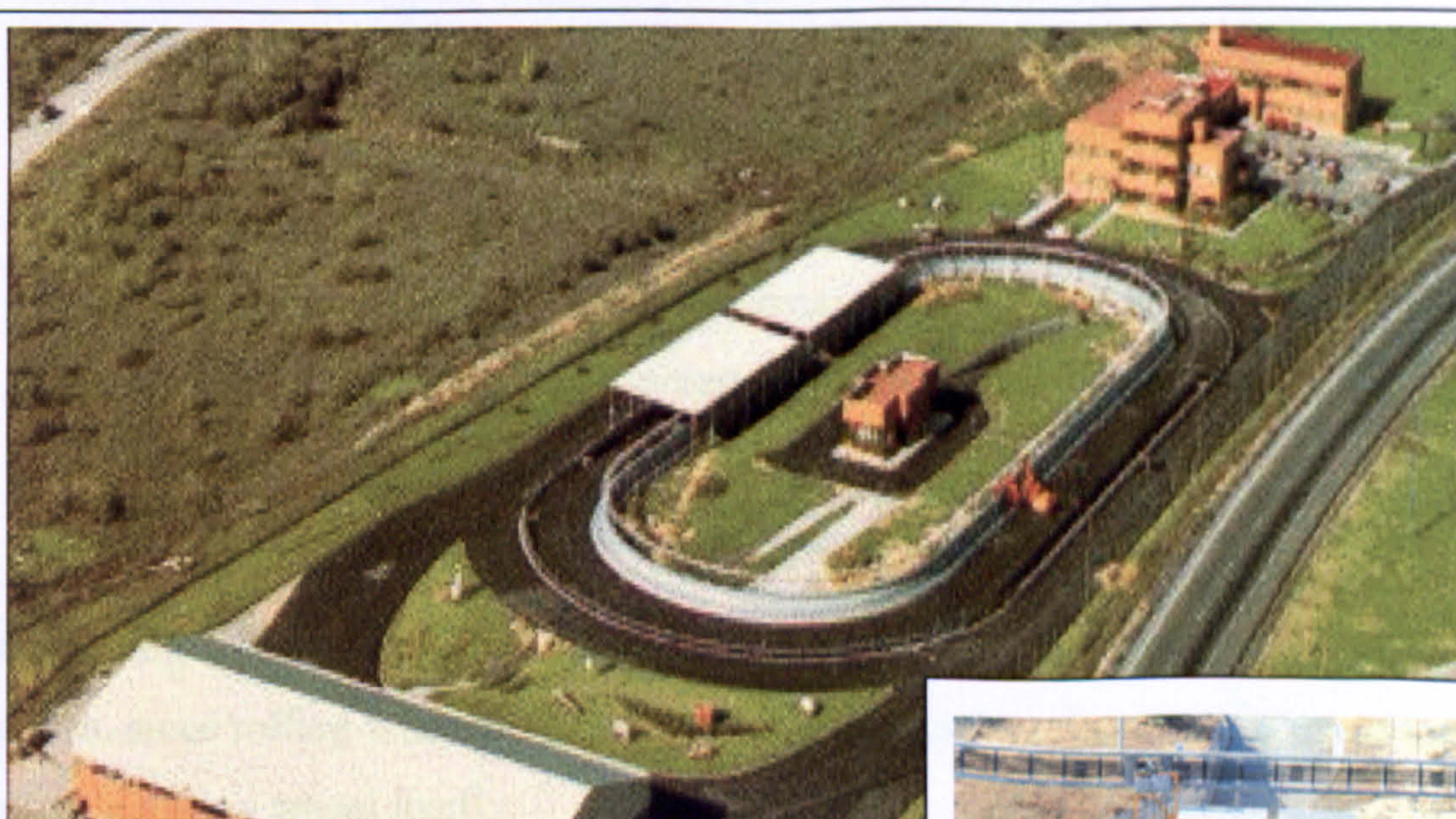


Plate 8-11: Spanish (CEDEX)

(CEDEX, 2000)

Circular testing track facilities also address most of the main testing parameters required, except the fact that the circular movement of the testing wheels creates a centrifugal force. Although an extended radius of 17.5m is currently in use to minimize the effect of that force, there is still a substantial centrifugal force created which of course affect the testing results (OECD, 1985) (Plate 8-10). To the best of the knowledge of the author no data has been reported on the use of APT to predict the environmental impact on HMA pavements such as wind and radiation and water in a verity of forms.

The previous review of accelerated full-scale pavement testing has given a basis for evaluating the UK road surface state, and that the majority of the developed countries, with a high-quality pavement infrastructure, have significantly used APT, particularly in recent years. This has allowed new developments from research to be put into action within relatively short timescales. The best examples of this progress are in South Africa and France. The mobility of the South African HVS gives it an extra flexibility of operation not possible with fixed facilities such as those at LCPC in Nantes and the TRL in the UK. The South African experience of using APT's has shown a considerable advantage in cost benefit effectiveness of 1:10 ratio, even though its high price and running costs. The HVS fleet has allowed the rapid development of new ideas and theoretically based design systems to be introduced.

8.4 Summary of the Weaknesses of the Existing PTF's

Pulse loading devices have the problem of lacking the presentation of realistic loading condition, since rolling wheels are replaced by pulse loading (i.e. a pulsed plate load is used to mimic a rolling wheel load).

Although the full-scale APT has impact on the following:

a) Evaluation, Validation, and Improvement of Structural Designs,
b) Vehicle-Pavement-Environment Interaction,
c) Evaluation of Materials and Tests,
d) Enhancement of Modeling in Pavement Engineering,
e) Development and Validation of Rehabilitation, Construction, and

Maintenance Strategies,

f) Pavement Engineering Applications and Issues, and

g) Improvement of Pavement Economics and Management through APT Applications.

Their usage is out of the reach of many researchers for the obvious reason of the cost of these facilities. Interestingly the minimum cost of an outdoor facility was for the Australian Accelerated Loading Facility (ALF that was mentioned in section 8.3.1.2.c), which was one million US$ in 1984 (NCHRP, 1996).

None of the indoor or the outdoor asphalt testing equipments that were described in section (8.3) of this chapter can provide a comprehensive cover of all the above factors. However the LoCATE, which can be defined as the controlled application of a wheel load to pavement structures for the purpose of simulating the effects of long-term in-service loading conditions in a relatively short period of time, does.

For all these reasons and as to what was mentioned, in section (8.2), by Brown (2004), the need for a versatile, compact, mobile asphalt testing equipment was necessary to be developed to enhance both educational and industrial issues.

8.5 Specifications of the Low Cost Asphalt Testing Equipment (LoCATE)

8.5.1 Introduction

As mentioned in the last few sections of this chapter, Pavement testing facilities (APT), which are operated either indoor or outdoor, are generally either limited to laboratory operating conditions or are highly expensive to purchase and operate. Therefore, the development of the LoCATE in this research work is aimed at providing the pavement industry with a mobile, compact, affordable indoor and outdoor road and highways testing facility. LoCATE is capable of covering a wide range of applications needed in pavement research studies as well as in the industry through accommodating different sample sizes and testing tyres. It can simulate real tyre-road surface interactions as accurately and economically as possible. The installation of Geophones like the ones used with the Falling Weight Deflectometer on the machine will give an extra ability to the researchers to use this device to measure pavement individual layers properties such as stiffness modulus, strain, stress and bonding coefficients.

These advantages will be highly appreciated by the market as the need for a more reliable simulating testing machine that can be used in laboratory and on roads environments is increasing day by day by road industry due to the increase in both:

1. Road maintenance costs and
2. The level of single axle loading exerted on the road by newer truck tyres as well as new loading regulations introduced by the European Union transport authorities.

Finally the design idea of duplicating the train motion mechanism was implemented for the LoCATE model to be considered. Many modifications and alterations have been introduced, and that led to the refinement of the final model. It can be seen from Appendix G that every single component of the model was designed individually according to the current specifications by the BS 5950: Part 1:2000 (SCI, 2001) and using the educational software SolidWorks (version 2005 SP4.1), and from these parts (components) a model was developed.

Following the completion of the research background and SolidWorks modeling stages, optimization stage of the model, using the educational software (COSMOS Works, version 2006) available at the School of Engineering, took place in order to optimize the main structural elements of the LoCATE. The use of this software made it possible to test individual machine components for stresses under different applied loadings, as well as how well the components are complementing one another in the overall design. This testing allowed for the much of the machine's design to be finalized prior to its physical manufacturing and testing.

For the purpose of designing parting in SolidWorks it is important to validate the design and analyze stress concentrations in the machine elements. This is done using a process called (FEA) or Finite Element Analysis with SolidWorks and COSMOS softwares; for analyzing single Solid parts (components). Each stress analysis performed on any part (element) of the proposed machine can then be studied in relation to its adjacent part (component) to achieve the overall stress free assembly. Figure (8-8) shows how stresses within a tested typical element can be analysed and visualized using the FEA technique and the COSMOS software. The coloured bar shown in Figure (8-8) represent the stress severity experienced by the tested element under the applied loading (blue: low stress, Red: high stress).

The final model is shown in Appendix G (G8-1 to G 8-6).

The driving motor, marked (3) in Figure (8-9) below, will be driving the lever-arm marked (6). The Chasse marked (5) is driven by the lever arm (6) that in turn is driving the testing wheel marked (1) in a forward and backward movement at a speed range of 3km/h to 7km/h. The loaded container marked (2) will be exerting the load on the testing wheel (1) via the rolling steel wheels marked (7) through the stiff supporting beams marked (4).

Only the testing wheel (1), the lever-arm and the chasse (5) are in motion during the test. This was decided to keep the load on the driving motor (3) to minimum and therefore reduce the size of the Power needed.

The top of the main frame marked (16) can be sealed off to control the temperature, air and environmental operational factors such as road wet and dry conditions, temperature and grit surface treatments and it can be towed to drive the machine for outdoor usage.

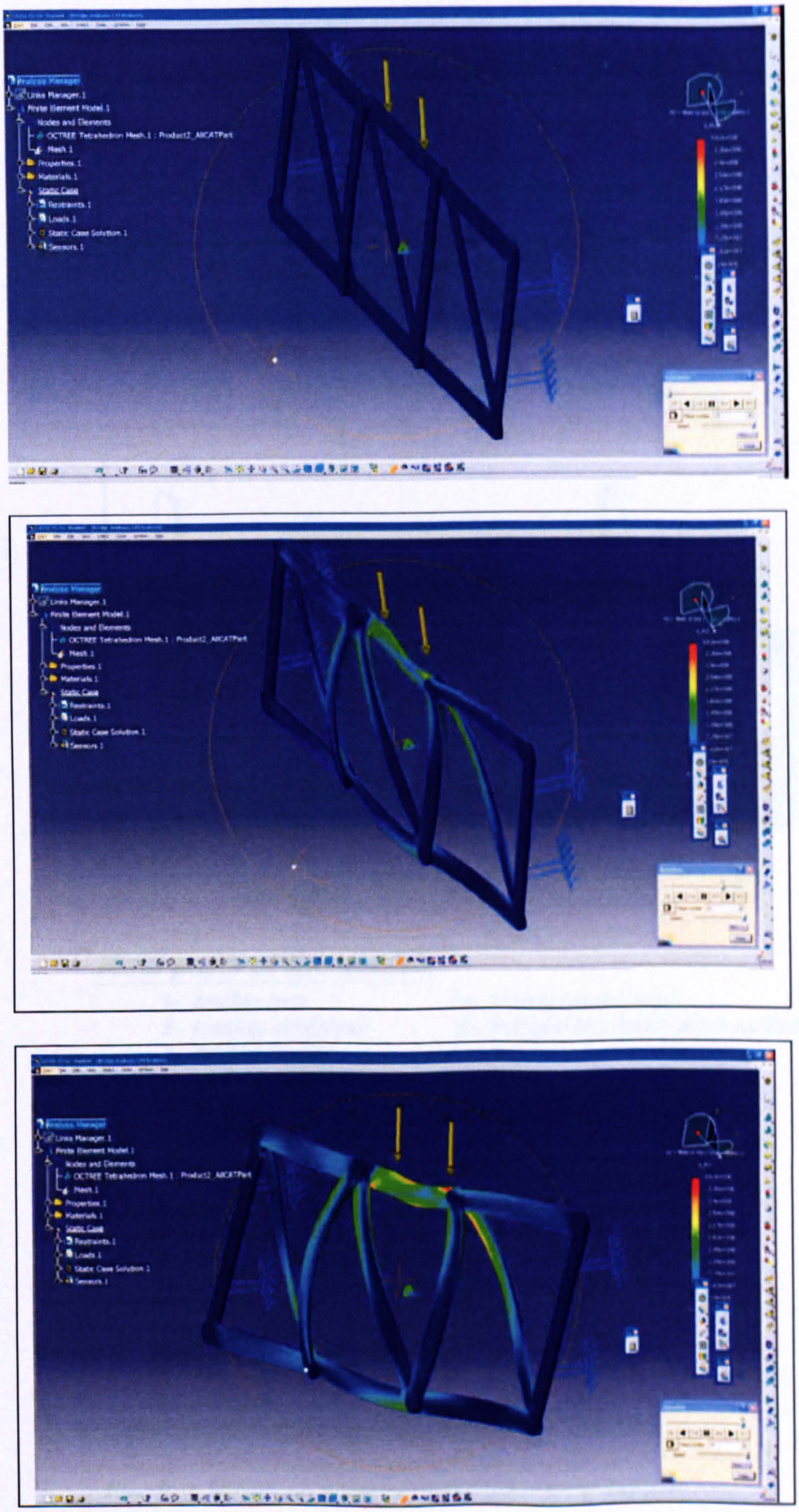
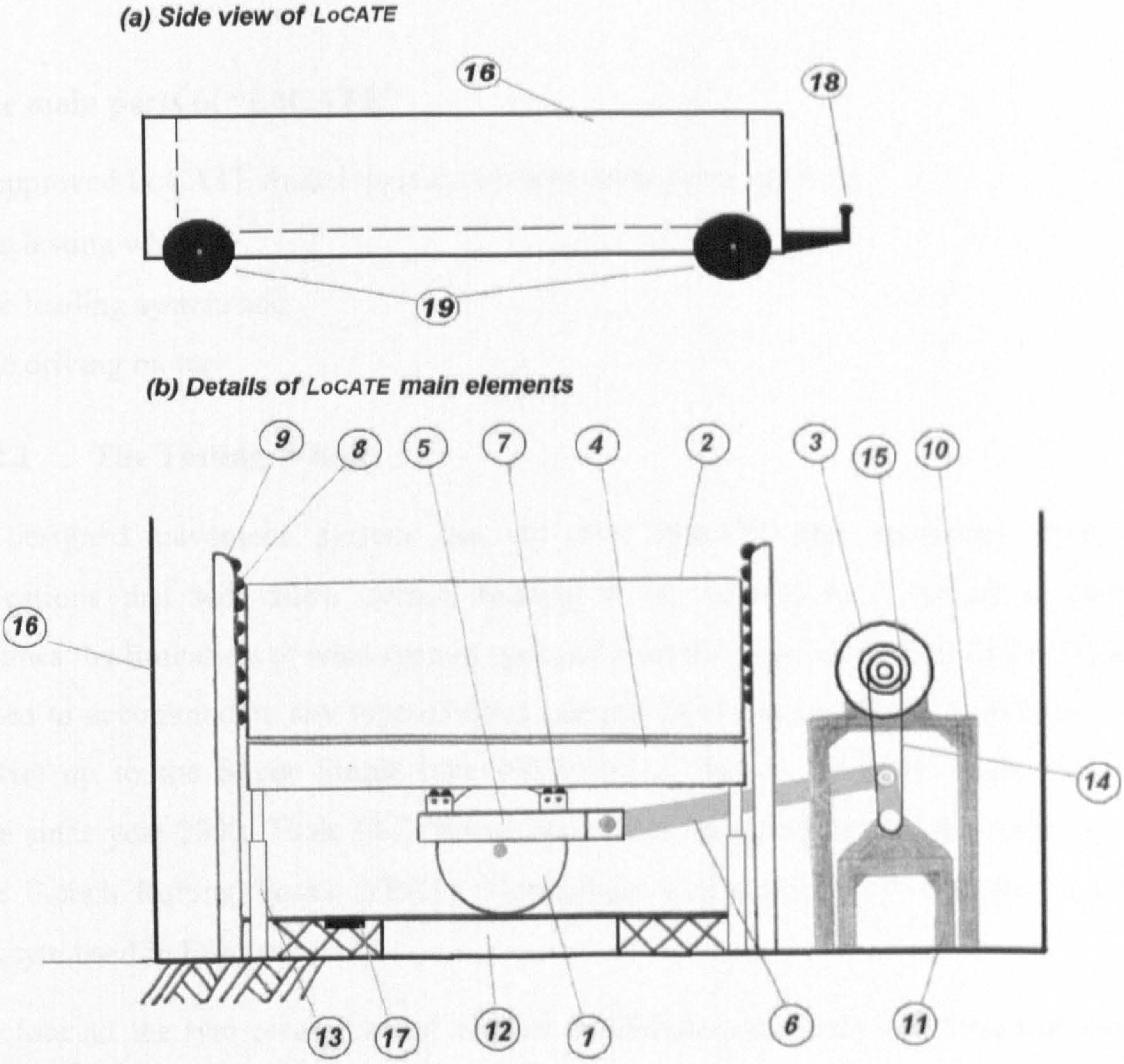


Fig. 8-8: Typical stress analysis diagram for an element using FEA & COSMOS software



Main parts of the model:

1- testing tyre
2- loading container
3- driving motor
4- stiff supporting beam
5- chasse
6- lever arm
7- steel rollers
8- supporting rubber rollers
9- loading frame
10- motor support structure
11- lever arm support mechanism
12- sample
13- jack
14- Driving belt
15- Set of pulleys to adjust motor speed
16- the main frame
17- Load cell
18- towing mechanism
19- transporting tyres ,silde up during testing

Fig. 8-9: Schematic view of the LoCATE facility

8.5.2 The main parts of "LoCATE"

The final approved LoCATE model consists of three main parts these are:

a) The testing wheel
b) The loading system and
c) The driving motor

8.5.2.1 The Testing Wheel

Each designed pavement system has its own material and pavement layer's specifications that will allow certain loading to be applied to it, which in turn determines the limitation of what type of tyre and load that can run over it. LoCATE is designed to accommodate any type of tyres ranging from the smallest (i.e. 400mm in diameter) up to the Super Single tyre 495/45R/22.5 that is currently employed in Europe since year 2000. Table (8-2) below shows the specifications for the tyres used by the French Rutting Tester (FPRT), Nottingham University (PTF) and the Super Single tyre used in Europe.

Therefore all the tyre pressures and contact conditions previously mentioned in this thesis can be accommodated. The testing tyre will be having a reciprocating movement during testing (bi-directional), and that has no significant effect on the obtained results. Uni-directional loading is more natural loading mode, but bi-directional loading was used for this study, because testing is then twice as effective as in uni-directional testing (Huhtala, 2000).

Tyre Type	Used by
(400 x 80) mm	French Rutting Tester BS EN 12697 pt 22: 2003
165 / 70R / 13	Nottingham University PTF
315 / 80R / 22.5	Dual axle trailers
495 / 45R / 22.5	Trailers fitted with Super Wide Single tyre

Table 8-2: Types of tyres may be used by the LoCATE

8.5.2.2 The Loading System

Referring to the schematic drawing, of the LoCATE facility in Figure (8-7) and Appendix G (8-1), the loading system consists of the loading container (2) and the main chasse (5). The loading container (2), which will be bearing the live load and transmitting it into the tyre via the main chasse (5), has dimensions of (1570 x 750 x 700) mm and its main components are made from stainless steel frame. The main frame elements of the container consist of rectangular cross section (RCS) of (50 x 30 x 2.6) mm, and the main elements of the chasse are made of (50 x 30 x 2.6) mm stainless alloy steel grade 275 as well.

The container will be secured and supported by the main frame elements, and it will be guided by spring-rollers (8). This arrangement has a purpose of smoothing the vertical movement of the load and securing the loaded container while the machine is in action. The loading container (2) will be having a maximum total load of 10kN (1tonne). A load cell (17) will be used to measure the precise loads on the tyre .The cell will be used at the start of the test to facilitate the adjustments of the wheel design load and will be removed during the testing.

8.5.2.3 The Driving Motor

Appendix G (8-2) shows the calculated values for the horsepower (HP) of the driving motor's Power at a range of operating speed between 1km/h to 20 km/h with a coefficient of friction between tyre and road surface of (0.65). To keep the cost of the LoCATE to minimum the research team recommended that the maximum speed of LoCATE facility should not exceed 7km/h. Contacts with different motor suppliers have been established to pick the most effective and suitable motor that is capable of running the machine smoothly and effectively.

Type of Motor	*BK series bevel-geared motor*
Driving Horse Power (HP)	11
Driving Torque (N.m)	3200
Testing tyre Speed	1m/s (3.6 km/h)
Motor & Inverter Supplier	*BAUER geared motors Co.*

Table 8-3: Specifications of the LoCATE Facility driving motor

The best choice was negotiated with the ***BAUER*** geared motors Co., as they were able to offer the most economical and effective type of geared motor and a frequency inverter (Table 8-3).

8.5.3 Testing and Calibration of LoCATE

As stated earlier in this thesis, the main objective of this project is:
To predict the mechanical properties of new bituminous mixes containing coated natural aggregates using a new machine capable of testing pavement deformations based on material characteristics, traffic load, mean speed of heavy vehicles and site climatic conditions.

Having selected the pavements to be tested, their rut depths will be measured using LoCATE and bituminous material slabs will be extracted from the selected pavements. The slabs will be fully analyzed for the purpose of determining their composition and compaction degree before and after testing.

By having the machine properly calibrated, testing of HMA will be undertaken at the LJMU laboratory, and then the machine will be towed out to test real road sections for the measurement of pavements rutting and stiffness. Tarmac Group has agreed to provide road trial sections with known rutting and stiffness characteristics. The next stage consists of creating a model able to represent the relationship between the in-situ rut depth, measured on pavement using LoCATE, and the rate of rutting measured in the laboratory, using

LoCATE too, operating under the same testing conditions of the FPRT facility, i.e. using the same tyre, speed and loading conditions the FPRT normally designed for.

Facility Dimensions	(2600 x 1750 x 1050)mm with no container movability (2600 x 2400 x 1050)mm with container movability
Total load of facility	~1250 kg (without container) ~1500 kg (with container)
Loading Container's Dimensions	(1600 x 750 x 750) mm
Loading Container Total Volume	0.9 m^3

Table 8-4: Some Specifications of the LoCATE Facility

8.6 Detailed specifications of the LoCATE testing facility are shown on the next five pages:

GENERAL

• Country:	UK
• City:	Liverpool
• Institution:	Liverpool John Moores University
• Name of Facility:	Low Cost Asphalt Testing Equipment (LOCATE)
• Construction (commissioned) year:	2007

FACILITY DESCRIPTION

- **TYPE**
  - ☒ Linear Test Track (rolling wheels)
  - ☐ Circular Test Track (rolling wheels)
  - ☐ Simulated Loading Device (e.g. pulse loading device)
  - ☐ Other (describe):

- **PLACEMENT**
  - ☐ Fixed inside
  - ☐ Fixed outside
  - ☒ Mobile

- **DIMENSIONS**
  - Pavement length:
    - Overall length: 1 m
    - Useful testing length: 0.8 m
    - Mean diameter (Loading diameter) for circular test tracks:
  - Concrete Test Pit: ☒ No
    - ☐ Yes ⇒ width:
    - height:

- **LOADING DESCRIPTION**
  - ☒ Rolling wheels:
    - Loading
      - ☒ Dead load
      - ☐ Hydraulic/pneumatic device
      - ☐ Mechanical device
      - ☐ Other (describe):
      - * How do you measure actual load? (e.g. known dead weight): Load cell below the testing tyre
    - Number of arms (circular test tracks):
    - Axle type:
      - ☒ Single axle / single wheel
      - ☐ Single axle / dual wheel
      - ☐ Tandem axle
      - ☐ Other (describe):

- ☐ Yes (complete axle)
- ☐ No (half-axle)

- Tire type:
  - ☐ Standard truck tires
  - ☐ Supersingle tires
  - ☐ Scale tires (e.g. fork-lift tires)
  - ☐ Solid tires
  - ☒ Other (describe):
- Range of tire pressure: 600 to 700 kPa
  - * How do you measure actual tire pressure? pressure gauge
- Suspension:
  - ☐ Pneumatic
  - ☐ Multileaf spring
  - ☐ Constant load by loading device
  - ☒ Other (describe): Free applied load
- Propulsion:
  - ☒ Driven axle or wheels
  - ☐ Towed axle or wheels
  - ☐ Central pivot motor (circular test tracks)
  - ☐ Other(describe):
- Loading direction:
  - ☐ One way
  - ☒ Two ways
  - ☐ Both (one way or two ways are possible)

- **ACCELERATED LOADING**
  - Range of Loads: 0 to 10 kN
  - Transverse distribution of loading:
    - Total lateral displacement: ***(distance between centres of wheel tracks, not between outer edges)***
    - Step intervals:
    - Automatic transverse distribution
      - ☐ No
      - ☒ Yes → Describe: A liver-arm will be guiding the backand forth movement of the testing wheel
  - Test speed:
    - ☒ Rolling wheels:
      - Speed (min., normal, max.; km/h): 0, 3, 7
      - Frequency (passes/hour/section): 3600

SECTIONS

- Number of pavement sections that can be tested simultaneously: 1
- Pavement section dimensions:

- Width:
- Pavement thickness (including surface, base and subgrade):
- Total mean length of a section:
- Useful mean testing length of a section:

- Construction procedure: ☐ Conventional procedure → ☐ Full scale plant ☐ Reduced scale plant
  ☒ Special procedure → ☐ Special plant ☒ Manual

ENVIRONMENTAL CONTROL

- Air temperature control:
  - ☐ No
  - ☐ Partial → Describe control process:
  - ☒ Yes → Range: **10 to 60 degrees celsius**
    Describe control process: **Heaters and coolers**
  - * How do you measure air temperature? **Thermocouple**
- Pavement temperature control:
  - ☐ No
  - ☒ Indirect control through air temperature
  - ☐ Partial → Describe control process:
  - ☒ Yes → Range: **16 to 30 degree celsius**
    Describe control process: **Heaters and coolers**
- Air moisture control:
  - ☒ No
  - ☐ Yes → Describe control process:
  - * How do you measure air moisture?
- Freezing-thaw cycles control:
  - ☒ No
  - ☐ Yes → Describe control process:
- Water table level control:
  - ☒ Not monitored
  - ☐ Monitored but not controlled
  - ☐ Monitored and Describe control process:
  - ☐ Monitored an Describe control process:
- Rainfall control:
  - ☐ No rainfall is possible (e.g. indoors facility)
  - ☒ Natural rainfall
  - ☐ Natural rainfall supplemented when needed
  - ☐ Fully controlled

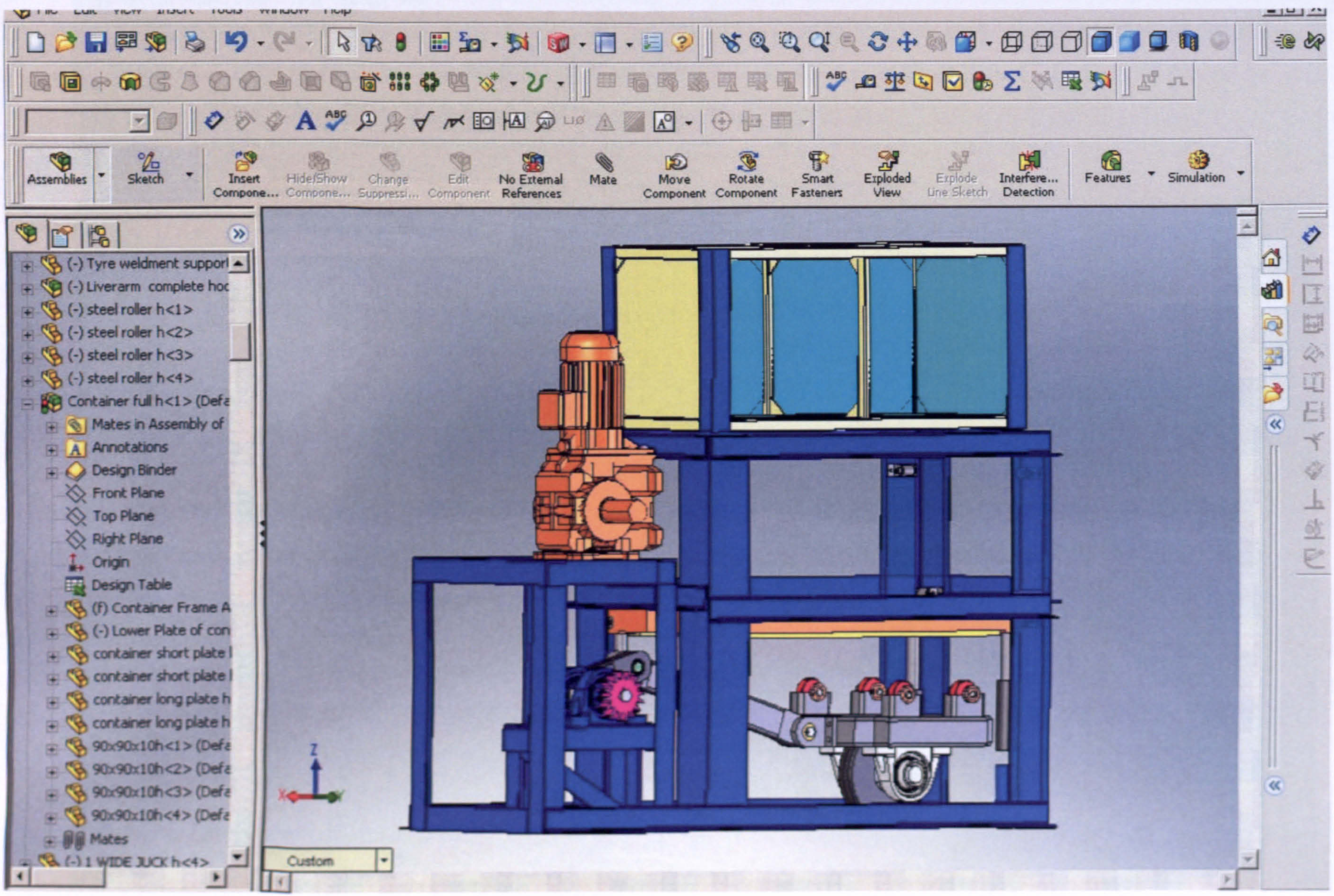


Fig. 8-10: A side view of the LoCATE Facility shows the positioning of the Motor

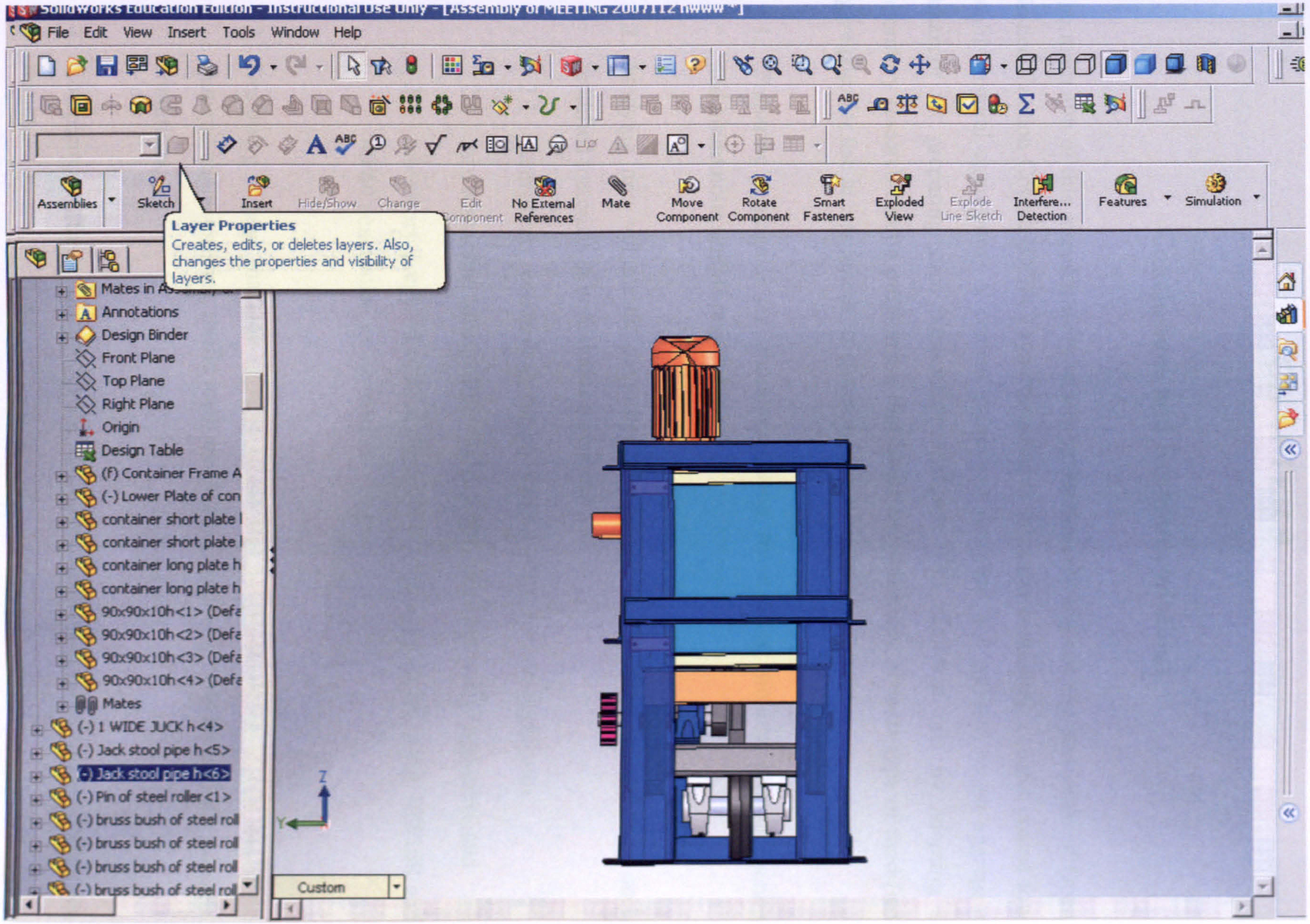


Fig. 8-11: A back view of the LoCATE Facility shows the positioning of the Motor

CHAPTER 9

Conclusions and Recommendations

The main aim of this research work was to investigate the engineering properties of Open Graded Friction Course (OGFC) type bituminous mixtures containing cement coated low quality natural aggregates. A cement paste was designed and utilized as an aggregate enhancer for preparing such bituminous mixtures. This aim was met by adopting a laboratory research testing programme that tested the suitability of such type of modified aggregates for use in bituminous mixtures, which includes:

1. Preparing aggregates for laboratory testing, by washing, sieving, cement coating and bituminous sample preparations.

2. Studying mix volumetric properties to understand the effect of inclusion such modified materials in bituminous mixtures:

   2.1 control mix containing primary crushed rock, Arcow aggregates,

   2.2 mix containing low quality natural aggregates, Croxden aggregates, and

   2.3 laboratory prepared cement coated Croxden aggregates, using 100/150 and 40/60 penetration grade binders.

3. Designing a new machine capable of testing pavement deformations based on material characteristics, traffic load, mean speed of heavy vehicles and site climatic conditions.

4. Comparing stiffness moduli for the mixtures mentioned above using ITSM tests results.

5. Comparing axial deformation for the mixtures by performing RLAT tests.

6. Performing index tests, namely, wheel tracking test on 305 x 305 x 50mm and 500 x 180 x 50mm shaped samples.

9.1 Conclusions

The results and analysis of the background studies and experimental investigations produced the following number of new and important findings and conclusions:

1 General:

1.1 The thesis clarifies the fundamental properties required of an aggregate to produce the physical properties required for a successful bituminous mixture. Furthermore it reveals the roughness and adhesion to the binder properties of a cement coating, a coating that was found to have a real impact on asphalt mixtures as it has improved the performance of the tested bituminous materials.

1.2 The cement coating behaved very well during laboratory testing, proving to be very amenable to bonding with bitumen. The physical properties of the aggregates tested were substantially improved with the addition of cement coating as surface roughness of the aggregates were improved considerably, and that led to enhancing the strength of bituminous products made from them.

1.3 Using coated aggregates for manufacturing porous asphalt, namely OGFC, have yielded very rewarding results. Elastic stiffness and resistance to axial deformation have improved by quite significant factors after the addition of cement paste. The literature review show that some improvement could be evidenced, and this improvement was expected to be of small magnitude; however the improvements that were found after carrying out the testing programme have shown much greater than anticipated.

2 ITSM Test programme:

From the results of the indirect stiffness modulus tests the following findings were worth mentioning:

2.1 Each mix's susceptibility to temperature was reflected by the change in its resilient modulus values as a function of temperature. For the binders used in this research, the binder type has proven to have an effect on the resilient modulus. At each of the testing temperatures, the mixes made from the harder binder (i.e. 40/60 pen graded binder) had higher resilient moduli for all the mixes studied.

2.2 The effect of binder type was clearly observed as the testing temperature was increased. It was noticed that when the test temperature increased up to 30^0C, the mixes with 40/60 pen graded binder show higher *Mr* compared with that obtained for the same mixes but with 100/150 pen graded binder, showing difference in *Mr* values of 85.1%, 29.6% and 26.8% , for (UC), (CC) and (A) respectively.

2.3 Differences in the calculated resilient modulus between the three mixes can be attributed to:

2.3.1 Internal mix's micro and macro structure reflected by their bulk densities and the corresponding percent of air voids content within the tested specimens.

2.3.2 The adhesion between the aggregate and the binders and in case of cement coated aggregate between the aggregate and the cement paste.

2.4 The addition of the cement paste to Croxden aggregates had influenced the angularity of the aggregate particles, as was shown in chapter Three, and had improved the adhesion between binder and thus subsequently affecting the bulk density of the corresponding mix gradation and the percent air voids of compacted specimens and resulted in significant increase in the mix stiffness moduli.

3 RLAT Test Programme:

The amount of axial deformation had decreased significantly by the introduction of cement coated aggregates within the bituminous mixtures, and that had led to the subsequent increase in their resistance to permanent deformation, (rutting). On the other hand, the reduction in bulk density of the coated mixtures was also thought to be closely related to this increase in the rutting resistance, which in turn is related to the increase in the coated aggregates' angularity and surface roughness. This project work's results have shown these relationships by comparing the changes in which is inter-related to testing temperature, bulk density, air voids and axial deformation due to the addition of cement paste to the aggregates within the tested mixtures.

4 Wheel Tracking Testing Programme:

From the results of the wheel-tracking test, it was found that Arcow mixes were the best in terms of rutting resistance. Cement Coated mixes proved to be second best. Uncoated Croxden mixes were the worst. Based on all these observations and the conditions specified by current standards, it is obvious that the use of cement paste as a coating agent for natural aggregates will be enhancing its performance. The wheel-tracking test is also considered to be a more simulative test. For this reason it was utilized in this study.

5. The results of the RLAT and the wheel-tracking test are very similar in relative terms. The results from the RLAT and the wheel tracking tests are highly related, at the corresponding testing temperature. It can be concluded that the amount of axial deformation had decreased significantly by the introduction of cement coated aggregates within the bituminous mixtures, and that had led to the subsequent increase in their resistance to permanent deformation, (rutting). On the other hand, the reduction in bulk density of the coated mixtures was also thought to be closely related to this increase in the rutting resistance, which is in turn related to the increase in the coated aggregates' angularity and surface roughness.

6. The wheel-tracking and RLAT results have conformed to the expected relative behaviour of the three types of materials under investigation. Porous asphalts have strong inter-particle contacts in the aggregate skeleton and experimental knowledge has shown that these materials have good resistance to deformation provided that they are restrained laterally. On the other hand, the uncoated Croxden mixtures' aggregate to aggregate contact was seriously reduced even though that it had the least air void within the mix and its binder content was at its optimum; this led to material instability and very low deformation resistance.

7. None of the indoor or the outdoor asphalt testing equipments that were described in this thesis can provide a comprehensive cover of all the above factors. However the LoCATE, which can be defined as the controlled application of a wheel load to pavement structures for the purpose of simulating the effects of long-term in-service loading conditions in a relatively short period of time, does.

Finally, outstanding new results were achieved; using cement paste for coating the waste natural aggregate and as thus the author is hoping that this will eliminate the restriction on their uses in road pavement by road engineers. This is an outstanding knowledge extension in this field and marks a start for more research in this ever growing industry.

9.2 Recommendations for Further Works

The following further work is highly worth doing;

1. Use of LoCATE to study the influence of each of the constituents of HMA on the response of an asphalt mix to deformation (elastic or plastic). This could lead to the utilization of the secondary aggregates in a wider range of applications.

2. Further studies can also be conducted on a variety of stabilizers and additives that may even improve the already enhanced physical properties of Hot Mix Asphalts containing cement coated aggregates.

3. Other aggregate gradations may be utilized to investigate the effect of cement paste of the asphalt mixture performance under different loading and environmental conditions according to the current British and European standards.

4. By using LoCATE, the impact of the following can be studied:

   4.1 For any selected pavement, rut depth can be measured after having samples/slabs of bituminous material extracted from that pavement. The samples/slabs can be fully analyzed for the purpose of determining their composition and compaction degree before and after testing.

   4.2 After calibrating the LoCATE machine, testing of bituminous pavement can be carried out indoors and then it can be towed out to perform outdoor testing on real road sections for the measurement of pavements rutting and stiffness. The next stage consists of creating a model able to represent the relationship between the in-situ rut depth, measured on pavement using LoCATE, and the rate of rutting measured in the laboratory, using LoCATE too, operating under the same testing conditions of the French Pavement Rutting Tester (FPRT) facility, i.e. using the same tyre, speed and loading conditions the FPRT normally designed for.

   4.3 Evaluation of coating and coating paste on pavement structural performance improvement.

   4.4 Evaluation of wearing and salt-road surface interaction (i.e. gritting) in icy weathers.

   4.5 Evaluation, validation and improvement of cold, medium and hot mix design.

5. It is also recommended that; in addition to the usage of one type of an asphalt mixer for preparing small sized specimens, a second machine was better to be used for making larger specimens. As difference in sample's preparation temperatures due to the difference in the temperature controlling techniques may result in some discrepancies in tests results. This can be eliminated by using one heating system for all sample's preparations.

6. Finally, further research on developing the cement coated aggregate should be carried out by utilizing a single testing machine (LoCATE) to eliminate any discrepancies due to testing samples by machines less representative to the actual pavement / tyre interaction, as (LoCATE) was designed for this purpose as well as being mobile.

Appendix A

Summary of limits that have been suggested for the mechanical properties of road-stone

Type of construction	Crushing strength (Ib./Sq.in) (min)	Aggregate Crushing Value ACV (% fines) (max)
Light or medium traffic		
Water bound		
Light traffic	15,000 (10.546 kg-force/sq. mm)	30
Medium traffic	25,000 (17.576 kg-force/sq. mm)	23
Concrete		
Body of road	10,000 (7.030 kg-force/sq. mm)	40
Surface of road	12,000 (8.436 kg-force/sq. mm)	35
Bituminous		
Body of road	15,000 (10.546 kg-force/sq. mm)	30
Surface of road	25,000 (17.576 kg-force/sq. mm)	23
Heavy traffic		
Water bound	40,000 (28.122 kg-force/sq. mm)	13
Concrete		
Body of road	12,000 (8.436 kg-force/sq. mm)	35
Surface of road	15,000 (10.546 kg-force/sq. mm)	30
Bituminous		
Body of road	15,000 (10.546 kg-force/sq. mm)	30
Surface of road	30,000 (21.092 kg-force/sq. mm)	17

Appendix B

Traffic Loadings and Maximum Aggregate Abrasion Values for

Flexible Surfacing

Traffic in commercial vehicles per lane per day	Under 250*	Up to 1 000	Up to 1 750	Up to 2 500	Up to 3 250	Over 3 250
Maximum AAV for chipping	14	12	12	10	10	10
Maximum AAV for aggregate in bituminous wearing courses	16	16	14	14	12	12

* For lightly trafficked roads carrying less than 250 commercial vehicles per lane per day aggregate of higher AAV may be used where experience has shown that a satisfactory performance is achieved by aggregate from a particular source. (Technical Memorandum H16/76 DOT, UK)

Appendix C

Categories of sites and minimum Polished Stone Value for Flexible Roads

Site	Approximate percentage of all roads in England	Definition	Minimum Polished Stone Value
A1 (difficult)	Less 0.1% than	i) Approaches to traffic signals on roads with 85% while speed of traffic greater than 40 mile/h (64km/h) ii) Approaches to traffic signals, pedestrian crossings and similar hazards on main urban roads	Less than 250 cv/lane/day: 60 250 to 1000 cv/lane/day:65 1000 to 1750 cv/lane/day:70 More than 1750 cv/lane/day:75
A2 (difficult)	Less 4% than	i) Approaches to and across major priority junctions on roads carrying more than 250 commercial vehicles per lane per day* (* the 350 cv/lane/day applies to each approach) ii) Roundabouts and their approaches iii) Bends with radius less than 150m on roads with an 85%ile speed of traffic greater than 40 mile/h (64 km/h) iv) Gradients of 5% or steeper, longer than 100m	Less than 1750 cv/lane/day: 60 1750 to 2500 cv/lane/day: 65 2500 to 3250 cv/lane/day: 70 Mare than 3250 cv/lane/day: 75
B (average)	Less 15% than	Generally straight sections of And large radius curves on: i) Motorways ii) Trunk and principal roads iii) Other roads carrying more than 250 commercial vehicles per lane per day	Less than 1750 cv/lane/day: 55 1750 to 4000 cv/lane/day: 60 More than 4000 cv/lane/day: 65
C (easy)	Less 81% than	i) Generally straight sections of lightly trafficked roads i.e. less than 250 cv/day ii) Other roads where wet skidding accidents are unlikely to be a problem	45

Appendix D

Current Mix Design methods

Property tested	Procedure	Applied Specification
Binder Run-Off	Binder Drainage Test	BS DD 232: 2005
Voids Contents	Volumetric measurement	BS EN 12697-8:2003
Bulk density	before compaction	BS 812 - 2 : 1995
Bulk density	after compaction	BS EN 12697 6:2003+A1:2007
Maximum Theoretical Density	(Loose sample)	EN 12697-5:2002+A1:2007 (E) (Procedure A)
Elastic Stiffness	RLIT	BS EN 12697-26:2004
Deformation	RLAT	BS DD 226 1996
Deformation	Wheel tracking	BS EN 12697-22:2003

Appendix E (CH.4)

Mix design and volumetric properties of laboratory prepared samples:

Table E 4-1
Bulk densities and Air Voids contents of (100 x 63.5 mm) samples with 100/150 pen grade Binder

Mix Type	Mass (g)	Average Height (mm)	Diameter (mm)	Volume of sample (mm^3)	Gmb	Corrected Gmb	Gmm	Air Void (%)
UC1	1055	62	101	496734.548	2.124	2.133	2.561	16.7
UC2	1050	62	101	496734.548	2.114	2.123	2.561	17.1
UC3	1070	61	101	488722.700	2.189	2.198	2.561	14.2
UC4	1075	62	101	496734.548	2.164	2.173	2.561	15.1
UC5	1065	60	101	480710.853	2.215	2.225	2.561	13.1
UC6	1055	62	101	496734.548	2.124	2.133	2.561	16.7
UC7	1070	61	101	488722.700	2.189	2.198	2.561	14.2
UC8	1060	62	101	496734.548	2.134	2.143	2.561	16.3
UC9	1065	63	101	504746.396	2.110	2.119	2.561	17.3
UC10	1075	62	101	496734.548	2.164	2.173	2.561	15.1
AVERAGE	**1064**	**62**		**494330.994**	**2.153**	**2.162**		**15.6**
CC1	1070	63	101	504746.396	2.120	2.126	2.540	16.3
CC2	990	61	101	488722.700	2.026	2.032	2.540	20.0
CC3	995	62	101	496734.548	2.003	2.009	2.540	20.9
CC4	990	63	101	504746.396	1.961	1.967	2.540	22.6
CC5	995	61	101	488722.700	2.036	2.042	2.540	19.6
CC6	1000	62	101	496734.548	2.013	2.019	2.540	20.5
CC7	995	62	101	496734.548	2.003	2.009	2.540	20.9
CC8	990	61	101	488722.700	2.026	2.032	2.540	20.0
CC9	970	59	101	472699.005	2.052	2.058	2.540	19.0
CC10	995	61	101	488722.700	2.036	2.042	2.540	19.6
AVERAGE	**1007**	**62**		**492728.624**	**2.028**	**2.034**		**19.9**
A1	1080	64	101	512758.243	2.106	2.115	2.660	20.5
A2	1080	63	101	504746.396	2.140	2.149	2.660	19.2
A3	1065	62	101	496734.548	2.144	2.153	2.660	19.1
A4	1055	62	101	496734.548	2.124	2.133	2.660	19.8
A5	1075	63	101	504746.396	2.130	2.139	2.660	19.6
A6	1080	63	101	504746.396	2.140	2.149	2.660	19.2
A7	1065	63	101	504746.396	2.110	2.119	2.660	20.3
A8	1050	62	101	496734.548	2.114	2.123	2.660	20.2
A9	1085	63	101	504746.396	2.150	2.158	2.660	18.9
A10	1055	62	101	496734.548	2.124	2.133	2.660	19.8
AVERAGE	**1069**	**62.6**		**502342.841**	**2.128**	**2.137**		**19.7**

Table E 4-2

Bulk densities and Air Voids contents of (100 x 63.5 mm) samples with 40/60 pen grade Binder

Mix Type	Mass (g)	Average Height (mm)	Diameter (mm)	Sample's Volume (mm^3)	Gmb	Corrected Gmb	Gmm	Air Void (%)
UC1	1050	61	101	488722.700	2.148	2.157	2.558	15.7
UC2	1020	61	101	488722.700	2.087	2.096	2.558	18.1
UC3	1050	62	101	496734.548	2.114	2.123	2.558	17.0
UC4	1050	62	101	496734.548	2.114	2.123	2.558	17.0
UC5	1030	59	101	472699.005	2.179	2.188	2.558	14.5
UC6	1045	60	101	480710.853	2.174	2.183	2.558	14.7
UC7	1050	62	101	496734.548	2.114	2.123	2.558	17.0
UC8	1050	62	101	496734.548	2.114	2.123	2.558	17.0
UC9	1060	61	101	488722.700	2.169	2.178	2.558	14.9
UC10	1060	63	101	504746.396	2.100	2.109	2.558	17.6
AVERAGE	**1040**	**62**		**491126.255**	**2.131**	**2.140**		**16.3**
CC1	995	62	101	496734.548	2.003	2.009	2.529	20.6
CC2	990	62	101	496734.548	1.993	1.999	2.529	21.0
CC3	990	61	101	488722.700	2.026	2.032	2.529	19.7
CC4	995	61	101	488722.700	2.036	2.042	2.529	19.3
CC5	995	62	101	496734.548	2.003	2.009	2.529	20.6
CC6	995	63	101	504746.396	1.971	1.977	2.529	21.8
CC7	990	61	101	488722.700	2.026	2.032	2.529	19.7
CC8	995	62	101	496734.548	2.003	2.009	2.529	20.6
CC9	1020	63	101	504746.396	2.021	2.027	2.529	19.9
CC10	995	62	101	496734.548	2.003	2.009	2.529	20.6
AVERAGE	**993**	**62**		**495933.363**	**2.008**	**2.014**		**20.3**
A1	1050	62	101	496734.548	2.114	2.123	2.652	20.0
A2	1070	64	101	512758.243	2.087	2.095	2.652	21.0
A3	1055	62	101	496734.548	2.124	2.133	2.652	19.6
A4	1065	62	101	496734.548	2.144	2.153	2.652	18.8
A5	1075	63	101	504746.396	2.130	2.139	2.652	19.4
A6	1055	63	101	504746.396	2.090	2.099	2.652	20.9
A7	1045	61	101	488722.700	2.138	2.147	2.652	19.0
A8	1055	62	101	496734.548	2.124	2.133	2.652	19.6
A9	1050	62	101	496734.548	2.114	2.123	2.652	20.0
A10	1045	62	101	496734.548	2.104	2.112	2.652	20.3
AVERAGE	**1057**	**62.4**		**499138.102**	**2.117**	**2.126**		**19.9**

Table E 4-3

Bulk densities and Air Voids contents of (305 x 305 x 50 mm) samples with 100/150 pen grade Binder

Type of Mix	h (mm)	l x w (mm)	Vol. x 10^{-3} (m^3)	Mass (kg)	Gmb	Gmm	Air Void (%)
UC1	51	305x305	4.744	9.561	2.015	2.321	13.2
UC2	51	305x305	4.744	9.526	2.008	2.321	13.5
UC3	51	305x305	4.744	9.551	2.013	2.321	13.3
UC4	51	305x305	4.744	9.535	2.010	2.321	13.4
UC5	52	305x305	4.837	9.567	1.978	2.321	14.8
Average:	**51.2**		**4.763**	**9.548**	**2.005**		**13.6**
CC1	52	305x305	4.837	8.561	1.770	2.241	21.0
CC2	52	305x305	4.837	8.557	1.769	2.241	21.1
CC3	51	305x305	4.744	8.568	1.806	2.241	19.4
CC4	52	305x305	4.837	8.583	1.774	2.241	20.8
CC5	52	305x305	4.837	8.558	1.769	2.241	21.0
Average:	**51.8**		**4.818**	**8.565**	**1.778**		**20.7**
A1	52	305x305	4.837	9.594	1.983	2.524	21.4
A2	51	305x305	4.744	9.568	2.017	2.524	20.2
A3	51	305x305	4.744	9.575	2.018	2.524	20.7
A4	52	305x305	4.837	9.636	1.992	2.524	21.5
A5	52	305x305	4.837	9.593	1.983	2.524	20.3
Average:	**51.6**		**4.800**	**9.593**	**1.999**		**20.8**

Table E 4-4

Bulk densities and Air Voids contents of (305 x 305 x 50 mm) samples with 40/60 pen grade Binder

Type of Mix	h (mm)	l x w (mm)	Vol. x 10^{-3} (m^3)	Mass (kg)	Gmb	Gmm	Air Void (%)
UC1	50	305x305	4.651	9.253	1.989	2.319	14.2
UC2	51	305x305	4.744	9.660	2.036	2.319	12.2
UC3	51	305x305	4.744	9.650	2.034	2.319	12.3
UC4	52	305x305	4.837	9.672	1.999	2.319	13.8
UC5	51	305x305	4.744	9.661	2.036	2.319	12.2
Average:	**51**		**4.744**	**9.579**	**2.019**		**12.9**
CC1	51	305x305	4.744	8.505	1.793	2.238	19.9
CC2	52	305x305	4.837	8.542	1.766	2.238	20.8
CC3	52	305x305	4.837	8.550	1.768	2.238	21.0
CC4	52	305x305	4.837	8.547	1.767	2.238	20.7
CC5	51	305x305	4.744	8.548	1.802	2.238	20.7
Average:	**51.6**		**4.800**	**8.538**	**1.779**		**20.6**
A1	52	305x305	4.837	9.654	1.996	2.522	20.9
A2	51	305x305	4.744	9.561	2.015	2.522	20.1
A3	50	305x305	4.651	9.483	2.039	2.522	19.2
A4	51	305x305	4.744	9.651	2.034	2.522	19.3
A5	52	305x305	4.837	9.655	1.996	2.522	20.9
Average:	**51.2**		**4.763**	**9.601**	**2.016**		**20.1**

Table E 4-5

Bulk densities and Air Voids contents of *(500 x 180 x 50 mm)* samples with 100/150 pen grade Binder

Type of Mix	h (mm)	l x w (mm)	Vol. x 10^{-3} (m^3)	Mass (kg)	Gmb	Gmm	Air Void (%)
UC1	51.0	500x180	4.590	9.345	2.036	2.321	12.3
UC2	52.0	500x180	4.680	9.275	1.982	2.321	14.6
UC3	52.0	500x180	4.680	9.349	1.998	2.321	13.9
UC4	50.0	500x180	4.500	9.174	2.039	2.321	12.2
Average:	**51.3**		**4.613**	**9.286**	**2.014**		**13.2**
CC1	52.0	500x180	4.680	8.310	1.776	2.241	20.8
CC2	52.0	500x180	4.680	8.285	1.770	2.241	21.0
CC3	50.0	500x180	4.500	8.150	1.811	2.241	19.2
CC4	52.0	500x180	4.680	8.250	1.763	2.241	21.3
Average:	**51.5**		**4.635**	**8.249**	**1.780**		**20.6**
A1	51.0	500x180	4.590	9.140	1.991	2.524	21.1
A2	51.0	500x180	4.590	9.225	2.010	2.524	20.4
A3	52.0	500x180	4.680	9.205	1.967	2.524	22.1
A4	51.0	500x180	4.590	9.228	2.010	2.524	20.3
Average:	**51.3**		**4.613**	**9.200**	**1.995**		**21.0**

Appendix F (CH. 5)

Repeated Load Indirect Tensile Tests' Results

Sample	Axis	Testing	Stiffness	Rise	Testing	Stiffness	Rise	Testing	Stiffness	Rise
		Temp ^{0}C	modulus (MPa)	time (ms)	Temp ^{0}C	modulus (MPa)	time (ms)	Temp ^{0}C	modulus (MPa)	time (ms)
UC1	1	10	2011	124	20	617	124	30	182	124
UC1	2	10	1843	124	20	574	126	30	176	124
UC2	1	10	2101	125	20	504	124	30	209	124
UC2	2	10	1954	124	20	486	124	30	168	124
UC3	1	10	1986	124	20	1058	124	30	182	124
UC3	2	10	1749	126	20	1012	124	30	161	125
UC4	1	10	1980	124	20	934	125	30	201	124
UC4	2	10	1723	124	20	870	124	30	186	124
UC5	1	10	1983	126	20	990	124	30	195	123
UC5	2	10	1752	124	20	867	124	30	181	124
UC6	1	10	1894	124	20	847	124	30	190	124
UC6	2	10	1669	124	20	825	124	30	159	124
UC7	1	10	1755	123	20	929	124	30	171	124
UC7	2	10	1539	124	20	898	124	30	168	124

Table F 5-1: Stiffness Moduli for Uncoated Croxden mixes with 100/150 pen grade Binder @ three different temperatures

Sample	Axis	Testing	Stiffness	Rise	Testing	Stiffness	Rise	Testing	Stiffness	Rise
		Temp ^{0}C	modulus (MPa)	time (ms)	Temp ^{0}C	modulus (MPa)	time (ms)	Temp ^{0}C	modulus (MPa)	time (ms)
CC1	1	10	2300	124	20	679	124	30	477	126
CC1	2	10	1937	124	20	643	124	30	410	124
CC2	1	10	2205	124	20	766	125	30	398	124
CC2	2	10	1864	126	20	653	124	30	336	124
CC3	1	10	2410	125	20	990	126	30	341	126
CC3	2	10	2028	121	20	982	124	30	286	124
CC4	1	10	2430	123	20	1094	126	30	341	124
CC4	2	10	2045	122	20	990	124	30	289	124
CC5	1	10	2211	125	20	1094	124	30	498	125
CC5	2	10	1862	124	20	981	124	30	418	124
CC6	1	10	2176	122	20	1085	124	30	523	124
CC6	2	10	1835	123	20	1060	124	30	446	124
CC7	1	10	2371	124	20	1097	124	30	322	124
CC7	2	10	1998	124	20	901	124	30	264	124

Table F 5-2: Stiffness Moduli for Cement Coated mixes with 100/150 pen grade Binder @ three different temperatures

Sample	Axis	Testing Temp ^0C	Stiffness modulus (MPa)	Rise time (ms)	Testing Temp ^0C	Stiffness modulus (MPa)	Rise time (ms)	Testing Temp ^0C	Stiffness modulus (MPa)	Rise time (ms)
A1	1	10	2540	125	20	1157	124	30	410	124
A1	2	10	2494	124	20	1142	126	30	398	124
A2	1	10	2562	124	20	1157	124	30	419	124
A2	2	10	2538	126	20	1146	125	30	407	124
A3	1	10	2563	125	20	1145	127	30	415	124
A3	2	10	2493	121	20	1130	125	30	411	125
A4	1	10	2556	123	20	1137	126	30	412	124
A4	2	10	2498	122	20	1096	123	30	397	124
A5	1	10	2530	125	20	1143	124	30	407	124
A5	2	10	2495	124	20	1138	125	30	399	126
A6	1	10	2536	123	20	1130	126	30	422	124
A6	2	10	2499	122	20	1088	124	30	404	124
A7	1	10	2549	124	20	1159	123	30	403	124
A7	2	10	2536	124	20	1145	126	30	398	124

Table F 5-3: Stiffness Moduli for Arcow mixes with 100/150 pen grade Binder @ three different temperatures

Sample	Axis	Testing Temp ^{0}C	Stiffness modulus (MPa)	Rise time (ms)	Testing Temp ^{0}C	Stiffness modulus (MPa)	Rise time (ms)	Testing Temp ^{0}C	Stiffness modulus (MPa)	Rise time (ms)
UC1	1	10	3418	124	20	1238	124	30	379	124
UC1	2	10	2864	124	20	1033	124	30	347	124
UC2	1	10	3831	123	20	1387	124	30	370	125
UC2	2	10	3225	124	20	1169	124	30	348	124
UC3	1	10	4952	124	20	1368	124	30	396	124
UC3	2	10	4163	124	20	1139	124	30	265	124
UC4	1	10	3420	124	20	1182	123	30	293	124
UC4	2	10	2868	124	20	995	124	30	314	124
UC5	1	10	3889	124	20	1259	124	30	349	124
UC5	2	10	3261	124	20	1089	124	30	272	124
UC6	1	10	4844	124	20	1523	125	30	382	124
UC6	2	10	4059	124	20	1273	124	30	305	124
UC7	1	10	3429	124	20	1153	124	30	358	123
UC7	2	10	2867	124	20	972	124	30	317	124

Table F 5-4: Stiffness Moduli for Uncoated mixes with 40/60 pen grade Binder @ three different temperatures

Sample	Axis	Testing Temp ^{0}C	Stiffness modulus (MPa)	Rise time (ms)	Testing Temp ^{0}C	Stiffness modulus (MPa)	Rise time (ms)	Testing Temp ^{0}C	Stiffness modulus (MPa)	Rise time (ms)
CC1	1	10	4552	124	20	1756	124	30	497	124
CC1	2	10	3827	124	20	1480	124	30	489	124
CC2	1	10	5621	124	20	1896	124	30	540	124
CC2	2	10	4736	124	20	1589	124	30	498	124
CC3	1	10	5687	124	20	1583	124	30	556	125
CC3	2	10	4787	125	20	1337	125	30	415	124
CC4	1	10	4576	123	20	1842	124	30	493	125
CC4	2	10	3850	124	20	1546	124	30	414	124
CC5	1	10	4556	124	20	1900	125	30	542	124
CC5	2	10	5428	126	20	1642	122	30	389	124
CC6	1	10	4066	124	20	1930	123	30	535	125
CC6	2	10	3458	124	20	1589	122	30	522	124
CC7	1	10	4539	124	20	1812	122	30	558	124
CC7	2	10	3801	124	20	1530	124	30	447	124

Table F 5-5: Stiffness Moduli for Cement coated mixes with 40/60 pen grade Binder @ three different temperatures

Sample	Axis	Testing	Stiffness	Rise	Testing	Stiffness	Rise	Testing	Stiffness	Rise
		Temp ^{0}C	modulus (MPa)	time (ms)	Temp ^{0}C	modulus (MPa)	time (ms)	Temp ^{0}C	modulus (MPa)	time (ms)
A1	1	10	5018	125	20	2391	124	30	532	124
A1	2	10	4219	124	20	2007	126	30	440	124
A2	1	10	7314	124	20	1701	124	30	586	124
A2	2	10	6139	126	20	1435	125	30	489	124
A3	1	10	5993	125	20	2014	127	30	481	124
A3	2	10	5040	121	20	1867	125	30	399	125
A4	1	10	5921	123	20	2527	126	30	573	124
A4	2	10	5010	122	20	2314	123	30	476	124
A5	1	10	4390	125	20	2266	124	30	551	124
A5	2	10	5219	124	20	1911	125	30	457	126
A6	1	10	4817	122	20	2391	124	30	628	124
A6	2	10	4044	123	20	2057	126	30	534	124
A7	1	10	4929	124	20	2347	123	30	589	124
A7	2	10	4136	124	20	1753	124	30	491	124

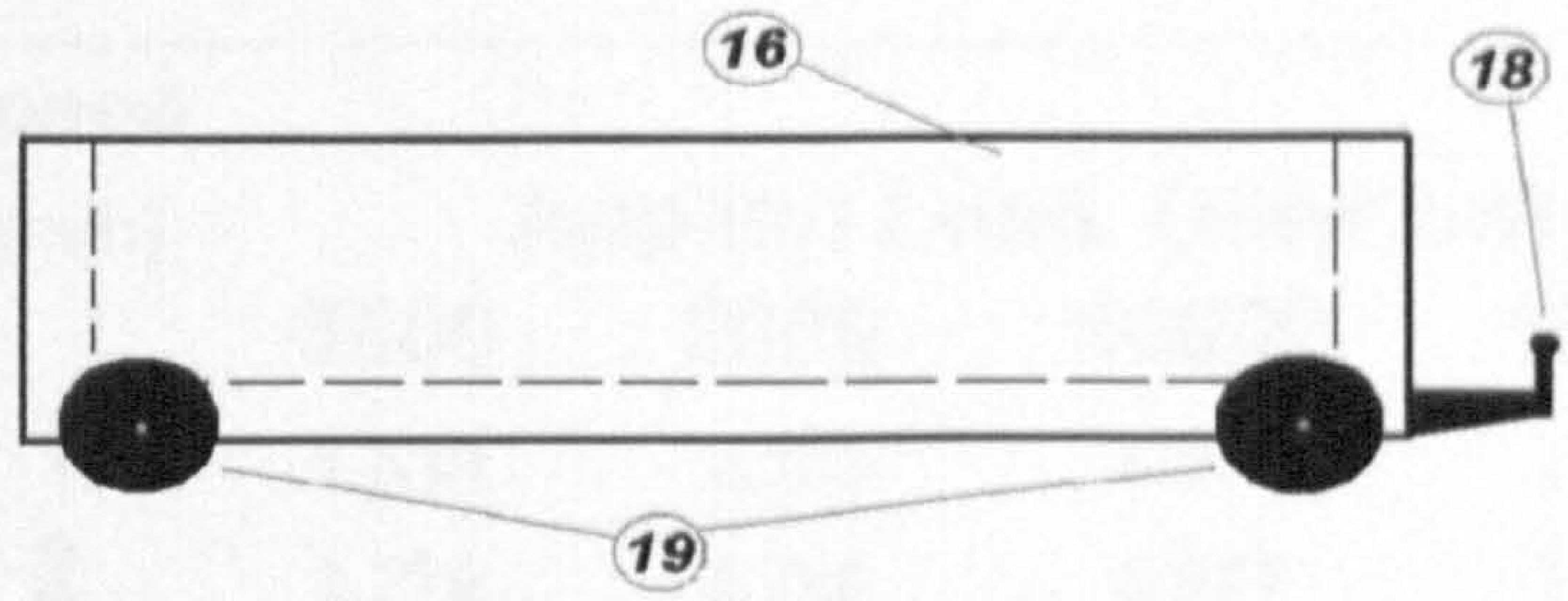
Table F 5-6: Stiffness Moduli for Arcow mixes with 40/60 pen grade Binder @ three different temperatures

Appendix G

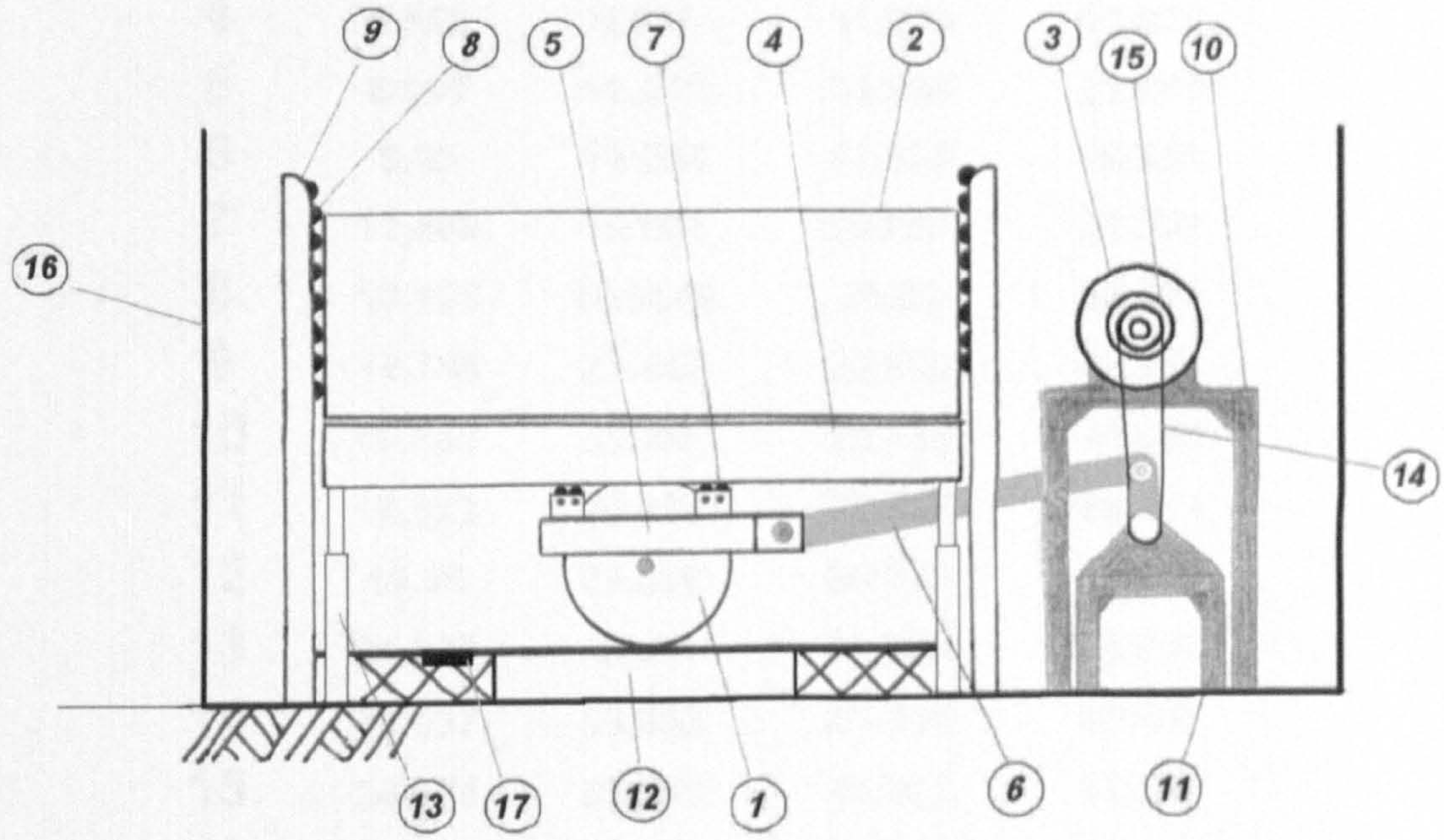
G 8-1

Schematic View of the LoCATE Facility

(a) Side view of LoCATE



(b) Details of LoCATE main elements



Main parts of the model:

1- testing tyre
2- loading container
3- driving motor
4- stiff supporting beam
5- chasse
6- lever arm
7- steel rollers
8- supporting rubber rollers
9- loading frame
10- motor support structure
11- lever arm support mechanism
12- sample
13- jack
14- Driving belt
15- Set of pulleys to adjust motor speed
16- the main frame
17- Load cell
18- towing mechanism
19- transporting tyres ,silde up during testing

Motor's Power with 25% extra for losses compensation P(HP)

Speed (km/h)	Applied Load (Newton) 5500	8000	10000	15000
1	1.638	2.383	2.978	4.468
2	3.276	4.766	5.957	8.936
3	4.915	7.149	8.936	13.404
4	6.553	9.532	11.915	17.873
5	8.191	11.915	14.894	22.341
6	9.83	14.298	17.873	26.809
7	11.468	16.681	20.851	31.277
8	13.106	19.0646	23.83	35.746
9	14.745	21.447	26.809	40.214
10	16.383	23.83	29.788	44.682
11	18.022	26.213	32.767	49.151
12	19.66	28.596	35.746	53.619
13	21.298	30.98	38.725	58.087
14	22.937	33.363	41.703	62.555
15	24.575	35.746	44.682	67.024
16	26.213	38.129	47.661	71.492

Calculated Values of the Motor's Horse Power (HP) related to variable loading values and speeds

Fig. G 8-3: An oblique view of the LoCATE Facility

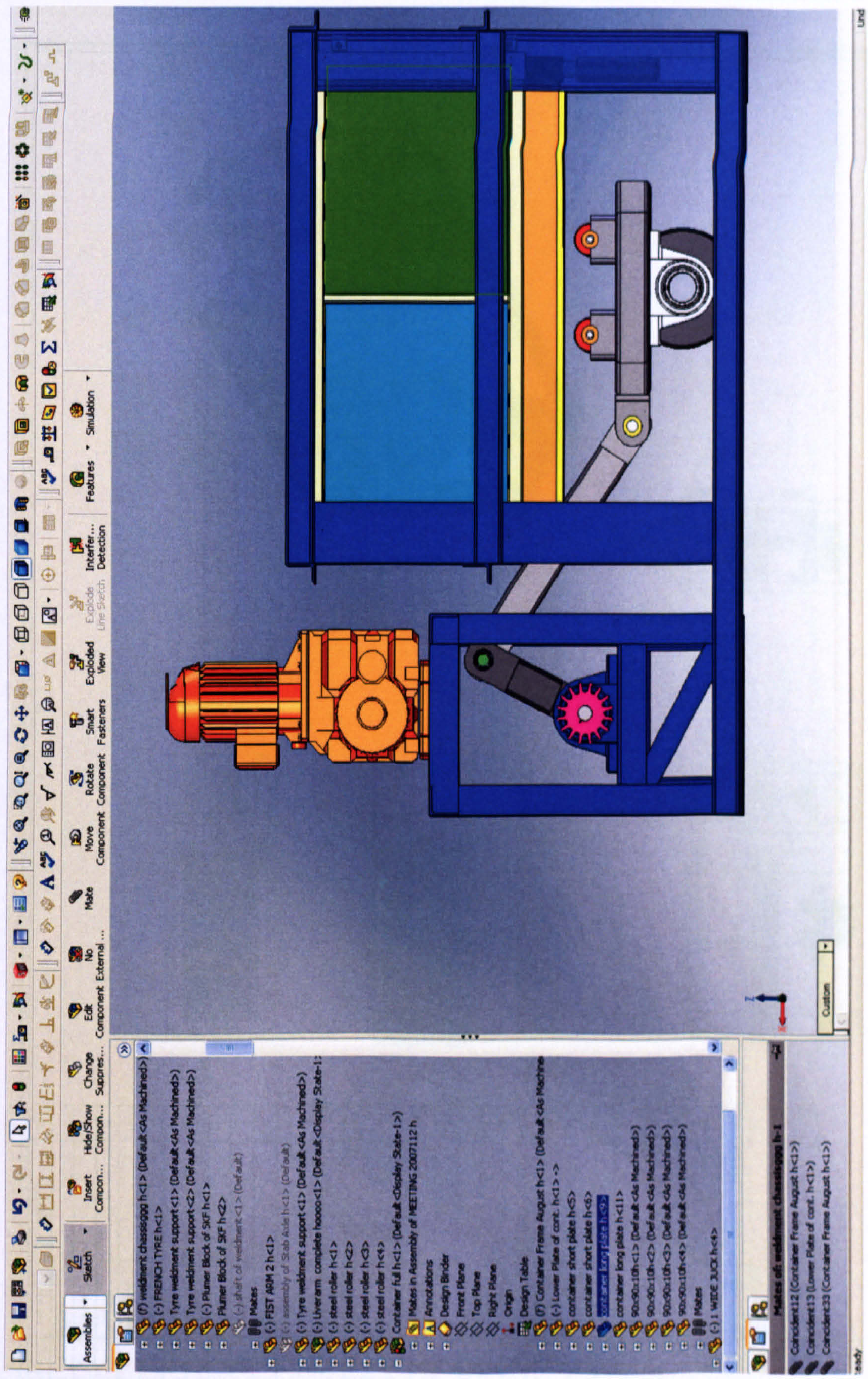


Fig. G 8-4: An oblique view of the LoCATE Facility

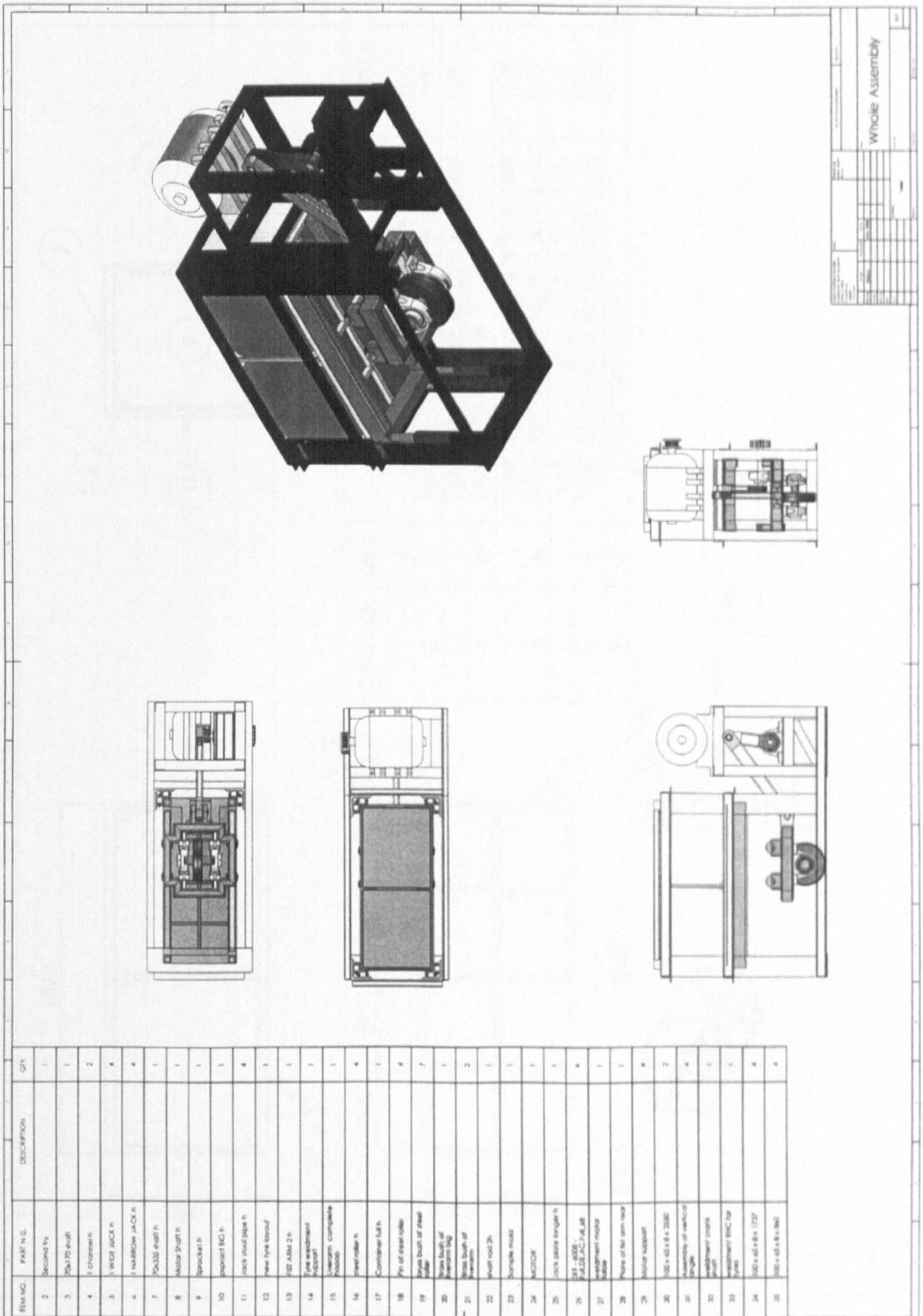
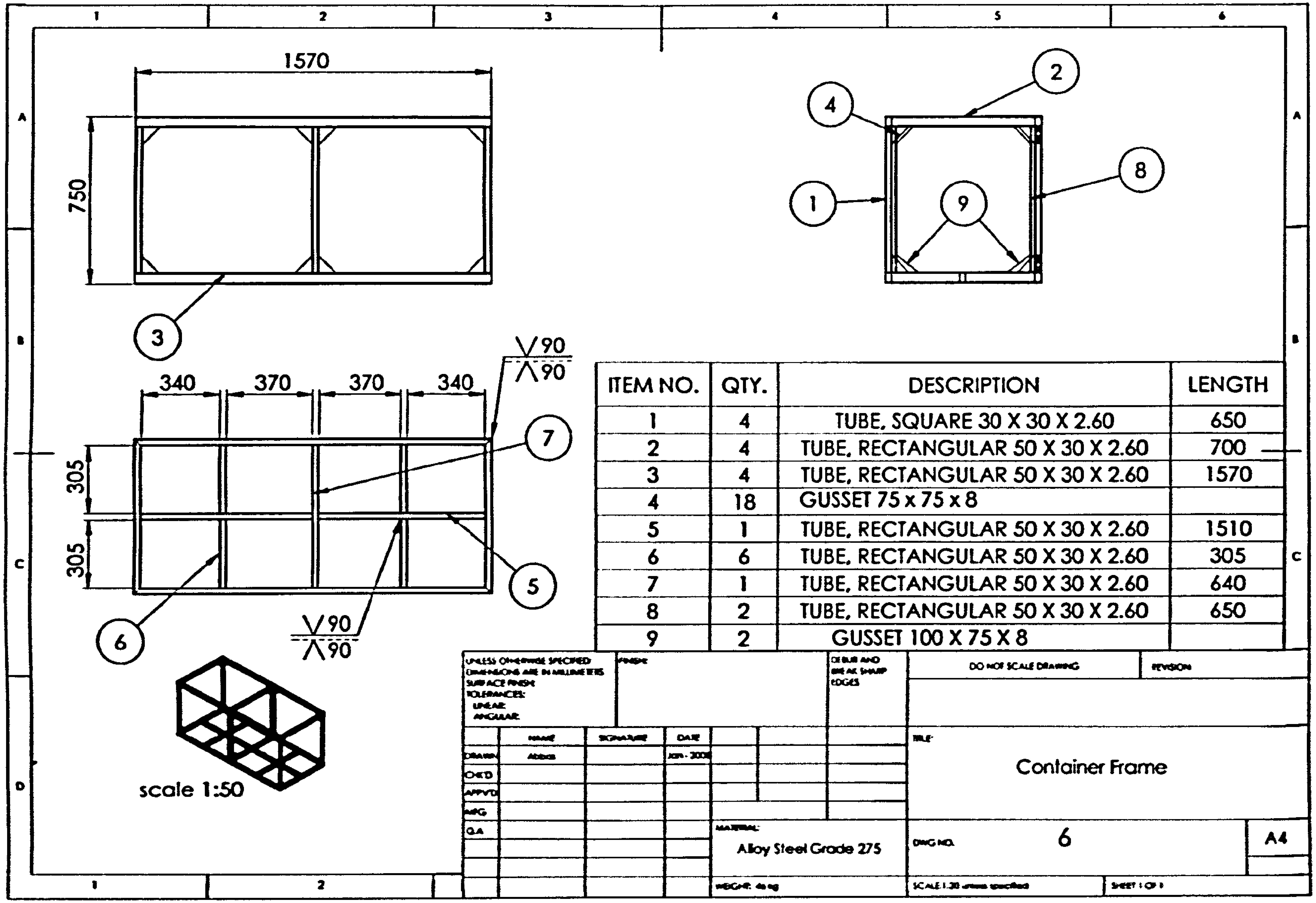


Fig. G 8-5: Drawing of the LoCATE facility from different angles



ITEM NO.	QTY.	DESCRIPTION	LENGTH
1	4	TUBE, SQUARE 30 X 30 X 2.60	650
2	4	TUBE, RECTANGULAR 50 X 30 X 2.60	700
3	4	TUBE, RECTANGULAR 50 X 30 X 2.60	1570
4	18	GUSSET 75 x 75 x 8	
5	1	TUBE, RECTANGULAR 50 X 30 X 2.60	1510
6	6	TUBE, RECTANGULAR 50 X 30 X 2.60	305
7	1	TUBE, RECTANGULAR 50 X 30 X 2.60	640
8	2	TUBE, RECTANGULAR 50 X 30 X 2.60	650
9	2	GUSSET 100 X 75 X 8	

Fig. G 8-6: Drawing of the LoCATE load carrying container

Appendix II

Laboratory Test Flow Chart

Wet sieve analysis to grade aggregates into different sizes
(BS EN 1015-1:1999)

Aggregates gradation of OGFC
(BS EN 13108-7:2006 (E) & BS PD 6691:2007)

Evaluation of volumetric properties of compacted samples
(BS EN 12697-8:2003) & (BS EN 12697-6:2003+A1:2007 (E))

Evaluation of the theoretical maximum density (TMD)
(BS EN 12697-5:2002+A1:2007 (E))

Preparing mixes of
AC 14mm open surf 100/150
& AC 14mm open surf 40/60
(BS EN 13108-7:2006 (E))

Binder Drainage Test
DS DD 232:2005

Evaluation of the indirect tensile Stiffness Modulus
using the NAT testing machine
(BS EN 12697-26:2004)

Method for determining resistance to permanent deformation of bituminous mixtures subject to unconfined dynamic loading
DD 226 1996

Evaluation of rutting using the
Large size device
at TARMAC Ltd.
(BS EN 12697-22:2003+A1:2007 (E))
Clause 8

Evaluation of rutting using the
Small size device
at TARMAC Ltd.
(BS EN 12697-22:2003+A1:2007 (E))
Clause 6

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