RECYCLING OF ASPHALT PAVEMENTS IN NEW BITUMINOUS MIXES

By

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DECLARATION

This thesis is submitted to Napier University, Edinburgh for the Degree of Doctor of Philosophy. The work described in this thesis was carried out under the supervision of Dr Fouad M Khalaf and Ms Geraldine Walsh. The work was undertaken at Napier University and Roadstone Dublin.

In accordance with Napier university regulations governing the Degree of Doctor of Philosophy, the candidate submits this thesis as his own original research unless otherwise stated.

During this period of research five papers have been written, presented, displayed as posters or published. Details are as follows:

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5. Byrne, D.A., Khalaf, F.M. and Walsh, G. 'Use of Recycled Asphalt Planings in 28mm Base', Presented at 2nd School of Built Environment Poster Exhibition, Napier University Edinburgh, 2003. (See Appendix C)

ABSTRACT

The recycling of Recycled Asphalt Pavement (RAP) in the production of new bituminous mixtures is an interesting possibility at a time when waste minimisation is to the fore. The economic cost of dumping RAP and other waste materials has increased which has led the drive by local authorities and contractors to investigate the recovery of RAP and other materials to reduce their spiralling waste disposal costs. The environmental effects of disposing RAP have posed questions to be asked about the pollution caused to the ground and streams by the leaking of leaches from RAP.

The present investigation consisted of experimental and theoretical studies of using various % of RAP aggregate in three different mixtures. The aggregates used have been recycled from bituminous mixtures.

The grading of the RAP material was firstly examined and the mix design was altered to produce specimens of the same composition as the base mixtures. Bituminous specimens and slabs were produced and stiffness, fatigue, tensile strength, Marshall, cantabro and wheel tracker tests were carried. The results showed that RAP could successfully be used in bituminous mixtures.

The effects of the increased stiffness as a result of adding the various amounts of RAP aggregate were calculated using the Shell Pavement Design Manual 3.0 (SPDM). The results showed that using RAP aggregate in the production of bituminous mixtures could reduce the overall thickness of the pavement.

Savings as a result of using RAP aggregate in bituminous mixtures were calculated and were offset against the cost of modifying the existing plant. From the results obtained the use of recycled materials in bituminous mixtures is economically viable. There are also financial benefits as a result of the reduced pavement thickness as calculated using SPDM 3.0.

Since the completion of this project Roadstone Dublin have started to use RAP in the production of bituminous mixtures. A number of trials have been planned in conjunction with Offaly County Council for the summer of 2005.

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ABBREVIATIONS

°C	=	Degrees celsius
μm	=	Micron
% v/v	=	Volume of voids expressed as a %
@	=	At
A	=	Annum
a	=	Area
AASHO	=	American Association of State Highway Officials
AAV	=	Aggregate Abrasion Value
Act	=	Actual
ACV	=	Aggregate Crushing Value
Agg	=	Aggregate
AIV	=	Aggregate Impact Value
Amt	=	Amount
Avg	=	Average
BANDS	=	Bitumen and Asphalt Nomographs Developed by Shell
BISAR	=	BItumen Stress Analysis in Roads
BS	=	British Standard
C&DW	=	Construction and Demolition Waste
CBR	=	California Bearing Ratio
cm	=	Centimetre
Comb	=	Combination
Comp	=	Compaction
Cost/km	=	Cost per kilometre
Cost/m	=	Cost per month
Cost/t	=	Cost per tonne
CRF	=	Crushed Rock Fines
D_{20}	=	Depth of 20mm Basecourse
D_{28}	=	Depth of 28mm Roadbase
DBFO	=	Design Build Finance Operate
DD	=	Draft for Development
D_{HRA}	=	Depth of Hot Rolled Asphalt

€	=	Euro
E	=	Stiffness
EU	=	European Union
h	=	Hour
HGV	=	Heavy Goods Vehicle
HRA	=	Hot Rolled Asphalt
IRR	=	Internal Rate of Return
ITFT	=	Indirect Tensile Fatigue Test
ITSM	=	Indirect Tensile Stiffness Modulus
ITST	=	Indirect Tensile Stiffness Test
Kg	=	Kilogram
Kg/m^3	=	Kilogram per metre cubed
Km	=	Kilometre
kN	=	Kilo-Newton
kPa	=	Kilo Pascal
L	=	Litre
LAAV	=	Los Angles Attrition Value
lg	=	Log
Lin	=	Linear
LVDT	=	Linear Variable Deformation Transducer
m	=	Metre
m^2	=	Metre squared
MAAT	=	Mean Annual Air Temperature
Maint	=	Maintenance
Max	=	Maximum
MDV	=	Micro-Deval Value
Min	=	Minimum
mm	=	Millimetre
MMAT	=	Mean Monthly Air Temperature
Mpa	=	Mega Pascal
mph	=	Miles per hour
ms	=	Millisecond
msa	=	Million standard axles
NAT	=	Nottingham Asphalt Tester

No.	=	Number
NPV	=	Net Present Value
NRA	=	National Roads Authority
OGL	=	Original Ground Level
Opt	=	Optimum
Ord	=	Ordinary
orig	=	Original
P	=	Parafilm
pen	=	Penetration
PQC	=	Pavement Quality Concrete
prEN	=	pre normative European de Normalisation
PSV	=	Polished Stone Value
RAP	=	Recycled Asphalt Pavement
Ref	=	Reference
RLAT	=	Repeated Load Axial Test
S.D.	=	Standard deviation
S1, S2......S9	=	System 1, System 2 ..... System 9
S_{20}	=	Stiffness of 20mm basecourse
S_{28}	=	Stiffness of 28mm roadbase
Savings/m	=	Savings per month
Sec	=	Second
S_{HRA}	=	Stiffness of Hot Rolled Asphalt
SHRP	=	Strategic Highway Research Program
Sm	=	Stiffness modulus
SMA	=	Stone Mastic Asphalt
So	=	Overall stiffness
SPDM	=	Shell Pavement Design Manual
Spec	=	Specification
T	=	Time
t	=	Tonne
t/m	=	Tonnes per month
t/yr	=	Tonnes per year
Temp	=	Temperature
TFV	=	Ten-percent Fines Value

UK	=	United Kingdom
VMA	=	Voids in Mineral Aggregate
Vol	=	Volume
vs	=	Versus
W	=	Load
WT	=	Wheel tracker
wt	=	Weight
ΔL	=	Change in Length
ε_x	=	Horizontal tensile strain
δ	=	Horizontal stress (kPa)

CHAPTER 1 - INTRODUCTION

1.1 General

Asphalt has been used as a construction material from the earliest days of civilisation, but its early use was as a waterproofing material in shipbuilding and hydraulic components, its use in road building was more recent (Roberts, Mohammad and Wang 2002).

The first roads were probably constructed over animal tracks and the only features on the route were markers to avoid marshes and other hospitable lands. The roads of the time tended to hold to high ground, such as on the Downs in the United Kingdom, to allow the traveller clear vision and, hence, safety.

The earliest roads were paved with bricks and stones in urban areas and corduroy or logs in soft ground conditions. Brick and stone paving seem to have started in the Middle East, with binders a bonding material. An existing example is the processional roads in Babylon that was constructed 620BC (Nicholls 1997).

Bituminous mixtures used in roads nowadays were evolved from dry stone mixtures developed by two pioneers Telford and Macadam. These inventors introduced individually dry bound mixtures for pavements, which were subsequently sprayed with a sealing tar blend to bind the aggregates together and provide a medium with good water proofing properties. These composite mixtures relied on stone interlocking for their strength. However, the advent of motor vehicles highlighted the weaknesses in their performance (Hunter 1994).

Other asphalt mixtures were developed which gave impermeable surfacings that would not produce dust and would also resist permanent deformation. The availability of Trinidad Lake asphalt resulted in producing asphalt mixtures for general use on city streets. The addition of a proportion of larger sized aggregate to asphalt mortar resulted in the concept of rolled asphalt, with the predominant binder changing through pitch bitumen to petroleum bitumen (Nicholls 1997).

The motor car quickly became a popular mode of transport and as their numbers increased, the damage to road pavements worsened. Higher speeds and the introduction of the pneumatic tyres broke up the surface of water bound macadam and tar covered roads.

In 1909, the UK parliament introduced the Road Board to advise Local Authorities on the best way to repave and repair damaged roads and the government provided grants to carry out this task. The treatment suggested for rural roads was to scarify (recycle) the existing water bound macadam layer, add more stone where necessary to give the road shape and to spray it with hot coal tar (Doncaster College 1999). It can be concluded that recycling of construction materials is not a new concept and has been around for many years more than what people customarily believed.

More recently, increased awareness of the environment and, in particular, the concern over guaranteeing sustainable development, and the pressing need to organise waste management have all contributed to enhancing the image of recycling as an important and even priority instrument for solving the problem of waste materials. Environmental awareness has also awakened recognition of the possible economic value of recyclable wastes through recycling into secondary raw materials. At the same time, a growth in demand for recycled products is likely to facilitate the exploitation of economies of the scale when producing these goods upstream, thus opening new prospects for the competitive development of the recycling industries and, in this way, for the promotion of the recycled products. The implementation of recycling in the context of an environmental policy has given rise to situations where the activity of recycling is not profitable unless some direct or indirect intervention takes place. The basic question is therefore as follows: is it possible for these policies to be reached with a recycling industry operating according to market rules? It can be stated that, if markets function correctly and in conditions of maximum efficiency and minimal costs, recycling may become profitable in an increasing number of cases (European Commission 1998).

To move towards sustainable development the amount of waste produced from industry in general must be reduced and the amount of waste recovered should be increased. Immediate challenges are to stabilise the quantity of waste generated and to tackle the "throw away" attitude. Sustainable development allows us to ensure that whatever waste recovery and disposal practices we engage in, does not compromise future generation's ability to meet their needs. In simple terms, we must deal with our wastes in our time and ensure that any sites used for recovery or disposal of waste are properly managed, comprehensively monitored and responsibly restored so that future generations are not lumbered with the economic and environmental cost of cleaning up after us (Environmental Protection Agency (EPA) Ireland1998).

Engineers need to find new products and materials. They must not just rely on traditional routes, but need ingenuity to develop new ideas to use and reuse resources in different ways, and to look for new solutions to old problems.

The world is at an environmental crossroads and to survive the whole approach to the use of our finite resources must be changed. Engineers made such incredible contributions to health and safety in the 19th century by ensuring supplies of clean water and providing proper sewage systems, which has done more for human health than almost any other single event. Engineers need to be at the fore in relation to environmental concerns and treat these issues with as equal importance as health and safety issues. They should lead the way in developing new materials and ways of reusing existing ones. They should be innovative, and actively involved in promoting the concept of sustainable development (The Structural Engineer 1999).

Since the reservoir of raw materials is finite, the flow of substances through the various stages of processing, consumption and use should be so managed as to facilitate or encourage optimum reuse and recycling, thereby avoiding wastage and preventing depletion of the natural resource stock (EPA Ireland 1998).

Recycling of bituminous mixtures has been carried out for several years but most of this material has been used as a lower class material such as a fill or blinding material in the construction of new roads and other paved areas.

Several investigations were carried out into the use of RAP. Most of these investigations concentrated on cold recycling and hot insitu recycling. Very little work has been carried out in offsite mixing plants.

The current investigation looks at using crushed RAP as an aggregate in the production of three different bituminous mixtures. A representative sample of RAP was taken from 8000 tonne stockpile and passed through a jaw crusher to produce a workable size of aggregate to produce bituminous mixtures.

Before any testing of bituminous mixtures could commence, the properties of the RAP material were explored. Grading and binder content tests were carried out to determine the composition of the RAP material. Mix designs were produced to ensure that the grading of the bituminous mixture was uniform throughout the investigation.

New tests as specified by British and European Standards were conducted to determine the effects of using RAP material in bituminous mixtures. Tests were carried out using different supplies of RAP and comparisons were made. The penetration of the binder was also changed to study its effects on the end results.

The main objectives and scope of this study are outlined as follows:

1. Review current knowledge of road building, recycling of Construction and Demolition Wastes (C&DW) and recycling of bituminous mixtures.
2. Determine the physical and mechanical properties of the constituent mixtures.
3. Examine the effects of using RAP in bituminous mixtures in terms of stability, flow, stiffness, fatigue, % wear and rutting values. .
4. Examine the effects of using harder penetration grade binder in mixtures containing different percentages of RAP.
5. Examine the effects of changing the source and method of crushing RAP in bituminous mixtures.
6. Effects of using RAP material in designing roads using the Shell pavement design package.
7. Economically determine the benefits/disadvantages of using RAP material in the production of bituminous mixtures.
8. Appraise the results to determine the advantages/disadvantages of using RAP in the production of bituminous mixtures.

The structure of the thesis can be summarised as follows:

Chapter 1 Introduction, scope and aim of current investigation.

Chapter 2 Literature review.

Section 1 History of road construction and design from ancient civilisations, middle ages and modern day.

Section 2 Recycling of C&DW in terms of economic, environmental aspects and various recycling methods.

Section 3 Literature review of the various methods of recycling bituminous mixtures, from in-situ recycling, to in-plant and the quality control measures needed to produce RAP aggregate.

Chapter 3 Tests commonly used to assess the quality and performance of bituminous mixtures.

Section 1 Description of the aggregate and bitumen tests used to determine the mechanical and physical properties.

Section 2 Review of the bituminous tests used to evaluate the various properties of bituminous mixtures in terms of stability, flow, stiffness, fatigue, % wear and wheel tracking tests.

Chapter 4 Experimental work

Section 1 Jaw and Cone crusher methods used to crush the recycled asphalt pavement.

Section 2 Experimental work used to determine amounts of different materials in each mix, mix density and methods of mixing the materials.

Section 3 Experimental and theoretical investigation to determine the properties of 30% Hot Rolled Asphalt (HRA), which were produced with various % of RAP aggregate.

Section 4 Investigation to determine the properties of basecourse which were produced with various % of RAP aggregate.

Section 5 Investigation to determine the properties of roadbase which were produced with various % of RAP aggregate.

Section 6 Investigation to determine the effects of adding cold RAP to superheated aggregate and the effects of using 50pen binder into roadbase.

Chapter 5 Study of the increased stiffness of asphalt as a result of adding RAP to basecourse and roadbase using the Shell Pavement Design Manual (SPDM 3.0).

Chapter 6 Investigation into the economics of recycling from the purchasing of the required plant, to the saving as a result of using RAP. The financial benefits of the reduced layer thickness are addressed in this chapter.

Chapter 7 General summary and conclusions with recommendations for further research.

CHAPTER 2 - LITERATURE REVIEW

SECTION 1: History of Road Construction

2.1 Ancient Civilisations and Road Construction

Ancient civilisations constructed fabulous buildings and monuments still standing to allow us witness their culture and achievements. Roads were also constructed by many of these civilisations in order to travel, trade, move armies etc. This section briefly reviews some of the known roads built through history by some of the well-known civilisations (Packer, Tenney and White 1980).

2.1.1 Egyptian Roads

Prior to the invention of the wheel, about 5000 years ago, people and materials were transported overland on foot or by animals. The Egyptians transported the heavy stones used to build the great pyramids along a series of log rollers. This reduced the frictional resistance between the stone and the ground as well as reducing the load intensity.

As inventors of the wheel, the Egyptians were the first to experience the constraints of its use. They found that it took less man power to move materials using the wheel and also found that the wheel removed the necessity of repositioning the logs rollers during the transportation process. Later they realised the need to create a hard surface on which to run their vehicles as a result of the reduced contact area and consequently were the first to construct a road pavement. It was 1km long and took 10 years to build. A section through a typical Egyptian pavement can be seen in Fig. 2.1.

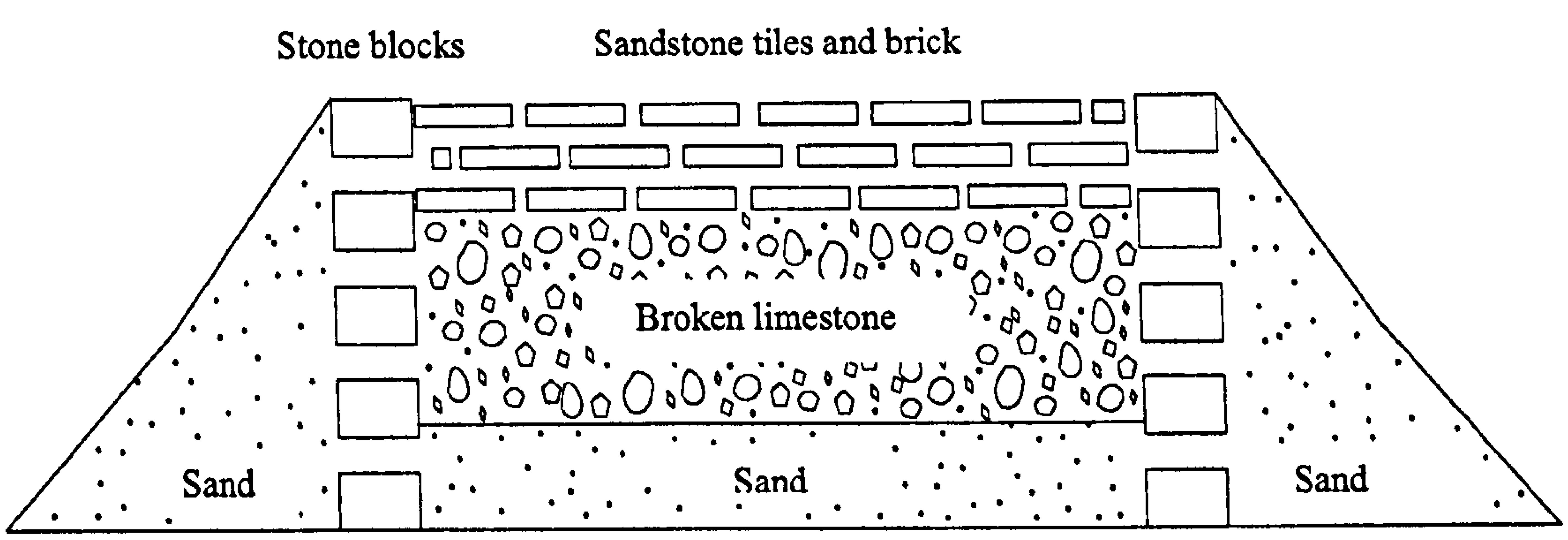


Fig. 2.1 - Egyptian road (3000BC)

2.1.2 Babylon royal processional route

The processional route was built in 620BC; it is the first recorded use of bitumen in a road pavement. The use of bitumen was for holding stone slabs in position. The road pavement was over 1Km long and the width varied between 10 and 20m (Doncaster College (A) 1999).

2.1.3 Persia royal road

The first road built over a significant distance is that of King Darius I of Persia. It was built about 500BC over a period of about 60 years. One of its great features was the use of a straight line as the shortest distance between two points. It also included the first recorded construction of town bypasses. This was something that the Romans were to exploit to great effect later (Packer, Tenney and White 1980).

2.1.4 India chandragupta's road

This was another great road building project of its time. It was built around 400BC, was 2400km long and took 30 years to build. It contained many features regarded nowadays as 20th century creations. Among these was the creation of a transport ministry responsible for construction and maintenance, milestones, refreshment centres and rest houses. The main purpose for the road was to improve trade and facilitate speedy troop movement's (Doncaster College (A) 1999).

2.1.5 Italy Roman roads

The Romans were prodigious road builders. They spent five centuries completing a road system that extended to every corner of their empire and eventually covered a distance equal to ten times the circumference of the Earth at the equator. This included over 80,000km of first class roads and 320,000km of smaller roads.

The Appian Way, which was the forerunner of many other Roman roads on three continents, began in 312BC as a road for use in the Saint Wars. Ultimately, the Appian Way reached southward 576km from Rome to Brundisium. The road system was gradually extended through efforts of numerous Roman Emperors. Along its route tunnels were built the longest of which is 0.75km. Some Roman roads have been used throughout the Middle Ages and into modern times. The Appian Way, on which Paul travelled to Rome, is still an important artery of western Italy. It is a mute reminder of the glory of the time when all roads lead to Rome (Packer, Tenney and White 1980). Fig. 2.2 shows a cross section of a Roman road.

Following the fall of the Roman Empire, the importance of cities diminished and, hence the need for roads reduced, as well as the loss of an organisation capable of constructing and maintaining them. Therefore, there was little true road building during the Middle Ages until the industrial revolution and the general movement away from agricultural economy (Nicholls 1997).

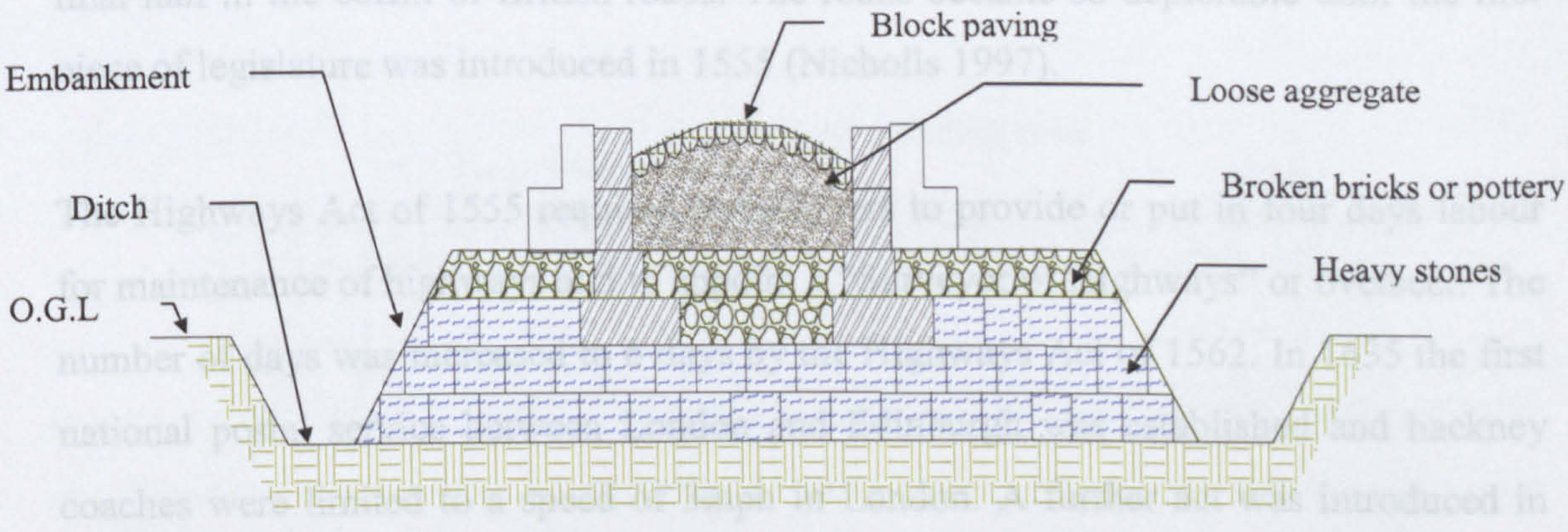


Fig. 2.2 - Cross section of a Roman road

2.2 History of British Roads Construction

The earliest roads constructed in Britain are dated back to 2500BC and were built along ridges in the chalk hills of the South Downs. By far the biggest contributors to the early British road building were the Romans. They built almost 78000km of roads across Europe and in the 150 years (around 250BC) of occupying Britain they built 5000km of road networks. After the Romans left, Britain was divided into separate kingdoms and it became unsafe to travel very far from home. There were no requirements to either build new or maintain the existing ones.

2.2.1 Middle age Britain

Mediaeval Britain was much more stable and travel, mainly for trading purposes, increased. Unfortunately, many of the Roman roads had been plundered for building materials and were left in a poor state of repair. Others were no more than muddy tracks in winter, which became heavily rutted and difficult to travel in the summer. They became virtually impassable to use in winter resulting in people building up provisions to last them through the winter months. As spring arrived, stocks were replenished and this was the origin of the spring and summer fairs which still exist in many parts of the Britain.

Roads in small towns, hamlets and villages were repaired locally under the guidance of the Church. The amount of maintenance was very small and crude, amounting to no more than filling in potholes. Even this small amount of maintenance stopped when Henry VIII announced the dissolution of the monasteries in 1540. This proved to be the final nail in the coffin of British roads. The roads became so deplorable until the first piece of legislature was introduced in 1555 (Nicholls 1997).

The Highways Act of 1555 required parishioners to provide or put in four days labour for maintenance of highways and to appoint a "Surveyor of Highways" or overseer. The number of days was increased to 6 days by the Highways Act of 1562. In 1635 the first national postal service between London and Edinburgh was established and hackney coaches were limited to a speed of 3mph in London. A further act was introduced in 1691 regarding the repair of highways and the control of charges for carrying goods and authorised the local justices to appoint "Overseers" at special "Highways Sessions" (Education Resources 2002).

2.2.2 The turnpike trusts of 1706

In 1706 an Act of Parliament introduced in excess of 1100 Turnpike Trusts, each responsible for the maintenance and improvement of highways in their charge. Over £7 million was allocated for the setting up of the trusts, which, were to raise money required for maintenance through a toll system. An additional £1.5 million was given towards preliminary maintenance work (Nicholls 1997).

Turnpike Trusts were made up of a group of people who would get together and ask the permission from parliament to take over a section of a road, or build a new road and maintain it for 21 years. They would pay for its maintenance by collecting tolls from people who used the road. These roads were commonly called Turnpike Roads. Many of the old roads that existed at the time were improved. The Trust also experimented with new ways to build roads, adding new methods of making roads stronger and last longer so that wheeled traffic could travel more easily (Highways Agency 2002). In 1749 the first stage coach service between Edinburgh and Glasgow began, and took 12 hours eachway (Education Resources 2002).

2.2.3 British road-building pioneers

The science and expertise of road building was almost completely lost from Roman times until the advent of Telford and MacAdam in the mid-eighteenth century. Much has been written about the types of construction used by each of these two great engineering pioneers. Figs 2.3 and 2.4 illustrate their techniques.

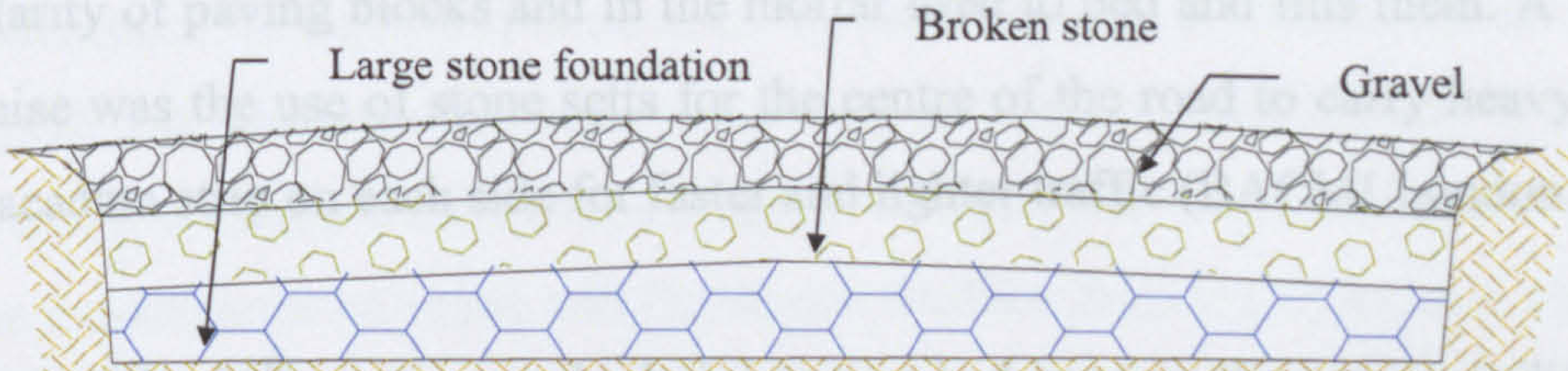


Fig. 2.3 - Typical Thomas Telford construction

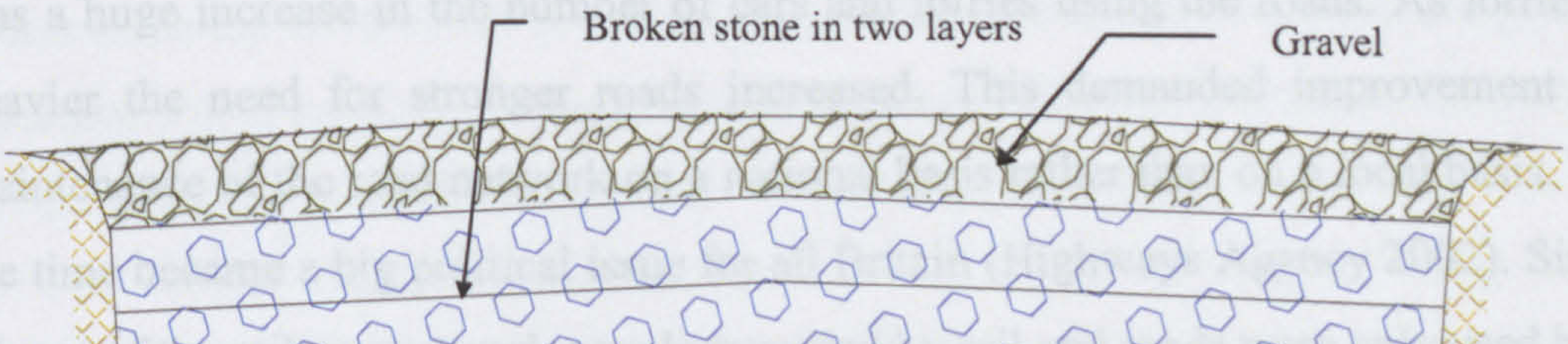


Fig. 2.4 - Typical MacAdam construction

Both engineers understood the basic principles necessary to ensure durability and ability to carry traffic in all weathers. They recognised the need for a good drainage system, stability of the sub-soil, layered construction, good compaction to ensure interlock of crushed rock particles and control of aggregate grading and its cleanliness. Perhaps most important of all, they provided a cambered surface consisting of small broken stones which would further break and compact under iron-shod wheels to form a strong and impervious surface layer. The essential difference between their methods lay in their approach to economics. Telford built thick to provide a road to last longer. MacAdam believed that provided the surface layer was rendered impervious by attrition and compacting therefore foundations could be dispensed with.

The rapid growth in railways since 1840 had linked all the major cities in Britain and revolutionised passenger and freight transport. In 1862 London to Edinburgh by rail usually took 10½ hours; this was reduced to 8½ in 1888 (Education Resources 2002).

With the rapid advances of the railways, the impetus went out of road building on a large scale and the attention turned to city streets. Here the improvement was to the size and regularity of paving blocks and in the mortar used to bed and fills them. A cheaper compromise was the use of stone setts for the centre of the road to carry heavy traffic, with a macadam strip on each side for faster and lighter traffic (BACMI London 1992).

By the end of the 19th century road administration had become the responsibility of the cities, towns and rural districts councils. At the beginning of the 20th century there were no fewer than 1900 local authorities concerned with highways and a national policy for roads did not exist. The motorcar had arrived in the late 1800's and in the 1900's there was a huge increase in the number of cars and lorries using the roads. As lorries were heavier the need for stronger roads increased. This demanded improvement in the maintenance of the road network on a national basis rather than on a local basis. This at the time became a big political issue for all Britain (Highways Agency 2002). Since the advent of the railway network people travelled by rail and roads were only used by local people. The motor car opened the countryside and returned vehicles to rural roads, which had been neglected for over a hundred years (Nicholls 1997).

2.3 Developments in Road Pavements

Road pavements passed different stages in their developments since the start of the 20th century. The introduction of standards for pavement construction along with the introduction of mechanical means of producing and laying materials helped develop the modern road network. The increased traffic, rise in construction costs and environmental impacts, have led to the introduction of performance specification and development of mixtures which reduce environmental impacts. This section briefly reviews some of the developments in road pavement construction over the last century.

2.3.1 Asphalt pavements

First used in Paris in 1854, they were made from natural rock asphalt. The rock was limestone impregnated with binder (traditionally called bitumen). The material was imported into Britain and used in London in 1870 and persisted until the 1930's. It was very expensive material and was consequently utilised at prestigious sites only (Nicholls 1997).

2.3.2 Tar covered pavements

After unsuccessful attempts to reduce dust by modifications to the motor car, it was soon realised that the only satisfactory way was to treat the road surface. Use of tar followed initial attempts with water, oil and even lime. Tar was then introduced as a binder for maintenance of roads and highways. Surface dressing of tar and chip, was first introduced as a dust prevention measure. The use of tar and chip coating to prevent dust did improve the quality of roads, but aggregate coated with tar and laid as a compacted layer known as tar macadam provided greater improvement to roads. Bituminous mixes, in the form of asphalt "sheet sand", had been used in some London streets since the 1870's, but it was the more wide spread use of the coarser-graded tar macadam which gradually change the whole nature of Britain's roads at the start of the 20^{th} century (BACMI London 1992).

2.3.3 Concrete roads

The first concrete roads were laid in Edinburgh in 1865. It was a dry mix concrete with very little water in it and was compacted by steamroller. They proved to be unsuccessful, breaking up due to frost action, and were abandoned. Improved materials and laying techniques saw their reintroduction in 1920. It was clear that the motor car was here to stay and something had to be done about the state of the roads (Doncaster College (B) 1999).

The road board set up by the British Government in 1909, increased vehicle licence duty and raised a fuel tax to provide funds for road improvements. In 1919-20 the amount credited to the Road Improvement Fund was £1.6 million. By 1913 the cost of road maintenance and reconstruction was £17.5 million or 43p per head of population. In 1919 the road board was absorbed into the newly created Ministry of Transport. There followed a period of high activity and the road network was increasingly surfaced with tarred mixtures of various forms, until the Wall Street crash in 1929 (BACMI 1992).

2.4 Early Methods Used for Producing Pavement Mixtures

Early 'tarmac' used was hand mixed cold aggregate and hot tar on fire-heated steel plates and was hand laid. The only concession to mechanisation was in the laying of the mixes, with the use of newly introduced steam roller for compaction, as road builders were quick to realise the need to compact well to ensure a good riding surface and long life. Hand mixing methods could not cope with the ever-increasing demand for the mixes. Mechanical mixers were introduced and although small by current standards, at half a tonne a batch or less, they could produce mixes more quickly and uniformly than hand mixing. From early day basic differences between coated macadam and asphalt had a notable influence on the type of plant designed to produce them. There was also a difference in ownership of the plants producing the two mixtures. Quarry owners who had previously sold dry stone for roads to contractors and local authorities, primarily undertook manufacture of coated macadams. They installed permanent mixing plants in their quarries. Hot asphalt was manufactured and laid by specialist contractors, many of whom had interest in natural deposits of asphalt. They purchased their aggregates and operated mobile plant that could be moved from job to job. This was the essential difference, along with the introduction of binder for asphalt production, until the advent of the motorway building programme in the early 60's.

2.5 Development of Asphalt and MacAdam

Over the years there was a gradual replacement of tar binder with bitumen as the supply of tar reduced and as the better performance characteristics of bitumen, resulting from its lower temperature susceptibility and greater resistance to oxidation, became recognised (BACMI 1992).

The first national specification for coated asphalt and macadam was issued in the early 1940's and roads using this material were very common in road construction during the

2nd World War. These specifications were issued as note by the then Road Research Laboratory. In 1945 a series of British Standards for tar macadam were published followed by standards for Bitumen MacAdam and Fine Cold Asphalt in 1950. Research at the Governments owned Road Research Laboratory led to the introduction, in the early 1960's, of dense coated macadams for the main bound layers of road pavements.

Bituminous mixtures are used in construction of almost all the flexible layers of a road. Asphalt is favourable for use in the layers due to its high binder content, whereas macadam is selected for the upper layers for heavily trafficked roads, which demand improved resistance to permanent deformation (rutting) (Fig. 2.5). The characteristics of asphalt and macadam road layers are:

- Better load spreading properties than in other layers.
- They can be quickly and accurately spread by machinery.
- Strength is available for use as soon as the material is cooled down.
- Impervious to water and not affected by frost.
- Cost is greater than uncoated materials.

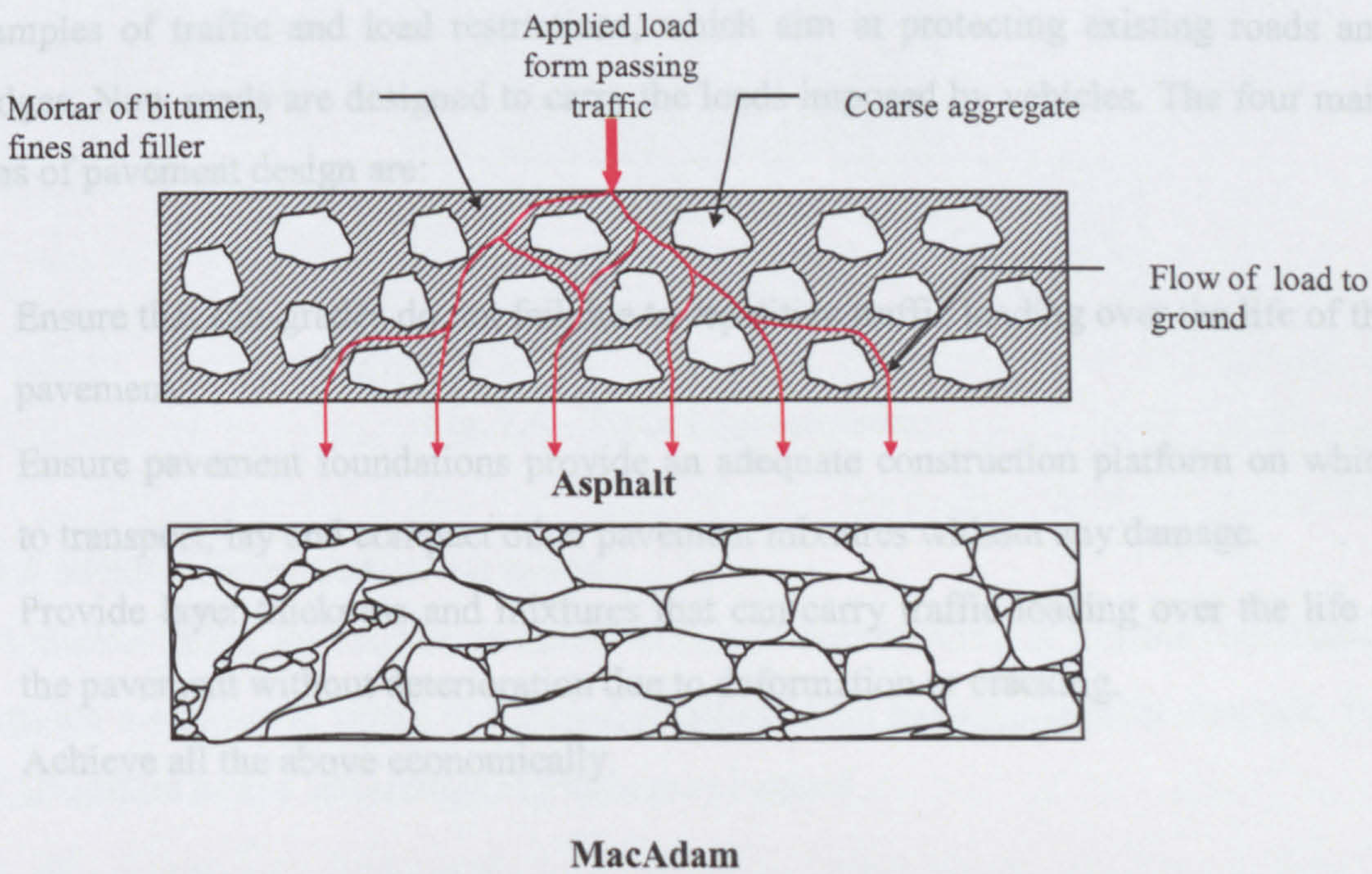


Fig. 2.5 - Schematic representation of asphalt and macadam

In recent years, communications have fostered a greater interchange of ideas between countries, as well as increase in the traffic levels imposing greater stresses on pavements, so that there is perceived benefit in developing further material types and then exporting them. These include Stone Mastic Asphalt (SMA) and the thin surface course systems. Further developments have occurred with pressures to reduce the environmental impacts of roads in term of less traffic noise and/or water spray, which can be achieved using mixtures such as porous asphalt. Similarly, there are different methods of managing the road network in Europe and elsewhere. Different bodies have different ideas as to what characteristics are required on their roads. Harmonisation through Europe may reduce the variation, but the increased number of different surfacing mixtures and treatments will allow clients to be more precise when specifying what they require (Doncaster College (B) 1999).

2.6 Modern Road Pavements Design

A pavement is a load bearing, multi-layered structure constructed so that the imposed surface load is transmitted and spread onto a natural, or formed, foundation in such a way that neither it, nor the structure itself, is overstressed or permanently deformed during a reasonable life span. For many years pavements were not designed to suit the requirements of traffic. The traffic was controlled to suit the road. There are many examples of traffic and load restrictions, which aim at protecting existing roads and bridges. Now roads are designed to carry the loads imposed by vehicles. The four main aims of pavement design are:

- Ensure that sub-grades do not fail due to repetitive traffic loading over the life of the pavement.
- Ensure pavement foundations provide an adequate construction platform on which to transport, lay and compact other pavement mixtures without any damage.
- Provide layer thickness and mixtures that can carry traffic loading over the life of the pavement without deterioration due to deformation or cracking.
- Achieve all the above economically.

The design process begins by deciding what type of pavement is required and what methodology is going to be applied in determining layer thicknesses (Doncaster College

(C) 1999). There are two main types of pavement construction, rigid and flexible. Over 90% of UK roads are flexible. Many rigid pavements are subsequently upgraded, repaired or have their lives extended by overlaying with flexible bituminous layer. There is a move in the UK and other countries towards flexible composite structures for heavily trafficked roads (Doncaster College (C) 1999).

2.6.1 Rigid pavement construction

Rigid construction is the term used for a pavement where both the roadbase and surfacing layers are constructed with concrete. The structural strength concrete is commonly referred to as Pavement Quality Concrete (PQC) (Doncaster College (D) 1999). On very heavily trafficked roads, including a number of motorways in the UK, continuously reinforced concrete roadbase has been used to avoid the undesirable effects of necessity joints (Hunter 1994). Fig. 2.6 shows a typical rigid pavement.

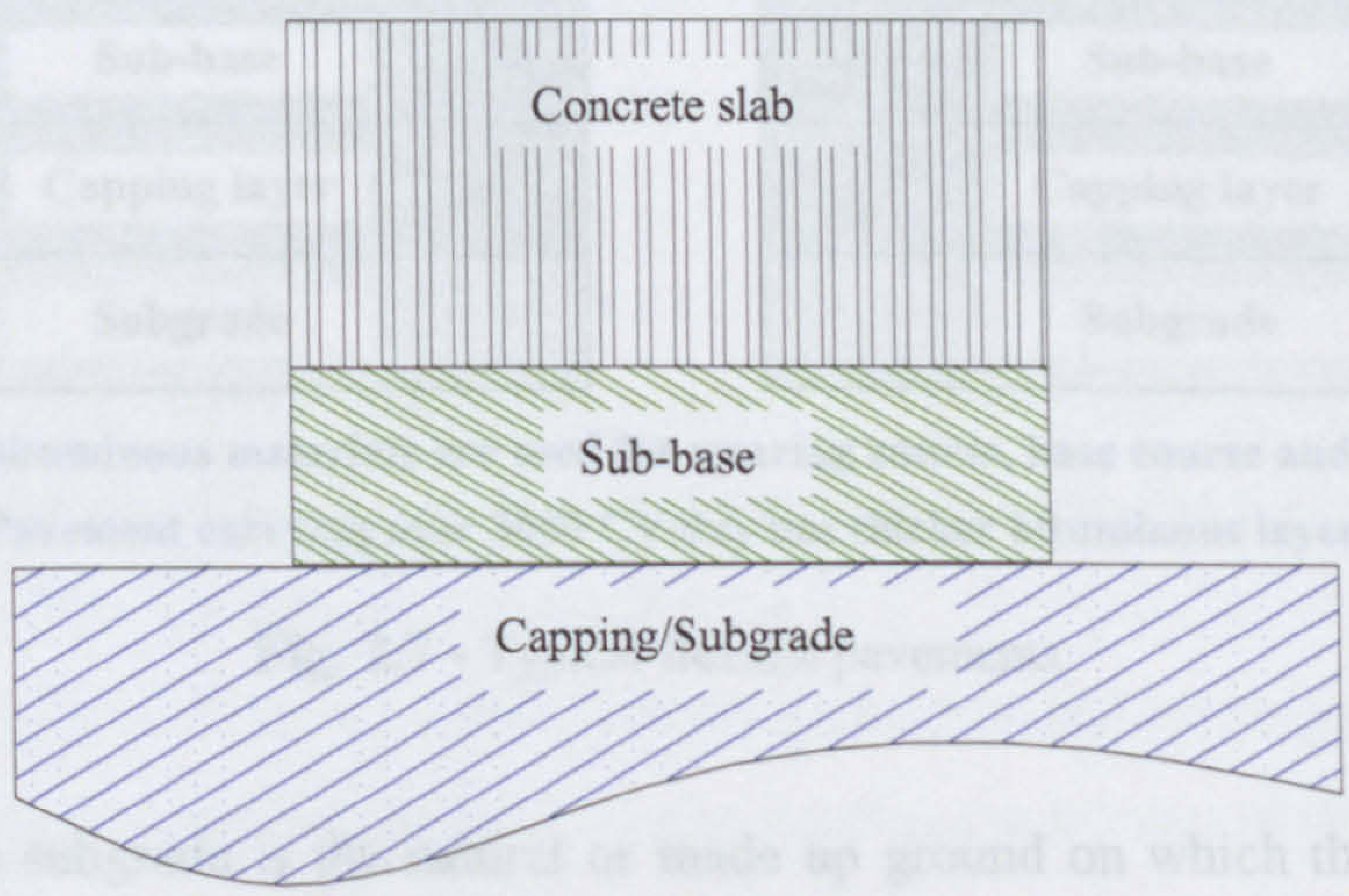


Fig. 2.6 - Typical rigid pavement

2.6.2 Flexible pavement construction

Bituminous mixes are used in so called "flexible" construction. In flexible construction there are a number of layers to the road structure, each having a specific function. Fig. 2.7 illustrates where bituminous mixtures may be used.

The nature of the bituminous mixtures will vary according to their position and function in the road structure. The wearing course and basecourse may be asphalts but the properties required of the wearing course at the surface are different from those required just below the surface in the basecourse. Wearing course asphalts differs from basecourse asphalts. Also particular types of material are selected according to their suitability. When traffic loads are very high, an asphalt roadbase may not provide sufficient resistance to deformation, but would give the necessary resistance to fatigue cracking. Macadam is used at the bottom of the roadbase where tensile stresses inducing cracking are greatest whereas asphalt is used for the upper part to provide improved resistance to deformation (Byrne 1999). The layers in a flexible pavement and their function can be summarised as follows:

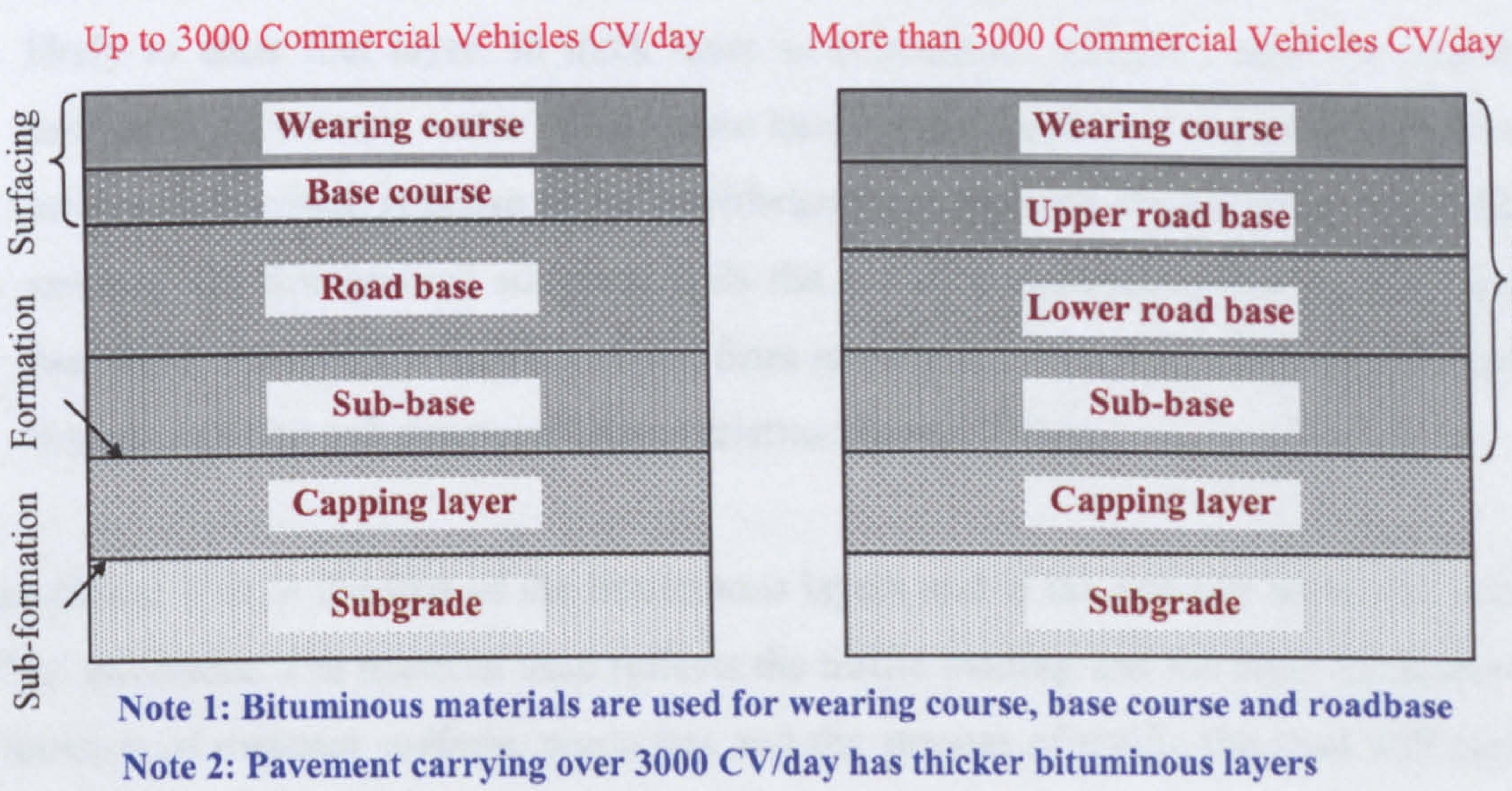


Fig. 2.7 - Typical flexible pavements

Subgrade: The subgrade is the natural or made up ground on which the pavement is constructed. On reconstructed ground it is usually well compacted by traffic, whereas on new roads it is carefully shaped and compacted to the appropriate level and profile. The surface of the subgrade is known as the formation.

Capping layer: A capping layer is sometimes laid over a weak subgrade to act as a sub grade improvement layer. This is usually a relatively low quality, cheap, locally available aggregate. With some soils, subgrade improvement can be achieved by treating the surface with lime or cement. In either case the aim is to ensure adequate support for plant used to lay the sub-base (BACMI 1992).

Sub-base: The sub-base is a layer of graded granular material which provides extra strength to the subgrade and assists in its drainage and general protection. The four main functions of the sub-base are:

- Provides a structural layer to distribute load to the subgrade. This layer is particularly important where the top bituminous layers are thin.
- Provides a working platform for construction traffic and a compaction platform for the subsequent laying of the bituminous mixtures.
- Acts as an insulating layer in conjunction with the bituminous mixtures to protect the subgrade from frost.
- May be used as a drainage layer to remove water from the pavement. The extent to which a sub-base material should be free draining depends on the amount of water likely to enter that layer. In thick layer of bituminous mixtures used for heavily trafficked pavements, water volumes are likely to be small. It is therefore crucial to have a layer which is dense and can withstand and transmit construction and traffic stresses. On fine-grained subgrade soils the use of a geotextile filter membrane is beneficial to prevent migration of soil fines into the sub-base material, destroying its drainage ability and structural characteristics (Hunter 1994).

Roadbase: This is the first of the bituminous layers and is the primary structural layer of the pavement. The material used reflects the traffic loading and the layer thickness is a function of material stiffness properties and the amount of traffic the road will carry during its service life. The functions of the layer is to:

- Reduce the vertical stresses on the pavement foundation and subgrade.
- Reduce flexural stresses in the surfacing by limiting the amount of deflection under load.
- Provide an accurate surface on which to lay basecourse material.

The roadbase must achieve all the above without cracking prematurely.

Basecourse: The basecourse is another layer made of a bituminous material. It is slightly denser, and has a smaller nominal size of aggregate to allow more accuracy and to finer tolerances than roadbase. The basecourse is responsible for creating smooth riding quality in the pavement.

Wearing course: Together with the basecourse, this layer makes up the road surfacing. In traditional constructions it has several functions. These are:

- Seal the surface against the ingress of water.
- Provides skidding resistance.
- Shed water into surface water drainage systems.
- Provide a quite running surface.
- Give good aesthetic qualities to the road such as colour.
- Carry road markings to aid safe usage by traffic.

New mixtures are now available that are pervious and allow the water through to an impermeable basecourse. These reduce water spray; reduce accidents caused by ponding or freezing water (Doncaster College (D) 1999).

2.7 The Future of Road Construction

The transport industry at the beginning of the new century faces some formidable challenges. Unlike the 19th century when the canals gave way to the railway or the 20th century where the railways gave way to the motorways, there seems to be no major modal development on the foreseeable horizon.

Whilst in recent times the railfreight industry in the UK has been undergoing a renaissance In the future it will be totally incapable of taking large quantities of freight transferred from different modes particularly road. In this respect it is interesting to note that the Republic of Ireland, despite its fairly thin railway network actually carries a higher percentage of total freight moved on its rail network (8%) than the UK (6%) (Institute of Asphalt Technology (IAT) 2000). Fig. 2.8 shows the percentage of goods transported by the different modes of transport in the EU in 1996 (European Union Road Federation (EURF) 2001).

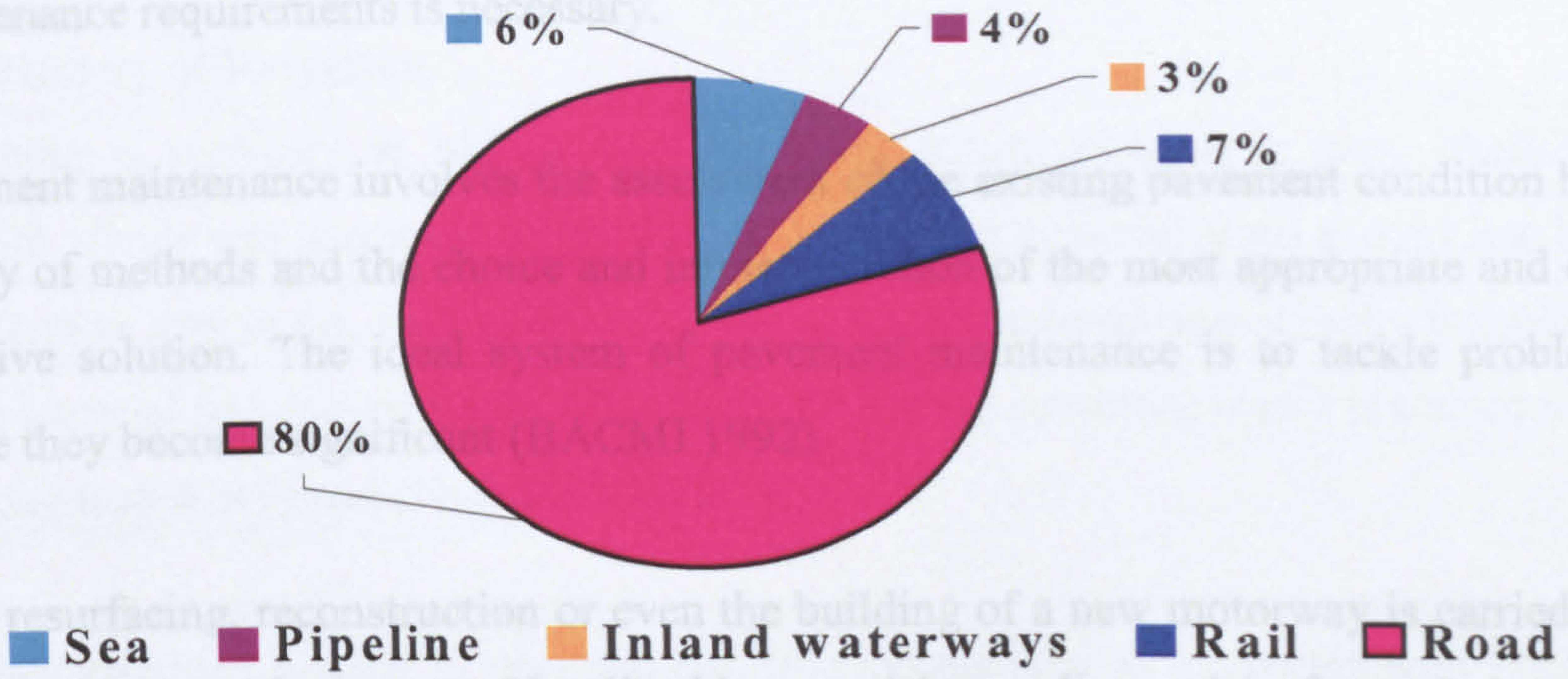


Fig. 2.8 – Percentage goods transported by mode of transport 1996

From Fig. 2.8 it can be seen that road transport is the main mode of transport within the EU. In the Republic of Ireland and the United Kingdom these figures are higher than the average with the percentage of goods moved by road standing at 92% and 83% respectively. As the reliance on road transport is so great, a good road network is required to keep traffic moving throughout the EU. For places on the peripheral areas of Europe i.e. Ireland, Scotland etc the roads are becoming more and more the arteries to survival as they provide the essential links to major markets. The introduction of the Euro has done away with another obstacle to conducting business within the EU.

Infrastructure is currently been built in member states to develop a European corridor, which will make trading between the different states more competitive. Along with the building of new motorways, old roads have to be resurfaced and in some cases reconstructed.

2.8 Road Maintenance

Road mixtures deteriorate progressively with time and require routine maintenance to ensure they continue to perform satisfactorily for as long as possible. For minor paved areas this is likely to involve only removal of detritus and a resurfacing or surface treatment when the existing surface which shows a sign of deterioration, but for regularly trafficked areas such as roads, routine inspection and assessment of maintenance requirements is necessary.

Pavement maintenance involves the assessment of the existing pavement condition by a variety of methods and the choice and implementation of the most appropriate and cost effective solution. The ideal system of pavement maintenance is to tackle problems before they become significant (BACMI 1992).

Once resurfacing, reconstruction or even the building of a new motorway is carried out there are going to be wastes. Usually this material was disposed in farmer's lanes or used to fill holes in land etc. As we enter a new millennium the question of sustainable development is coming more to the fore. This in turn has put the spotlight on waste management and the recycling of renewable resources.

SECTION 2: Recycling of Construction and Demolition Wastes (C&DW)

2.9 History of Recycling

Recycling is probably as old as road pavement construction. It is most likely that material which was supposed to be thrown away after removal from a road has been used to pave new areas, such as drives or car parks, for enterprising or lucky individuals as the material became available. This kind of informal recycling is still practised today and long may it continue (Nicholls 1997).

The first recorded mixing of recycled construction material as aggregate with Portland cement was in Germany in 1860 for the manufacture of concrete products. Systematic investigations have been carried out since then into the possibilities of using crushed concrete and masonry rubble as an aggregate in concrete. Most of this work however was carried out in the 1940 and 1950's using the type of materials, which were available at the time. There is very little knowledge of how modern recycled construction materials with impurities perform when crushed to aggregates and used to produce new concrete and asphalt (Khalaf 2000).

A limited amount of re-use of old bituminous surfacings has been carried out in the UK for many years. Material removed from roads prior to resurfacing has been used as fill material, as well as for regulating and blinding purposes in the construction of paved areas for light traffic use (BACMI 1992).

Although it is not recognised, asphalt is the most recycled product in several countries. In the Netherlands up to 50% or more of reclaimed asphalt is used in new asphalt mixtures. One reason for asphalt recycling is the shortage of mineral aggregate resources in the Netherlands. The other reason is that there is no space to landfill the materials due to the population density (Shell Bitumen 2001).

The recycling of old asphalt pavements has significant economic benefits because binder represents more than 30% of the cost of a new pavement and in times when oil prices are rising, so is the cost of binder. Old asphalt that is not recycled gets used in a lower value pavement application or as a granular base, or goes to landfill (Institute of Asphalt Technology (IAT) 2002).

Recycling is very much a key part of the aggregates industry in the UK. Over the past 10 years it has invested heavily in recycling operations. The recycling sector is now producing an estimated 45-50 million tonnes a year. This represents 19% of the total aggregate market. The most widely used recycled and secondary aggregates are from demolition wastes and road planings, which are generated in urban areas where major construction markets exist (Quarry Products Association (QPA) 2001).

2.10 Environmental Aspects

The environmental impacts of aggregate extraction are a source of increasing concern in many parts of the UK. These impacts have been itemised as loss of mature countryside, visual intrusion, heavy lorry traffic on unsuitable roads, noise dust and blasting vibration.

In the 30-year period to 1991 the production in the UK of newly quarried aggregates increased from 110 million tonnes to a peak of 300 million tonnes in 1988, falling back to about 250 million tonnes by 1991 (Sherwood 1995). This figure reduced further to 215 million tonnes in 1996 (European Commission 1998).

Despite the drop in demand in recent years, the production of quarried aggregates is expected to increase still further. The Department of the Environment has published forecasts, which indicated that by 2011 the annual consumption of aggregates in England and Wales would increase to about 400 million tonnes.

Road building plays a significant role in the demand for aggregates as it accounts for about one-third of the total production. In 1996 approximately 96 million tonnes were used and even if the amounts used by local authorities in road construction are excluded, it is estimated that the current road building plans estimated by Department of Transport in the 1990's will use 510 million tonnes. The implications of this have been considered by the Royal Commission on Environmental Pollution, which reported that: "We are concerned that extensive damage to the environment would be caused through extraction of the aggregates to carry out the present road building programme. We do not consider that the implied rate of consumption can be regarded as sustainable" (Structural Engineer 1999).

Quite apart from depleting the stock of natural resources, the quarrying and processing of primary aggregates involves the generation of obvious environmental and amenity impacts most of them limited to the local area surrounding the quarry. The impacts of quarrying include: noise, dust, air pollution, vibration, pollution to surface or groundwater, visual or aesthetic impacts, changes in land form, changes to natural habitats and possible destruction of historical artefacts.

Some of these impacts are primarily environmental, but others depend on the presence of people and can more correctly be termed amenity impacts. Quarries in remote areas cause fewer and less severe amenity impacts than those in urban or suburban settings. By contrast remote quarries rely on transport links to deliver their aggregates to the final user (Symonds Group 1999).

Aggregates, crushed rocks, sands and gravel are essential to our way of life. They literally underpin our society, providing us with places to live, work, play and much more. Quarrying for aggregates does, however, have environmental implications, requiring a responsible approach from the industry and a considerate attitude to its neighbours (Quarry Products Association (QPA) 2001). Fig. 2.9 shows the break down of land use in the UK.

Recycling of C&DW may avoid some of the above impacts, as shown before, but it could introduce others. Moreover, because both C&DW and the potential locations for their use are likely to occur in more urban settings, therefore any impacts will affect people.

The transport impacts associated with delivering C&DW by roads are essentially similar to those associated with primary aggregates. But if C&DW are processed on site, in such cases the transport impacts are taken out of the equation altogether. Rail transport would suit quarrying more whereas recycling C&DW moves from site to site using heavy lorries. (Symonds Group 1999).

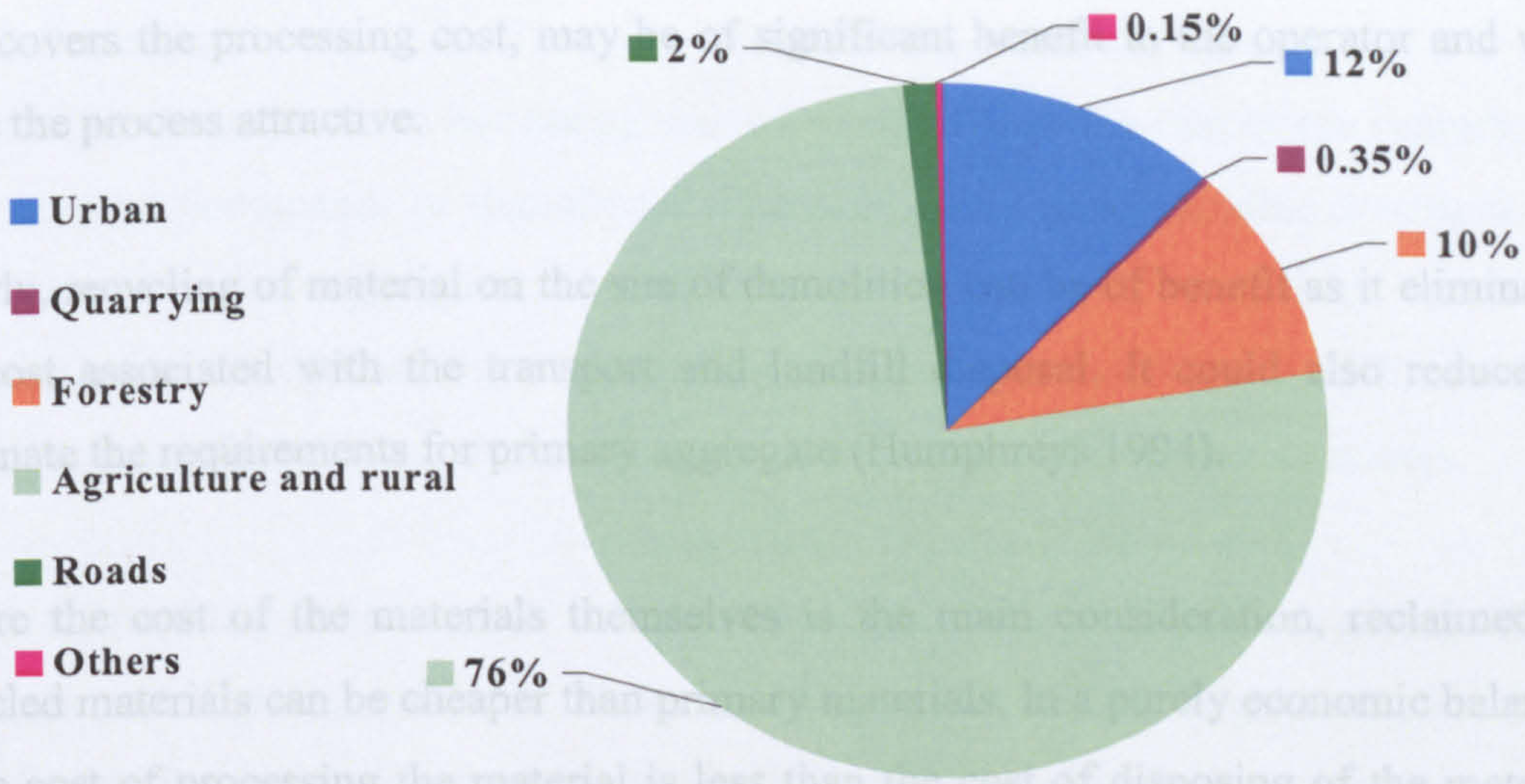


Fig. 2.9 - Land use in the UK

Environmental impact assessments are normally required for quarrying of mineral and rocks. Such documents are designed to ensure that every possible impact is considered and, dealt with. A typical report provides a comprehensive assessment of noise, dust, traffic, water, ecology and landscape. It adds up to a vital safeguard for local communities as well as for the environment (Quarry Products Association (QPA) 2001).

2.11 Economic Aspects

The total costs of recycling C&DW is very specific to the individual projects on which demolition and construction wastes arise. They depend on the programme constraint, requirements for materials on/or near the site, proximity of landfill sites and the cost of transport and disposal. Recycling is selected when this offers the most economic means of disposal. This occurs where there is a positive benefit arising from consideration of the following: site fill requirements, separation of materials, cost of disposal of rejects to landfill site, processing to produce a marketable material, presence of a market for the products, costs of collection, transport and price of primary aggregates.

The relationship between these may be quite complex and the economic viability of recycling may vary depending on the geographic location of the operation and particular contractual relationship between the various parties who are involved in the process and indeed other business interests of operators. In many cases it is a demolition contractor who is preparing waste materials for re-use as an alternative to paying for landfill and transport. In this particular circumstance, sale of the material to a user at a rate, which only covers the processing cost, may be of significant benefit to the operator and will make the process attractive.

Clearly, recycling of material on the site of demolition can be of benefit as it eliminates the cost associated with the transport and landfill disposal. It could also reduce or eliminate the requirements for primary aggregate (Humphreys 1994).

Where the cost of the materials themselves is the main consideration, reclaimed or recycled materials can be cheaper than primary materials. In a purely economic balance, if the cost of processing the material is less than the cost of disposing of the material and producing new material then it is worth recycling that material.

As more reclaimed or recycled materials are used in construction the cost of such materials will be reduced, while sources and production of virgin materials are inevitably going to become fewer and therefore more expensive due to taxation such as an aggregate tax. Processing and manufacturing costs of both primary and recycled materials are also likely to rise because of their environmental impact and the consequent need to provide abatement technology (D.E.T.R 1999). The balance of economics for or against recycling can be small, and specific to individual projects (Humphreys 1994).

2.12 Recycling Construction and Demolition Wastes (C&DW)

The different types of materials that can be found in C&DW are: soil and made up ground, concrete, bricks, stones, bituminous mixtures, architectural features, metals, timber, glass and other materials.

2.12.1 Soil and made ground

Soil and made up ground arise from the stripping of a site before construction begins. The types of materials usually associated with soil and made up ground are topsoil, gravel, rock and sand. However, made up ground may contain significant amounts of unhazardous contaminants like concrete, bricks, glass, ceramics, pottery, etc or hazardous materials like asbestos (Humphreys1994).

2.12.2 Concrete

Concrete has been put to an increasing use in buildings from the turn of the century and is a common component of demolition wastes. It is the most valuable feedstock for producing secondary aggregates. Generally concrete appears in two forms: reinforced concrete in structural elements of a building, such as columns beams and floors slabs and mass unreinforced concrete in roadwork, foundations and building structures.

Concrete products which arise from maintenance works includes: paving slabs, kerbstones and lighting standards. These materials are normally of high strength and can be attractive for recycling processes. High-density concrete blocks can also be suitable, although lightweight blocks generally crush into fine material and therefore are unsuitable as coarse aggregate (Humphreys 1994).

2.12.3 Bricks

Bricks have been reclaimed for some time and they have been in strong demand for the past 30 years. They are available from most salvage yards in various quantities. However, only a small fraction of the estimated 2.5 billion new bricks sold are recycled each year. Reclaimed bricks are commonly used in conservation work and for new buildings. Bricks arising from demolition may be contaminated with mortar, render, plaster or mixed with other materials such as timber and concrete. Reclaimed masonry contains a mixture of whole bricks/block units along with other contaminants making the cleaning cost too high. In many cases bricks are crushed and used as fill material (D.E.T.R 1999).

2.12.4 Stones

Stones arise in some parts of the UK during the excavation of ground and can be valuable as fill. Dressed stone from the demolition of old buildings can be reused in refurbishment work (Humphreys 1994).

2.12.5 Bituminous mixtures

Bituminous mixtures arise from road planings, surplus or reject bituminous mixtures and excavations for services, utilities from major highway improvements.

The first two are often recovered in a relatively uncontaminated form and are thus suitable for recycling. Excavation-derived material may be mixed with a significant amount of other material, such as soils and made up ground (Humphreys 1994), therefore less suitable for recycling.

2.12.6 Architectural features

Architectural features salvaged from demolished structures are finding an increasing market. The items recovered are large, ranging from fireplaces, timbers and wooden panelling used to carved doorways and ornate masonry. There are a number of specialist outlets for these and operators appear to be benefiting from demolition contractors willingness to allow them to pick over sites. In tonnage terms the operations are small, but may be important in preserving some of the more aesthetic and architecturally important features of demolished structures (Humphreys 1994).

2.12.7 Metals

Metals are recovered and recycled almost as a matter of course. This is a well-developed industry with trade associations and recognised markets. Metal, predominantly steel, is used as structural material in many forms of construction. This can be an important by product from many operations. Lightweight steel materials are used in ventilation systems, conduits, cladding, roofing etc. Copper is extensively used in water and power distribution systems. Aluminium is present as cladding and cast iron is found in drainage systems. In construction operations, surplus minor quantities of material can arise as packaging in the form of steel drums. Metal products, which cause handling problems and/or generate a low yield when recycled, may be sent to landfill. One of these is window frames which arise from both demolition and refurbishment of old buildings, where due to the awkwardness of handling glass and the amount of metal that they yield these are generally sent to landfill, often with associated wastes (Humphreys 1994).

2.12.8 Timber

Timber is used as structural components in floors, roof trusses, internal fixtures and fittings. Wastage of timber components in construction occurs when the material is over ordered or surplus not cut to size and spoilt. Timber sections arise from demolition can be offered for resale but much of it is sent to landfill (Packer, Tenney and White 1980).

A recent Scottish report suggested that up to 25% of the C&DW generated in Scotland is wood waste. The composition of C&DW varies greatly from one site to another depending on what type of activity, and it is dependant on whether the waste is construction and demolition or not (D.E.T.R 1999).

2.12.9 Glass and other materials

Glass is present in windows and often in internal partitions. Plasterboard and plaster materials, wallboards and plastic products are extensively used in internal buildings finishes and floor coverings. Whilst glass is a material which is widely recycled in other forms, it is not usually recycled from demolition sites. Lightweight plasterboards and wallboards are not recycled from demolition operations. They are usually extensively damaged during removal and have little potential for recycling (Humphreys 1994).

The use of recycled glass as an alternative construction aggregate has grown in the last decade as greater volumes of waste glass are becoming available. Cullet is used for many construction applications in the United States where detailed specifications studies have been done to identify the specific cullet requirements for the various uses.

Roadway applications include the use of glass aggregate in basecourse, sub-base, sub-grade and embankments. Cullet can be added to natural aggregate and the mixed material will have adequate strength and resistance to abrasion and traffic loads (Environmental Protection Agency Ireland (EPA) 2002). Best estimates for C&DW arising in some European countries as a perecentage can be seen in Table 2.1. (Symonds Group 1999).

2.13 Recycling Operations

Recycling operations can be divided into two broad categories: recycling of C&DW (i.e. concrete, bricks, stones and soil etc) recycling of bituminous mixtures.

2.13.1 Recycling of C&DW

Plants for the production of recycled aggregates from C&DW are not much different from that used in the production of crushed aggregate from other sources. They incorporate various types of crushers, screens, transfer equipment and devices for the removal of foreign matters. The basic method of recycling is one of crushing the debris to produce a granular product of a given particle size. The degree of reprocessing carried out after this is determined by the level of contamination of the initial debris and the application for which the recycled material will be used as general fill, base or fill in drainage projects, sub-base or surface material in road construction or new concrete. A number of different processes are possible for the crushing and sieving of demolition wastes which mainly consist of concrete, such as would be the case for example on a pavement rehabilitation project. There are two systems used to produce recycled aggregate. These are the closed and open systems as illustrated in the block diagrams in Figs 2.10 and 2.11. The closed system is usually recommended as the size of the end product is better defined. The advantage of the open system is in its large handling capacity because in the closed system the 0–40mm fraction from the secondary screen is re-screened over the 40mm screen and put back through the secondary crusher.

Table 2.1 - Best estimates for C&DW arising in some European countries

Country/year	**Concrete (%)**	**Other core C&DW (%)**	**Soil (%)**	**Recycled Asphalt Pavements %**	**Total (%)**
Germany/94-96	15	5	72	9	100
UK/96	33	12	44	11	100
Netherlands/96	52	3	31	13	100
Belgium/90-92	18	1	78	3	100
Austria/97	14	4	76	6	100
Denmark/96	17	7	72	4	100
Sweden/96	19	10	25	46	100
Finland/97	5	9	85	1	100
Ireland/95-97	21	11	68	0	100
Average	**22**	**7**	**61**	**10**	**100**

Recycling plants working according to the principles of one of these schemes are regarded as first generation processing plants. They are characterised by the fact that there are no facilities for removing contaminants, with the possible exception of a magnet for the separation of reinforcement and other ferrous material. Such plants are frequently used on pavement rehabilitation and recycling projects.

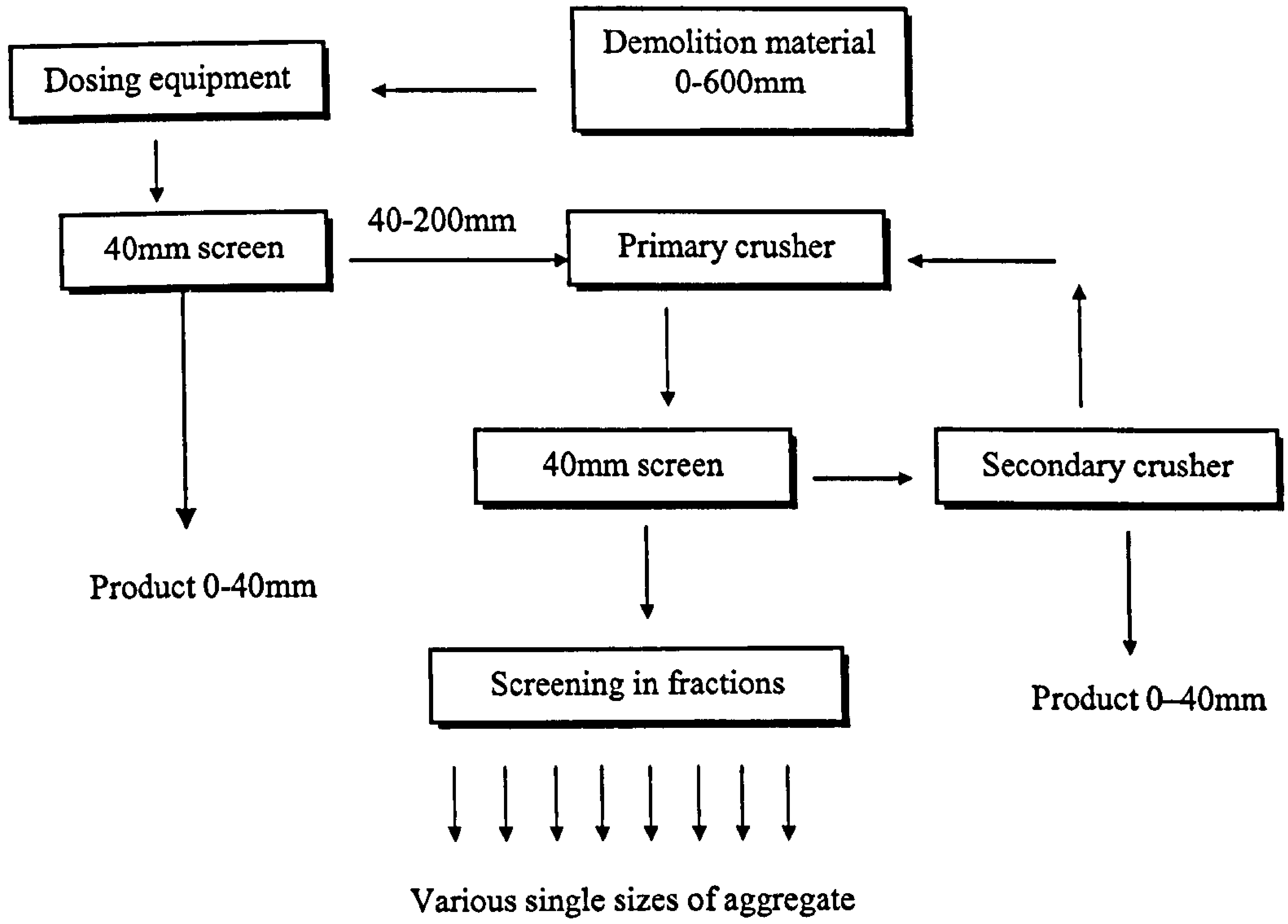


Fig. 2.10 - Flow chart of typical closed system plant for production of recycled aggregate from concrete debris, free from foreign matter (Hansen 1992)

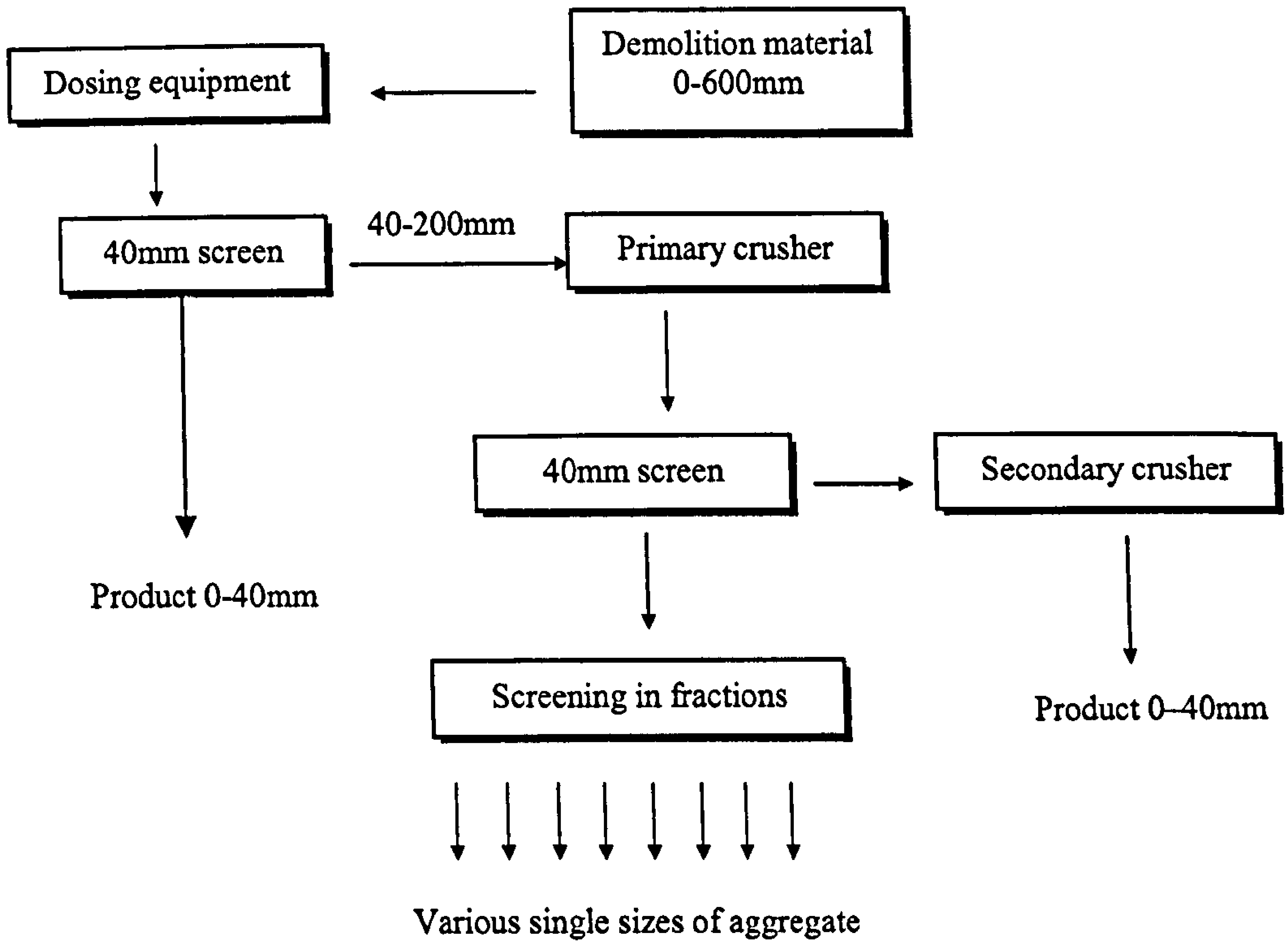


Fig. 2.11 - Flow chart of typical open system plant for production of recycled aggregate from concrete debris, free from foreign matter (Hansen 1992)

Fig. 2.12 shows a typical second-generation plant where large pieces of debris arriving from demolition sites are reduced to 400-700mm maximum size by means of a wrecking ball or hydraulic shears to cut reinforcement. Large pieces of steel, wood, plastics and paper are removed by hand (Fig. 2.13). Incoming material is then crushed in a primary crusher, which is usually of jaw or impact type.

Products from the primary crusher are screened on a deck consisting of a 10mm scalping screen. Material below 10mm is wasted in order to eliminate fine contaminants such as dirt and gypsum. All iron and steel is removed by self-cleaning magnets, which are placed at one or more critical locations above conveyor belts. Material above 40mm material is passed through a secondary jaw, cone, hammer or impact crusher in order to reduce all products to 40mm maximum size. Particles ranging from 40-100mm material from the primary crusher bypasses the secondary crusher. All material is then washed or air-sifted (Fig. 2.14) in order to remove remaining lightweight matter such as wood, paper and plastics. The clean product is screened into various size fractions according to customer specifications.

Recycled and processed aggregates which are made from mixed construction rubble usually contain less than 1% impurities, which may be good enough for road construction purposes, but not necessarily acceptable for use as concrete aggregates. However when recycled aggregates are made from raw materials, which contain more than 95% old concrete, the end product is usually clean enough to meet specifications for concrete aggregates without washing. In ideal future third generation plants the demolished materials should be supplied to the recycling plant, processed and sold without the need to transport large quantities of residual matter to city dumps either from demolition site or from the processing plant. This would be an ideal situation from an environmental and economic point of view (Hansen 1992).

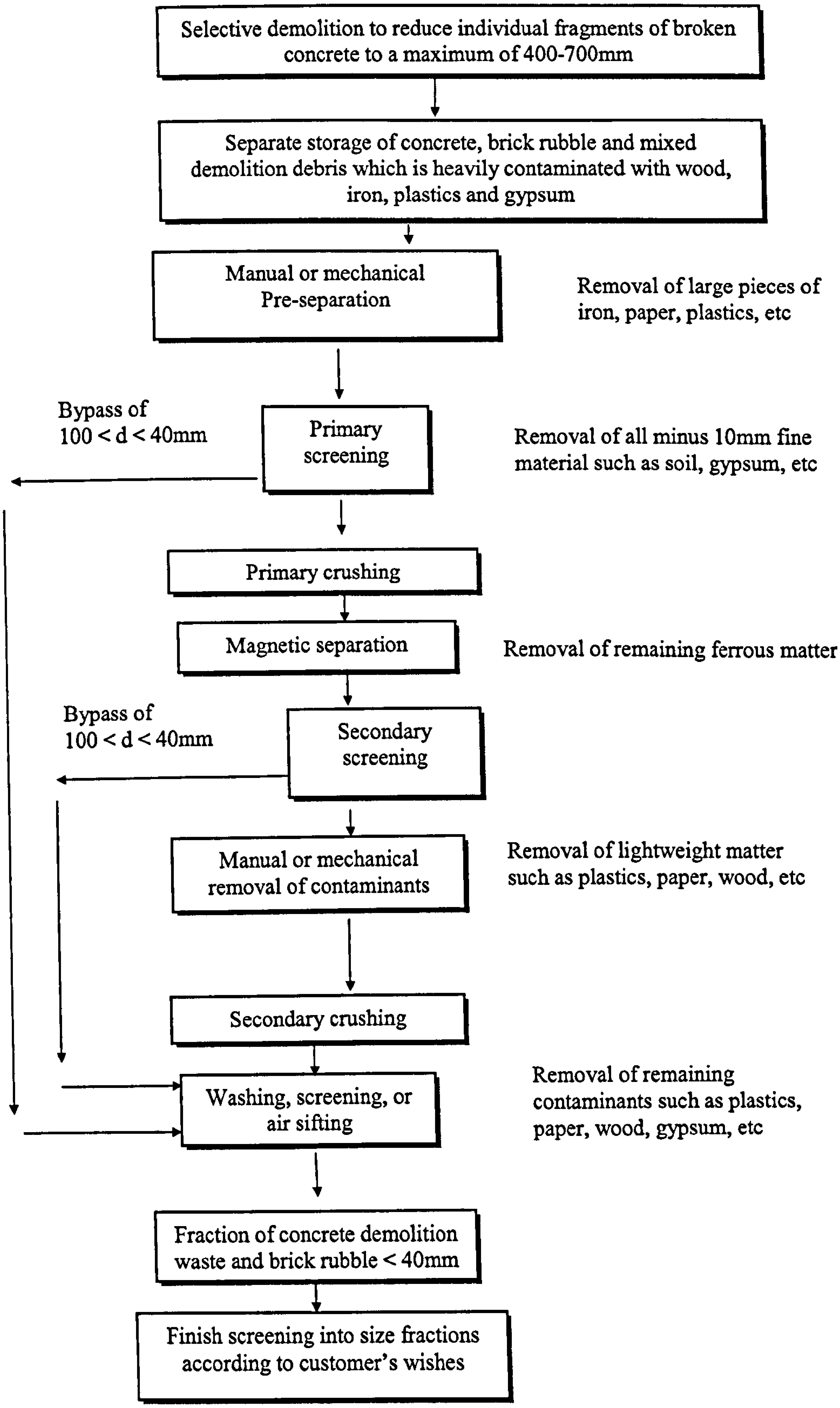


Fig. 2.12 – Second generation processing procedures for building and demolition waste (Hansen 1992)

Fig. 2.13 - Crusher with manual picking belt (Humphreys 1994)

Fig. 2.14 - Recycling system with air separation table in the foreground and a over belt magnet for removing steel in the background (Humphreys 1994)

SECTION 3: Recycling of Bituminous Mixtures

2.14 General

Recycling of used asphalt and other waste mixtures as replacements for virgin aggregate is on the increase worldwide. The UK is ahead of Europe in terms of the amount of material recycled. The highways agency has already revised the specification for recycling of used asphalt into new basecourse mixtures from 30% to 50% (Asphalt Industry Alliance (AIA) 2001).

The recycling of asphalt mixtures are marketed under a large number of names. This may be somewhat bewildering to professionals who are not familiar with asphalt recycling. All recycling processes fall into one of four possible categories as follows: hot recycling, cold recycling, in-situ recycling and in-plant recycling. It follows that both the in-situ and in-plant-recycling processes can be either hot or cold (Sherwood 1995). Table 2.2 shows the different methods used for hot and cold, in-situ or in-plant recycling (D.E.T.R 1999).

Table 2.2 - Road recycling techniques

Location	Hot	Cold
In-situ	Repave	Retread
	Remix	Deep in-situ
In-plant	Central plant hot recycling	Central plant cold recycling

2.15 Hot In-situ Recycling

2.15.1 Repave

Repave is an in-situ technique suitable for use when the road pavement surface has deteriorated without excessive hardening of the binder. The wearing course is heated, levelled and scarified by a repaving machine. A new material is overlaid without having to prepare the surface in the normal way. This overlay is added while the old surface is still hot to ensure that a good bond is established between the old and new paving mixtures (D.E.T.R 1999).

2.15.2 Remix

Remix is a process where, the existing wearing course is firstly softened by a pre-heater on the remixer then a rotating scarifiers on the remixer loosen the bituminous material, which is augured to the centre of the machine where it enters a pugmill mixer and mixed thoroughly with virgin materials. The new mix is then placed evenly to grade and slope by a compacting screed. Fig. 2.15 shows the layout of a remixer. The advantages of remix are: in-place rehabilitation by recycling existing mixtures, very slight traffic disruption as resurfacing is completed in a single pass, fast, cost effective, kind to the environment and economical, 20–30% savings in construction time, transport costs, energy and raw materials, high performance, by which resurfacing up to 7,000m^2/day and no overheating of the mix (Writgen 1989).

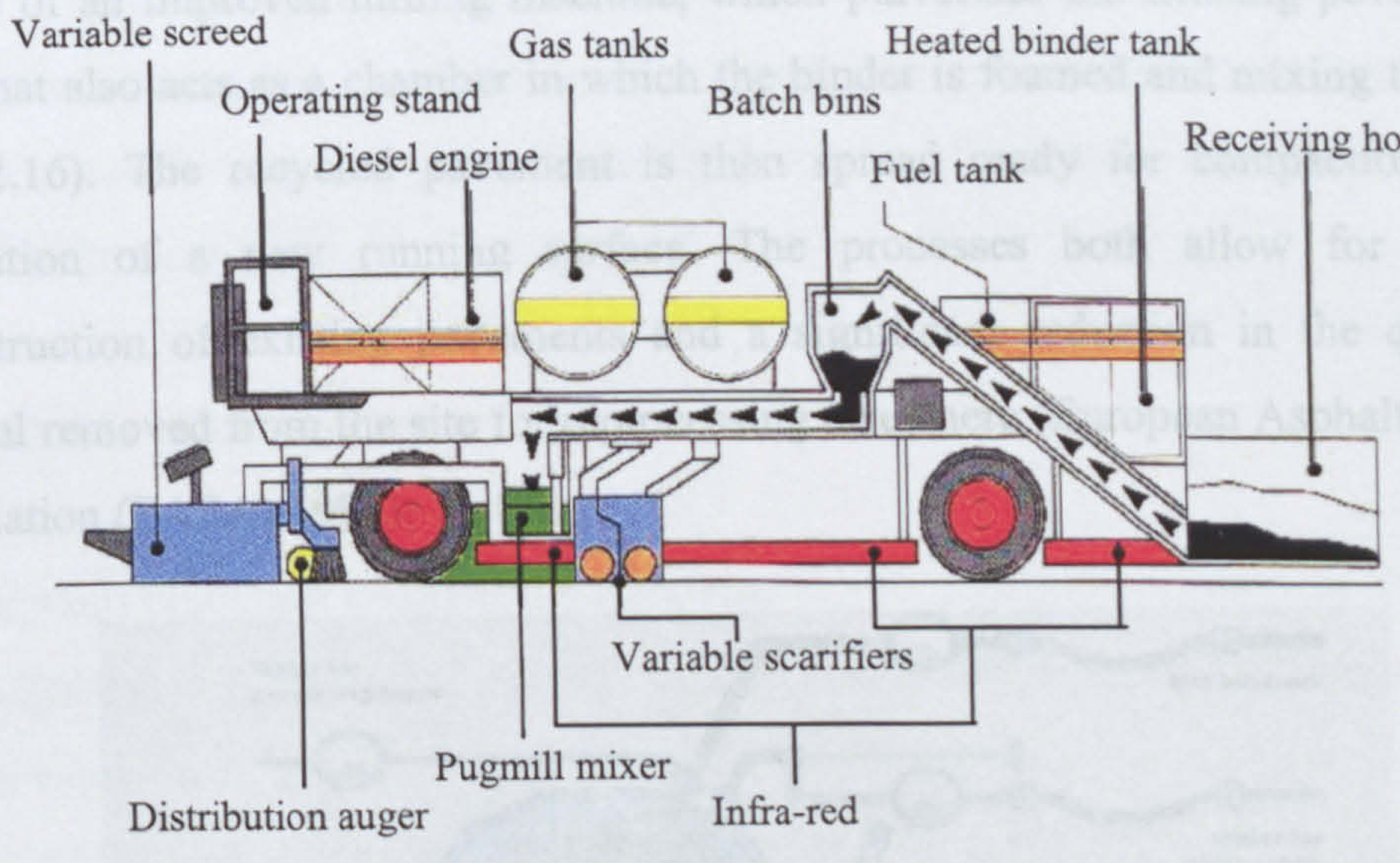


Fig. 2.15 - Layout of a remixer

2.16 Cold In-situ Recycling

2.16.1 Retread

The retread process is a method of cold in-situ recycling restricted to a shallow depth. Pavement surfacing layers are scarified in-situ to a depth of about 75mm, broken up levelled, sprayed with binder, reharrowed and rolled before being surface dressed. The advantages of this method are: well liked by public because of its speed, reshape rejuvenates and restores skid resistance to affected surfaces, reuses 100% of the existing pavement and cheaper than alternative methods and equipment is small-scale and lowtech so mobilisation and downtime costs are low. The disadvantages associated with retread are: restricted to lightly trafficked residential streets as the technique produces an open graded binder-rich material, which has low stability and rut resistance and provides no structural improvements to the pavement directly.

2.16.2 Deep in-situ recycling

Deep in-situ recycling is a cold process that involves pulverising the top layers of damaged or failed road pavements to a depth of 350mm with a special rotavating machine and mixing in specific quantities of either cement slurry or foamed binder (D.E.T.R. 1999). The foamed binder emulsion based system involves the scarification and rotivation of the top layers with binder emulsion and compaction of the existing surface before overlay with a new wearing course. The foamed binder process requires the use of an improved milling machine, which pulverises the existing pavement in a hood that also acts as a chamber in which the binder is foamed and mixing takes place (Fig. 2.16). The recycled pavement is then spread ready for compaction and the application of a new running surface. The processes both allow for the rapid reconstruction of existing pavements and a significant reduction in the quantity of material removed from the site for reprocessing elsewhere (European Asphalt Pavement Association (EAPA) 1995).

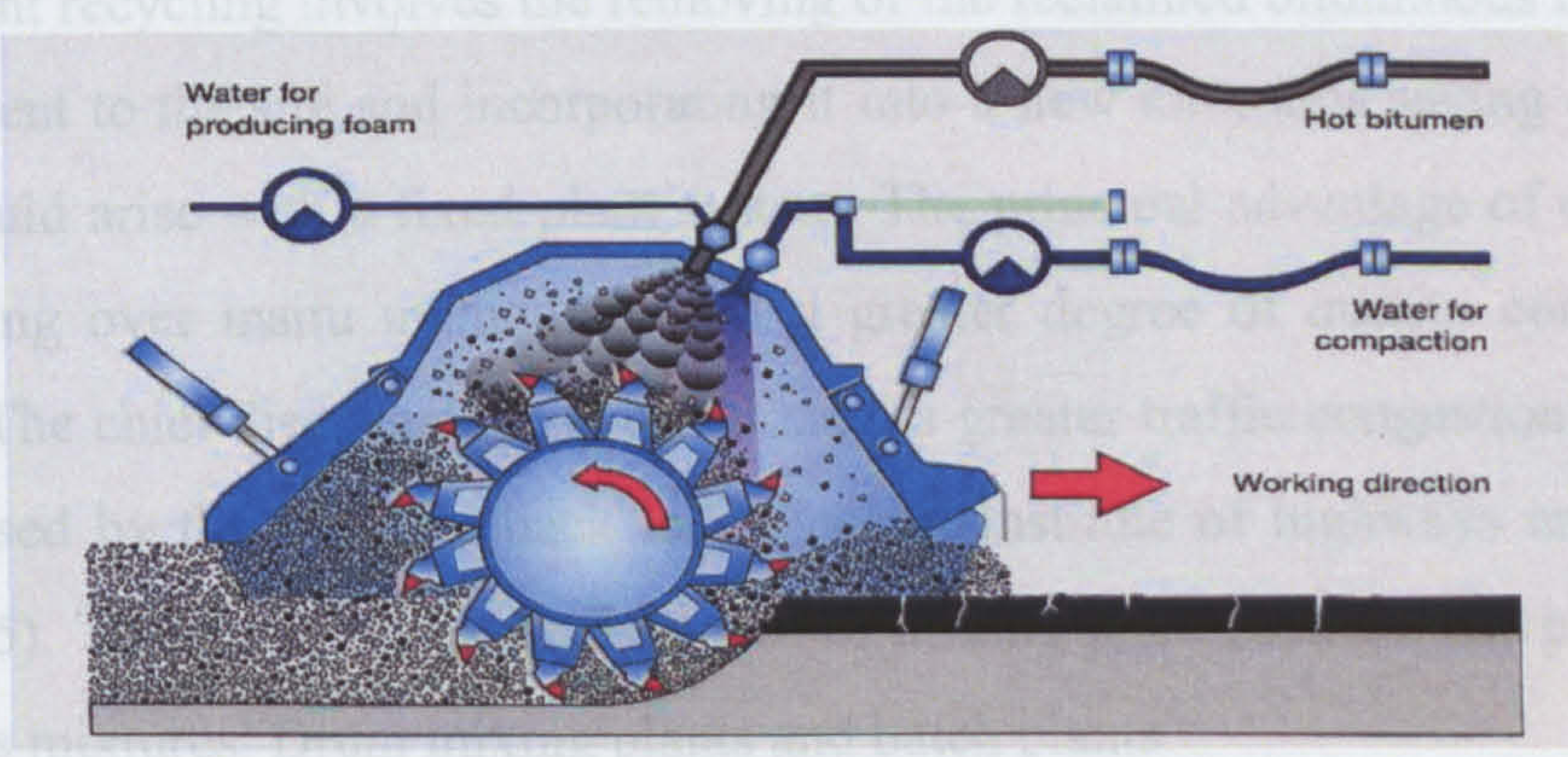


Fig. 2.16 - Chamber in which binder is foamed

Deep in-situ recycling does not require removing spoil to tip, paying landfill tax, importing new quarried materials or road-clogging fleets of trucks, quick paving, in excess of 1000m^2/day, or 1km/day of haunching works, thus saving on time labour and plant hire costs, work can be done on single carriageways within traffic management safety, no excavation required which means there is no need for sub-base and the pavement level before and after pulverising/mixing can be the same, 100% of existing pavement mixtures are used with savings in transport, disposal of old material, mixing and laying of new material, process can take place in urban areas without relaying kerbs or gullies, can save up to 30% of the cost and 60% of time involved in replacing the asphalt road material. There is no significant height difference between the section of road under reconstruction and the normal running surface, allowing ease of traffic management and access to adjacent properties.

The constraints associated with deep in-situ recycling are: low take up can lead to high mobilisation costs, minimum contract size of $5000m^2$ is generally required, contractors include a risk element with any unfamiliar process to allow for possible remedial work, downtime and use of a specialised contractor, complaints from residents as a result of heavy compaction plant necessary to achieve the high refusal density and low air voids and initial task of lowering the frames and covers of manholes and gullies to avoid conflict with machinery.

2.17 Hot In-plant Recycling

2.17.1 Central plant hot recycling

Central plant recycling involves the removing of the reclaimed bituminous material to a plant adjacent to the site and incorporating it into a new mix, thus saving on transport cost as would arise with a fixed plant system. The principal advantage of central plant hot recycling over insitu methods is that a greater degree of quality control can be achieved. The chief disadvantage is that it causes greater traffic congestion but this can be minimised by the use of a back load system (Institute of highways and Transport (IHT) 1995). There are two principal types of mixing plant used for the production of bituminous mixtures: Drum mixing plants and batch plants

1. Drum mixing plants: Drum mixers produce asphalts by continuous process, which dries the aggregate and mixes it with binder in a specially designed drum. The aggregates are introduced to the drum in the vicinity of the burner, which provides the heat to dry the aggregate and bring them to mixing temperature. Early experiments with recycling in this type of plant, in which the reclaimed asphalt pavement (RAP) was added with the new aggregate, indicated that the very high temperatures in the vicinity of the burner, which are of the order of 1500^0C, caused unacceptable emissions from the plant. These emissions, which became known as blue smoke, were the result of the formation of very small particles generated when the binder on the reclaimed material either boiled or burnt due to the very high temperatures in the vicinity of the burner flame. Various systems were investigated by plant manufactures and the one eventually adopted and most widely used is to introduce the reclaimed material about half way down the drum through a ring specially and cleverly designed openings in the drum (Fig. 2.17). In some equipment, a circular heat shield was also fixed inside the drum, across its centre, to ensure that there was no direct contact with the burner flame (Fig. 2.18).

It is possible to mix 100% recycled material in a drum mixer however it is not advisable and is very unlikely to be commercially viable because recycling at this level requires significant increases in dwell time in the drum. Again, specific limits for the quantity of recycled material to be incorporated into a mixture will vary between installations, but it seems that operating costs increase when more than 70% reclaimed material is included and that 50% is a reasonable operating figure for this type of equipment (Sherwood 1995).

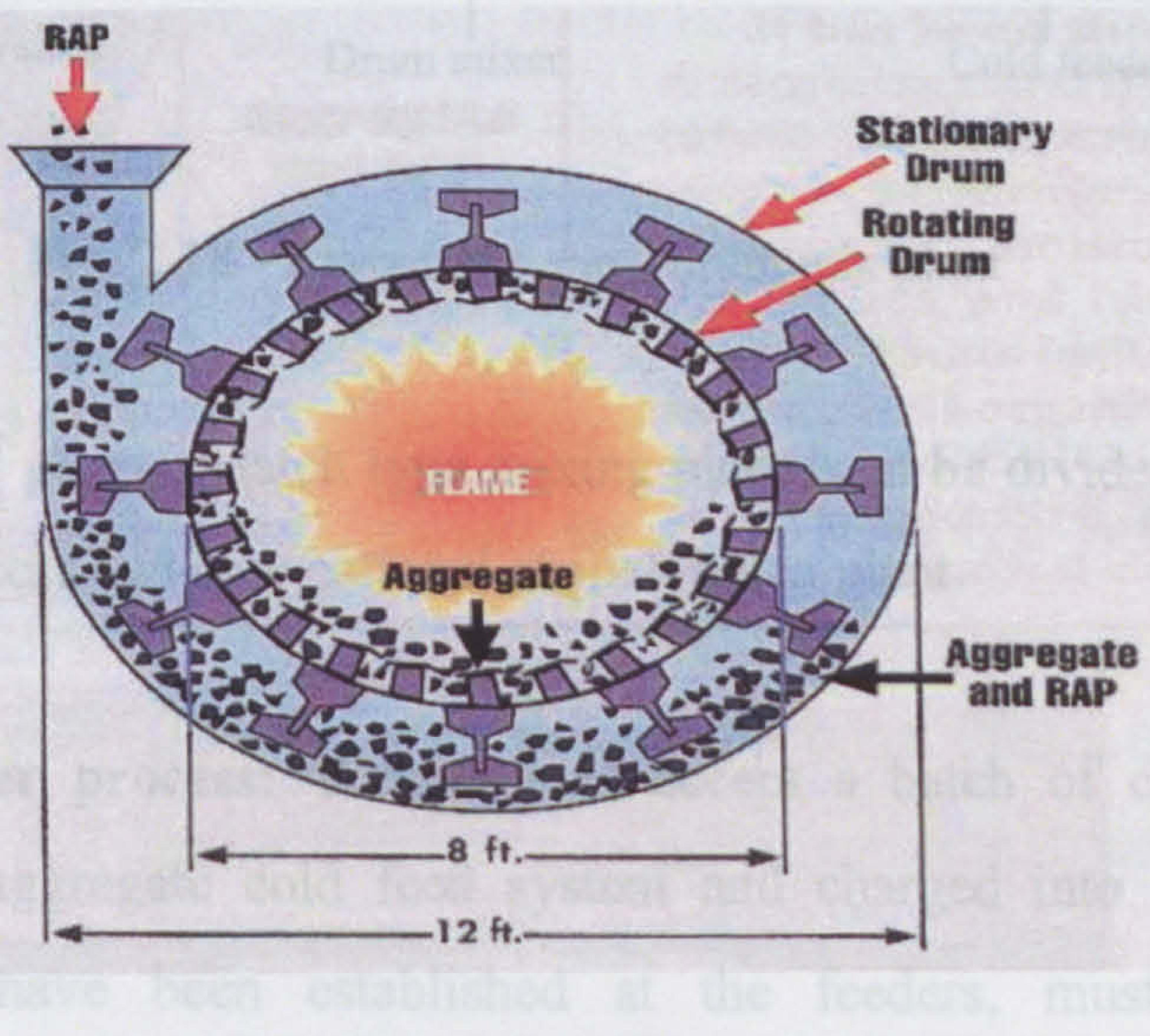


Fig. 2.17 - Cross section of a double drum mixer

The advantages of central plant hot recycling are: environmental savings in both quarrying of raw materials and industrial pollution caused by binder processing, energy saving energy due to less mixing time, resulting mixture is indistinguishable for all practical purposes from material made of 100% virgin aggregates/binder and the process allows plants to process their own reclaimed material which is particularly necessary for the drum mix plant process.

The constraints of central plant hot recycling are: hot mix processing of bituminous material has the potential to cause emissions of noxious fumes, this may be avoided by ensuring that the heating of the bituminous material takes place away from the direct flame of the burner, transportation of the RAP off site to the plant, contractors may include a risk element with any unfamiliar process to allow for additional remedial works, implementation of waste management regulations and may need planning permission to stockpile.

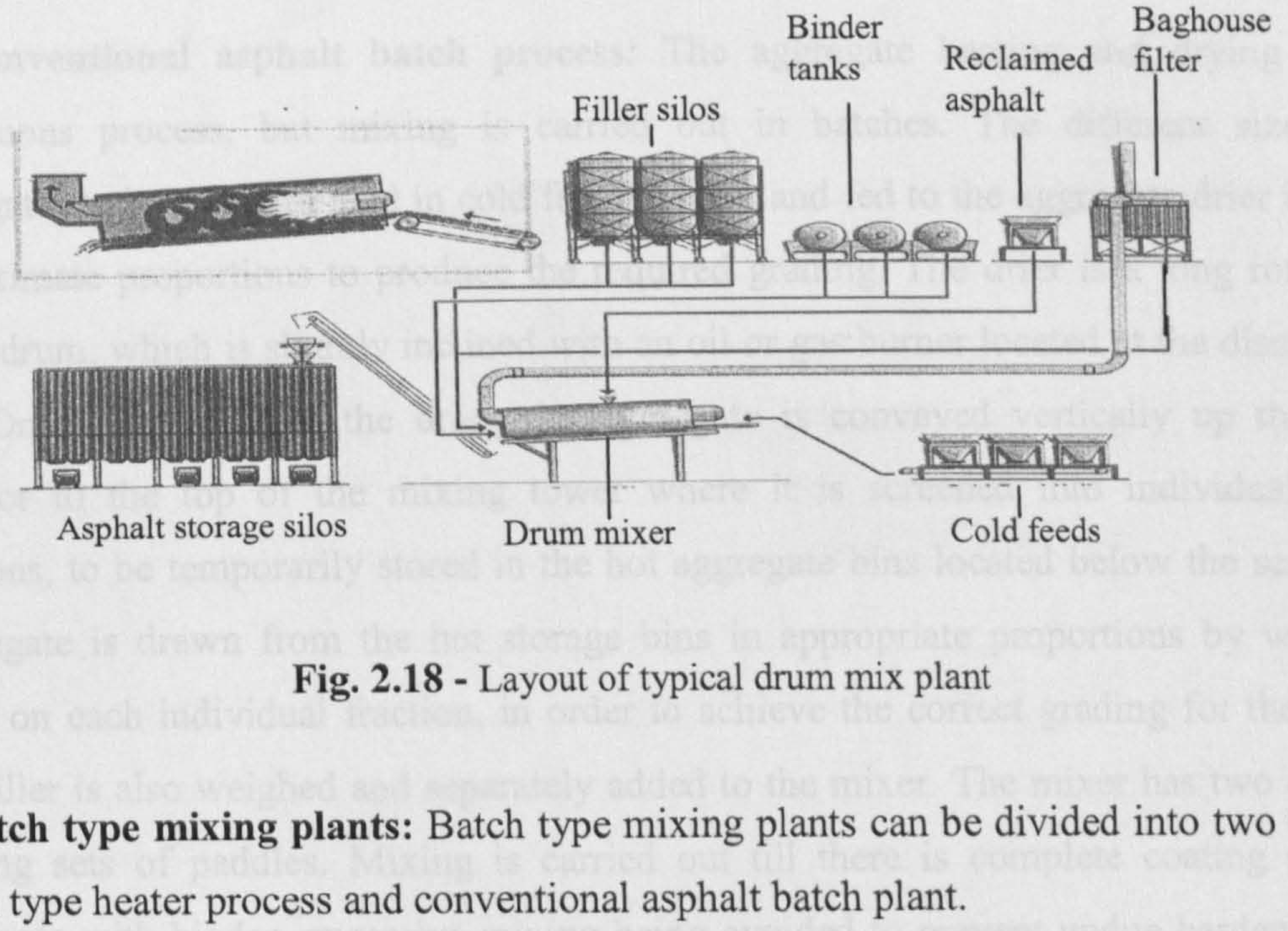


Fig. 2.18 - Layout of typical drum mix plant

2. Batch type mixing plants: Batch type mixing plants can be divided into two types: Batch type heater process and conventional asphalt batch plant.

A. Batch type heater process: These plants accept a batch of cold proportioned aggregate from the aggregate cold feed system and charged into the dryer. These proportions, which have been established at the feeders, must be within the requirements of the final specification because they cannot be corrected at a later stage.

The aggregates after weighing are charged, into a batch heater. This has a large diameter compared with its length and is heated using gas or fuel oil heating unit. The batch heater acts as a dryer and also raises the temperature of the aggregate to that required by the final mix specification. As the batch heater process is not an efficient means of removing moisture from the aggregates a pre-dryer must be used. This is particularly important where a mix contains a lot of fine material. These fines may be treated in a rotary drier first and stored separately. The dry fines are then metered and fed in the correct proportion onto the cold feed belt which also carries the proportioned coarse aggregate to the check weigh hopper. The dried heated and weighed batch is delivered directly into the pugmill where the liquid binder and filler are added. Dust collection, binder and filler metering, binder storage, hot mix storage etc are similar to those found in the indirectly heated batch plant. Between 10% and 15% reclaimed material can be added to the mixer by means of a belt feeder and weigh conveyor (Doncaster College (E) 1999). Fig. 2.19 shows the layout of a typical batch heater process.

B. Conventional asphalt batch process: The aggregate heating and drying is a continuous process, but mixing is carried out in batches. The different sizes of aggregates to be used are held in cold feed hoppers and fed to the aggregate drier in the approximate proportions to produce the required grading. The drier is a long rotating metal drum, which is slightly inclined with an oil or gas burner located at the discharge end. On discharge from the drier, the aggregate is conveyed vertically up the hot elevator to the top of the mixing tower where it is screened into individual size fractions, to be temporarily stored in the hot aggregate bins located below the screens. Aggregate is drawn from the hot storage bins in appropriate proportions by weight, based on each individual fraction, in order to achieve the correct grading for the mix. The filler is also weighed and separately added to the mixer. The mixer has two contra rotating sets of paddles. Mixing is carried out till there is complete coating of the aggregate with binder, excessive mixing being avoided to prevent undue hardening of the binder at the relatively high mixing temperature (BACMI 1994). Between 10% and 15% of reclaimed material can be added by feeding the material directly into the batch weigh hopper. Up to 50% percent can be added to the mix using the parallel-drum method. The material is heated to between 80°C and 120°C, weighed in a separate vessel and then passed by means of a screw conveyor into the mixer. The exhaust from the reclaimed dryer is directed into the discharge-end box of the virgin drum where the fumes are incinerated (Doncaster College (E) 1999). Fig. 2.20 shows a typical layout of a conventional asphalt batch process.

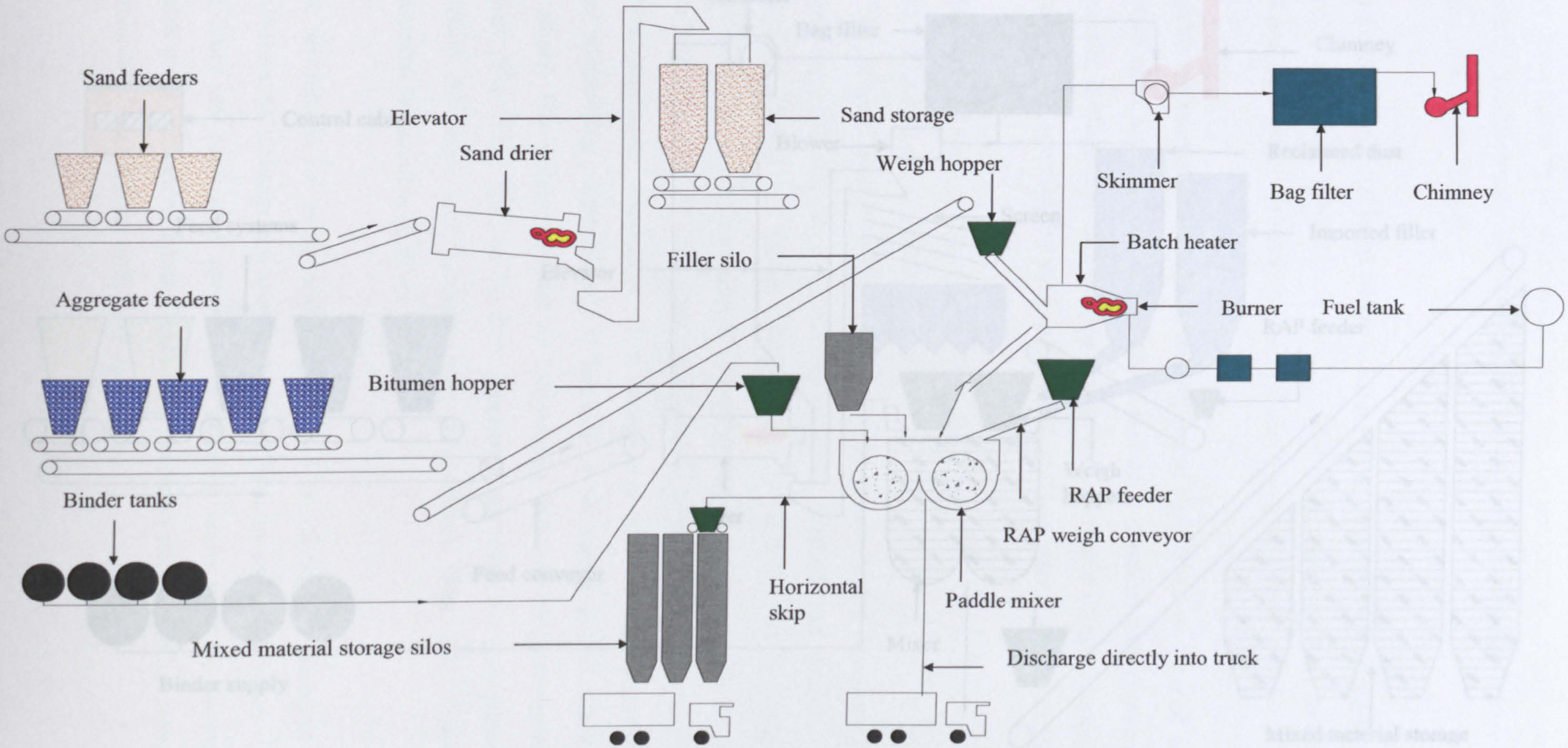


Fig. 2.19 - Layout of typical batch heater process

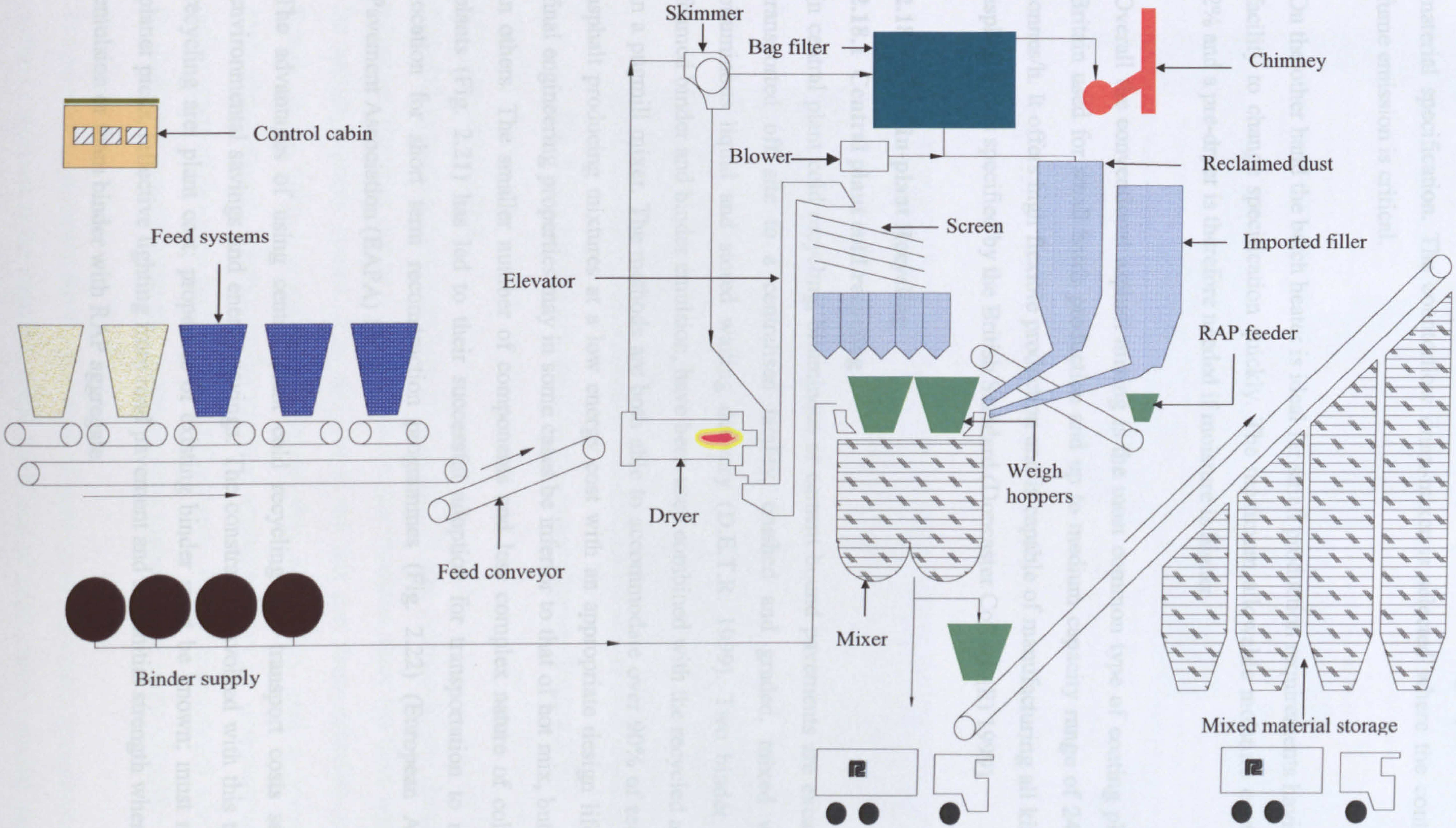


Fig. 2.20 - Typical layout of a conventional asphalt batch process

The drum-mixer is an economical plant for the production of large quantities of one material specification. The counterflow drum-mixer is selected where the control of fume emission is critical.

On the other hand the batch heater is ideal for small production requirements having the facility to change specification quickly. The maximum allowable moisture content is 2% and a pre-dryer is therefore needed if moisture is higher.

Overall the conventional asphalt mixing is the most common type of coating plant in Britain used for small batch production and up to medium capacity range of 240-300 tonnes/h. It offers high flexible production and is capable of manufacturing all kinds of asphalt mixes specified by the British Standard (Doncaster College (E) 1999).

2.18 Cold In-plant Recycling

2.18.1 Central plant cold recycling

In central plant cold recycling, bituminous or cement bound pavements are excavated, transported off site to a centralised facility, crushed and graded, mixed with a bituminous liquid and stored waiting delivery (D.E.T.R. 1999). Two binder types, foamed binder and binder emulsion, have been used combined with the recycled asphalt in a pugmill mixer. The methods are both able to accommodate over 90% of recycled asphalt producing mixtures at a low energy cost with an appropriate design life. The final engineering properties may in some cases be inferior to that of hot mix, but equal in others. The smaller number of components and less complex nature of cold mix plants (Fig. 2.21) has led to their successful adoption for transportation to remote location for short term reconstruction programmes (Fig. 2.22) (European Asphalt Pavement Association (EAPA) 1995).

The advantages of using central plant cold recycling are: transport costs savings, environmental savings and energy savings. The constraints involved with this type of recycling are: plant cost; properties of existing binder must be known; must remove planer picks, reflective lighting from road pavement and low initial strength when using emulsion or foam binder with RAP aggregate.

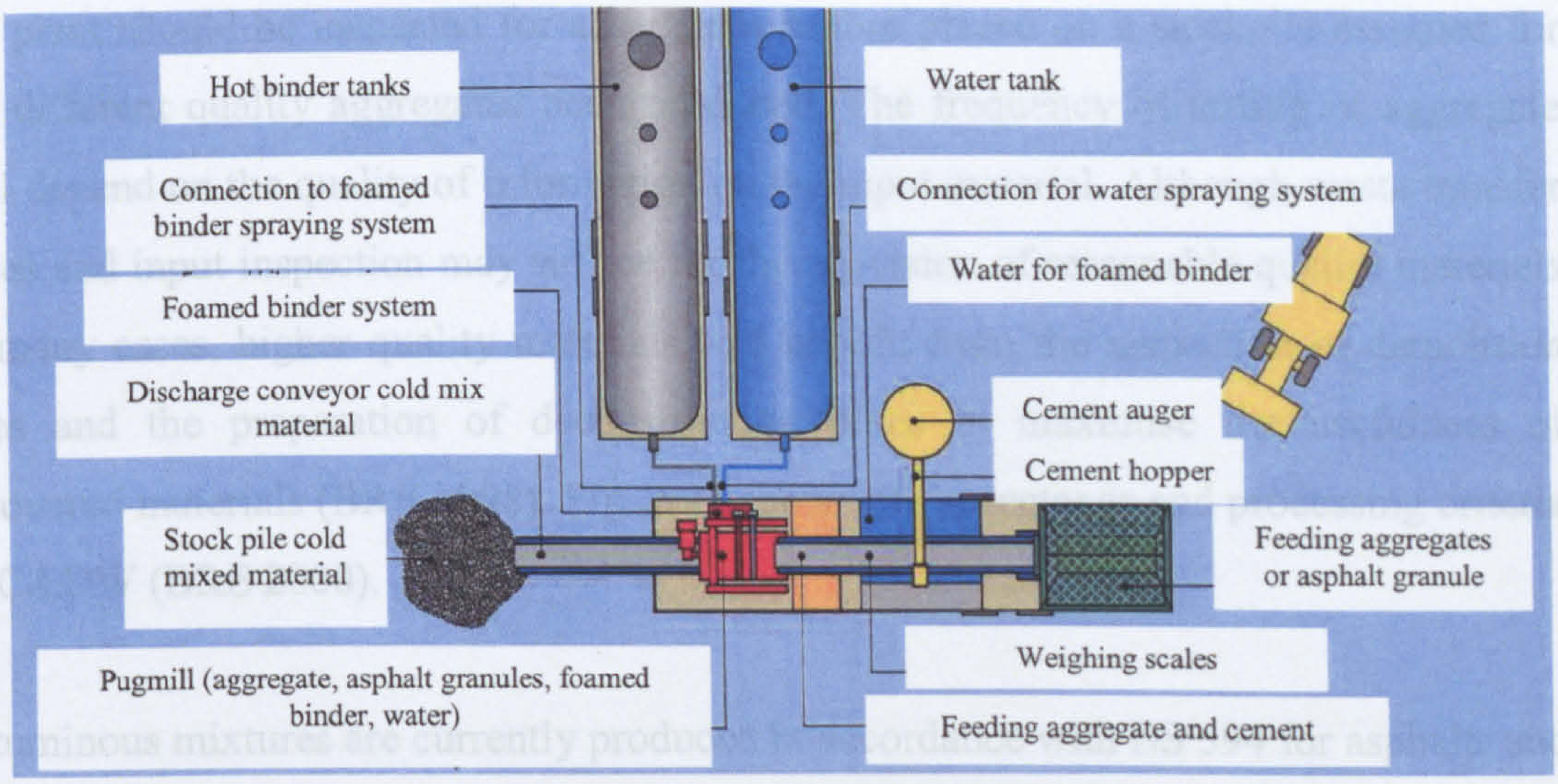


Fig. 2.21 - Typical layout of central plant cold recycling (Wrigten 2000)

Fig. 2.22 - Portable cold mix plant used to make foamix (Wrigten 2000)

2.19 Quality Control

Recycled aggregates have long been used in the UK and overseas by the construction industries. However due to a lack of suitable specifications, there has been little basis for applying quality control. It is also due to the fact that these materials are often thought to be inferior to natural aggregates, they are mainly used in lower grade applications, though, suitable quality recycled aggregates may be used successfully in higher grade applications such as structural concrete. Recent international advances in the drafting of specifications now enables greater usage of RAP in the UK (BRE 1998).

The primary requirement for the provision of good quality products is input control for materials received at the recycling plant. Each load of unprocessed material received at the plant should be inspected for acceptance before placed on a stockpile assigned for the different quality aggregates been produced. The frequency of testing of aggregate will depend on the quality of information on the input material. Although waste transfer notes and input inspection may suffice for the provision of reasonable quality materials in many cases, higher quality materials will benefit from the inspection of demolition sites and the preparation of deconstruction plans to maximise the usefulness of recovered materials (BRE 1998). Fig. 2.23 shows the acceptance and processing criteria of C&DW (BRE 2000).

Bituminous mixtures are currently produced in accordance with BS 594 for asphalts and BS 4987 for macadams. These specifications covers recycled material from roads. Prior to use, the reclaimed material is tested to determine its grading, binder and moisture content, which must conform to the above standards. Clause 902 of the Department of Transport (DOT) Specification for Highway Works 7th edition allows a percentage addition of 10% of reclaimed bituminous material to a recycled roadbase, basecourse or wearing course mix, provided the final mix fulfils the above British standards. The specification does not allow for the use of tar or tar based binders to be recycled. Practical considerations generally limit the percentage addition to a maximum of 30% reclaimed material. The percentage that can be added is linked primarily to the temperature that the virgin aggregate can be heated to. The higher the temperature that the aggregate can be heated the more RAP can be added. Dust extraction can be a problem with recycling. The moisture content of the reclaimed material is also an important factor; the heating temperature required being proportional to this. It is therefore recommended that planings are used immediately or covered before use (Institute of Highways and Transport (IHT) 1995).

The principal advantage of central plant hot recycling over insitu methods is that a greater degree of quality control can be achieved. The chief disadvantage is that it causes greater traffic congestion but this can be minimised by the operation of a back load system where applicable (Institute of Highways and Transport (IHT) 1995). As the RAP material has to be tested to determine its properties, some kind of storage facility may be required in order to keep the moisture content of the RAP material to a minimum. In some bituminous plant even the virgin aggregates are kept in a covered area to keep the moisture to a minimum. Fig. 2.24 shows an aggregate storage and supply system for the different aggregate used in a bituminous mixing plant.

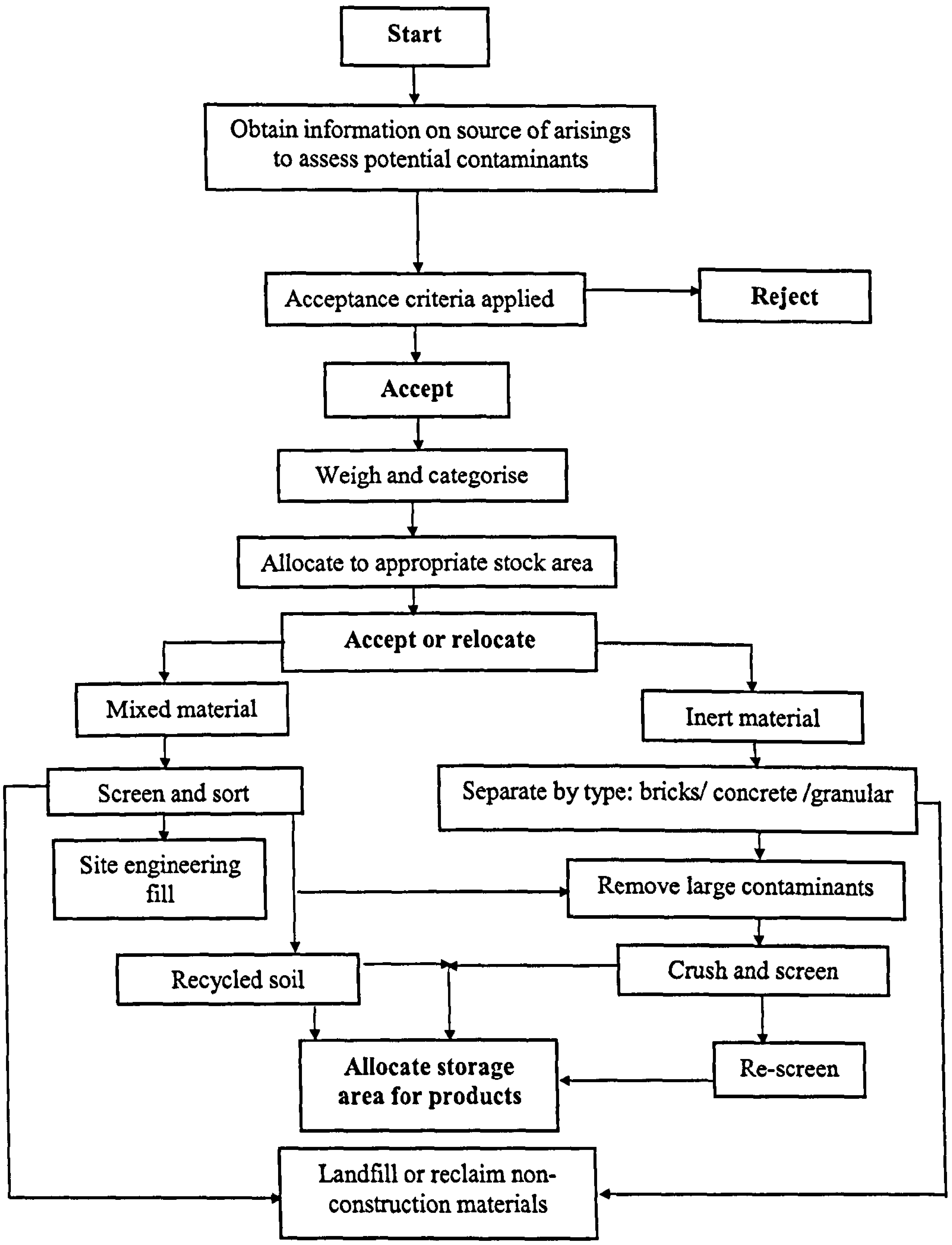


Fig. 2.23 - Flow chart for the acceptance and processing of C&DW (BRE 2000)

Fig. 2.24 - Aggregate storage and supply system for a bituminous mixing plant

The essential requirements for recycled materials are that it must have a performance similar to that of a new material and cost effective. One way of judging its fitness for example is to carry out tests to determine if it complies with the relevant standards.

CHAPTER 3 – TESTS COMMONLY USED TO ASSESS THE QUALITY OF BITUMINOUS MIXTURES

SECTION 1: Aggregate and Binder Tests

3.1 General

The results of measuring British and European testing procedures throughout the process of flexible road construction are relied on for guidance. Decisions are taken based on the data generated at virtually every stage of the operation, including initial investigation, cost estimation, production control, installation, acceptance and assessment of performance. Confidence in the accuracy of the data being analysed is therefore important. This can be achieved by ensuring sufficient care has being taken by skilful, knowledgeable individuals during sampling and testing to produce reliable result. Errors can lead to costly, false assumptions being made about the quality of materials and products.

The interpretation of the data provided is essential in the light of precision of the test methods and the reliability of test equipment. This can be achieved with the aid of appropriate statistical techniques, many of which involve simple calculations. Good interpretation of data leads to the correct decision being taken and good roads being built at the right cost (Hunter 1994).

In order to ensure that bituminous mixtures are capable of providing the characteristics required of road pavements, the properties of the individual components must first be ascertained before producing a bituminous mixture.

3.2 Aggregate Testing

Aggregate testing can be broken into two main categories: physical tests and mechanical tests.

3.3 Physical Tests

The physical tests carried out on aggregates include: particle size distribution, aggregate shape, relative density, bulk density and aggregate shrinkage.

3.3.1 Particle size distribution

Determination of the aggregate grading is carried out in accordance with BS 812: Part 103: 1985. This is a fundamental property, which governs how an aggregate will perform in the mixture.

Using a nest of sieves, it is possible to classify the material by grading analysis. Fig. 3.1 shows previous results comparing dry and wet gradings of aggregate. The figure shows how routine dry sieving could underestimate the amount of material passing through the 75μm sieve. Wet sieving washes the finer particles off the coarser particles therefore it gives more realistic results.

Previous research has shown that particle size has a direct relationship to skid resistance and the grading of aggregates is a fundamental factor in the design of asphalt mixtures (Smith and Collins 1993).

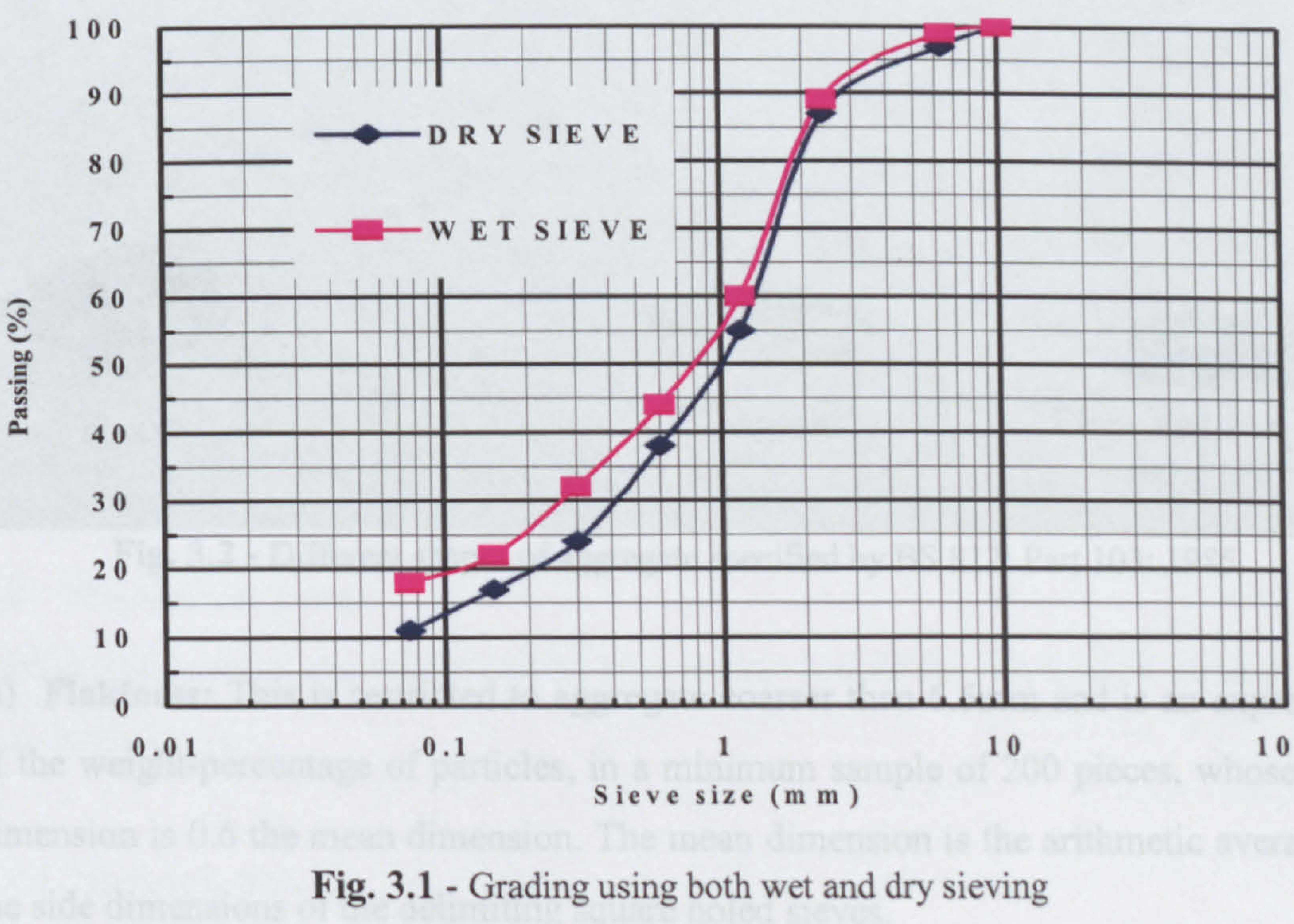


Fig. 3.1 - Grading using both wet and dry sieving

3.3.2 Aggregate particle shape

In both natural and crushed rock aggregates the particles within a particular size fraction have a range of shapes. The shape reflect intrinsic petrological-petrographic characteristics of the material, environmental effects plays a role in their formation and process. BS 812: Part 103: 1985 groups the aggregates particles into six shapes: rounded, irregular, angular, flaky, elongated and elongated/flaky. Fig. 3.2 shows the different shapes of aggregate specified in BS 812: Part 103: 1985.

Rounded, irregular and angular particles are approximately equidimensional and for many assessments are grouped together as one and termed equidimensional or cuboidal. The description of shape in BS 812: Part 112: 1985 emphasises departure from equidimensional in indices of flakiness and elongation.

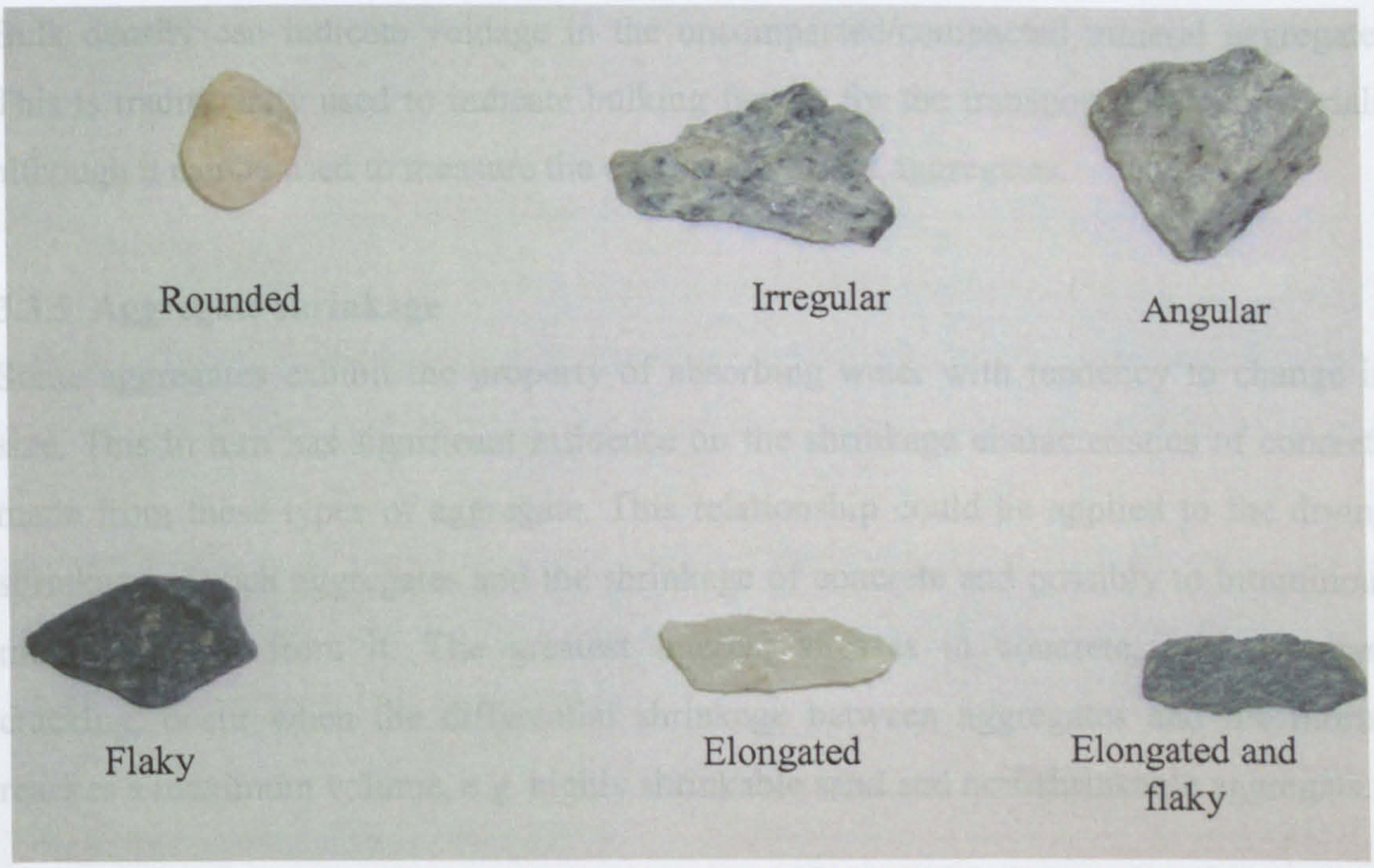


Fig. 3.2 - Different shapes of aggregate specified by BS 812: Part 103: 1985

(a) Flakiness: This is restricted to aggregate coarser than 6.5mm and is an expression of the weight-percentage of particles, in a minimum sample of 200 pieces, whose least dimension is 0.6 the mean dimension. The mean dimension is the arithmetic average of the side dimensions of the delimiting square holed sieves.

(b) Elongated: This is the weight percentage of particles whose long dimension is greater than eight times the mean dimension. Measurement can be made with a standard gauge (BS 812: Part 112: 1985).

3.3.3 Relative density

Relative density is a parameter used in highway engineering as a requirement when dealing with asphalts and their mix design since it relates to the volume of an aggregate and hence the area of ground covered per unit volume of material. Relative density on a saturated surface dry (SSD) basis is used in the calculation of the aggregate abrasion value (AAV) to adjust for the loss of volume. High values of water absorption in the SSD are generally believed to be indicative of durability problems such as frost susceptibility or soundness (BS 812: Part 112: 1985).

3.3.4 Bulk density

Bulk density can indicate voidage in the uncompacted/compacted mineral aggregate. This is traditionally used to indicate bulking factors for the transportation of materials although it can be used to measure the compactability of aggregates.

3.3.5 Aggregate shrinkage

Some aggregates exhibit the property of absorbing water with tendency to change in size. This in turn has significant influence on the shrinkage characteristics of concrete made from these types of aggregate. This relationship could be applied to the drying shrinkage of such aggregates and the shrinkage of concrete and possibly to bituminous mixtures made from it. The greatest internal stresses in concrete, with resultant cracking, occur when the differential shrinkage between aggregates and the mortar reaches a maximum volume, e.g. highly shrinkable sand and non-shrinkable aggregate.

3.4 Mechanical Tests

Mechanical tests of aggregates can be broken into two further categories: strength tests and durability tests.

3.4.1 Strength tests

Current methods of assessing strength are quite simple and are contained in several parts of BS 812: Part 112: 1985. All these tests require a sample of dry aggregate of size 10 to 14mm. The strength is expressed as a value of strength based on determining the amount of fine aggregate produced after impacting or crushing. The three parameters, which are typically obtained are aggregate impact value (AIV), aggregate crushing value (ACV) and ten-percent fines value (TFV).

3.4.2 Aggregate impact value (AIV)

The AIV gives a relative measure of the resistance of an aggregate to sudden shock or impact. In the current BS 812: Part 112: 1985 method, a test sample of single size dry 10 to 14mm aggregate is placed in a steel mould and subjected to 15 blows from a 13.5kg weight dropped from a standard height. The resistance to sudden impact is assessed by sieving the test sample using a 2.36mm test sieve and determining the mass of material passing as a percentage of the original sample weight. Two samples are assessed and the AIV expressed as the mean of two results.

3.4.3 Aggregate crushing value (ACV)

The ACV gives a relative measure of the resistance of an aggregate to crushing under an applied compressive load. In the BS 812: Part 110: 1990 method, a test sample of single size dry 10 to 14mm aggregate is placed in a steel mould and subjected to slowly applied compressive load of 400kN for a period of 10 min. The resistance to crushing is assessed by sieving the test sample through a 2.36mm test sieve and determining the amount of materials passes as a percentage of the original mass of the sample. Two samples are assessed and the ACV expressed as the mean of two results. A value of < 25% is typically regarded as acceptable.

3.4.4 Ten-percent fines value (TFV)

The TFV as described in BS 812: Part 111: 1985 is the load expressed in kN required to produce 10% fine material when subjected to a gradually applied compressive load. The equipment used in the TFV test is similar to that used to determine the ACV. A test sample of single size dry 10 to 14mm aggregate is placed in a steel mould. An increasing load is applied according to the expected strength of aggregate. By varying the load the 10% fine value of the material is determined (Smith and Collins 1993). Preliminary estimates of this load are first obtained and then the following formula is used to calculate the load that gives the 10% fines value:

$$TPF = \frac{14\,a}{b+4} \qquad \textbf{Eq. 3.1}$$

Where:

a = maximum force (kN)

b = mean percentage fines from two tests at the same load (%)

3.5 Durability Tests

3.5.1 Aggregate abrasion value (AAV)

The AAV reflects hardness and brittleness of the constituent minerals, the influence of mineral cleavage and the strength of intergranular bond. Igneous and non-foliate metamorphic rocks yield the highest values within comparable composition ranges. Inadequate resistance to abrasion of road surfacings aggregates causes an early loss of the surface texture, which is necessary to maintain high-speed skid resistance. BS 812: Part 113: 1995 describes the test procedure carried out to determine the AAV.

3.5.2 Los Angeles attrition value (LAAV)

A sample of aggregates is charged with 6 to 12mm steel balls in a rotating steel cylinder for 500 or 1000 revolutions at 33rpm for material less or greater than 19mm respectively. This causes attrition through mutual tumbling and impact of the aggregates with the steel balls. After testing the sample is screened on a 1.7mm aperture sieve. The coarser fraction washed and oven dried to find its weight. The loss in weight, as a percentage of the original weight is the LAAV. Values derived for this test are numerically similar to those of the Aggregate Crushing Value (ACV) test (Smith and Collins 1993).

3.5.3 Polished stone value (PSV)

The polished stone value was designed to measure the susceptibility of a stone to polishing, when used in the wearing surface of a road. An accelerated polishing apparatus simulates the action of dust laden tyres on samples of aggregate set in polyester resin backing, mounted in standard moulds on a rotating wheel. A 200mm diameter x 38mm broad tyre bears on the aggregate with a total force of 725N. Corn emery and water are continuously fed to the surface of the tyre. This is repeated using emery flour as abrasive. The polish of the specimens is then measured using a standard pendulum arc fraction tester. The deflection of a pointer on a calibrated scale shows the coefficient of friction expressed as a percentage, and this is the polished stone value. A higher value signifies greater resistance to polishing (Smith and Collins 1993).

3.5.4 Micro-deval value (MDV)

The abrasion of the aggregate is produced by a charge of stainless steel balls in a rotating cylinder. The test may be carried out either wet or dry adding 2.5L of water to the test cylinder prior to testing in the wet state. The Micro-deval value is calculated as:

$$\mathrm{MDV} = \frac{100\ \mathrm{m}}{\mathrm{M}} \qquad \textbf{Eq. 3.2}$$

Where:

M = weight of the aggregate tested (g)

m = weight of aggregate that passes through the 1.6mm sieve after the test (g)

3.6 Binder Tests

3.6.1 General

A wide range of tests is performed on binders, from specification tests to more fundamental rheological and mechanical tests. As an almost infinite variety of binders could be manufactured, it is necessary to have tests, which can characterise different grades. Two tests used in the UK to characterise binders are the penetration and softening point tests. These two tests are used to specify different grades of binder. Although they are arbitrary empirical tests, it is possible to estimate important engineering properties, such as high temperature viscosity and low temperature stiffness of the binder from these two simple tests (Shell Bitumen 1991).

3.6.2 Penetration test

In this test a needle of specified dimensions as described in BS 2000: Part 49: 1995 is allowed to penetrate into a sample of binder, under a known load (100g), at a fixed temperature (25°C), for a known time (5 seconds). The distance the needle penetrates, in units of decimillimetre (0.1mm) is termed the penetration. Therefore the greater the penetration of the needle the softer the binder. This test is the basis upon which penetration grade binders are classified into standard penetration ranges (Shell Bitumen 1991). The two grades of binder used in this investigation were 50 and 100pen, the latter being the softer of the two.

3.6.3 Softening point test

In this test a steel ball (3.5g) is placed on a sample of binder contained in a brass ring; this is suspended in a water or glycerol bath as outlined in BS 2000: Part 58: 1993. Water is used for binders with a softening point of 80°C or below, and glycerol is used for softening points greater than 80°C. The bath temperature is raised at 5°C per minute, the binder softens and eventually deforms slowly with the ball through the ring. At the moment the binder and steel ball touch a base plate 25mm below the ring, the temperature of the water is recorded. The test is performed in duplicate and the mean of the two measured temperatures is reported, to the nearest 0.2°C for penetration grade binder and 0.5°C for oxidised binder. If the difference between the two results exceeds 1°C the test must be repeated (Shell Bitumen 1991).

SECTION 2: Bituminous Mixtures Tests

3.7 General

The composition of a bituminous mixture has a definite influence on its behaviour in service therefore confidence in the accuracy of the data being analysed is vital. This can only be achieved by demonstrating that sufficient care has being taken by skilful, knowledgeable individuals in the following prescribed methods for proper sampling and testing that yields reliable results.

Since the introduction in the 1930's all types of bituminous mixtures have been specified in terms of a grading analysis and binder content range, which is better known as a recipe specification. In the early days this was very simple, with crude proportions for grading and target binder content. This became more sophisticated and nowadays the properties of the constituents are specified. The alternative method of specification is to detail the performance required from the material or "a performance specification".

There is a positive move towards performance specifications for coated roadstone throughout the world (Shell Bitumen 1991). There are several reasons for this: the shortcomings of specifying by recipe, the need to better utilise resources and to achieve maximum performance, transfer of risk from customer to supplier, the customer role, Design Build Finance Operate (DBFO) projects, the advent of the European Standards and to ensure harmonisation.

As the move towards performance specifications, knowledge of wheel tracking, creep and elastic stiffness will become mandatory. All the tests carried out on bituminous mixtures can be divided into three-categories (Curtis 1997): empirical, fundamental and simulative.

3.8 Empirical Tests

Despite the advances in fundamental testing of asphalts some empirical test methods remain to be specified in both pavements and mix design procedures which are in use around the world. The Marshall test is still widely used as a mix design method. Although the test offers guidelines on the material it cannot give basic information on stress/strain characteristics of the material under test and therefore prohibits engineering based prediction of subsequent performance in terms of stiffness and fatigue (Curtis 1997).

3.8.1 Marshall test

The Marshall test consists of manufacturing of cylindrical specimens 102mm in diameter and 64mm in height using a standard compaction hammer and a cylindrical mould as can be seen in Fig. 3.3. The specimens are tested for their resistance to deformation at 60°C at a constant rate of 50mm/min. During the applying of the load the jaws of the loading ring confine the majority but not the entire circumference of the specimen; the top and bottom of the cylinder being confined as can be seen in Fig. 3.4. Thus the stress distribution in the specimen during testing is extremely complex. Two properties are determined: the maximum load the specimen will carry before failure (Marshall stability) and the amount of deformation of the specimen before failure (Marshall flow). The ratio of stability to flow is known as the Marshall quotient, a sort of pseudo stiffness, which, is a measure of the mixtures resistance to permanent deformation (Curtis 1997).

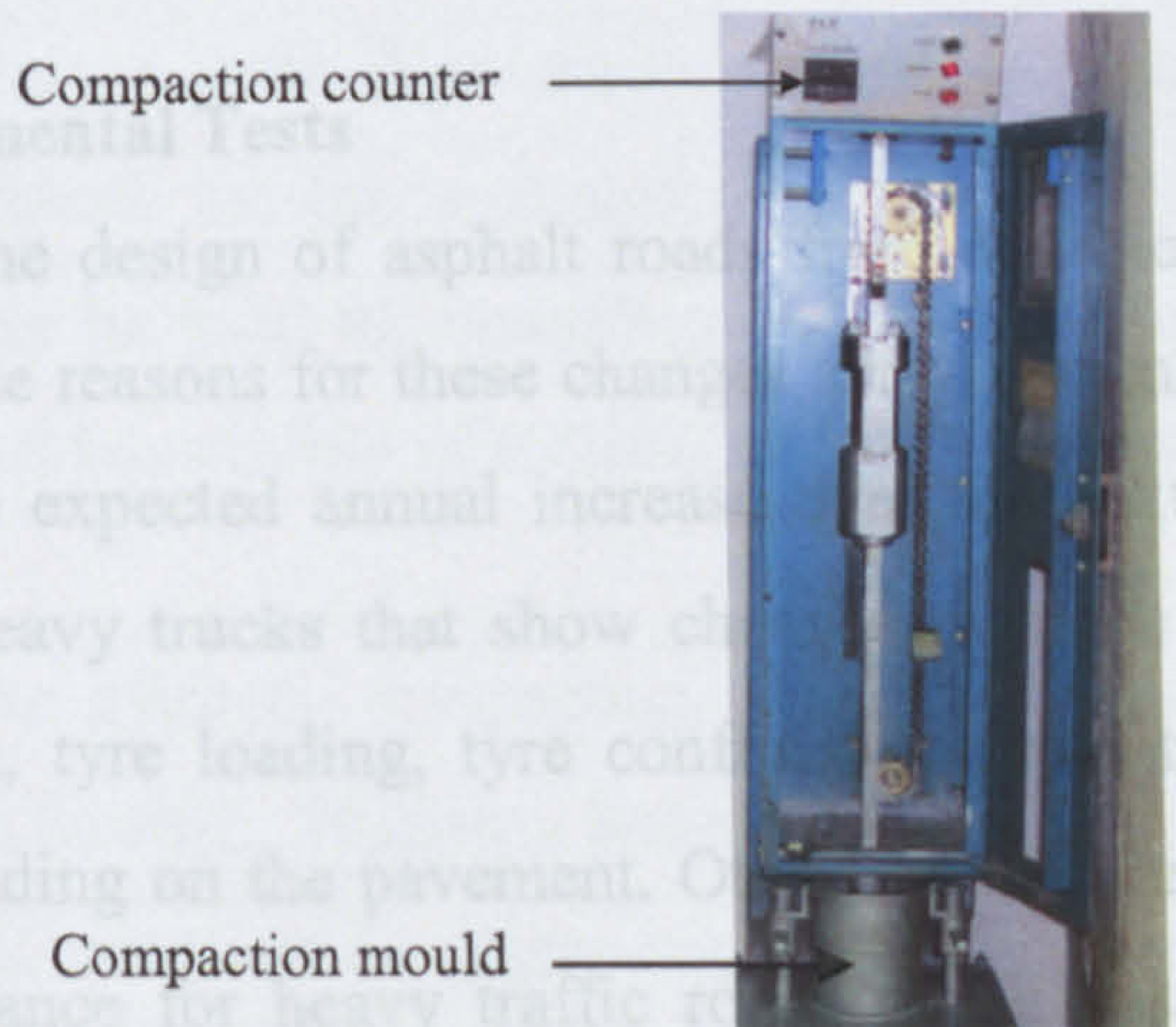


Fig. 3.3 - Marshall compaction hammer

For many engineering mixtures, the strength of the material frequently is thought of as representing the quality of the material. However, this is not necessarily the case for hot-mixed asphalt paving. Extremely high strength is often obtained at the expense of lowered wear and vice versa. Therefore, in evaluating and adjusting mix designs the gradation of aggregate and asphalt content the final mix design must strike a favourable balance between the strength and wear requirements. Moreover, the mixture must be produced as practical as possible and be economically viable (Asphalt Institute 1984).

Fig. 3.4 - The Marshall test apparatus

The data from the Marshall test is used to determine the optimum binder content for a bituminous mix. The mean value at each binder is plotted on a graph against binder content. Optimum or design binder contents are then derived from the mean of the values determined from a number of graphs (BACMI 1992).

3.9 Fundamental Tests

A move in the design of asphalt roads from empirical to fundamental mix design is occurring. The reasons for these changes are pavement loading has reached a very high level and the expected annual increase over the next few decades. It is not only the number of heavy trucks that show changes over the years but the axle loading, axle configuration, tyre loading, tyre configuration and tyre pressure. These all result in increased loading on the pavement. Other reasons for these changes include providing low maintenance for heavy traffic roads which has led to the development of new mixtures like Stone Mastic Asphalt (SMA) and modified binders, road authorities are confronted with decreasing budgets therefore they must get the best value for money.

Pavement science has developed significantly and the insight into the stiffness/fatigue parameters has increased thus allowing more opportunities for the use of RAP in bituminous mixtures. In some countries the annual amount of recycled asphalt laid is between 25 and 50% of the annual production of hot bituminous mixtures. For political reasons recycling at the highest level is required therefore the level of recycled asphalt in individual mixtures may vary between 50-90% from country to country (Heide 1997).

The need for a simple test apparatus to measure fundamental properties of asphalt mixtures was addressed in the late 1980s with the development of the Nottingham Asphalt Tester or NAT. A sister apparatus to the NAT is the NU-10 which was developed by Cooper Research Technology Limited. The machine allows deformation, stiffness and fatigue testing to be carried out by employing different subsystems and test software. The test rig is housed in a temperature control cabinet to give accurate environmental control during testing. The most common fundamental tests are: repeated load triaxial test, uniaxial creep test, Swedish indentation test, vacuum triaxial test, stiffness, fatigue, tensile strength and cantabro test. In order to carry out these tests specimens have to be prepared and compacted using a gyratory compactor.

3.9.1 Gyratory compactor

Gyratory compaction is considered to be the best method of preparing specimens. The resultant specimens are more representative of the material laid and compacted in the road. While only a vertical compaction stress is applied, a gyratory action generates horizontal stresses on the specimens simulating the action of a roller. Gyratory compaction can be used to: manufacture test specimens, determine target site density and assess the compatibility of a mix (Cooper Research (A) 1999).

The use of the gyratory compaction is on the increase and the endorsement of the method by the Strategic Highway Research Programme (SHRP) in the United States has accelerated its adoption in many countries. The method involves applying a static vertical stress to the material in a cylindrical mould while the axis of the mould is inclined at an angle to the vertical. The axis of the mould is then caused to gyrate about the vertical without allowing the mould to rotate (Fig. 3.5). While compaction takes place, the height of the material within the mould is recorded so that the relationship between mixture density and number of gyrations can be determined (Cooper K.E. 1997). The gyratory compactor can be used to compact either 100 or 150mm diameter specimens with the height of specimens ranging from 70 to 170mm. The new European

Standard prEN 12697: Part 31: 1998, has been written to allow for the use of the gyratory compactor.

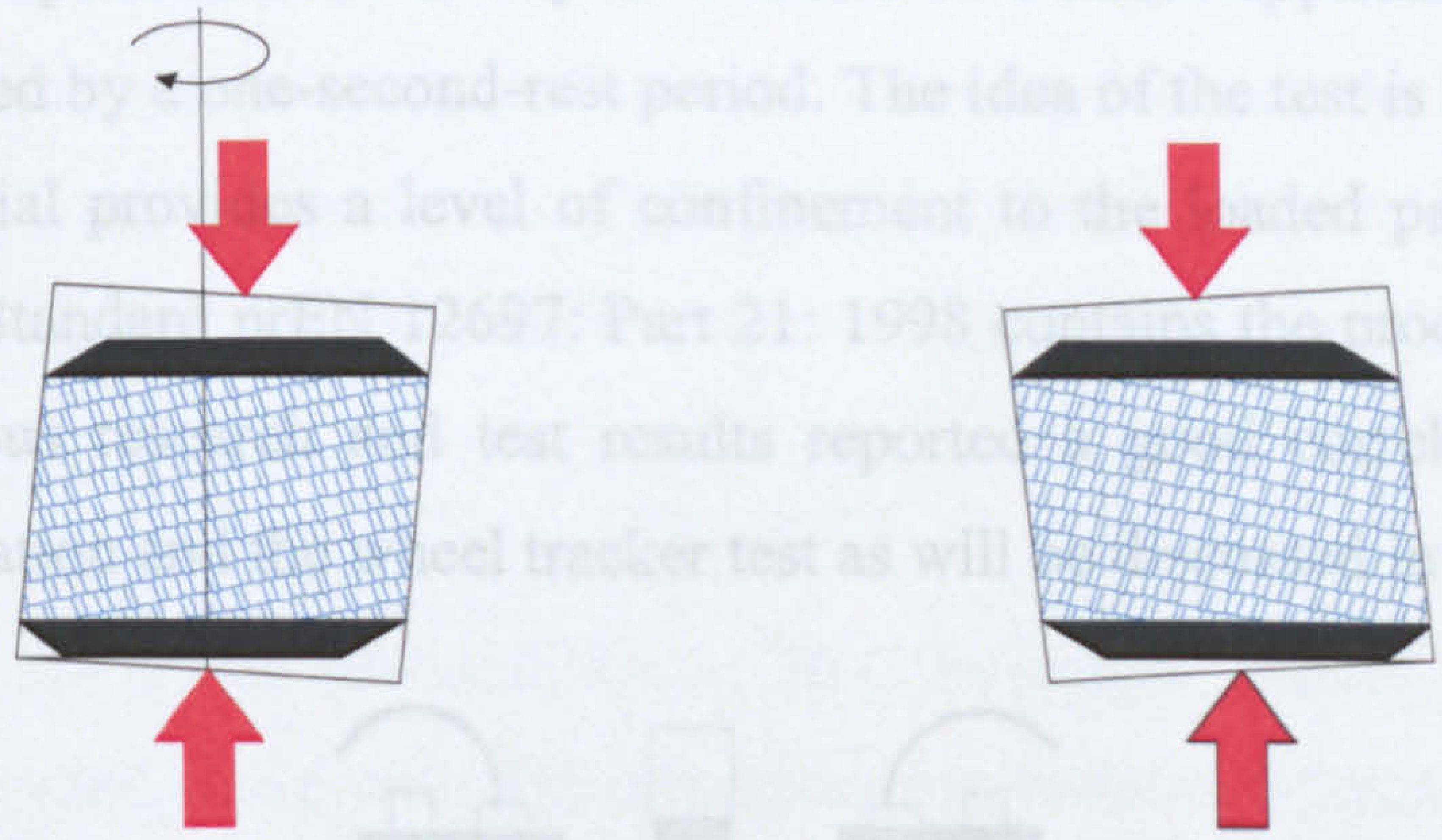


Fig. 3.5 - Principal of gyratory compaction

3.9.2 Repeated load axial test

In this test the specimen is subjected to a repeated application of axial stress. This is considered to be more simulative of traffic loading than static creep. A load cycle consists of a stress application of 1 second duration followed by 1 second rest period. In the UK the standard test consists of 1800 load cycles with a maximum axial stress of 100kPa at a test temperature of 30°C The final value of axial strain recorded at the end of the test is taken as a measure of resistance to permanent deformation (Cooper K.E. 1997). Fig. 3.6 shows the set-up of the apparatus used to carry out the Repeated load axial test.

3.9.3 Uniaxial creep test

In the uniaxial creep test a cylindrical bituminous specimen is subjected to an axial stress for a period of time at a specified temperature. The resultant axial strain is monitored and recorded at different intervals during the test. At the end of the test the superimposed axial stress is set to zero and the relaxation in axial strain monitored (Cooper K.E. 1997). Fig. 3.6 also shows the set-up of apparatus used to carry out the uniaxial creep test. The test is covered by prEN 12697: Part 25b: 1998.

3.9.4 Swedish indentation test

The Swedish indentation test is similar to the repeated load triaxial test with a 15cm diameter specimen loaded using a 10cm diameter upper platen (Fig. 3.7). During the test the specimen is subjected to repeated applications of axial stress. This is considered to be more simulative of traffic loading than static loading which resulted in the creeping of the specimen. A load cycle consists of a stress application of one-second duration followed by a one-second-rest period. The idea of the test is that the annulus of unloaded material provides a level of confinement to the loaded part of the material. The European Standard prEN 12697: Part 21: 1998 contains the procedure to carry out the test. Previous research and test results reported a good correlation between the Swedish indentation and the wheel tracker test as will be discussed in section 3.10.2.

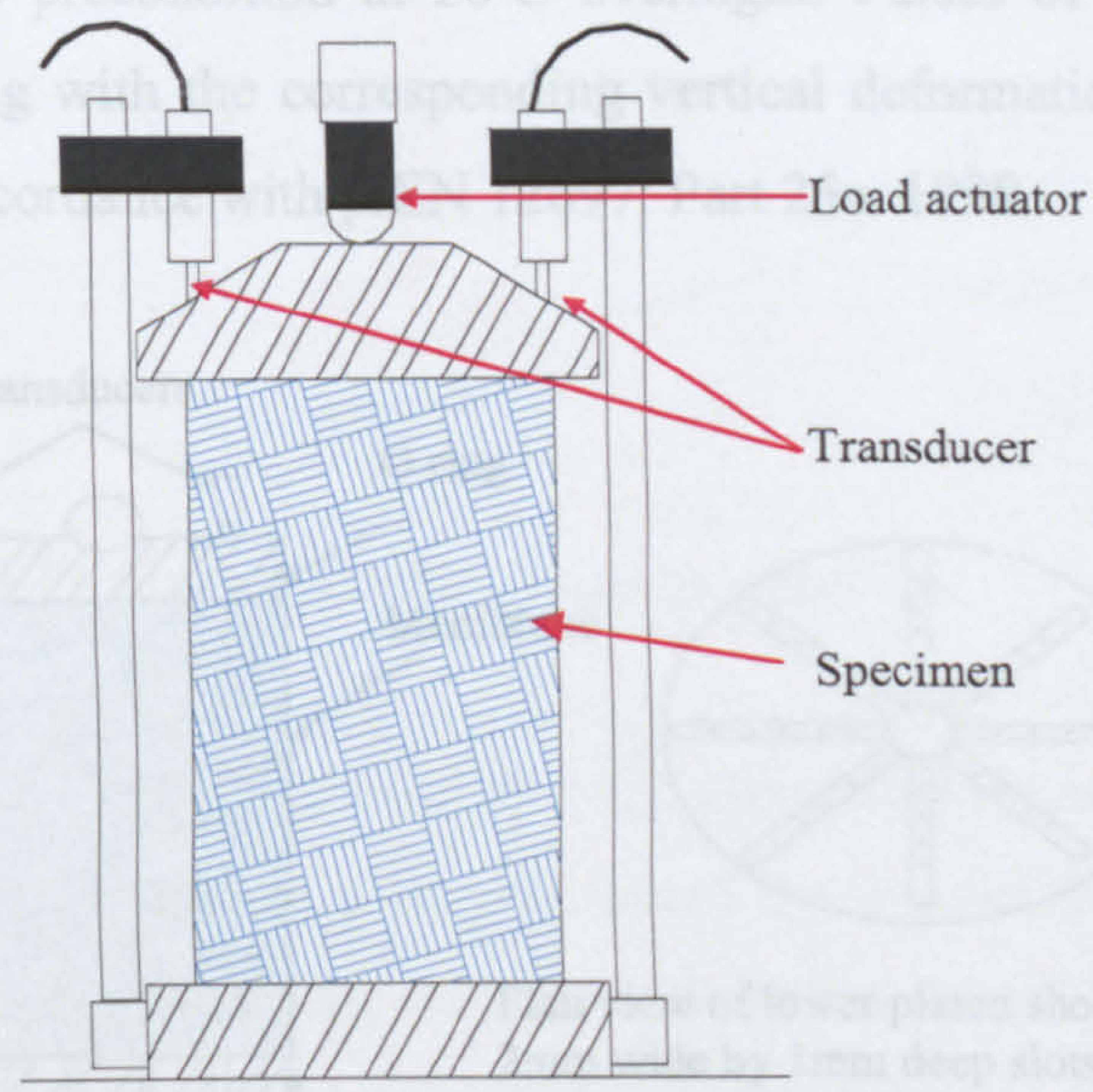


Fig. 3.6 - Apparatus used and specimen set-up adopted to carry out the repeated load axial test and the uniaxial creep test

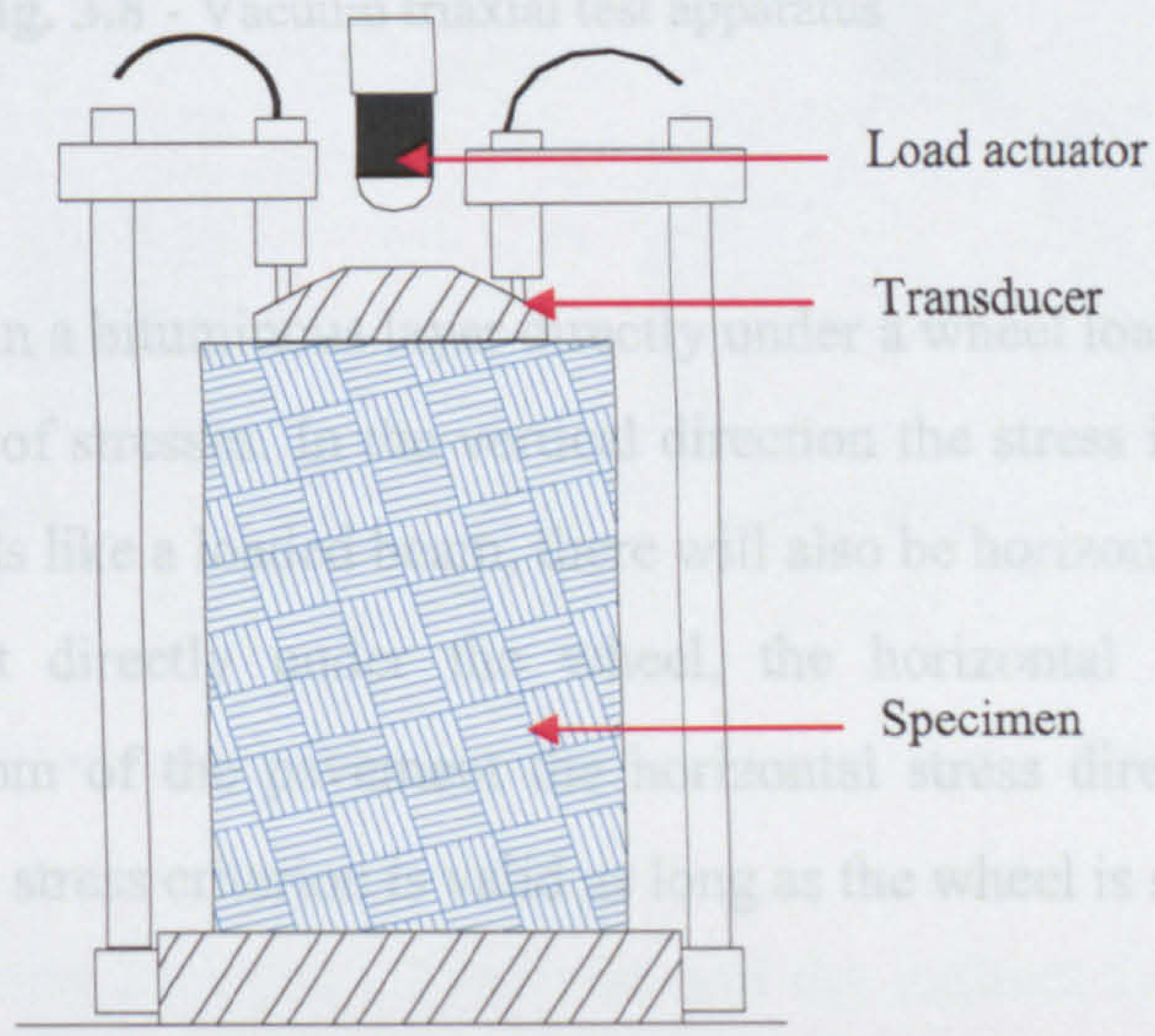


Fig. 3.7 - Swedish indentation test

3.9.5 Vacuum triaxial test

In this test the specimen is enclosed in a membrane sealed with two end platens as shown in Fig. 3.8. The lower platen is perforated and designed to allow a vacuum to be applied. This causes a pressure differential between the outside and inside of the membrane which applies confining pressures of < 1bar). However, initial work has shown that even 0.7bar is rather high and more practical levels are < 0.5bar (Cooper K.E. 1998).

The specimens are left to precondition at 20°C overnight. Pulses of vertical load is applied and recorded along with the corresponding vertical deformation. The vacuum triaxial is carried out in accordance with prEN 12697: Part 25a: 1998.

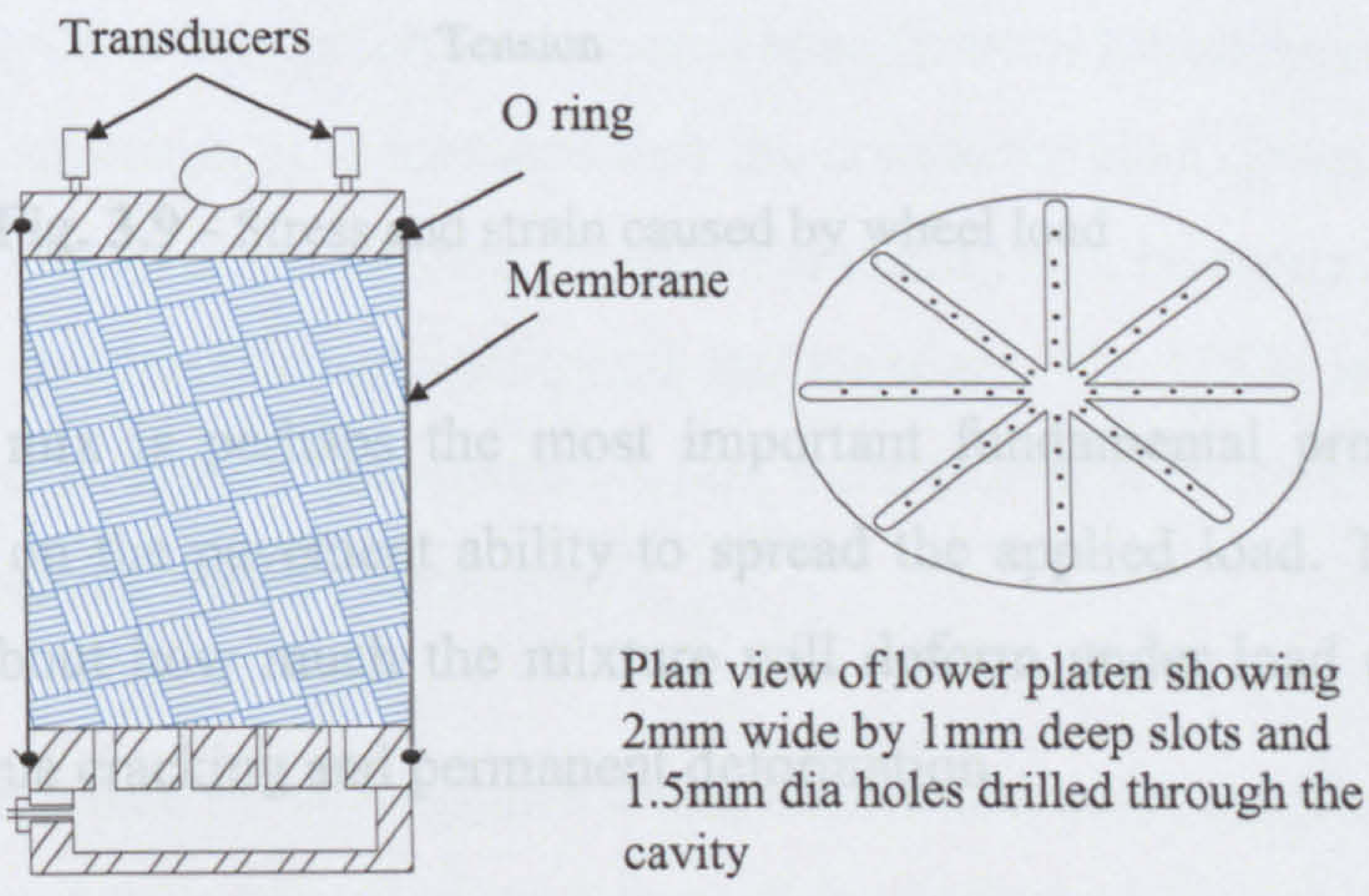


Fig. 3.8 - Vacuum triaxial test apparatus

3.9.6 Stiffness

The stresses, which occur in a bituminous layer directly under a wheel load, comprise of a series of different types of stresses. In the vertical direction the stress is compressive and because the layer bends like a loaded beam, there will also be horizontal stresses. At the top of the pavement directly under the wheel, the horizontal stress will be compressive. At the bottom of the pavement the horizontal stress directly under the wheel will be tensile. This stress criterion is valid as long as the wheel is stationary.

Fig. 3.9 shows that as the wheel moves along a pavement there is a stress distribution pattern. As the diagram below shows that horizontal stresses alternate between compression and tension. The strain in the material will change in a similar manner.

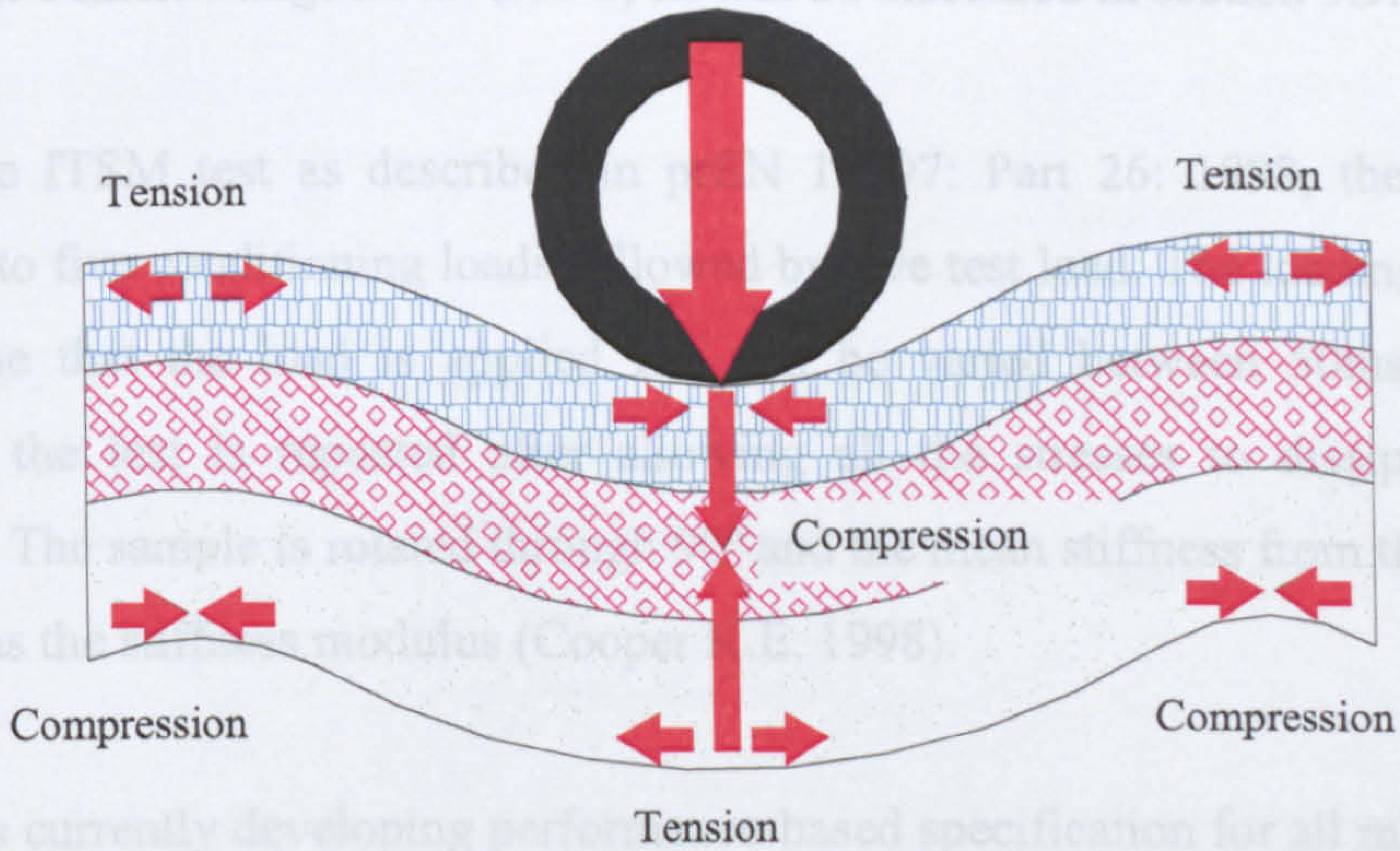


Fig. 3.9 - Stress and strain caused by wheel load

The stiffness of the mix is perhaps the most important fundamental property as it provides information on the pavement ability to spread the applied load. This in turn gives an indication about how much the mixture will deform under load (Fig. 3.10). This is related to fatigue cracking and permanent deformation.

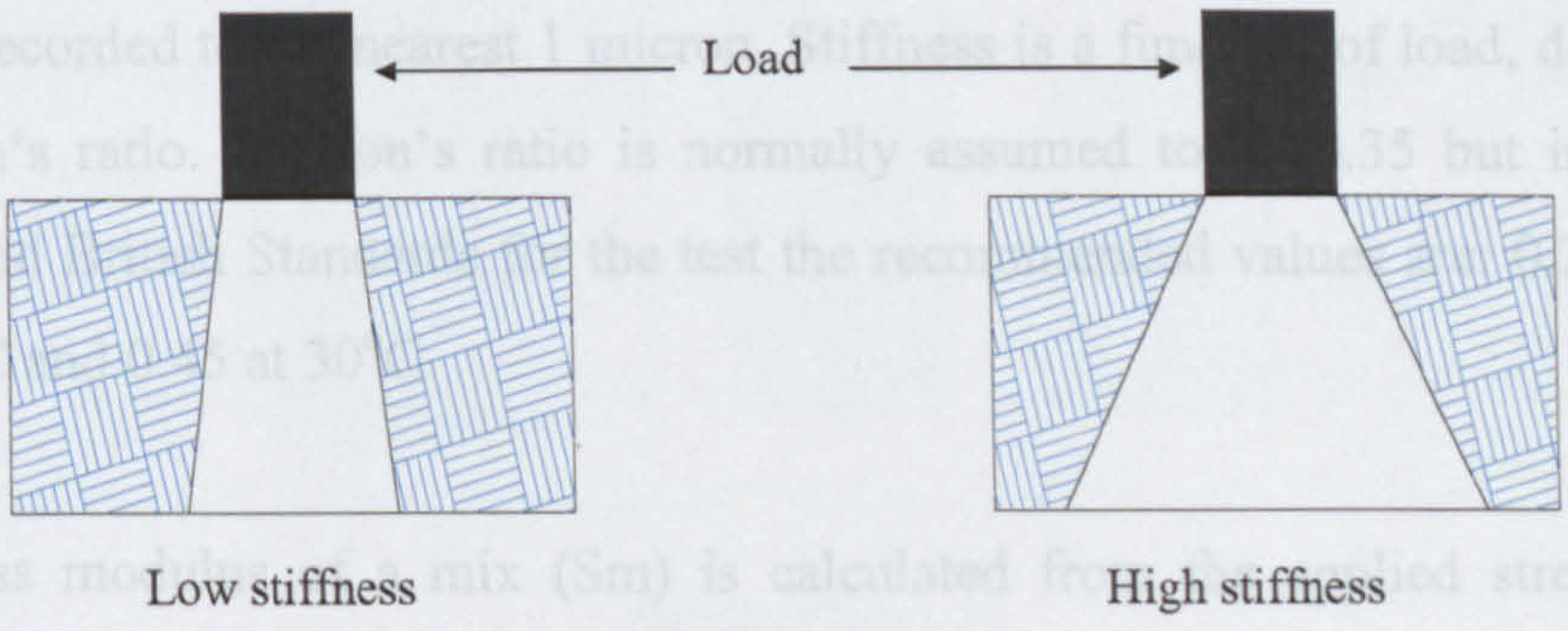


Fig. 3.10 - Load spreading ability as effected by pavement stiffness

The Indirect Tensile Stiffness Modulus (ITSM) test and the Indirect Tensile Stiffness Test (ITST) are two of the new methods recommended by the new European Standards to determine pavement stiffness. However, care must be taken to ensure that the specimen is at a correct temperature as an error of 1°C could result in a 10% difference in stiffness (Cooper K.E. 1998). The difference in the two tests is the target horizontal stress load can be altered in the ITST to coincide with the horizontal stresses applied in the Indirect Tensile Fatigue Test (ITFT) as will be discussed in section 3.9.8.

During the ITSM test as described in prEN 12697: Part 26: 1998, the specimen is subjected to five conditioning loads followed by five test load. The loading time, which is the time that the load is applied for, can be varied between 50ms and 160ms. Normally the test is repeated after allowing all the stresses to dissipate from the specimen. The sample is rotated through 90° and the mean stiffness from the two tests is recorded as the stiffness modulus (Cooper K.E. 1998).

The UK is currently developing performance-based specification for all mixtures laid in the pavement. In this approach, it is intended that the contractor shall design a material that has to meet specified criteria and then compliance testing will be carried out during the contract to ensure that the laid material meets that standard. The ITSM is well suited to this work provided it can be demonstrated that measurements correlate well with other fundamental laboratory tests (Nunn 1997).

In the ITSM and ITST a load pulse is applied along the vertical diameter (Fig. 3.11) of a cylindrical specimen and the peak transient deformation along the horizontal diameter is accurately recorded to the nearest 1 micron. Stiffness is a function of load, deformation and Poisson's ratio. Poisson's ratio is normally assumed to be 0.35 but in the draft European and British Standards for the test the recommended values are: 0.25 at 10°C, 0.35 at 20°C and 0.45 at 30°C.

The stiffness modulus of a mix (Sm) is calculated from the applied stress and the resultant strain (University of Nottingham 1998/9). Fig. 3.12 shows the determination of stiffness modulus of a mix.

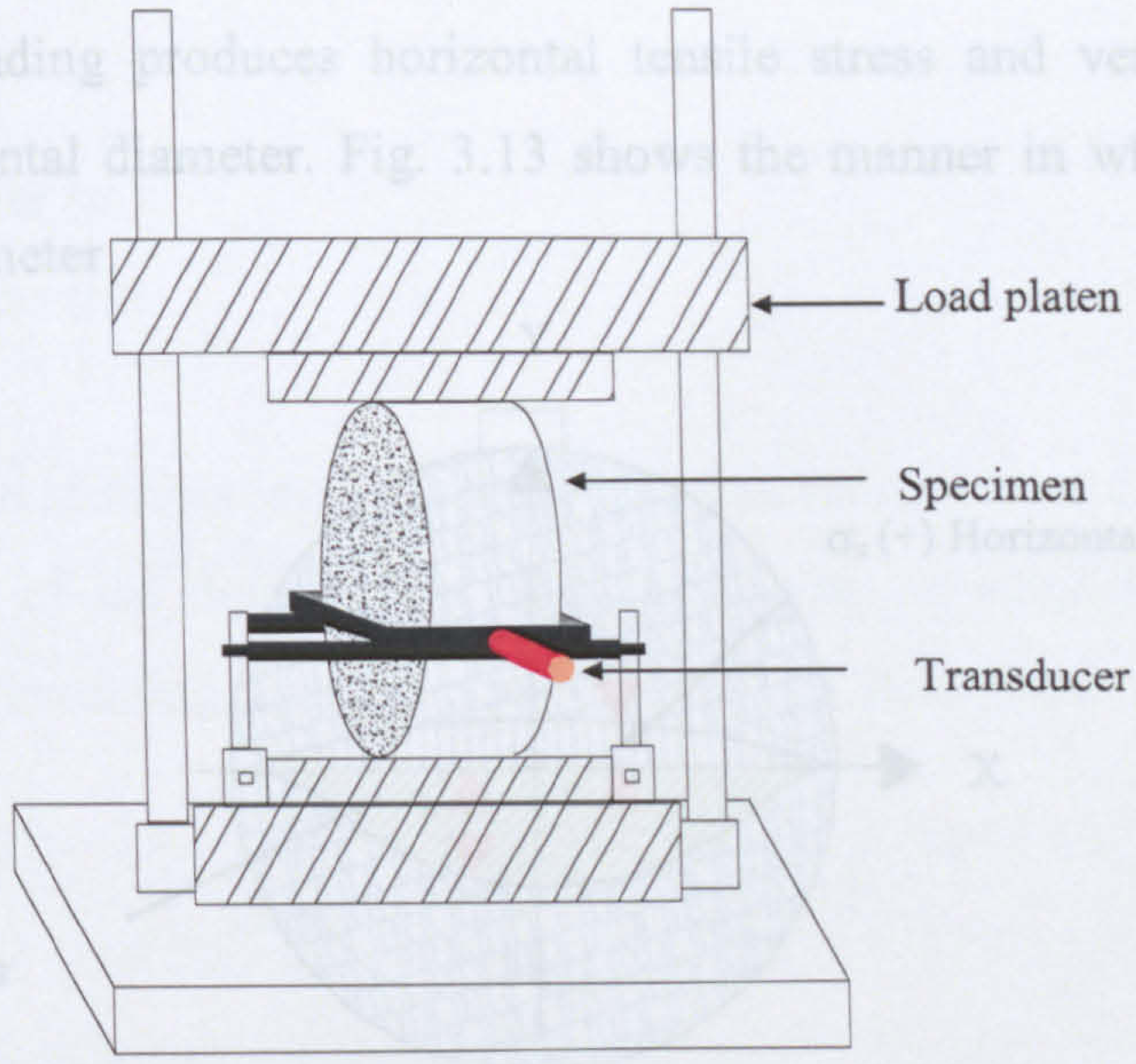
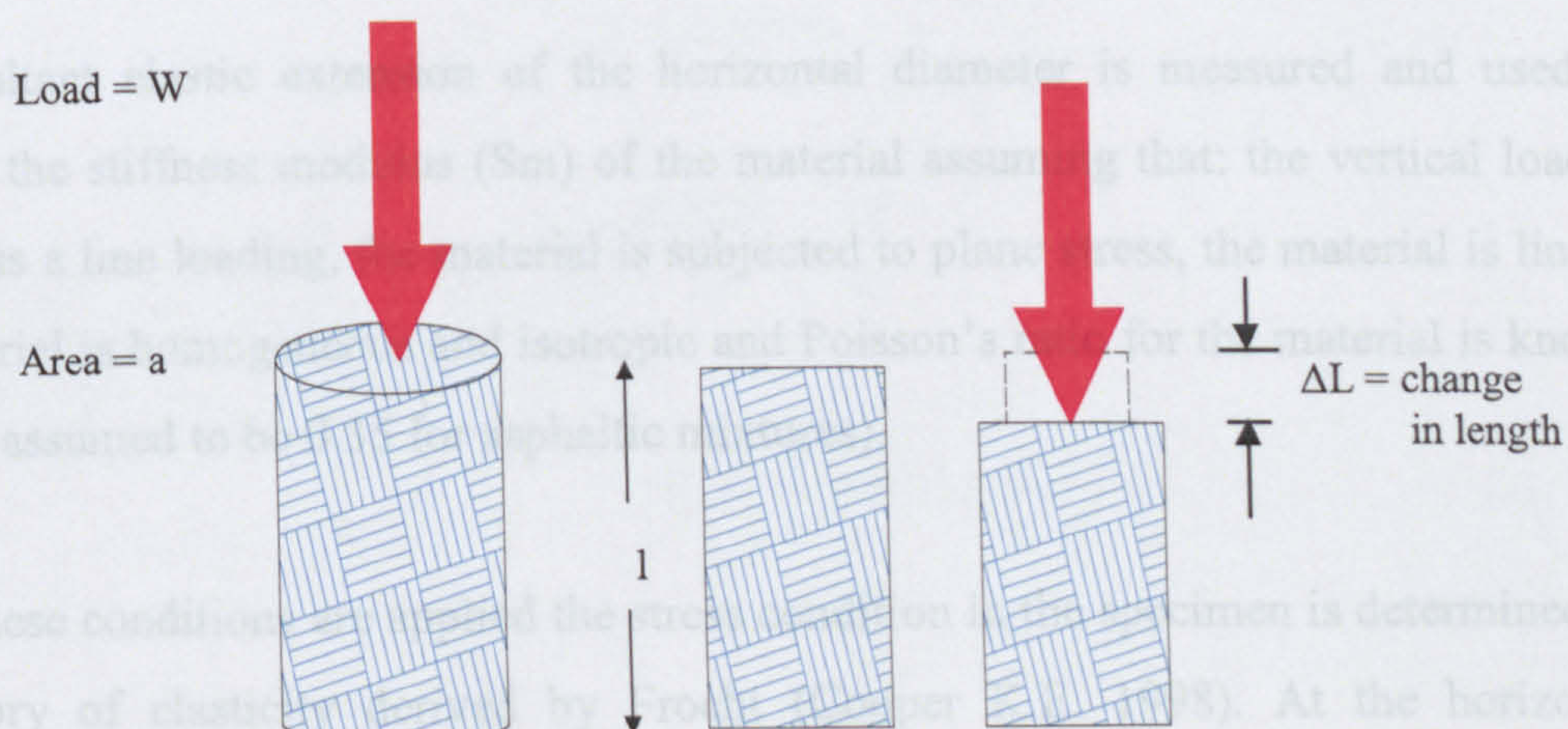


Fig. 3.11 - ITSM and ITST jig and test set-up



$$\text{Stress } (\sigma) = \frac{W}{a} \qquad \textbf{Eq. 3.3}$$

$$\text{Strain } (\varepsilon) = \frac{\Delta L}{l} \qquad \textbf{Eq. 3.4}$$

$$\text{Stiffness of mix (Sm)} = \frac{\text{Stress}}{\text{Strain}} \qquad \textbf{Eq. 3.5}$$

Fig. 3.12 - Determination of stiffness modulus of the specimen

3.9.7 Stiffness modulus theory

In the diametrical method of measurement of the stiffness modulus of the pavement mixtures a repeated line loading is applied across the vertical diameter of a cylindrical specimen. This loading produces horizontal tensile stress and vertical compressive stress at the horizontal diameter. Fig. 3.13 shows the manner in which these stresses vary across the diameter.

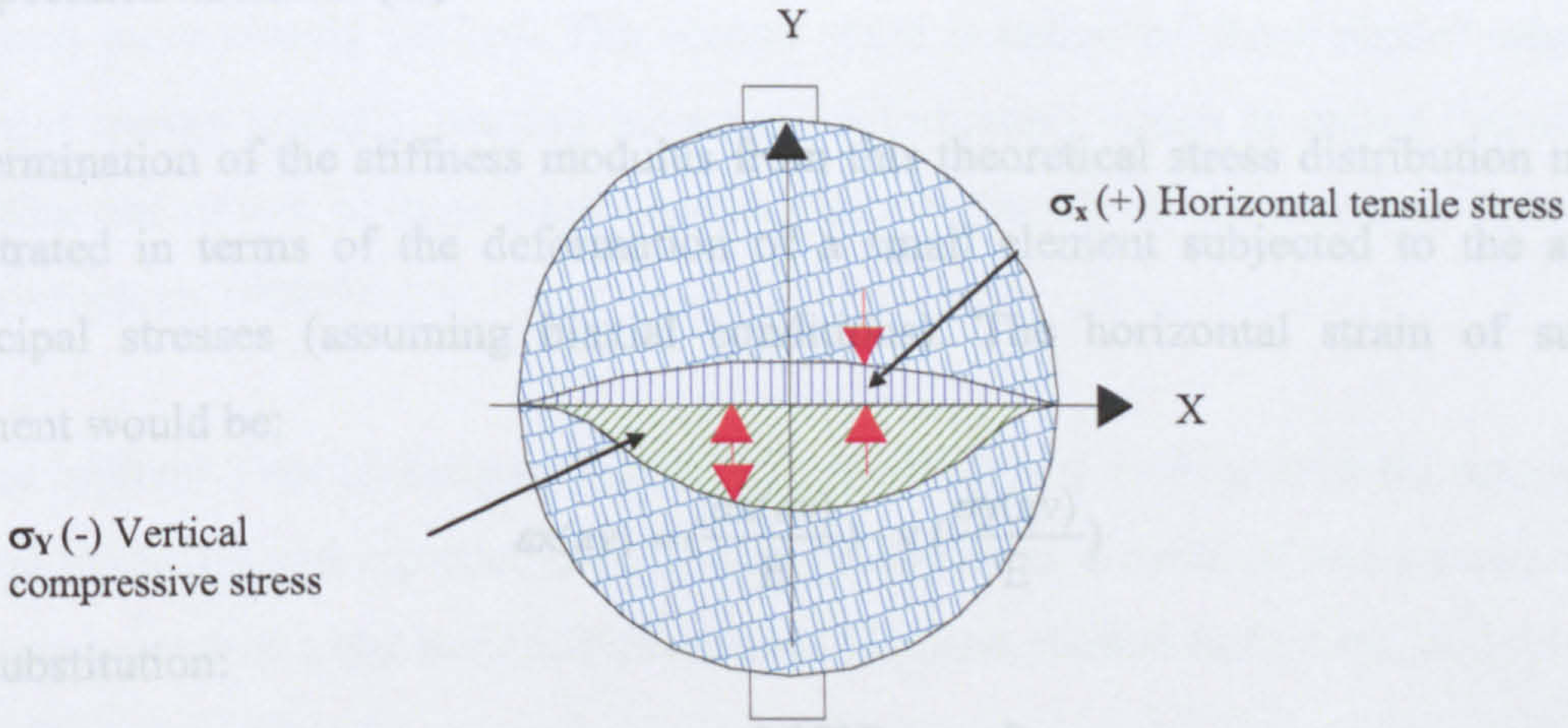


Fig. 3.13 - Distribution of stress in the indirect tensile specimen (Cooper K.E. 1998)

The resultant elastic extension of the horizontal diameter is measured and used to estimate the stiffness modulus (Sm) of the material assuming that: the vertical load is applied as a line loading, the material is subjected to plane stress, the material is linear, the material is homogeneous and isotropic and Poisson's ratio for the material is known (usually assumed to be 0.35 for asphaltic mixtures).

When these conditions are applied the stress condition in the specimen is determined by the theory of elasticity derived by Frocht (Cooper K.E. 1998). At the horizontal diameter the maximum and average stresses in the X and Y directions are as follows:

$$\sigma x(\max) = \frac{2P}{\pi dt} \qquad \textbf{Eq. 3.6}$$

$$\sigma y(\max) = \frac{-6P}{\pi dt} \qquad \textbf{Eq. 3.7}$$

$$\sigma x(av) = \frac{0.273P}{dt} \qquad \textbf{Eq. 3.8}$$

$$\sigma y(av) = \frac{P}{dt} \qquad \textbf{Eq. 3.9}$$

Where:

P = vertical load (N)

d = specimen diameter (m)

t = specimen thickness (m)

Determination of the stiffness modulus from this theoretical stress distribution may be illustrated in terms of the deformation of a small element subjected to the average principal stresses (assuming biaxial conditions). The horizontal strain of such an element would be:

$$\varepsilon x(av) = (\frac{\sigma x(av)}{E}) - v\,(\frac{\sigma y(av)}{E})$$

by substitution:

$$\varepsilon x(av) = (\frac{0.273P}{Edt}) - (\frac{vP}{Edt})$$

horizontal deformation:

$$\Delta = \varepsilon x(av)d$$

therefore:

$$\Delta = \frac{0.273P}{Et} + \frac{vP}{Et}$$

therefore:

$$E = \left(\frac{P}{\Delta t}\right)(0.273 + v) \qquad \textbf{Eq. 3.10}$$

Where:

E = stiffness modulus (Sm) of the material (kPa)

v = Poisson's ratio

3.9.8 Fatigue test

The use of the Indirect Tensile Fatigue Test (ITFT) is an attractive method of carrying out fatigue testing as the method is easy to carry out and no special specimen preparation techniques are necessary. The test is carried out in accordance with prEN 12697: Part 24: 1998.

In bituminous pavements, fatigue is the phenomena of cracking. It consists of two main phases, crack initiation and crack propagation which is caused by tensile stresses generated in the pavement not only by traffic loading but also by temperature variations and construction practices (Woodward 1996). It has been suggested that fatigue could occur in two stages as illustrated in Fig. 3.14. The first stage is densification, which could be considered as a secondary compaction stage where the aggregate skeleton becomes more closely packed. The second stage is called a "shear phase" where the material moves laterally causing pavement deformation which is called "shoulders". Whether one or two of these stages occurs depends largely on the mix type (University of Nottingham 1998/9).

In the Indirect Tensile Fatigue Test (ITFT) as illustrated in Fig. 3.15 the specimen is placed in the jig and a predetermined load is applied at a cycle of one second applied and one second of a rest period. The number of pulses it takes before the sample fails is recorded.

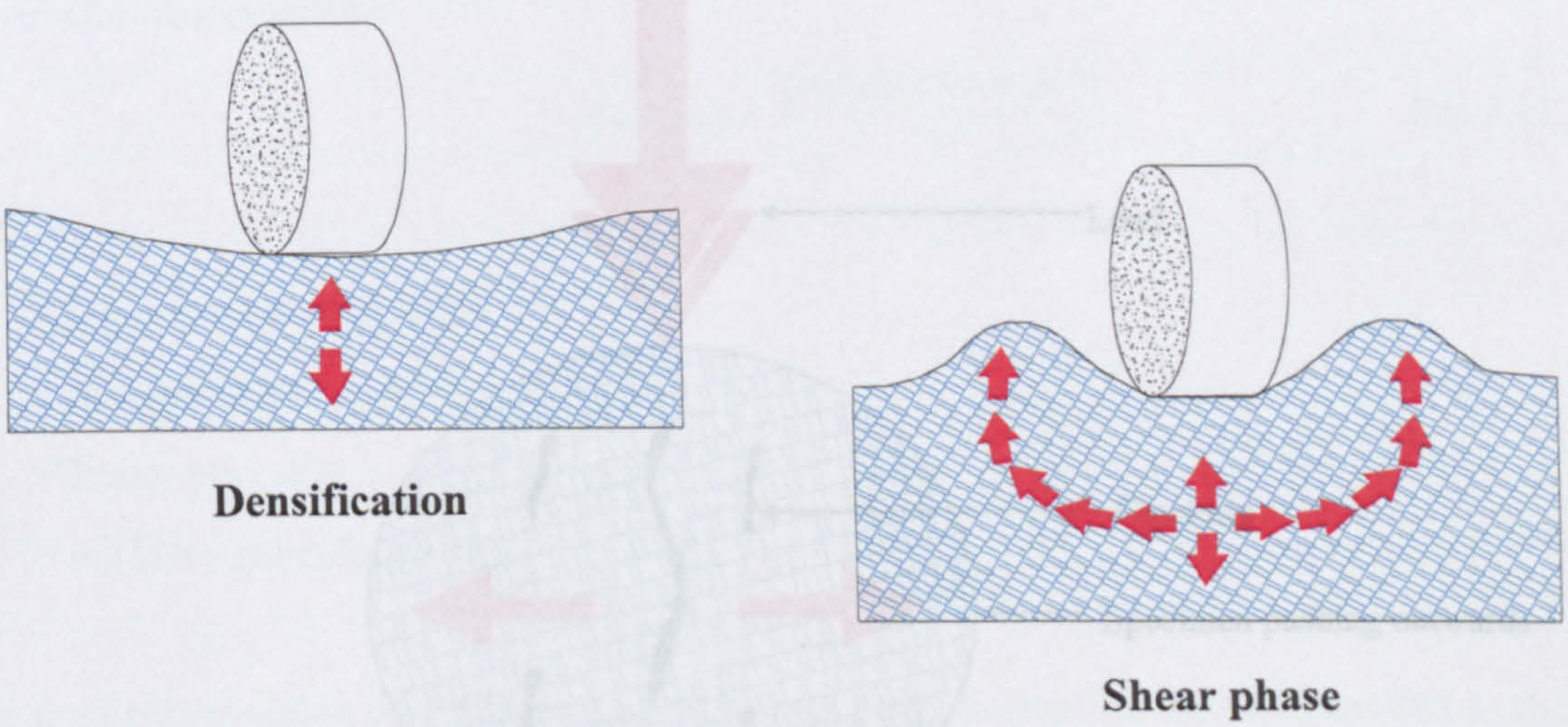


Fig. 3.14 - Permanent deformation in two stages

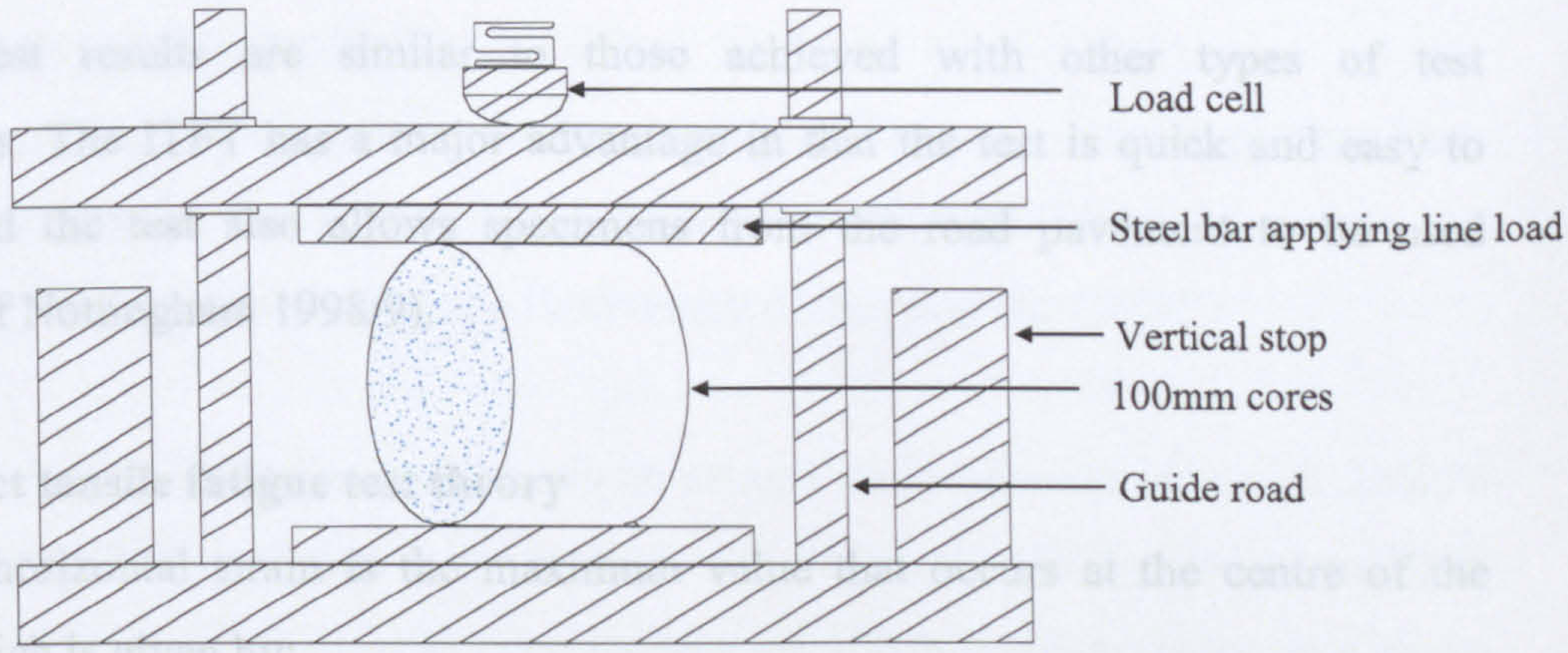


Fig. 3.15 - Diagram of ITFT apparatus and test set-up

The principle of the ITFT test is illustrated in Fig. 3.16. A repeated load is applied along the vertical diameter of a specimen generating repeated applications of an indirect tensile stress and accordingly strain along the horizontal diameter. This regime will eventually result in the initiation of a fatigue crack along the vertical diameter and, if continued the specimen will be split in two parts.

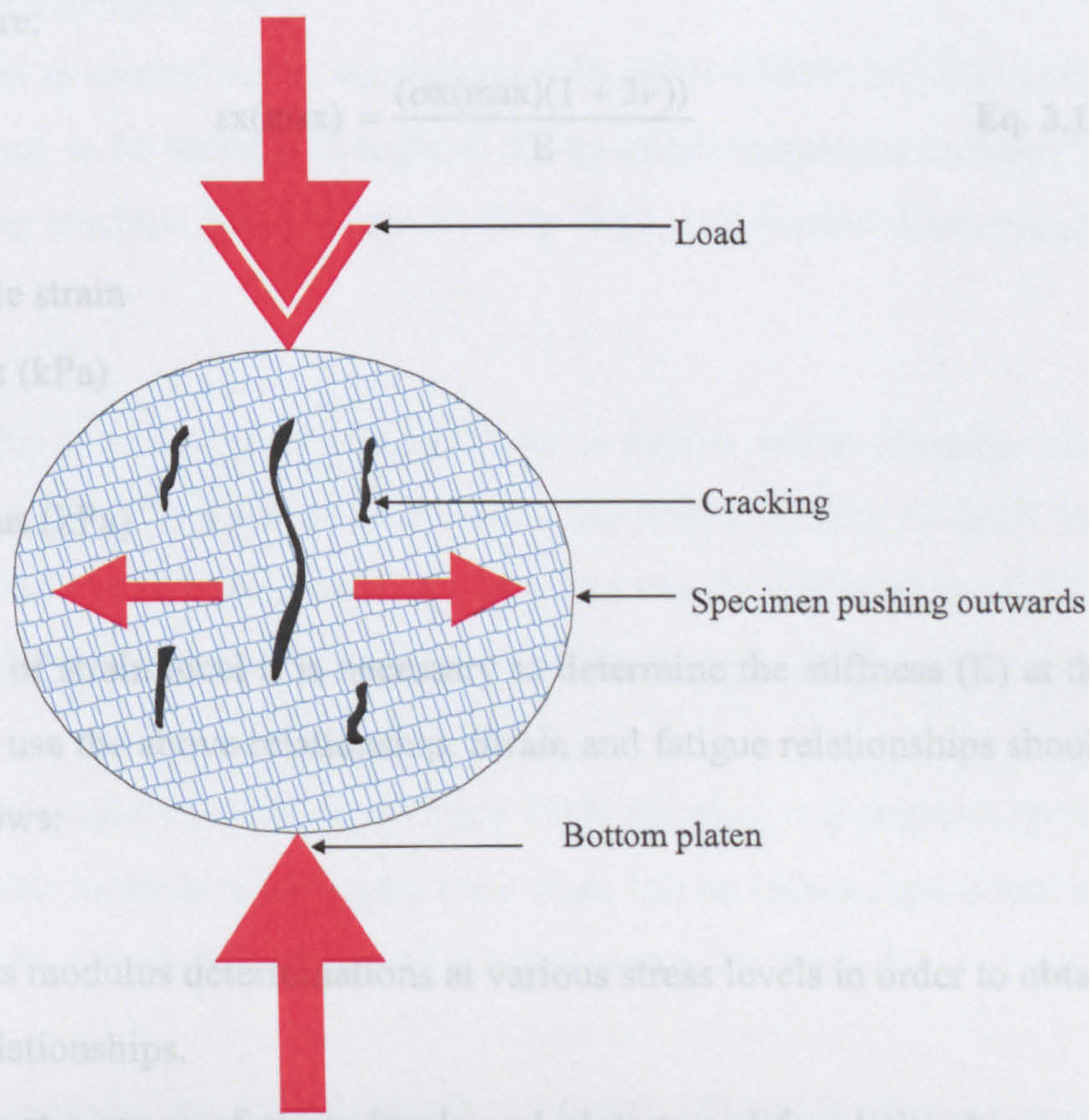


Fig. 3.16 - ITFT principle

Although the stress reversal, which is applied in actual practice, is not as intensive as in the ITFT, test results are similar to those achieved with other types of test configurations. The ITFT has a major advantage in that the test is quick and easy to carry out and the test also allows specimens from the road pavement to be used (University of Nottingham 1998/9).

3.9.9 Indirect tensile fatigue test theory

The critical horizontal strain is the maximum value that occurs at the centre of the specimen which is given by:

$$\varepsilon x(\max) = \left(\frac{\sigma x(\max)}{E}\right) - v\left(\frac{\sigma y(\max)}{E}\right)$$

by substitution:

$$\varepsilon x(\max) = \left(\frac{2P}{\pi dtE}\right) - v\left(\frac{-6P}{\pi dtE}\right)$$

therefore:

$$\varepsilon x(\max) = \frac{(2P(1+3\nu))}{\pi dtE}$$

by substitution therefore:

$$\varepsilon x(\max) = \frac{(\sigma x(\max)(1+3\nu))}{E} \qquad \textbf{Eq. 3.11}$$

Where:

εx = horizontal tensile strain

σx = horizontal stress (kPa)

v = Poisson's ratio

E = stiffness modulus (kPa)

For the determination of strain level it is necessary to determine the stiffness (E) at the appropriate stress and use the above relationship. Strain and fatigue relationships should be determined as follows:

- Carry out stiffness modulus determinations at various stress levels in order to obtain stress stiffness relationships.
- Carry out fatigue at a range of stress levels and plot stress life relationships on a log-log basis.

- Use the above equation to calculate maximum tensile strain and plot strain life relationships on a log-log basis
- Use linear regression analysis on the log strain/log life data in order to plot the best-fit line through the data (noting the correlation coefficient). Linear regression can be carried out on most excel sheets (University of Nottingham 1998/9).

The ITFT test is used to fatigue each specimen using a range of stress levels i.e. 200kPa –900kPa. Any stress that causes a specimen to fail before the 200th pulse is too high. Any stress, which causes the test to continue beyond 100,000 pulses, is too low as the whole series of tests will take too long.

When plotted as number of load applications to failure versus horizontal stress on log-log axes, the individual results should lie about a straight line. Always test at the same conditions of temperature and rise time. This will enable a database to be set up and the resistance to fatigue of different mixtures to be compared (Cooper K.E. 1998).

3.9.10 Indirect Tensile strength

The indirect tensile test is carried out in accordance with prEN 12697: Part 23: 1998. The cylindrical specimen to be tested is brought to the specified temperature, placed in the compression-testing machine between the loading strips, and loaded diametrically until a breakage occurs.

The compression testing machine, is of Marshall type or similar which complies with prEN 12697: Part 34: 1998. The machine must have a minimum capacity of 28kN and capable of applying loads to test specimens at a constant rate of deformation of 50 ± 2mm per minute. The jig is similar to that used in the indirect tensile fatigue test.

As the standard test temperature is low, an ordinary 28kN Marshall compression testing machine may not be able to produce sufficient load when 150 or 160mm specimens are tested. In such cases a high-load 40kN Marshall compression must be used.

The ITT is used to determine the indirect tensile strengths of asphalt mixtures at low and intermediate pavement temperature. These measurements can be used in performance prediction models such as superpave, to predict the intermediate temperature fatigue cracking potential of asphalt pavements (Turner 2002).

3.9.11 Cantabro test

This test, which is normally used in the design of porous asphalt, can be applied to assess the adhesion between binders and aggregate. The Cantabro test was developed to assess the cohesion of open bituminous mixtures in the laboratory. The cohesion of bituminous mixtures is achieved from a coating of the aggregate chippings and providing that there is good adhesion between the aggregate and the binder, the bituminous mix will have good resistance to wear.

The test procedure is simple and involves the preparing of samples in the Marshall hammer or Gyratory compactor. These samples are then placed in the Los Angles abrasion apparatus as can be seen in Fig. 3.17. The samples are weighed and then subjected to a number of revolutions, usually 300. The samples are taken out and weighed. The weight lost is expressed as a percentage of the initial weight and this can be calculated using the following formula:

$$P = \frac{100 \text{ x } (P1 - P2)}{P1} \qquad \textbf{Eq. 3.12}$$

Where:

P = the abrasion loss (%)

P1 = initial weight of sample (g)

P2 = final weight of sample (g)

3.10 Simulative Tests

As the stress conditions in a pavement loaded by a rolling wheel are extremely complex and cannot be precisely replicated in a laboratory test on a sample of asphalt. Simulative tests have been used to compare the performance of different mixtures (Shell 2001). The wheel tracker test is used to simulate this phenomenon. Before the wheel tracker test can be carried out samples have to be obtained. Two different methods of obtaining samples for the wheel tracker test are by taking cores from the road and by the production of slabs using the slab compactor. The second method was used in this investigation, as there was no road or trial strip laid using the mix, therefore cores could not be taken.

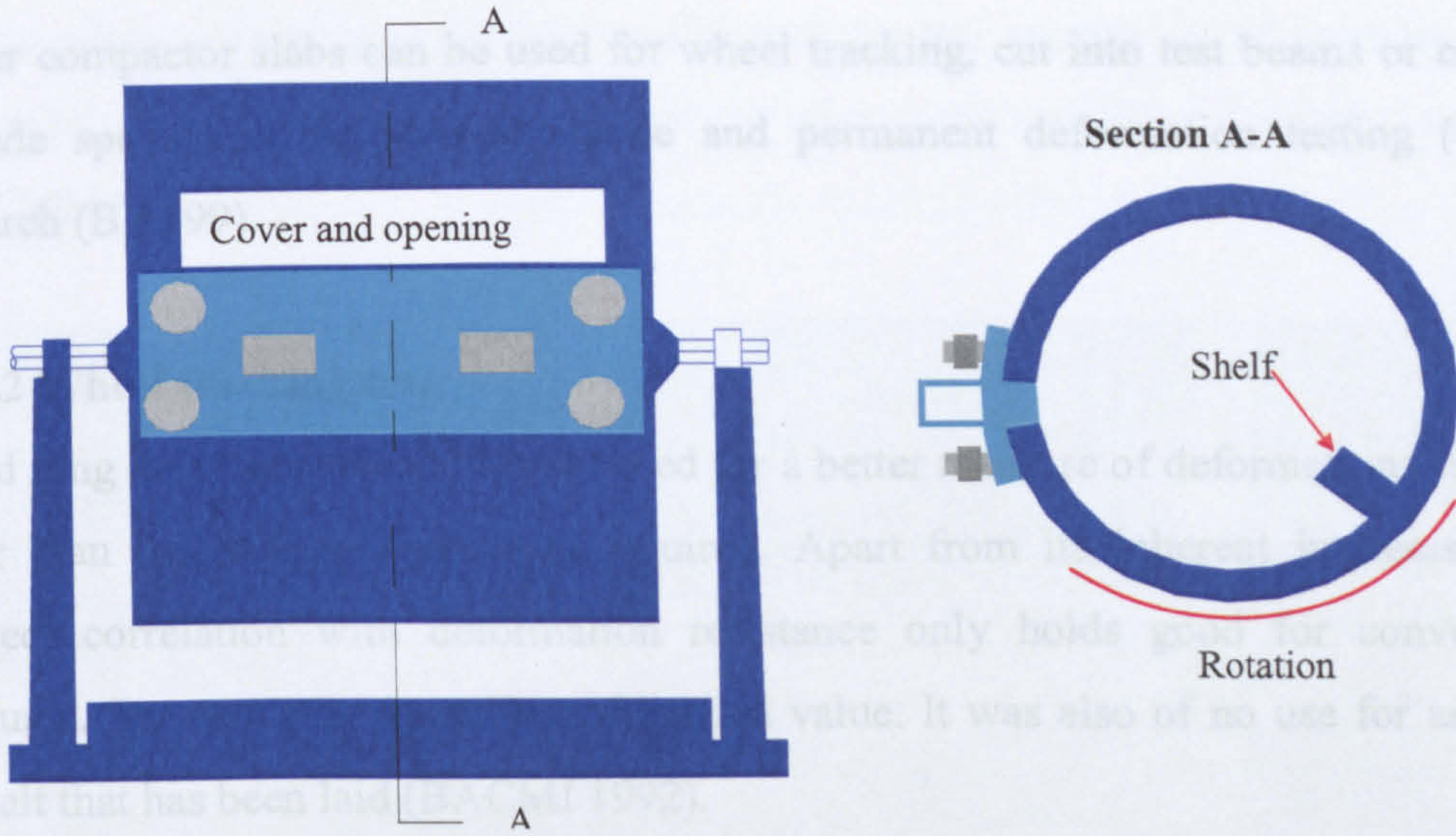


Fig. 3.17 - Diagram of Los Angles abrasion apparatus

3.10.1 Slab compactor method of producing slabs

The slab compactor (Fig. 3.18) provides a pneumatically powered means of compacting slabs of asphaltic material in the laboratory under conditions, which simulate insitu compaction. Four different levels of vertical force can be selected up to approximately 30kN. As the roller is 305mm wide, the compactive effort of the largest static site roller can be reproduced.

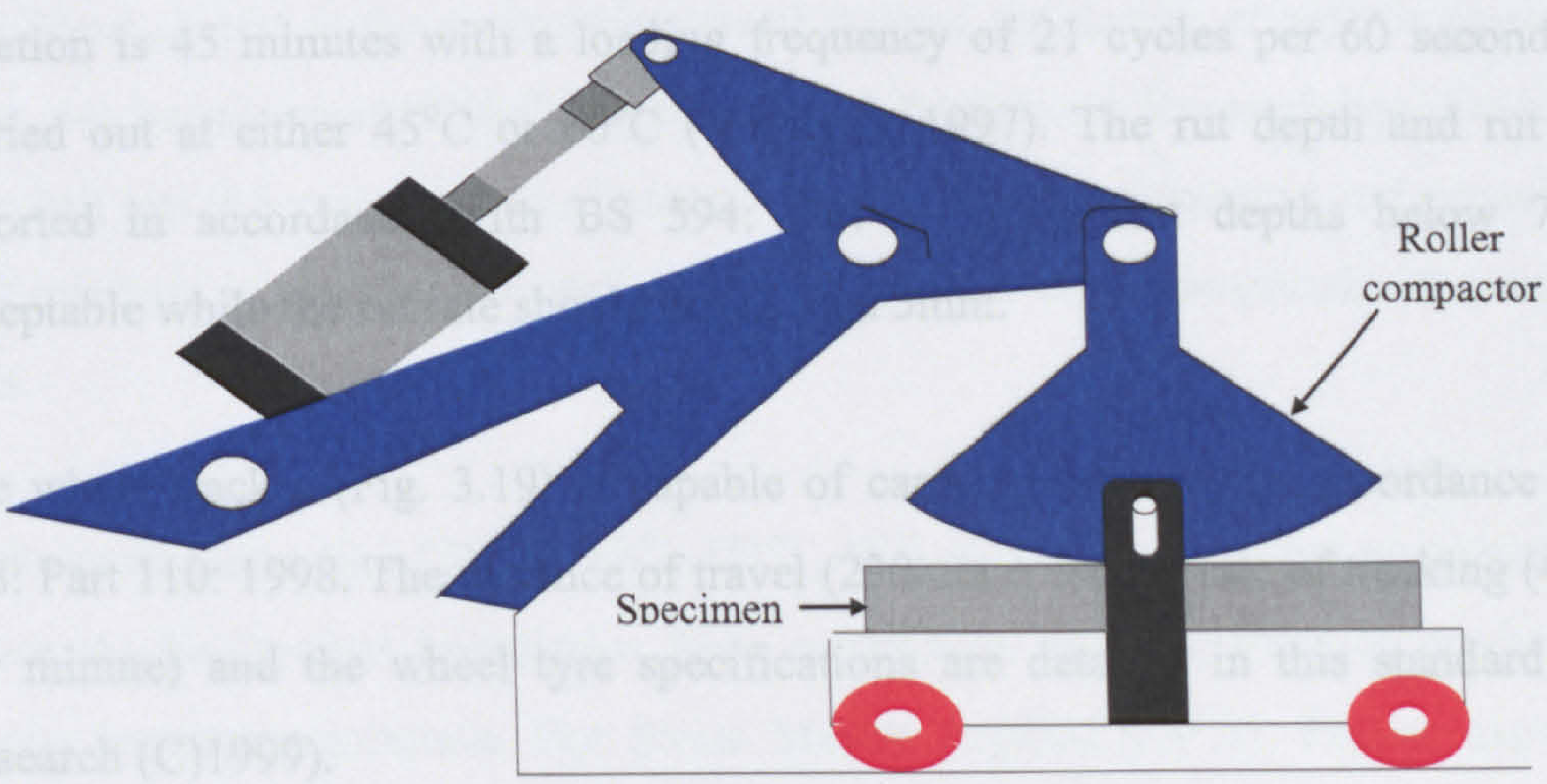


Fig. 3.18 - Roller compactor apparatus

Slabs produced with the roller compactor measure 305 by 305mm and from 50mm to 100mm thick. The precise depth of the roller can be set allowing the user to compact to a target density. The roller compactor method is included in the new European standards and is covered by prEN 12697: Part 33: 1998.

Roller compactor slabs can be used for wheel tracking, cut into test beams or cored to provide specimens for indirect tensile and permanent deformation testing (Cooper research (B)1999).

3.10.2 Wheel tracking test

It had long been recognised that the need for a better measure of deformation resistance other than the Marshall test was required. Apart from its inherent imprecision the indirect correlation with deformation resistance only holds good for conventional mixtures. The test was somewhat of limited value. It was also of no use for assessing asphalt that has been laid (BACMI 1992).

The simple reproduction of a loaded wheel passing over asphalt forms a convenient method of assessing how different mixtures perform. The laboratory scale facility, generally referred to as the 'Wheel Tracker' is a good discriminator between asphalts and is now included in a performance test for wearing courses on heavily trafficked roads. The laboratory facility comprises of a wheel of fixed dimensions and construction, loaded to 520N, which travels in a reciprocating motion over a constrained sample of asphalt, either in a laboratory manufactured slab or a 200mm core. The test duration is 45 minutes with a loading frequency of 21 cycles per 60 seconds and is carried out at either 45°C or 60°C (Whiteoak 1997). The rut depth and rut rate are reported in accordance with BS 594: Part 1: 1994. Rut depths below 7mm are acceptable while the rut rate should not exceed 5mm.

The wheel tracker (Fig. 3.19) is capable of carrying out tests in accordance with BS 598: Part 110: 1998. The distance of travel (230mm ± 5mm), rate of tracking (42 passes per minute) and the wheel tyre specifications are detailed in this standard (Cooper Research (C)1999).

Deformation is measured using Linear Variable Deformation Transducer LVDT having a nominal linear range of 20mm. The transducer amplifier provides an output such that deformation can be measured to accuracy better than ±0.05mm.

The wheel tracker test clearly correlates very closely with conditions in the road and should, when refined, prove a very useful tool for the assessment of resistance. It is particularly appropriate for assessment of the many modified binder's available (BACMI 1992).

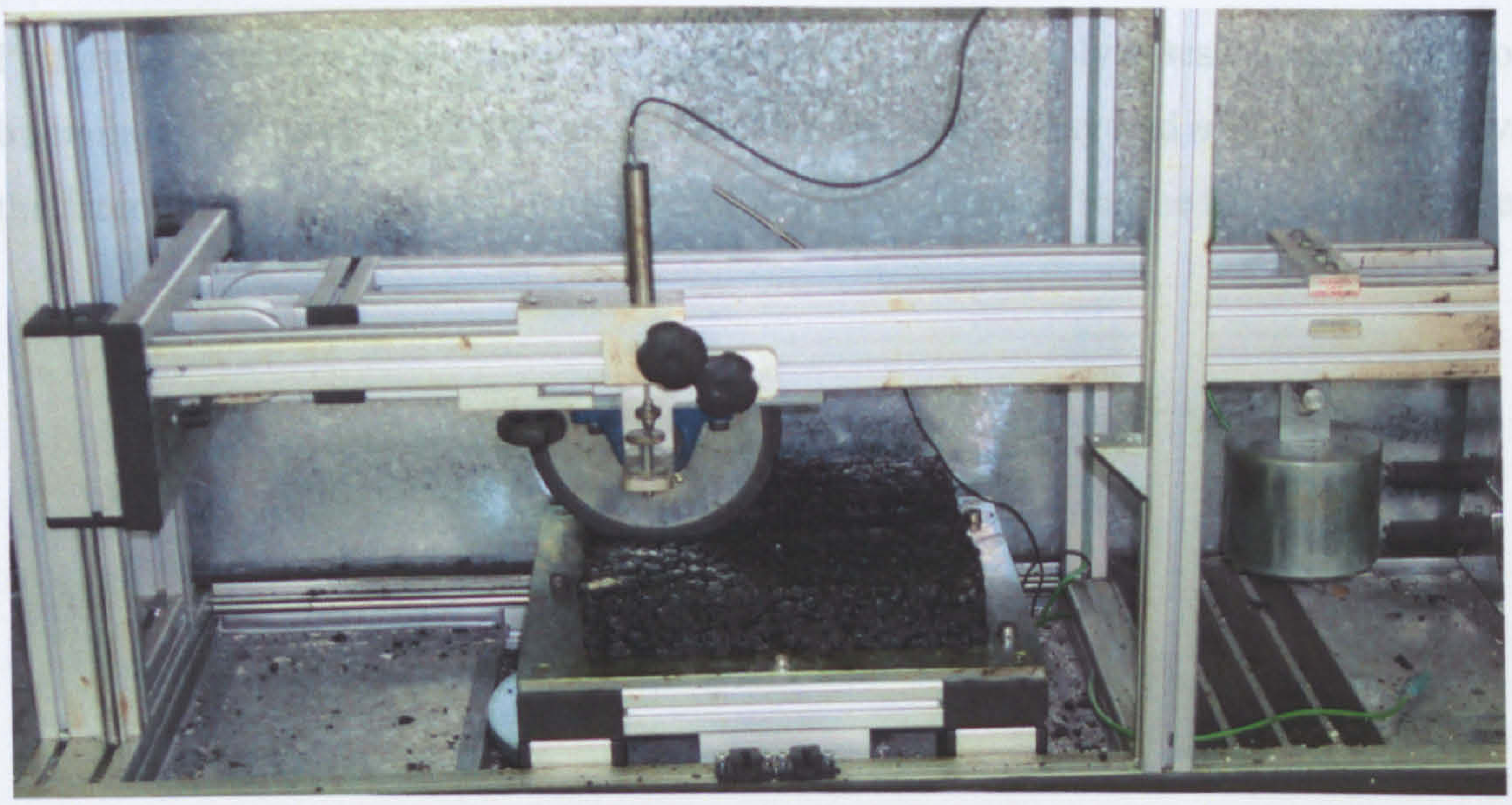


Fig. 3.19 - Wheel tracker apparatus

3.11 Overview

The Marshall test is the most widely used method for design of asphalt pavements. It must also be recognised that this method has it limitations. Research has shown that the Marshall test is a poor method of measuring permanent deformation and cannot reliably rank mixtures in order of their deformation resistance when compared with performance tests.

Performance tests like ITSM, ITST, ITFT, ITT, % wear and wheel tracker are slowly removing the 'recipe' boundaries from contractors enabling them to produce higher quality pavement mixtures like Stone Mastic Asphalt (SMA), Porous asphalt, high stiffness roadbases etc. The widespread acceptance of these tests has led to the inclusion of standards to carry out these tests in the new European standards. These standards also allow deformation resistance, stiffness and fatigue to be carried out on traditional asphalts, which are the most important parameters for these mixtures.

These more precise methods of testing and assessing the quality and viability of mixtures have given the designer the opportunity to use alternative materials in the design of new mixtures. Recycled Asphalt Pavements (RAP) can be added to design mixtures and assessed to determine whether the mix is a viable option.

Performance tests are a cheap, quick and easy way to accurately assess the properties of bituminous mixtures. Performance specification is here to stay and will no doubt pave the future in the design and construction of new and old roads for the foreseeable future.

CHAPTER 4 – EXPEIRMENTAL WORK

SECTION 1: Recycled Asphalt Pavements

4.1 General

Recycling road material has the potential to save money, energy and scarce resources. These advantages can be realised provided the in-service performance of the recycled material is similar to that of comparable new material.

4.2 Reclaimed Asphalt Pavements (RAP)

Reclaimed asphalt pavement were added to the virgin mixture at different percentages to determine the optimum amount of RAP that would yield the best results for new bituminous mixtures. At the start and before any mixing, the RAP was crushed to produce usable sizes. Two methods of crushing were investigated: hammer and cone crusher and jaw crusher.

4.2.1 Hammer and cone crusher

Samples of roadbase and basecourse were put through this system. This crusher was situated offsite and was mostly used for crushing stone cores. The maximum size of aggregate that the crusher could handle was 100mm. It was also found that the material had to be passed through the crusher twice to obtain a good grading. Three different grades of material were coming off the crusher 28-0mm, 14-0mm, and 5-0mm. Tests were carried out in accordance with BS 812: Part 102: 1999 and binder recovery tests were carried out in accordance with BS DD 250: 1999. Table 4.1 shows the results of sieve analysis and binder recovery test for the different RAP materials crushed.

The problems with using the hammer and cone crusher system are: crushing system used is of a low output therefore it could not be used for full-scale production, material had to be passed through the crusher twice to obtain a good grading, cost of purchasing this type of crushing system would make it less attractive to be used for recycled materials.

4.1.2 Jaw crusher

This crushing system was situated onsite and was previously used in the production of Clause 804 (75mm down 10mm nominal size). Instead of crushing roadbase and basecourse mixtures, a representative sample of RAP was taken from the stockpile and put through the crusher. The material was not screened as it was going to be used in different mixes. After crushing the material was bagged and brought back to the laboratory. Here the oversize material was extracted according to what mixture was being produced and two grades of RAP aggregate were obtained: 28-0mm and 20-0mm. Sieve analysis and recovered binder contents were determined for the two grades of RAP aggregate, the results of which can be seen in Table 4.2.

Table 3.1 - Sieve analysis and recovered binder from crushing RAP using hammer and cone crusher (offsite)

BS Sieve Size	28mm Roadbase (% passing)			20mm Basecourse (% passing)		
	28-0 mm	14-0mm	5-0mm	20-0mm	14-0mm	5-0mm
20	100	100	100	100	100	100
14	36.2	98	100	74.8	98.8	100
10	12.2	79.6	100	60.6	84.3	100
6.35	8.6	28.3	99.9	51.1	31.4	98.9
3.35	7.0	12.6	81.8	38.5	15.7	82.9
75 μm	1.0	1.2	4.0	2.5	1.1	4
Pan	0	0	0	0	0	0
(%) Recovered binder	1.32	2.34	5.42	3.78	4.89	6.32

The advantages of using this system of crushing are: crusher system can be used to produce RAP for mass production purposes; crushing system is onsite therefore transport costs are lower and cost of crushing can be calculated easier.

During the investigation into the use of RAP, a question arose about the consistency of the RAP material. As there was a substantial amount of RAP in a sister quarry (Quarry No. 2), samples were taken over a three month period and gradings done to determine differences in: grading from different ends of the stockpile and gradings from the two different quarries.

Table 4.2 - Sieve analysis and recovered binder from crushing RAP using jaw crusher (onsite)

BS Sieve Size	% Reclaimed asphalt planings 28-0mm	20-0mm
20mm	98.1	100
14mm	92.8	90.3
10mm	85.4	78.95
6.35mm	71.9	62.83
2.36mm	41.6	36
600μm	–	12.6
300 Um	–	6.3
150 μm	–	2.4
75 μm	4.9	0.6
Pan	0	0
Recovered binder	2.98	2.71

As the samples were taken, material from the stockpile was sold to customers, therefore a random sample could be taken easily. Fig. 4.1 shows how a typical random sample should be taken. The crusher that was used to crush this material was a jaw crusher also, which allowed us to correlate the results obtained for the two sources of RAP.

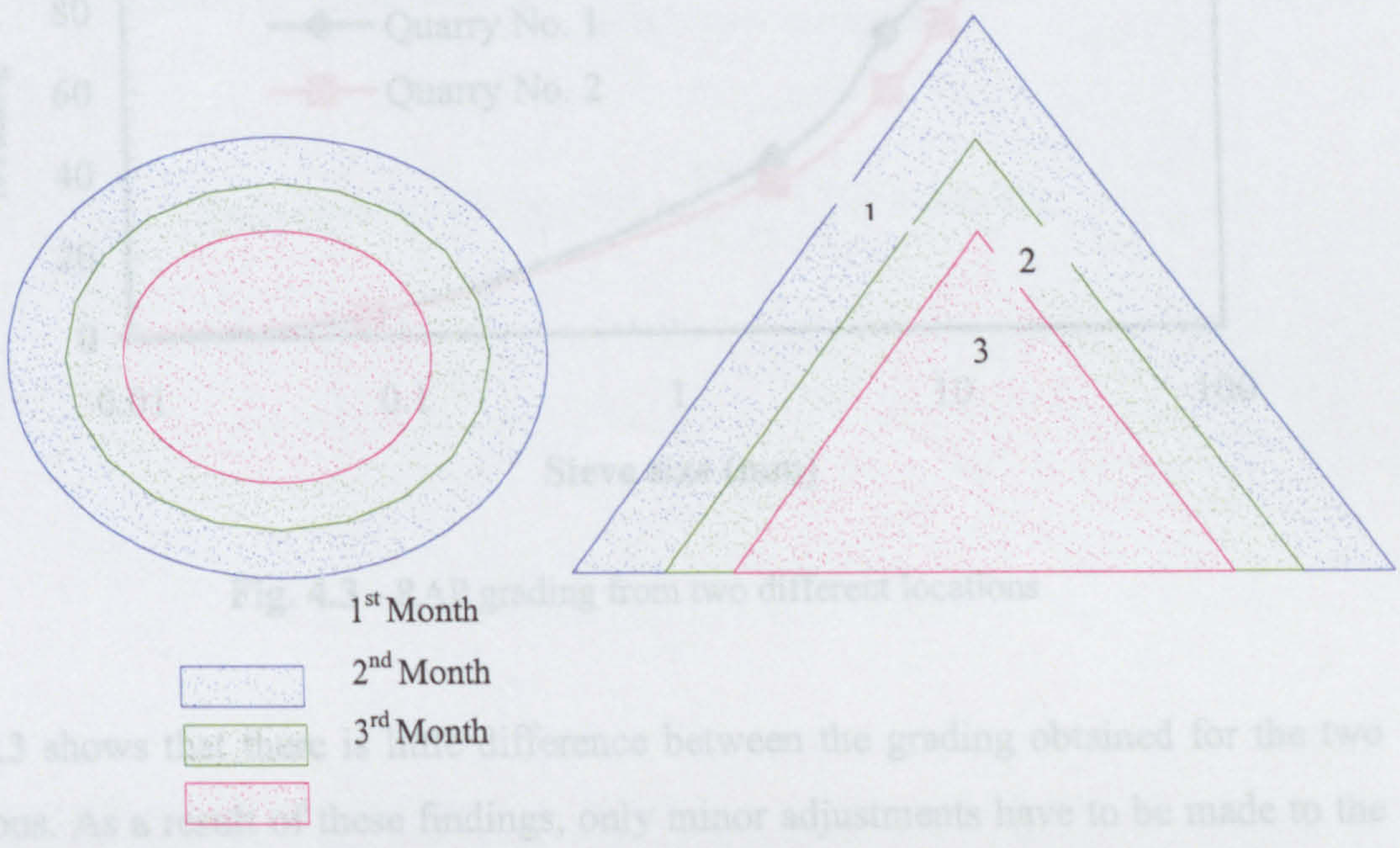


Fig. 4.1 - Random sampling from a stockpile

In order to determine the variability of the stockpile a graph was produced and the three grading curves drawn (Fig. 4.2). The grading obtained for the RAP material taken from the adjoining quarry (Quarry No.1) was compared with the average result for the material tested from the sister Quarry No. 2. Fig. 4.3 shows the comparison of these results.

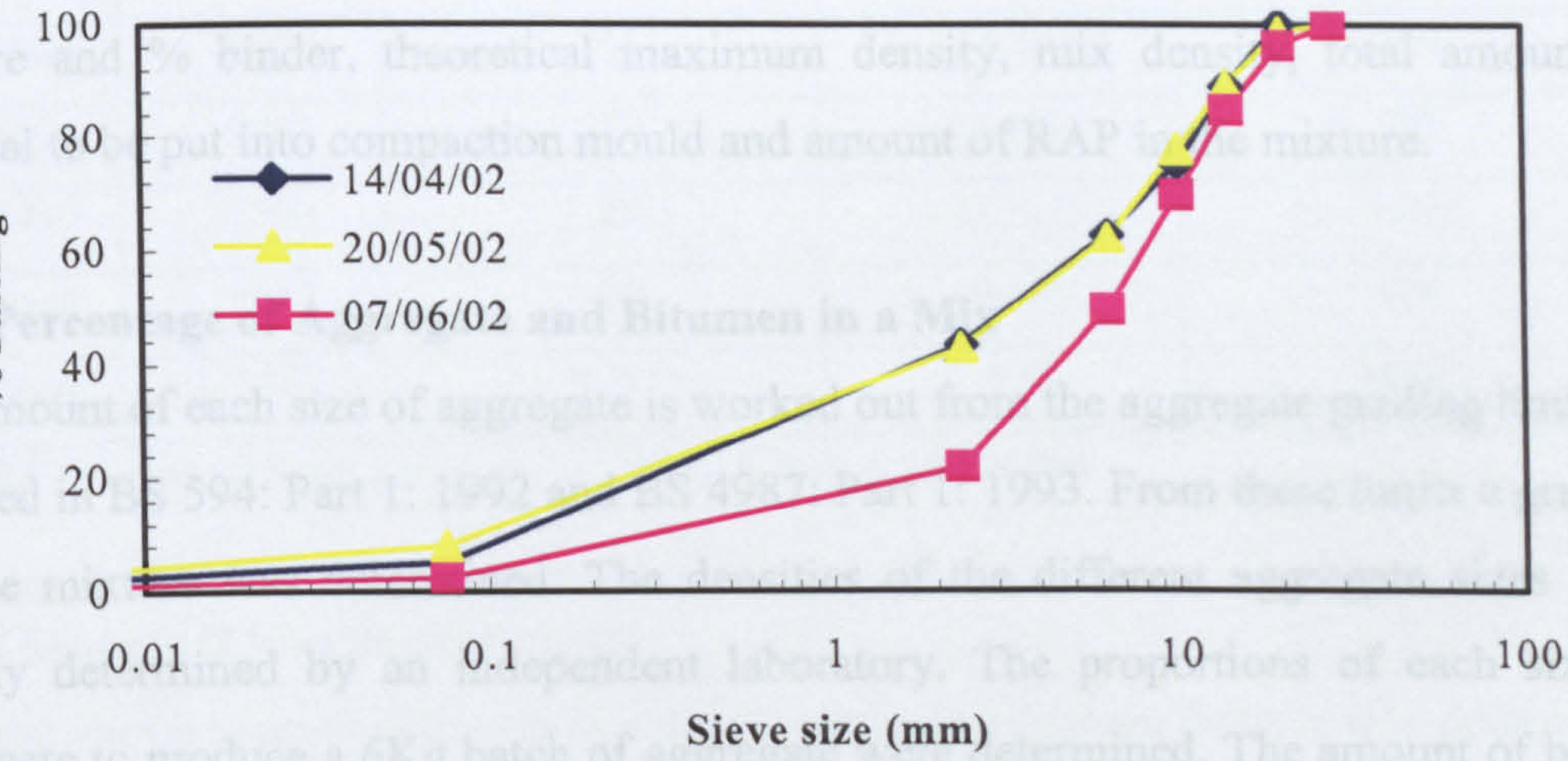


Fig. 4.2 - Grading for material taken from Quarry No. 2

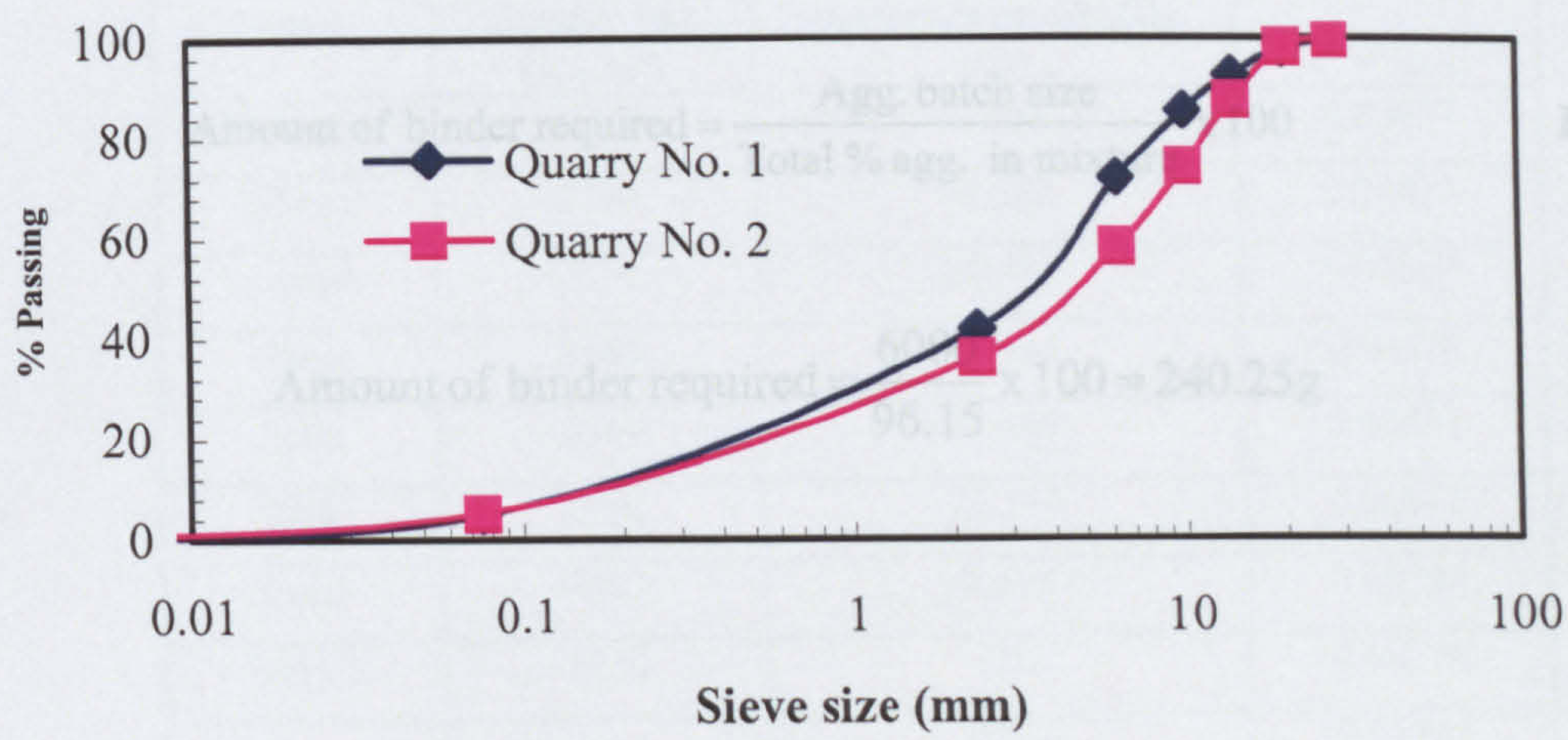


Fig. 4.3 - RAP grading from two different locations

Fig. 4.3 shows that there is little difference between the grading obtained for the two locations. As a result of these findings, only minor adjustments have to be made to the original mix design to produce similar bituminous mixtures. The effects of using this adjusted mix will be explored in more detail in the next chapter.

SECTION 2: Mix Design and Mixing

4.3 General

After determining the different properties of the RAP material, mix designs were established. The information required before blending of the materials to produce the different bituminous mixtures took place was: amount of each size of aggregate in the mixture and % binder, theoretical maximum density, mix density, total amount of material to be put into compaction mould and amount of RAP in the mixture.

4.4 Percentage of Aggregate and Bitumen in a Mix

The amount of each size of aggregate is worked out from the aggregate grading limits as outlined in BS 594: Part 1: 1992 and BS 4987: Part 1: 1993. From these limits a grading for the mixture was determined. The densities of the different aggregate sizes were already determined by an independent laboratory. The proportions of each size of aggregate to produce a 6Kg batch of aggregate were determined. The amount of binder needed to coat the mixture was then determined. This increases the batch size above 6Kg. The grading, amount of aggregate and bitumen required are presented in Table 4.3. The amount of binder required is calculated as follows:

$$\text{Amount of binder required} = \frac{\text{Agg. batch size}}{\text{Total \% agg. in mixture}} \times 100 \qquad \textbf{Eq. 4.1}$$

$$\text{Amount of binder required} = \frac{6000}{96.15} \times 100 = 240.25\text{g}$$

Table 4.3 Grading, amount of aggregate and bitumen required

Aggs and binder	Materials density (kg/m^3)	Aggs in bituminous mixture (%)	Agg and binder in mix (%)	Amount of agg & binder required (g)
28mm	2700	12	11.538	720
20mm	2700	12	11.538	720
14mm	2690	14	13.461	840
10mm	2660	16	15.384	960
6mm	2760	13	12.4995	780
CRF*	2700	23	22.1145	1380
Sand	2697	10	9.615	600
Binder	1030	N/A**	3.85	240.25

* CRF = Crushed Rock Fines ** N/A = Not Applicable

4.5 Theoretical Maximum Density

Once the above properties are known the theoretical maximum density was determined as can be seen in Table 4.4.

Table 4.4 - Calculation of theoretical maximum density

Aggs and binder	Materials density (A) (Kg/m^3)	Aggregate and other materials in mix (B) (%)	C = B/A
28mm	2700	11.538	0.00427
20mm	2700	11.538	0.00427
14mm	2690	13.461	0.00560
10mm	2660	15.384	0.00578
6mm	2760	12.499	0.00452
CRF	2700	22.114	0.00819
Sand	2697	9.615	0.00356
Binder	1030	3.85	0.00370
		ΣC =	0.03937
Theoretical density = $100/\Sigma C$		100 / 0.03937	2540Kg/m^3

4.6 Compacted Mix Density

After determining the theoretical maximum density, the % voids, which were incorporated in the mix, was addressed. The % voids in the mix was estimated at 6% as a compacted site density in the region of 93-97% would be acceptable. This in turn means that in order to accommodate the 6% voids the material must be compacted to 94% of the theoretical maximum density, therefore the mix density can be calculated by:

$$\text{Theoretical maximum density} = \frac{2540\,(100-6)}{100} = 2388\text{Kg/m}^3 \qquad \textbf{Eq. 4.2}$$

4.7 Amount of Bituminous Mixture to be placed in Mould

In order to determine exactly the amount of material required in the mould, information required for the height of the finished specimen and the radius of the mould were needed. It was decided that 100mm diameter moulds would be used and a finished height of 70mm as this is within the specimen height range set out in prEN12697: Part 26: 1998. From this information the volume could be determined:

$$\text{Volume of specimen} = \pi r^2 h \qquad \textbf{Eq. 4.3}$$

$$\text{Volume of specimen} = ((3.14)(50 \times 50)(70))$$

$$\text{Volume of specimen} = 549778.3\text{mm}^3$$

Now that the volume and mix density is known the amount of material to be put into the mould can be determined:

$$\text{Density} = \text{mass /volume}$$

Therefore:

$$\text{Mass} = \text{density x volume} \qquad \textbf{Eq. 4.4}$$

$$\text{Mass} = 2388 \text{ x } (549778.3\text{x}10^{-9})$$

$$\text{Mass} = 1.313\text{Kg}$$

4.8 Amount of RAP Added to Mix

After finding out the amount of material that is put into the mould, samples were made up and tested for the control mixtures i.e. the mixtures containing 0% RAP. The mix design to allow the use of different percentages of RAP would have to be altered. The proportions of the different aggregates would also change, as the RAP was not screened into single size aggregate. Table 4.5 shows how the different proportions were calculated for a roadbase mixture with 10% RAP.

The amount of the different sizes of RAP material in the mix can be seen in column number 3. As the RAP material was only screened into a 28mm down, the proportion of the different sizes of aggregate cannot be changed. Therefore the amount of virgin aggregate has to be adjusted to ensure that the overall grading was achieved. The amount of virgin aggregate for each of the different sizes is therefore calculated. An example calculation for 28mm virgin aggregate can be seen below.

$$\text{Amount 28mm virgin agg.} = ((\text{Batch size} \times \%\ 28\text{mm as a decimal}) - \text{amount of RAP agg.})) = (6000 \times 0.12) - 11 = 709\text{g} \qquad \textbf{Eq. 4.5}$$

Table 4.5 - Proportions of material required for roadbase with RAP added

Batch size			6000g		
Virgin Aggregate		90%	Amount of virgin Aggregate		5400g
RAP Aggregate		10%	Amount of RAP Aggregate		600g
Aggregate	**RAP grading (%)**	**Amount RAP (g)**	**Aggregate**	**Grading virgin aggregate (%)**	**Amount virgin aggregate (g)**
28mm	1.9	11	28mm	12	709
20mm	5.3	32	20mm	12	688
14mm	7.4	44	14mm	14	796
10mm	13.5	81	10mm	16	879
6mm	30.3	182	6mm	13	598
CRF	36.7	220	Sand	23	1160
<75μm	4.9	29	CRF	10	571
Total		600			5400

This equation is used for 20mm, 14mm and all the other different aggregate sizes. Another element considered was the amount of binder recovered and utilised. From BS DD 250: 1999 the recovered binder was 2.98% of the mix. The following formula was then used to determine the binder required:

$$\text{Binder required} = \frac{\text{Amount virgin agg. x 100}}{\text{100 - Binder content}} - \text{Amount virgin agg.}$$

$$+$$

$$\frac{\text{Amount RAP agg. x 100}}{\text{100 - Binder content}} - \text{Amount RAP agg.}$$

$$-$$

$$\frac{\text{Amount RAP agg. x 100}}{\text{100 - Recovered binder}} - \text{Amount RAP agg.} \qquad \textbf{Eq. 4.6}$$

By substitution:

$$\text{Binder required} = \frac{5400 \times 100}{100 - 3.85} - 5400 + \frac{600 \times 100}{100 - 3.85} - 600 - \frac{600 \times 100}{100 - 2.98} - 600$$

Therefore:

$$\text{Binder required} = 216.22 + 24.86 - 18.42 = 222.66\text{g}$$

A trial batch was produced and mixed for examination. During mixing it was visible that not all the aggregate was coated. This meant that not all the recovered binder was utilised. It was decided to reduce the amount of recovered bitumen utilised by 10% increments to see the improvements in the coating of the aggregate. It was found that utilisation of about 50% recovered bitumen was achievable. Therefore the amount of binder in the mix had to be increased in order to accommodate this. The equation for the calculation of the amount of bitumen was then altered to facilitate this. The revised equation can be seen below:

$$\text{Binder required} = \frac{5400 \times 100}{100 - 3.85} - 5400 + \frac{600 \times 100}{100 - 3.85} - 600 - \frac{600 \times 100}{100 - (2.98 \times 0.5)} - 600$$

Therefore:

$$\text{Binder required} = 216.22 + 24.86 - 9.21 = 231.87\text{g}$$

The theoretical maximum density and mix density for mixes containing RAP were assumed to be the same as the base mix. The reason for this assumption was the density of the RAP material would fluctuate more frequently than virgin materials. RAP materials could comprise of material from a number of different road projects. In turn the aggregates used to make the initial mixture for the different road project could have came from different quarries.

A mix design sheet for a typical roadbase containing 10% RAP can be seen in Table 4.6. A complete set of mix design charts for all the different mixtures at the different % of RAP can be seen in Appendix A.

4.9 Mixing of Materials

The materials were placed in the oven for three to four hours before mixing. Roadbase and basecourse mixtures were mixed at 155°C whereas, 30% Hot Rolled Asphalt (HRA) mixtures which is the predominately used in wearing course were mixed at 165°C. To ensure that the materials were mixed at their specific temperature the solid materials were heated to + 5°C above mixing temperature. The mixing of the material had to be carried out quickly so to ensure minimal loss of temperature. The procedure in which the materials were mixed is as follows:

(1) Take the mixing bowl, aggregate and bitumen out of the oven.
(2) Place half the aggregate into the mixing bowl followed by half the amount of bitumen then add in the other half of the aggregate and the bitumen.
(3) Place bowl into the mixer and mix the material for one minute.
(4) Inspect mix to see if it is mixed thoroughly, if not repeat Step 3.
(5) Empty the mixture into a tray and place back into oven and allow it to return to compaction temperature.

Table 4.6 - Calculation of mix proportions

28mm Roadbase				
Batch size (g)				**6000**
Virgin aggregate (%)	**90**	**Amount virgin Aggregate (g)**		**5400**
RAP (%)	**10**	**Amount RAP aggregate (g)**		**600**
Aggregate size	**RAP passing (%)**	**Amount RAP (g)**	**Amount of total aggregate (g)**	**Amount virgin (g)**
28mm	1.9	11	720	709
20mm	5.3	32	720	688
14mm	7.4	44	840	796
10mm	13.5	81	960	879
6mm	30.3	182	780	598
Dust	36.7	220	1380	1160
Filler	4.9	29	600	571
Total		**600**		**5400**
Binder content (%)		**3.85**	**Theoretical maximum density = 2540Kg/m³**	
Theoretical binder required (g)		**222.6**	**Mix density = 2388Kg/m³**	
Actual binder required (g)		**231.87**	**Total mass into mould = 1.313Kg**	

SECTION 3: Wearing Course

4.10 General

The wearing course used in this investigation was 30% Hot rolled asphalt (HRA). HRA is a gap-graded blend of mineral aggregate (stone), sand, filler and binder complying with both the Irish National Roads Authority (NRA) and BS 594: Part 1: 2002 specification, used extensively to surface major roads. It is predominantly sand based bituminous mix with a wide range of surfacing from rural and national roads to car parks and urban streets. Pre-coated chips are normally rolled into the surface to provide skid resistant properties.

This type of wearing course is the most commonly used surfacing material in Ireland and the UK. A typical HRA mix composition for a wearing course (Fig. 4.4) consists of 32% coarse aggregate, 54% fine aggregate, 6.5% filler and 7.5% binder by mass of the total mix. The strength of the material is obtained through the mortar of sand, binder and filler. Additional strength is achieved with the use of 50pen binder as discussed in section 3.6.2..

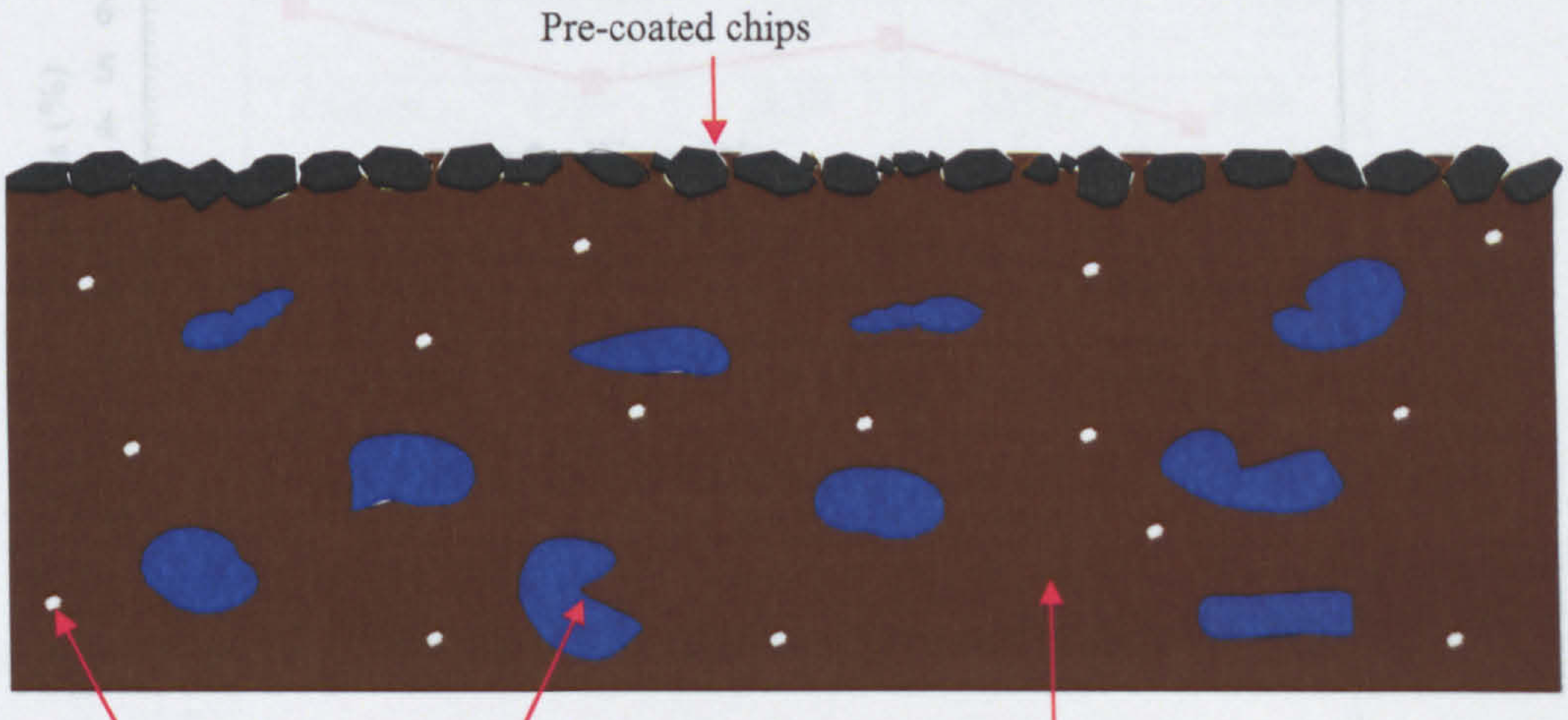


Fig. 4.4 - Typical composition of a HRA mixture

HRA with 30% coarse aggregates, as specified in BS 594: Part 1: 2002, is firmly established as a reliable heavy-duty material, eminently satisfactory in the moderate climate of Ireland and Great Britain (Illston 1992). For road purposes, 30/14 that is 30%

of 14mm single size aggregate is the most commonly used surfacing material and is usually laid in a 40mm layer.

In order to determine the optimum % of RAP for the HRA, a testing regime was established using Marshall, Gyratory and wheel tracker samples. Before any of the above tests were carried out, the voids content of the specimens had to be determined.

4.11 % Air Voids

The % of air voids in a mix plays a big role in its performance. Too many air-voids can be indicative of a poorly compacted material, while on the other hand a lack of air voids may indicate over compaction of the mix. The air-voids were calculated using two different methods: water/air method and the parafilm method prEN12697: Part 14: 1998. The water/air method is the easier of the two to carry out but can underestimate the air void content due to water getting into the air pockets on the surface. Fig. 4.5 shows the results obtained from air-voids measurements conducted on specimens before the ITSM, ITST and ITFT were carried out.

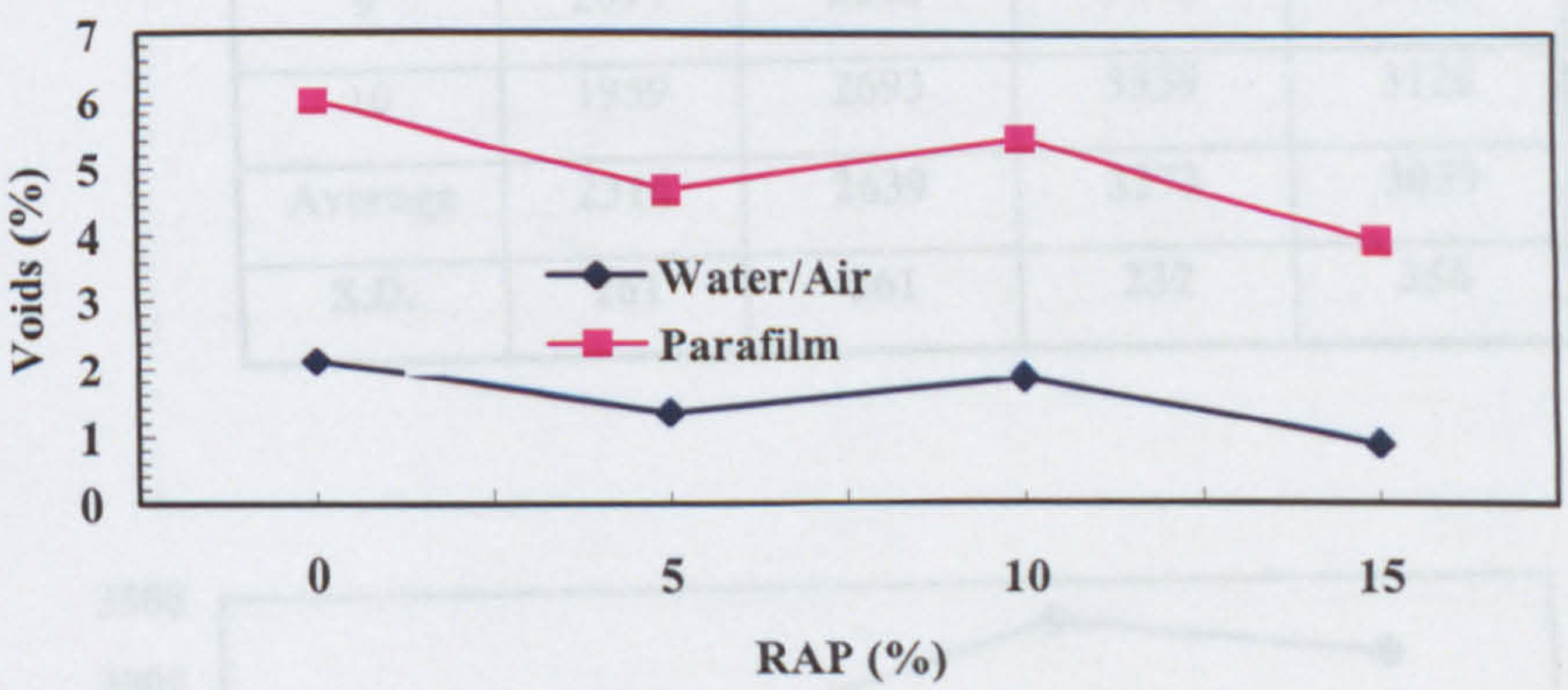


Fig. 4.5 - Void content for specimens produced at different % RAP

Fig. 4.2 shows that the parafilm method gives higher values of air voids compared to the water/air method. The figure also shows that the specimens containing RAP had less air voids than the base specimens. Although the air void content did not steadily decrease the difference between air voids measured by the two methods followed the same pattern.

4.12 Indirect Tensile Stiffness Modulus (ITSM) test

After ascertaining the % air voids the ITSM test was carried out. Table 4.7 shows the stiffness results obtained for the different % of RAP added. The average stiffness and standard deviation (S.D.) were determined for each mixture and a graph was produced (Fig. 4.6) of average stiffness versus % RAP.

Table 4.7 - Stiffness results in Mpa obtained from ITSM test at different % RAP

Specimen No.	Stiffness (Mpa)			
	0% RAP	**5% RAP**	**10% RAP**	**15% RAP**
1	2804	2486	3115	2688
2	2713	2274	3150	3579
3	2343	2511	2762	3232
4	2197	2585	3543	3074
5	2194	2790	3449	2434
6	2189	2895	3288	2965
7	2323	3151	3267	3324
8	2344	2665	3269	2652
9	2097	2346	3545	3324
10	1959	2693	3339	3128
Average	**2316**	**2639**	**3273**	**3039**
S.D.	**261**	**261**	**232**	**356**

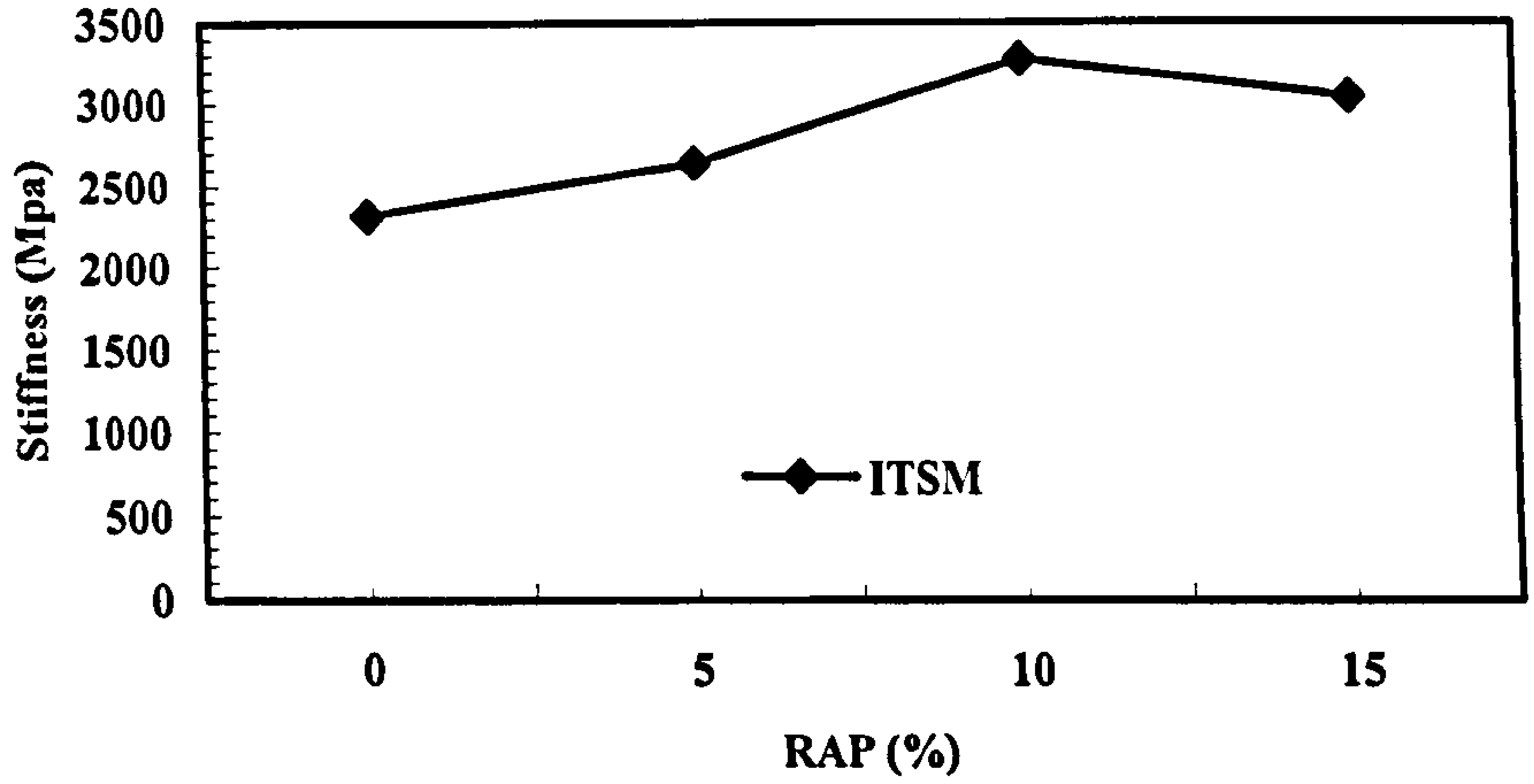


Fig. 4.6 - ITSM results for HRA specimens at different % RAP

Fig. 4.6. shows that the stiffness of the mix increases up to the addition of 10% RAP. With the addition of 15% RAP the stiffness of the mix decreased. From the graph the stiffness obtained at 15% was still higher than the base mix. The increase in stiffness of the mixtures that contain RAP could be as a result of the reduced penetration of the recovered binder.

4.13 Indirect Tensile Stiffness Test (ITST)

The next test to be conducted on the specimens was the ITST. Table 4.8 shows the results obtained for the specimens at the different % RAP whereas Fig. 4.7 shows the results obtained in graphical form.

Table 4.8 Stiffness results in Mpa obtained from ITST test at different % RAP

Load (KN)	Stiffness (Mpa)			
	0% RAP	5% RAP	10% RAP	15% RAP
200	2428	2726	3013	2779
300	2518	2306	2891	3650
400	1838	2432	2536	2860
500	1755	2122	2697	2661
550	1365	2126	2545	2004
600	1768	2221	2531	2173
650	1284	2078	2531	2146
700	1533	2114	2215	2310
800	1622	1930	2189	2465
900	1226	1819	2068	1978

Fig. 4.7 shows that the majority of the specimens containing RAP performed better than the base mix. From the findings above an addition of between 10 and 15% RAP would yield the optimum stiffness results. These results are very encouraging from a practical point of view.

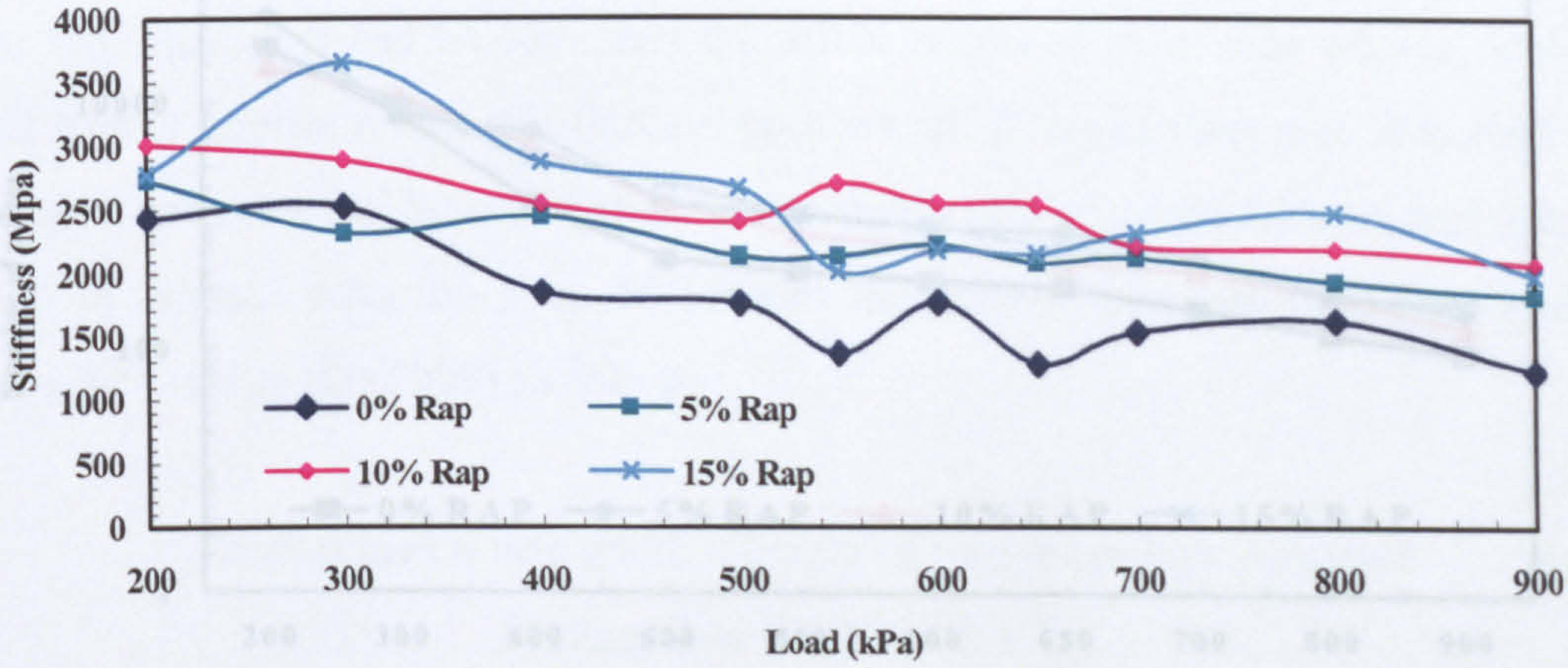


Fig. 4.7 ITST results for 30% HRA specimens at different % RAP

4.14 Indirect Tensile Fatigue Test (ITFT)

The last test to be carried out on these specimens was the ITFT, which is a destructive test compared the ITSM and ITST tests. Table 4.9 shows the results obtained from the ITFT test whereas Fig. 4.8 shows a graphical representation of the results.

Table 4.9 - Results obtained from the ITFT test

Load KN	Number of pulses to failure			
	0% RAP	**5% RAP**	**10% RAP**	**15% RAP**
200	27578	50476	19032	27275
300	7742	7782	11068	9491
400	1499	5676	4368	3393
500	519	1474	1476	2210
550	417	1184	824	1200
600	351	916	680	992
650	320	902	500	650
700	209	546	433	522
800	132	286	270	276
900	94	199	150	241

From these ITFT results it is evident that all the mixtures containing RAP performed better than the base mix in relation to the number of pulses to fatigue failure.

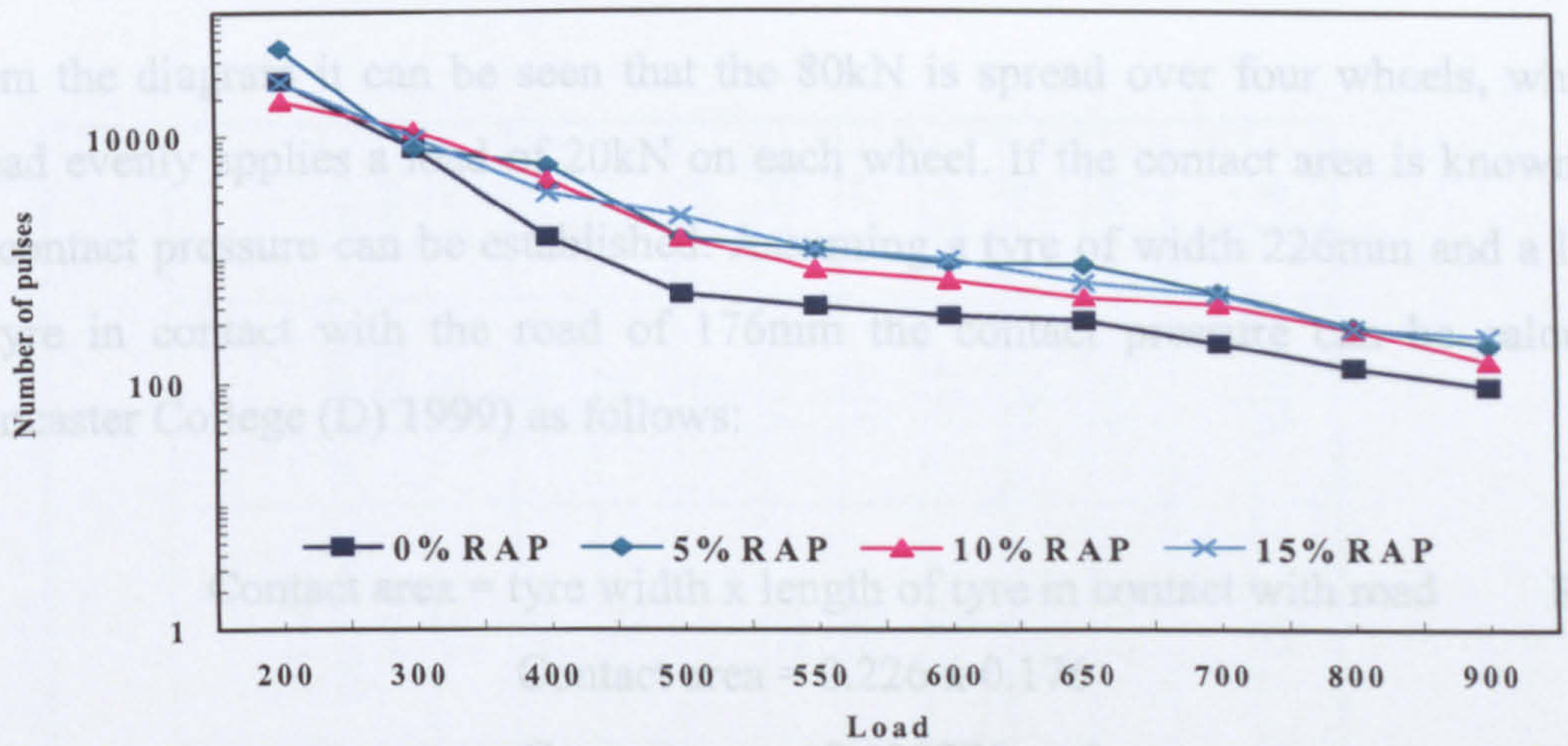


Fig. 4.8 - ITFT results for 30% HRA at different % RAP

In order to relate these stresses to a real life situation a comparison to a wheel on a Heavy Goods Vehicle (HGV) can be carried out. A standard axle is taken as an axle that carries a load of 80kN (Doncaster college (D) 1999). The load that each wheel imposes on the pavement can therefore be determined. Fig. 4.9 shows the arrangement and provision for separately analysing the effect of twin wheel loading on the wearing course material.

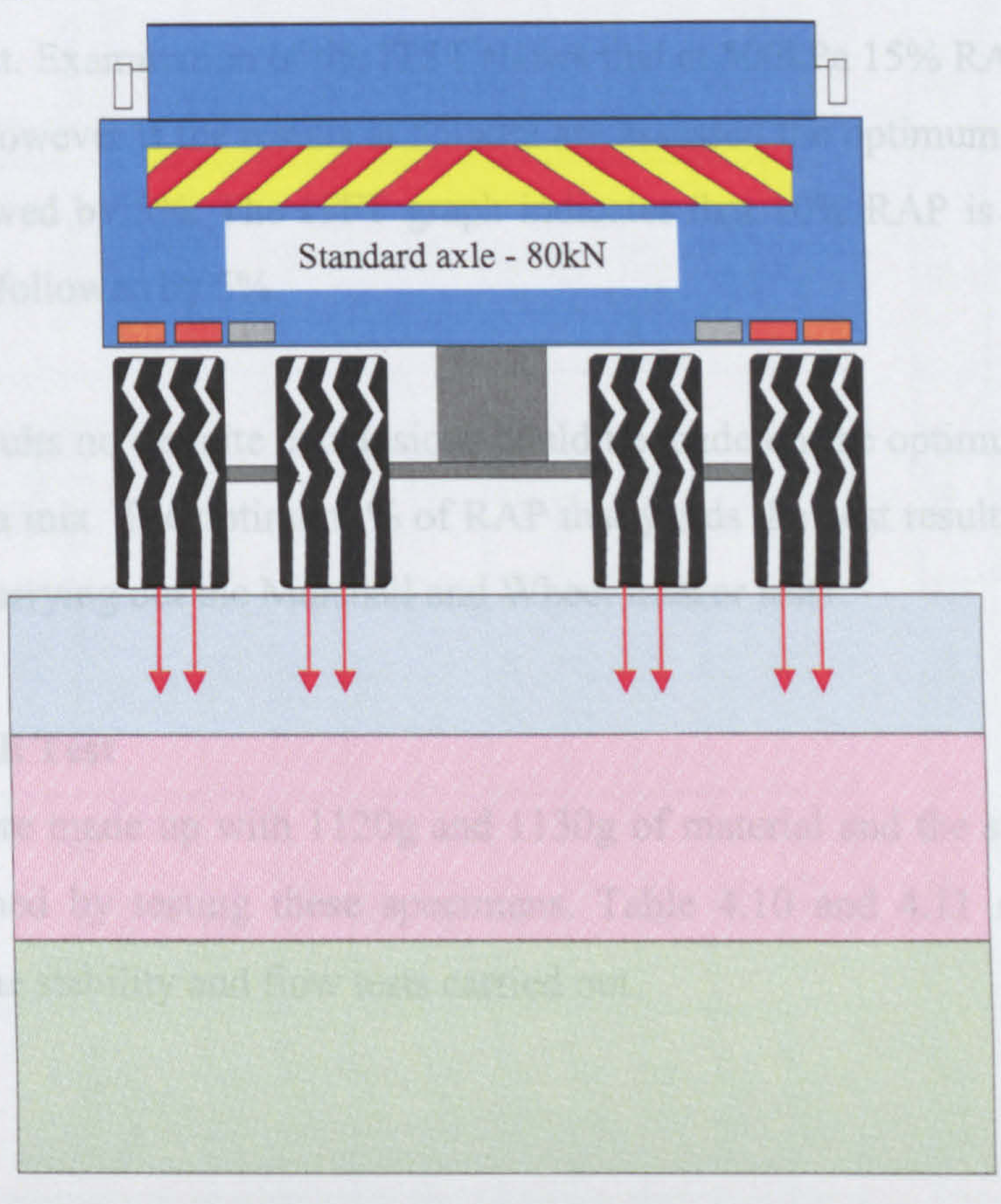


Fig. 4.9 - Pavement structure and rear wheel loading

From the diagram it can be seen that the 80kN is spread over four wheels, which if spread evenly applies a load of 20kN on each wheel. If the contact area is known then the contact pressure can be established. Assuming a tyre of width 226mm and a length of tyre in contact with the road of 176mm the contact pressure can be calculated (Doncaster College (D) 1999) as follows:

$$\text{Contact area} = \text{tyre width x length of tyre in contact with road} \qquad \textbf{Eq. 4.7}$$

$$\text{Contact area} = 0.226 \times 0.176$$

$$\text{Contact area} = 0.039776\text{mm}^2$$

From this the contact pressure can be worked out:

$$\text{Contact pressure} = \text{load/contact area} \qquad \textbf{Eq. 4.8}$$

$$= 20/0.039776$$

$$= 502\text{kPa}$$

Results between 500 and 550kPa can therefore be examined to see which material is performing best. Examination of the ITST shows that at 500kPa 15% RAP appears to be the best mix, however if the results at 550kPa are assessed the optimum % RAP is 10% and then followed by 5%. The ITFT graph indicates that 10% RAP is the optimum at 500kPa this is followed by 5%.

From these results no definite conclusions could be made on the optimum % of RAP to be added into a mix. The optimum % of RAP that yields the best results might be more evident after carrying out the Marshall and Wheel tracker tests.

4.15 Marshall Test

Specimens were made up with 1120g and 1130g of material and the stability and flow were determined by testing these specimens. Table 4.10 and 4.11 show the results obtained for the stability and flow tests carried out.

Table 4.10 - Results of Marshall test using on average 1120g of material (30% HRA and 0% RAP)

Mix density = 2.195Kg/m^3 (30% HRA + 0% RAP)												
Sample no.	**Weight in air (g)**	**Weight In water (g)**	**Weight Parafilm air (g)**	**Weight Parafilm water (g)**	**Volume Water/air (ml)**	**Volume Parafilm (ml)**	**Density water/air (Kg/m^3)**	**Density parafilm (Kg/m^3)**	**Void water/air (%)**	**Void parafilm (%)**	**Stability (kN)**	**Flow (mm)**
1	1123	607	1130	591	515	530	2.18	2.13	0.73	2.91	698	2.19
2	1122	605	1127	577	518	545	2.17	2.07	1.23	5.79	549	2.45
3	1123	607	1127	594	516	529	2.18	2.13	0.91	2.9	601	2.63
4	1115	601	1120	583	514	532	2.17	2.10	1.18	4.11	792	2.68
5	1125	611	1129	595	515	530	2.19	2.13	0.37	2.99	777	3.13
6	1118	607	1122	586	512	532	2.19	2.11	0.45	3.93	612	3.76
7	1119	602	1124	574	517	545	2.16	2.06	1.45	6.03	670	2.5
8	1119	595	1123	573	524	545	2.13	2.06	2.75	6.13	593	2.79
9	1113	594	1117	569	519	543	2.14	2.06	2.34	6.13	497	4.94
Average	**1120**	**603**	**1124**	**582.5**	**517**	**537**	**2.17**	**2.095**	**1.27**	**4.57**	**643**	**3.0**

Table 4.9 - Results of Marshall test using on average 1130g of material (30% HRA and 0% RAP)

Mix density = 2.195Kg/m^3 (30% HRA + 0% RAP)												
Sample no.	**Weight in air (g)**	**Weight In water (g)**	**Weight Parafilm air (g)**	**Weight Parafilm water (g)**	**Volume Water/air (ml)**	**Volume Parafilm (ml)**	**Density water/air (Kg/m^3)**	**Density parafilm (Kg/m^3)**	**Void water/air (%)**	**Void parafilm (%)**	**Stability (kN)**	**Flow (mm)**
1	1130	619	1133	603	511	526	2.21	2.15	-0.72	1.92	643	2.49
2	1132	621	1135	605	511	526	2.22	2.15	-0.94	1.74	596	2.46
3	1134	623	1137	601	511	532	2.22	2.13	-1.02	2.66	576	1.77
4	1132	620	1135	590	512	540	2.21	2.10	-0.69	4.28	629	2.35
5	1130	617	1134	596	513	533	2.20	2.12	-0.33	3.12	560	2.75
6	1132	618	1136	593	514	538	2.20	2.11	-0.3	3.82	617	2.25
7	1125	614	1129	586	512	538	2.20	2.09	-0.22	4.43	482	2.63
8	1129	612	1133	584	518	545	2.18	2.07	0.59	5.31	577	2.56
9	1129	613	1133	596	517	533	2.19	2.13	-0.4	3.16	444	1.96
Average	**1130**	**617**	**1134**	**595**	**513**	**535**	**2.20**	**2.1**	**-0.36**	**3.39**	**569**	**2.4**

Tables 4.10 and 4.11 show that the specimens prepared using more material yielded lower stability and flow values. It is also evident from the tables that the specimens prepared using 1120g of material gave a higher air void content compared to samples prepared using 1130g. The author recommends that the parafilm method of measuring % void content be used as the results obtained give more realistic values. It would have been expected that the specimens prepared using 1130g, which had lower % air voids, would have given higher stability and flow results compared to samples prepared using 1120g. This may be due to the higher amount of materials used in preparing the 1130g samples resulting in a crushing of some of the coarse aggregate during compaction using the Marshall hammer and a subsequent reduction in stability and flow. These results will be compared later with results obtained from specimens of 30% HRA with different % of RAP added.

Six specimens were compacted at the different % RAP and stability and flow results were determined. Table 4.12 shows the average stability and flow at different % RAP. Graphical representations of these results can be viewed in Figs 4.10 and 4.11 respectively.

Table 4.12 - Average stability and flow at different % RAP

RAP (%)	Stability (kN)	Standard deviation (S.D.)	Flow (mm)	Standard deviation (S.D.)
0	643	102.4	3.0	0.88
5	956	140.1	3.07	0.82
10	1321	105.4	3.13	0.85
15	898	62.99	3.63	0.39

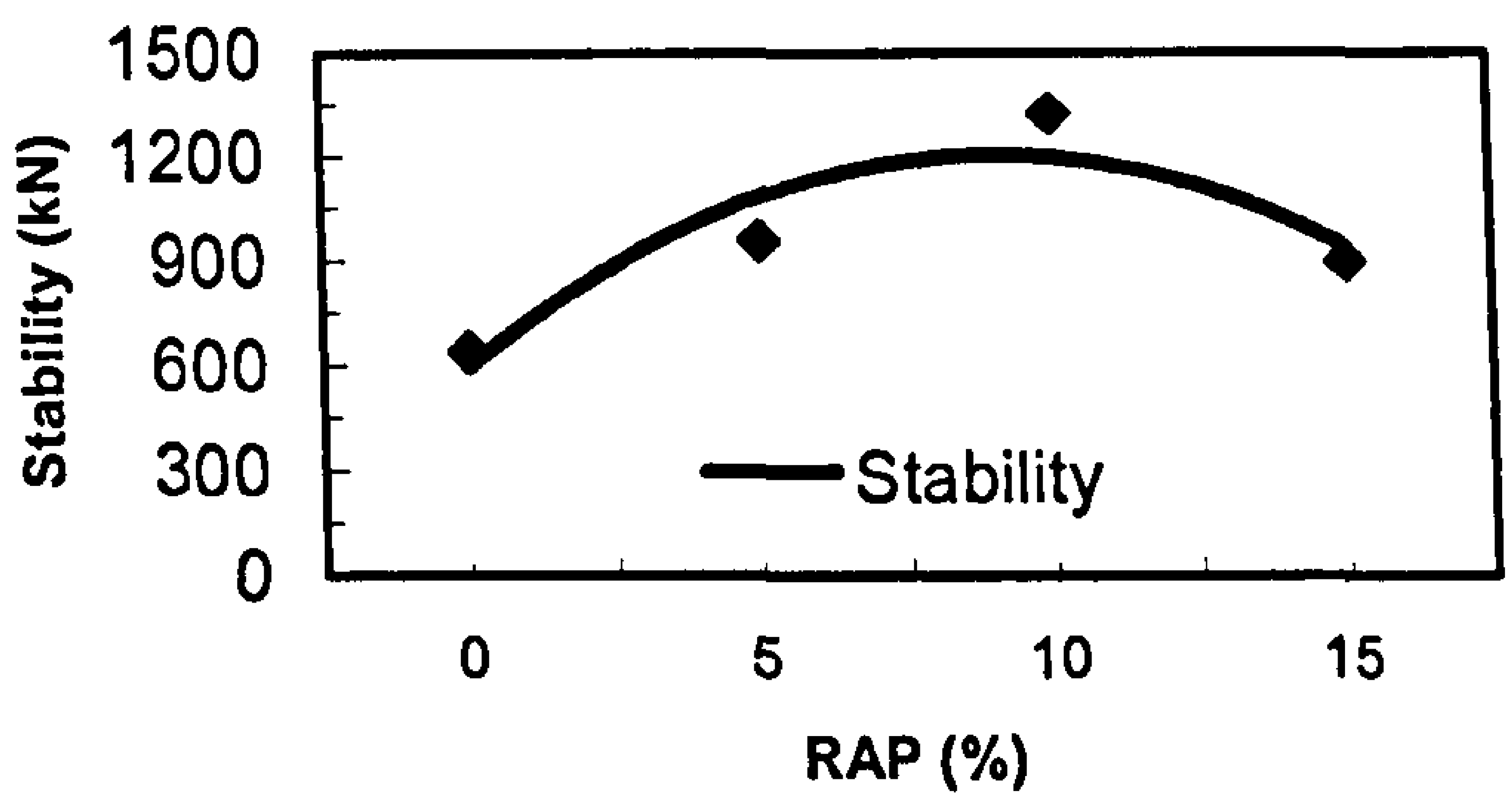


Fig. 4.10 - Average stability results at different % RAP

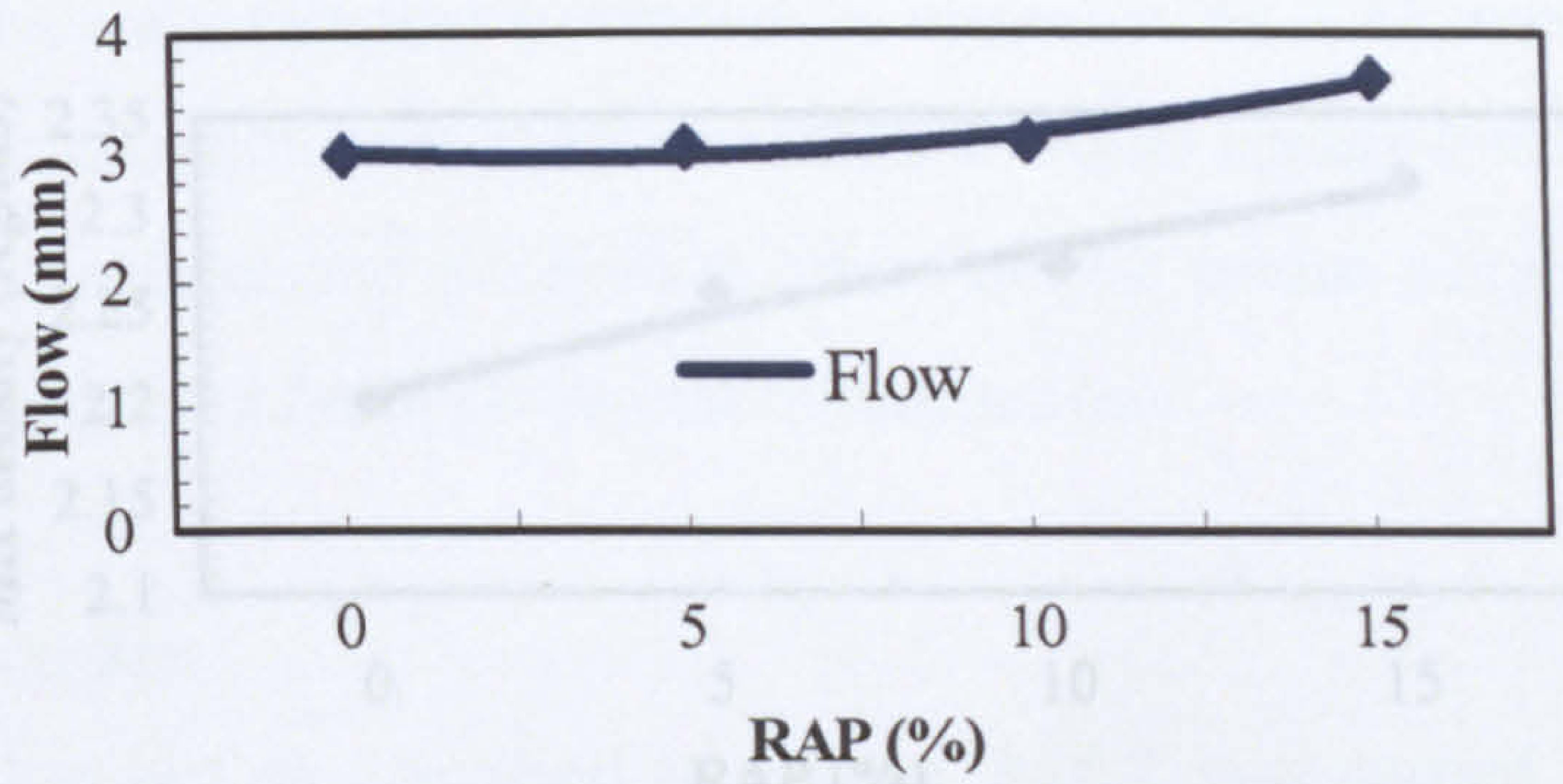


Fig. 4.11 - Average flow results at different % RAP

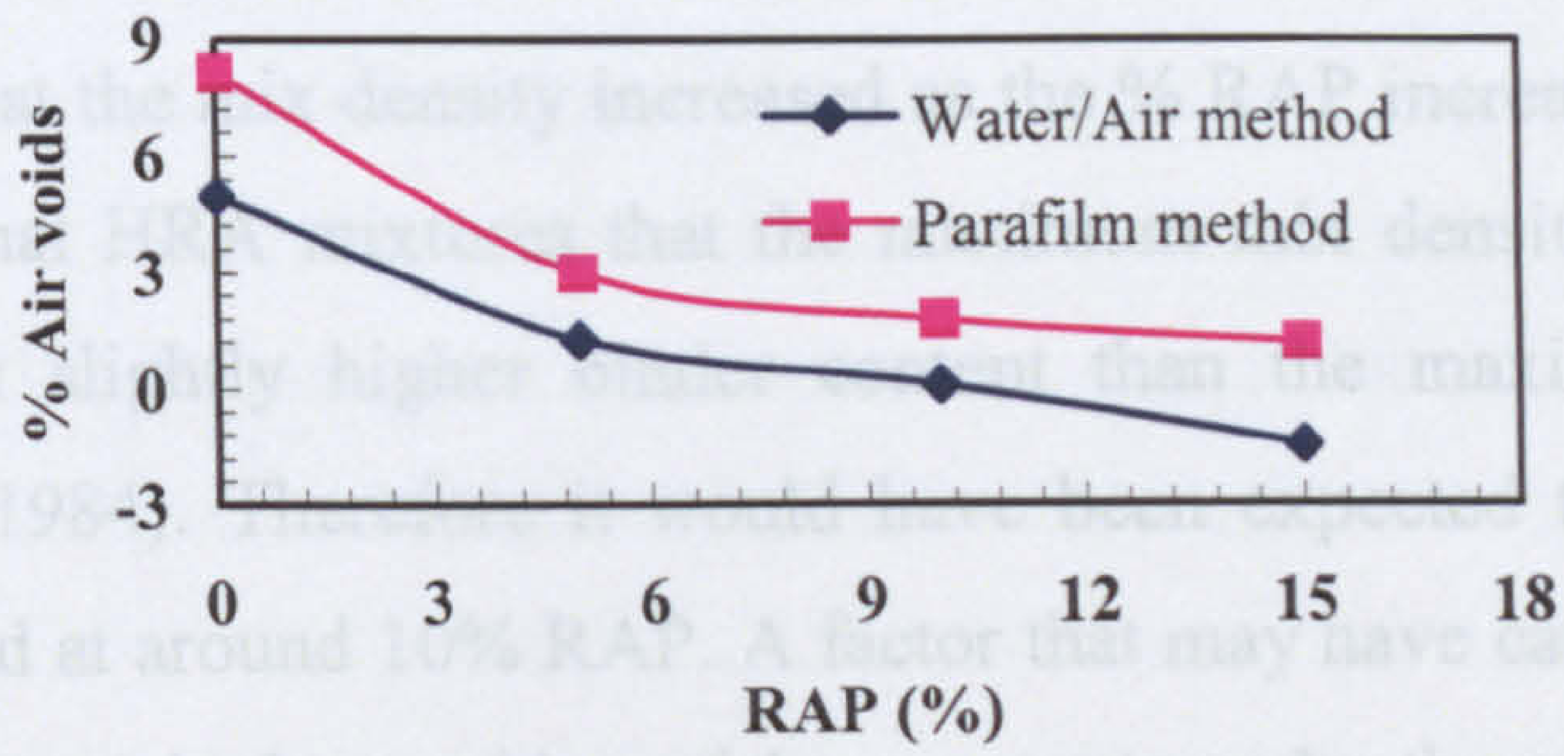


Fig. 4.12 - Air voids vs % RAP

Fig. 4.10 shows that the stability of the mix increased to a maximum value at 10% and started to decrease after this. The increase in the stability values of the mix could be due to the increase in stiffness of the recovered binder over time as discussed in section 4.12. The flow of the material increased with increasing % of RAP material. Fig. 4.12 shows the relationship between the % air voids and % RAP for the specimens tested. The graph was plotted to determine whether the % air voids increased or decreased with the increasing % of RAP. Fig. 4.12 shows that the % air voids decreased as the % RAP increases. The crushing of the RAP proportion of the mix when being compacted in the Marshall hammer seemed to result in negative % air voids being reported. Another reason for the reduction in air voids at higher % RAP may be due to the lower ACV of the RAP aggregate compared to the sand aggregate thus causing the increased crushing of specimens containing RAP prepared in the Marshall hammer. Fig. 4.13 shows the relationship between reported mix density and % RAP.

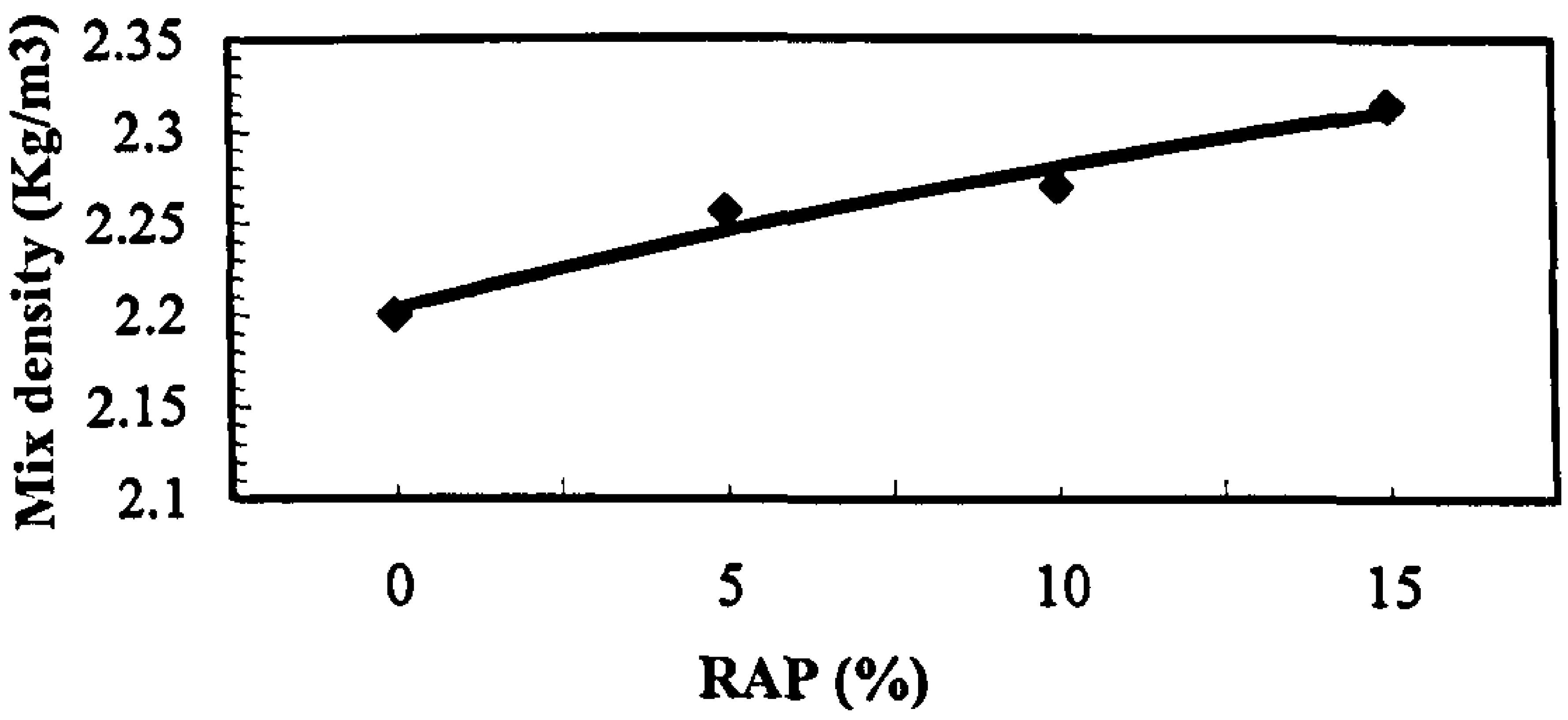


Fig. 4.13 - Mix density vs % RAP

Fig. 4.13 shows that the mix density increased as the % RAP increased. It is usually the case with traditional HRA mixtures that the maximum mix density normally (but not always) occurs at slightly higher binder content than the maximum stability (The Asphalt Institute 1984). Therefore it would have been expected that the mix density would have peaked at around 10% RAP. A factor that may have caused the mix density to continue to increase is the crushing of the aggregate under the action of the Marshall hammer. This crushing could have increased as the % RAP increased causing the mix density to increase. The author concludes that in order to determine the correct stability the density of the RAP would have to monitored and taken into account continuously. Fig. 4.14 shows a graph of voids in mineral aggregate (VMA) Vs % RAP.

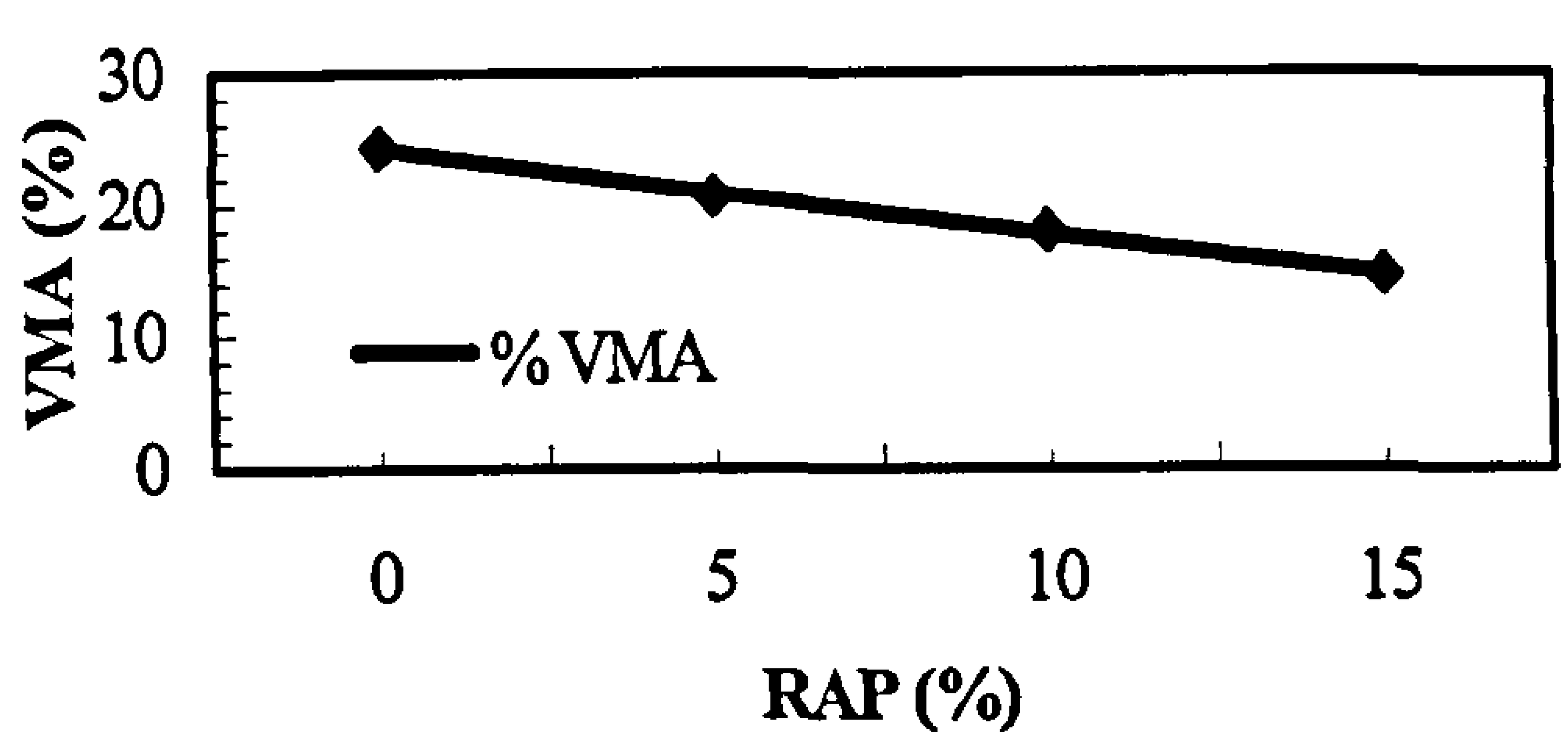


Fig. 4.14 - VMA vs % RAP

It was expected that the % VMA would decrease to a minimum value then increase with increasing RAP contents. Fig. 4.14 shows that the VMA decreased as the % RAP increased. The crushing of the RAP aggregate could have caused these phenomena.

From the results obtained there is no clear evidence to suggest that the optimum % RAP is 10% as the stability results suggested. From the results obtained after carrying out the

Marshall test, the only test that produced an optimum % RAP was the stability test. From this graph the optimum % RAP was 10%. It would appear that the crushing action of the Marshall hammer had a negative effect on the results obtained. The extent of these effects is perhaps a basis for further investigation.

4.16 Wheel Tracker

The wheel tracker test as discussed in section 3.10.2 was carried out on HRA slabs containing RAP aggregate, which were compacted using the roller compactor. Tests were conducted to determine if the rut depth and rut rate were within 5mm and 7mm respectively as outlined in the NRA's Notes for guidance on specification for road works 2000. Ruts of 15mm were obtained from the slabs containing no RAP made with 10kg of material. This 15mm rut suggests that the sample has failed. An investigation was then carried out in order to find the cause of failure. The investigation was carried out in two stages:

- Make slabs with different amounts of material and check the air void content to determine at what amount of material the 6% air voids could be achieved.
- Apply more compactive effort to the specimens made with different amounts of material and check the % air void content.

Slabs were made with 10 and 10.8kg of material. Air void content calculations were carried out using the weight in water/air and the parafilm methods. Table 4.13 shows the results obtained from the % air void tests.

Table 4.13 shows that as more material is added the % air voids decreased. This would indicate that the amount of material required to ascertain the 6% air voids as calculated in Appendix (A) lies between 10 and 10.8kg

Table 4.13 - Results obtained from air void calculations at ordinary compaction effort

Amount of material (g)	Voids air/water method (%)	Voids Parafilm method (%)
10000	3.8	7.3
10000	4.3	8
10000	4.3	8.1
10000	3.9	7.5
10800	-1.4	1.4
10800	-1.7	0.7
10800	-1.2	1.9
10800	-0.9	2.4

More slabs were produced containing 10 and 10.8kg. The compactive effort used to produce the slabs was increased. The number of passes to compact the slabs was not changed but the pressure that was exerted on the specimen increased. Table 4.14 shows the two different levels of compaction used to prepare the slabs. Air void contents were taken using both the weight in air/water and parafilm methods. Table 4.15 shows the air void content results obtained for specimens produced with a higher compactive effort.

Table 4.14 - Different levels of compaction used in preparing 30% HRA slabs

Compactive effort	Cycle no.		Pressure (bars)		
			2.5	4	6
Ordinary compaction	1	No. of passes	11	5	0
	2		2	9	0
	3		2	0	5
More compaction			Pressure (bars)		
	Cycle no.		3.5	4.5	6
	1	No. of passes	11	5	0
	2		2	9	0
	3		2	0	5

Table 4.15 – Air void results from slabs produced using higher compactive effort

Amount of material (g)	Voids air/water method (%)	Voids Parafilm method (%)
10000	3.9	7.5
10000	3.7	7
10800	-1.4	3.3
10800	-0.7	4

From Table 4.15 it is evident that the target air void content lies between 10 and 10.8kg of material placed into the mould. If comparing the results in Tables 4.13 and 4.15 it is evident that the increased pressure applied did not alter the air voids significantly enough to warrant extra compaction. In order to determine the amount of material required to achieve 6% air voids a graph was drawn of air voids vs mass of material which can be seen in Fig. 4.15.

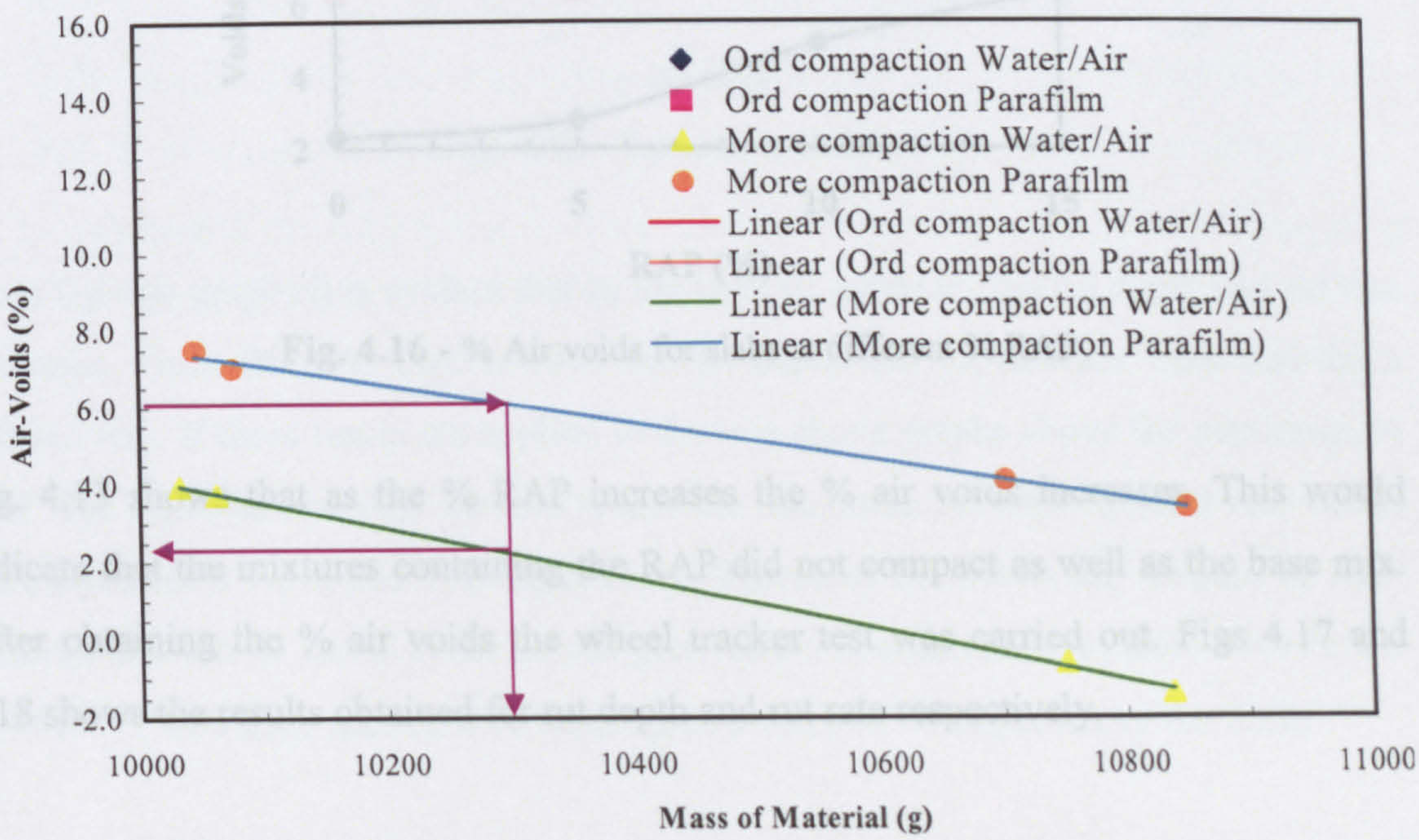


Fig. 4.12 - Air-voids vs compaction

From Fig. 4.15 it is transparent how similar the void content results are for the two different levels of pressure applied. Fig. 4.15 shows that to achieve 6% air voids 10.3kg of material must be placed in the mould. An air void content of 2.5% would be expected using the weight in air/water method for slabs made with 10.3kg of material.

Slabs with 10.3kg of material were produced using ordinary compaction. The % air voids test carried out on the slabs gave values of 2.3% and 6.4% for the weight in air/water and parafilm method respectively. Identical samples were then reproduced and tested to determine the rut depths and rut rates. Rut depths of 5-6mm were obtained for these slabs in the wheel tracker. The rut rate was found to be between 3-4mm. Both the rut depths and rut rates were found to be within the prescribed limit as set out in BS 594: Part 1: 1994.

More slabs were compacted containing 5%, 10% and 15% RAP. % air voids and rut rates were determined. % air void calculations were carried out on the slabs and the results are presented in Fig. 4.16 below.

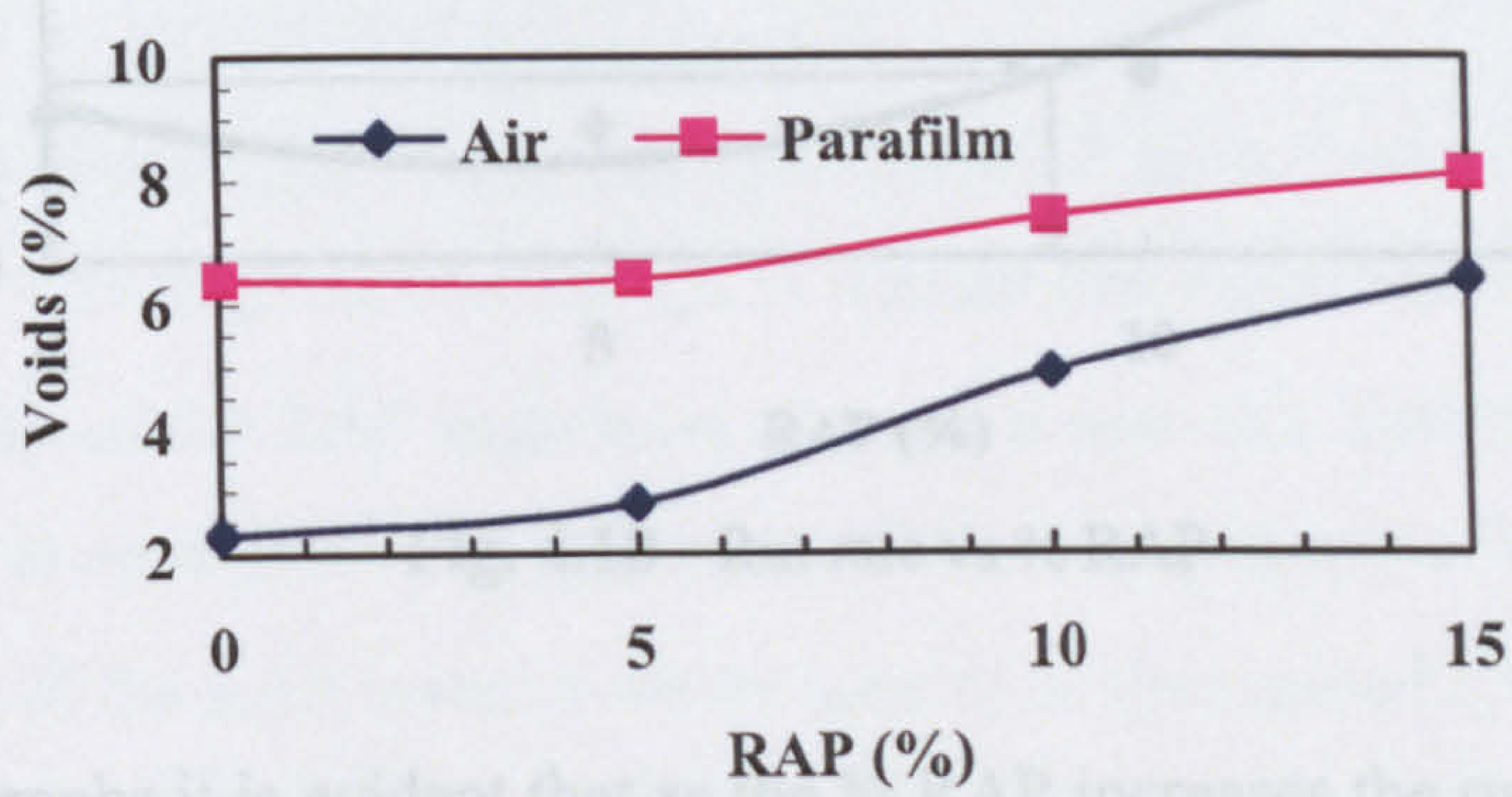


Fig. 4.16 - % Air voids for slabs at different % RAP

Fig. 4.13 shows that as the % RAP increases the % air voids increases. This would indicate that the mixtures containing the RAP did not compact as well as the base mix. After obtaining the % air voids the wheel tracker test was carried out. Figs 4.17 and 4.18 shows the results obtained for rut depth and rut rate respectively.

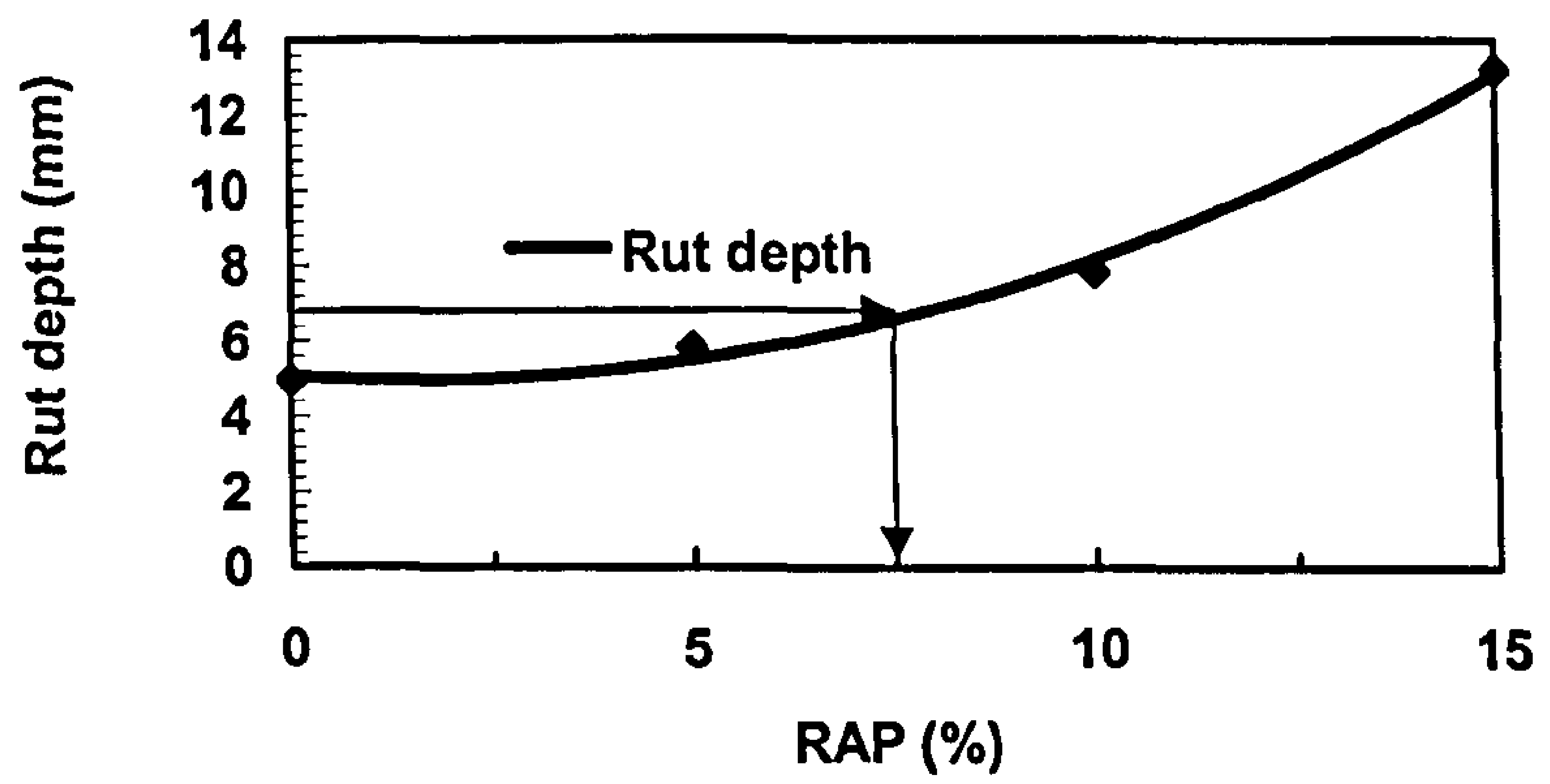


Fig. 4.17 - Rut depth vs % RAP

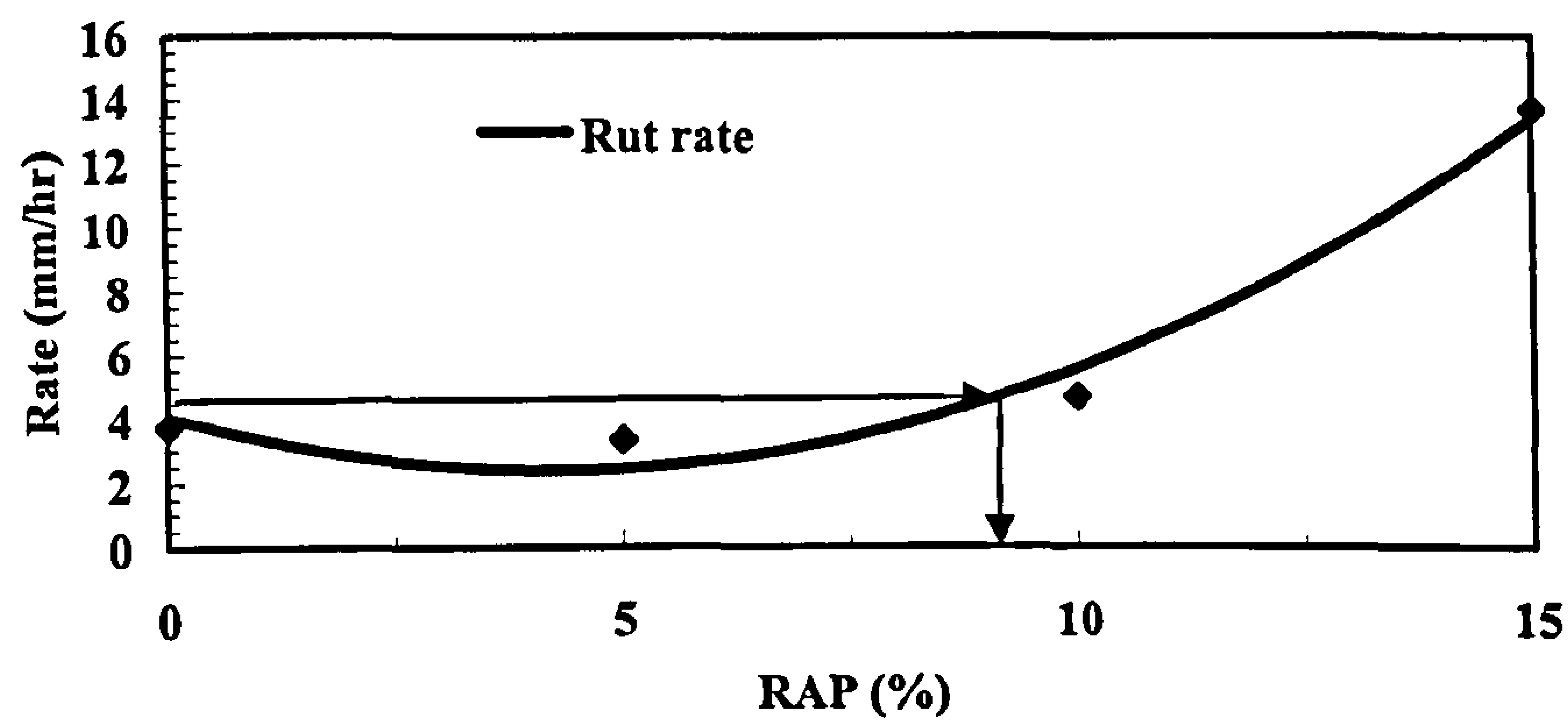


Fig. 4.18 - Rut rate vs % RAP

From the two graphs it is evident that as the % RAP increases the rut depth and rut rate increases. From BS 594: Part 1: 1994 the rut depth and rut rate are 7mm and 5mm respectively. If these limits are applied to the two above graphs above the maximum % RAP that could be added to the mix before the maximum rut depth and rut rate values are surpassed is 7.5% and 9% respectively. Therefore the maximum amount of RAP material that can be added to the mix to give results in accordance with BS 594: Part 1: 1994 is 7.5% because adding 10% would give a value of rut depth above the limit.

4.17 Summary

Table 4.16 summarises the results obtained from the three different tests carried out on 30% HRA with different % RAP added.

Table 4.14 - Optimum % RAP determined from various tests carried out

Test carried out	Optimum % RAP
ITSM	10
ITST	15
ITFT	10
Marshall stability	10
Rut depth	7.5 (max)
Rut rate	9 (max)

The final conclusions obtained from the testing programme carried out on 30% HRA are as follows:

- The maximum amount of RAP material that can be added to this 30% HRA mix is 7.5%. Any % above that will cause the specimens to fail due to high rut depth.
- In terms of practical use the most amount of material that could be added would probably be in the region of 5%.
- The economic cost of adopting a plant to use RAP in its mixtures is high, therefore adding 5% RAP might not be enough to warrant this expenditure.
- If the quality of the RAP aggregates changes a new mix design would have to be carried out to determine if the mix meets all the requirements of the relevant tests.
- The density of the RAP material would have to be determined if the RAP was going to be used in 30% HRA..

SECTION 4: Basecourse

4.18 General

The wearing course and the basecourse make up the upper layer of road surfacing, which should provide a level surface with good riding characteristics. The basecourse is normally dense binder macadam but HRA could also be used. 20mm dense basecourse is typically composed of 60% coarse aggregate, 21% crushed rock fines (CRF) 14.4% sand and 4.6% binder by mass of the total mix. A schematic of this composition is illustrated in Fig. 4.19.

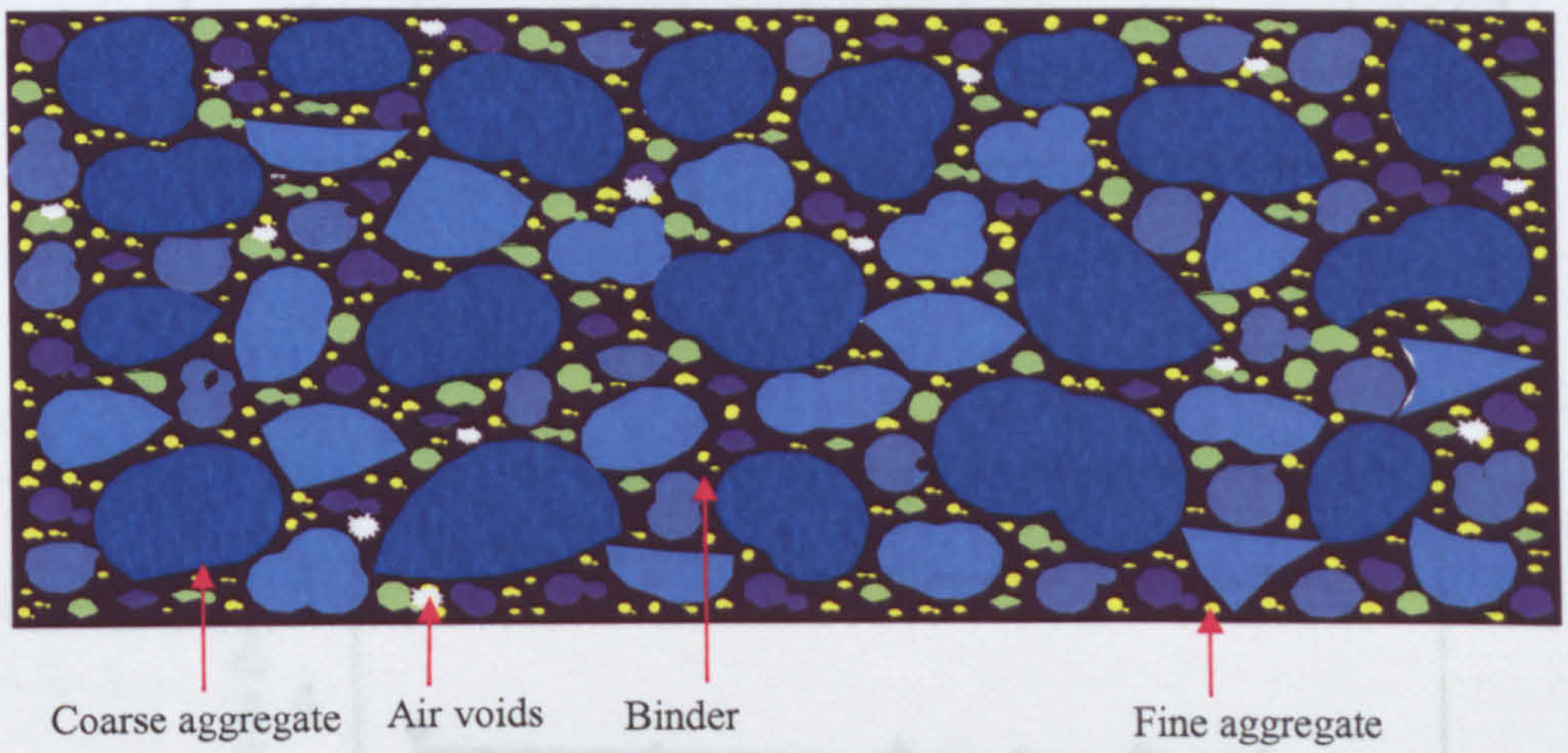


Fig. 4.19 - Typical composition of a basecourse

The main task of a basecourse material is to transfer the load from the upper wearing course to the roadbase and therefore should be a stable and stiff material to minimise crack propagation.

4.19 % Air voids and ITSM test on specimens made using materials from Quarry No.1

Gyratory specimens were made up with varying % of RAP. The % air voids and ITSM were subsequently determined. Table 4.17 shows the % air void content results obtained for the weight in air/water and parafilm method. Fig. 4.20 shows these results in graphical form.

Table 4.17 - Air void content results

RAP (%)	Voids (air/water) (%)	S.D.	Voids (parafilm) (%)	S.D.
0	3.4	0.83	7.2	1.67
10	2.6	0.77	6.2	2.01
15	3	0.89	6.2	1.71
20	2.8	0.95	6.4	1.97
25	3	1.01	6	1.75
30	2.9	0.75	6.4	1.16
35	3	0.56	6	2.14
40	2.9	1.09	6.1	1.86
50	2.7	0.67	5.8	1.49

Note: 6 specimens were produced and tested for each % RAP

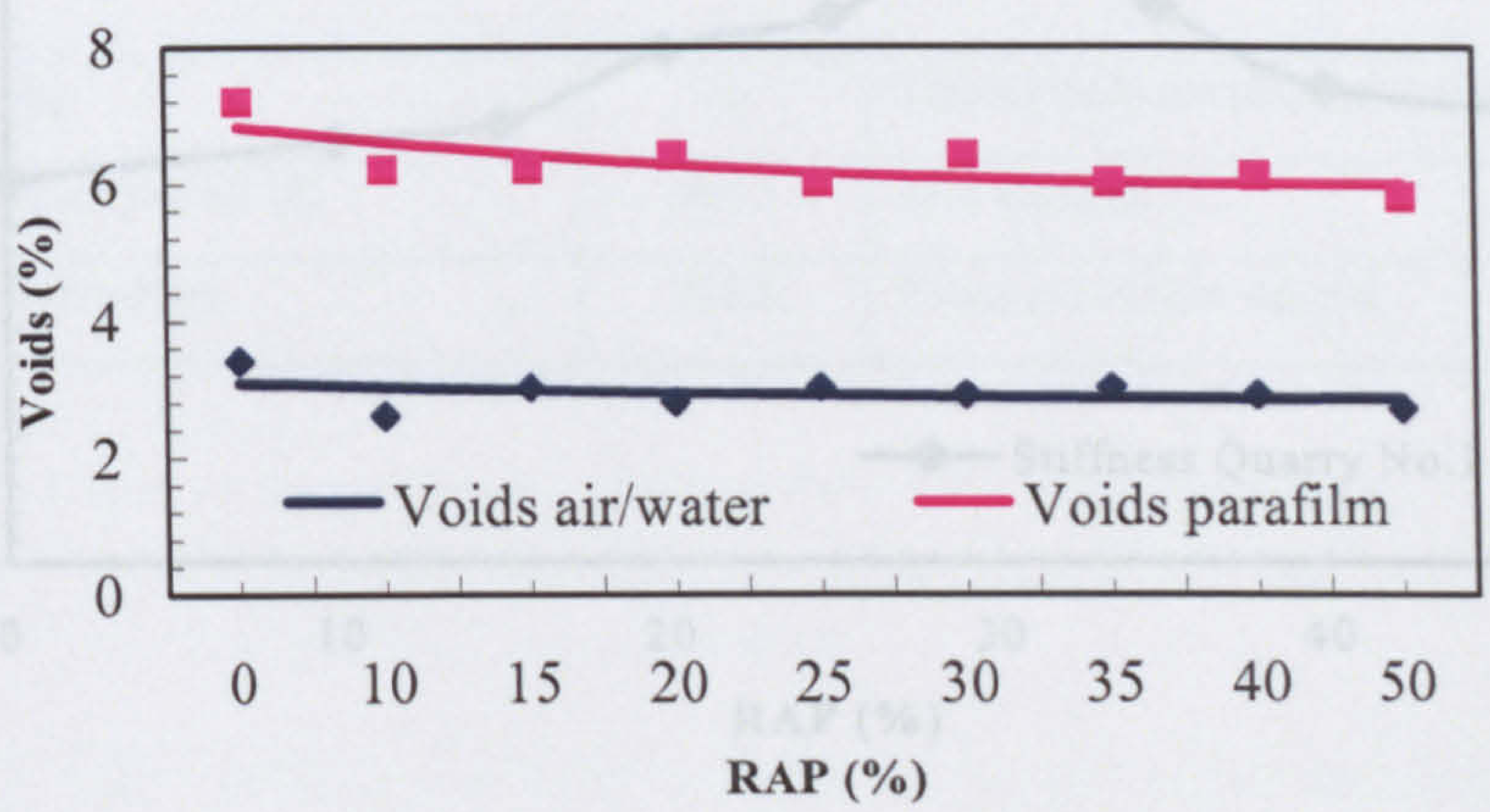


Fig. 4.20 - % Air voids vs % RAP

Fig. 4.20 shows that the specimens containing RAP had lesser % air voids than the one without. The figure also shows that the difference in results between the two methods used is consistent. The ITSM test was then carried out. The results of this test are shown in Table 4.18. Fig. 4.21 shows the ITSM results for the different % RAP in graphical form.

Table 4.18 - ITSM results at different % RAP material

Specimen No	ITSM (Mpa)								
	0% RAP	**10% RAP**	**15% RAP**	**20% RAP**	**25% RAP**	**30% RAP**	**35% RAP**	**40% RAP**	**50%R RAP**
1	2401	2686	3294	4129	4046	4652	3843	3753	3435
2	2403	3408	3596	3400	4099	5002	3686	3441	3629
3	2518	3563	3500	4024	3815	4687	4236	3417	3029
4	2843	3322	3259	3902	3382	4714	4202	3572	3227
5	3354	3216	3469	4371	4392	5001	4180	3461	3373
6	3429	2964	3518	3343	3891	4734	3583	3728	3500
7	3204	3329	3158	4231	4811	5200	5276	3890	3522
8	3379	3400	3209	4184	4665	4993	5145	3590	3593
Average	**2943**	**3236**	**3375**	**3948**	**4200**	**4873**	**4269**	**3648**	**3413**
S.D.	**455**	**283**	**164**	**381**	**378**	**200**	**631**	**264**	**200**

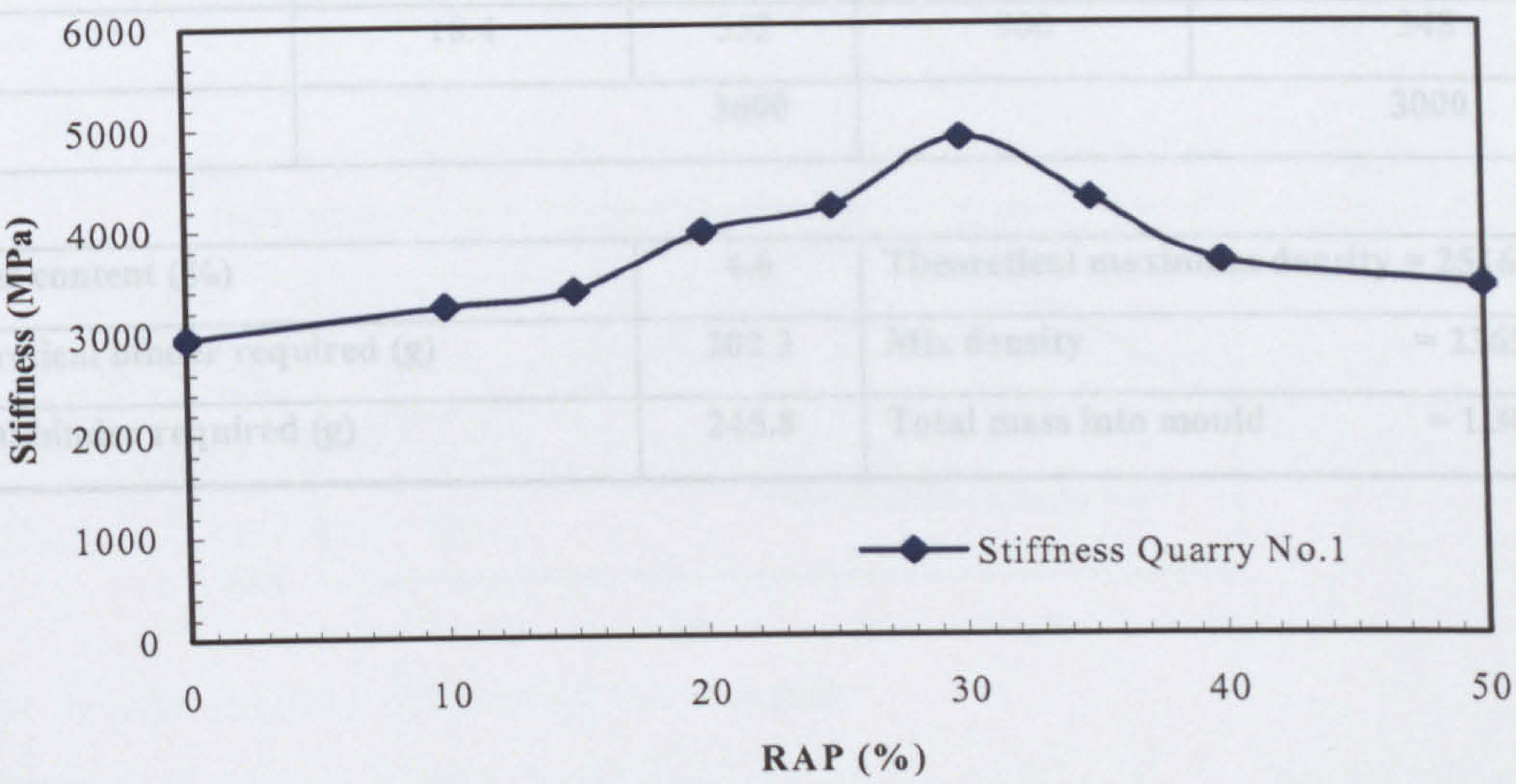


Fig. 4.21 - ITSM results at various % RAP

Fig. 4.21 shows that the stiffness of the material increases with the % of RAP added. At 30% RAP the stiffness reaches its maximum value of 4873Mpa. The addition of 50% RAP showed interesting results. The stiffness of the material containing 50% RAP was higher (3413Mpa) than the base mixture (2943Mpa). An examination of Table 4.19 found that the grading of the material had altered from the original grading and therefore result in an out of specification material.

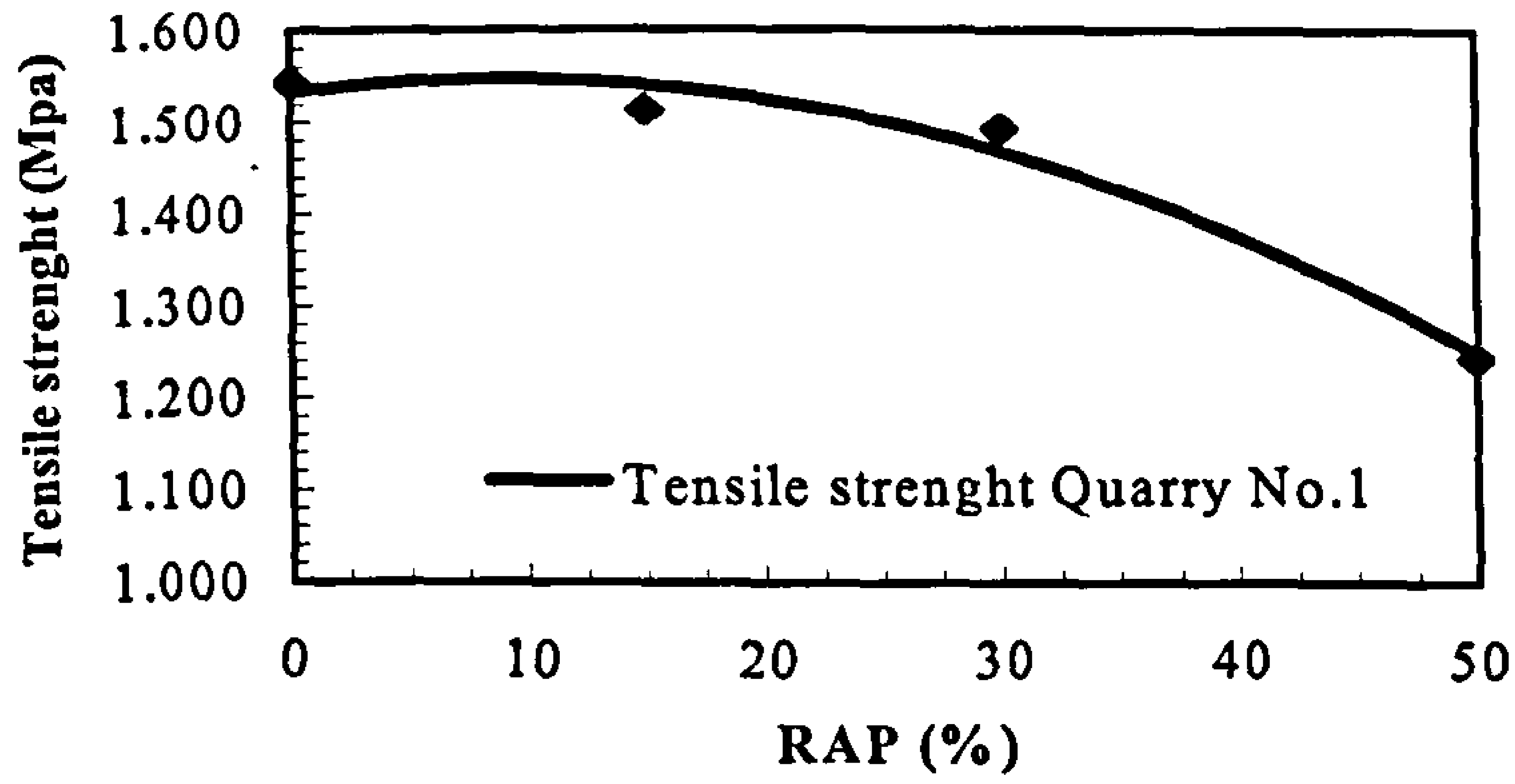


Fig. 4.23 - Tensile strength results at different % RAP

Fig. 4.23 shows that the tensile strength of the basecourse specimens reduces as more RAP added. The reduction in tensile strength decreased rapidly above 30% RAP. This would indicate that the tensile strength reduces rapidly after the optimum % RAP is surpassed.

The research so far has concentrated on RAP material from one location only. In an attempt to further validate the results, RAP material produced from Quarry No. 2 was included in this investigation. The methodology and test programme adopted was similar to that for Quarry No. 1.

4.21 % Air Voids and ITSM Quarry No. 2

Table 4.21 shows the results obtained after carrying out the % air void test calculation using the air/water method and the ITSM results. Figs 4.24 and 4.25 show a graphical representation of these results respectively.

4.20 Indirect Tensile Strength

After carrying out stiffness tests more specimens were prepared and compacted containing 0, 15, 30 and 50% RAP to carry out the ITSM test and Tensile strength. Table 4.20 shows the results for the ITSM and Tensile strength tests. Figs 4.22 and 4.23 shows a graphical representations of these results respectively.

Table 4.20 - Results for the ITSM and tensile strength

% RAP	ITSM (Mpa)	S.D.	Tensile strength (Mpa)	S.D.
0	4863	497	1.543	0.068
15	5028	826	1.513	0.162
30	5821	610	1.493	0.102
50	4917	317	1.244	0081

Note: Six specimens were produced at each of the varying % of RAP

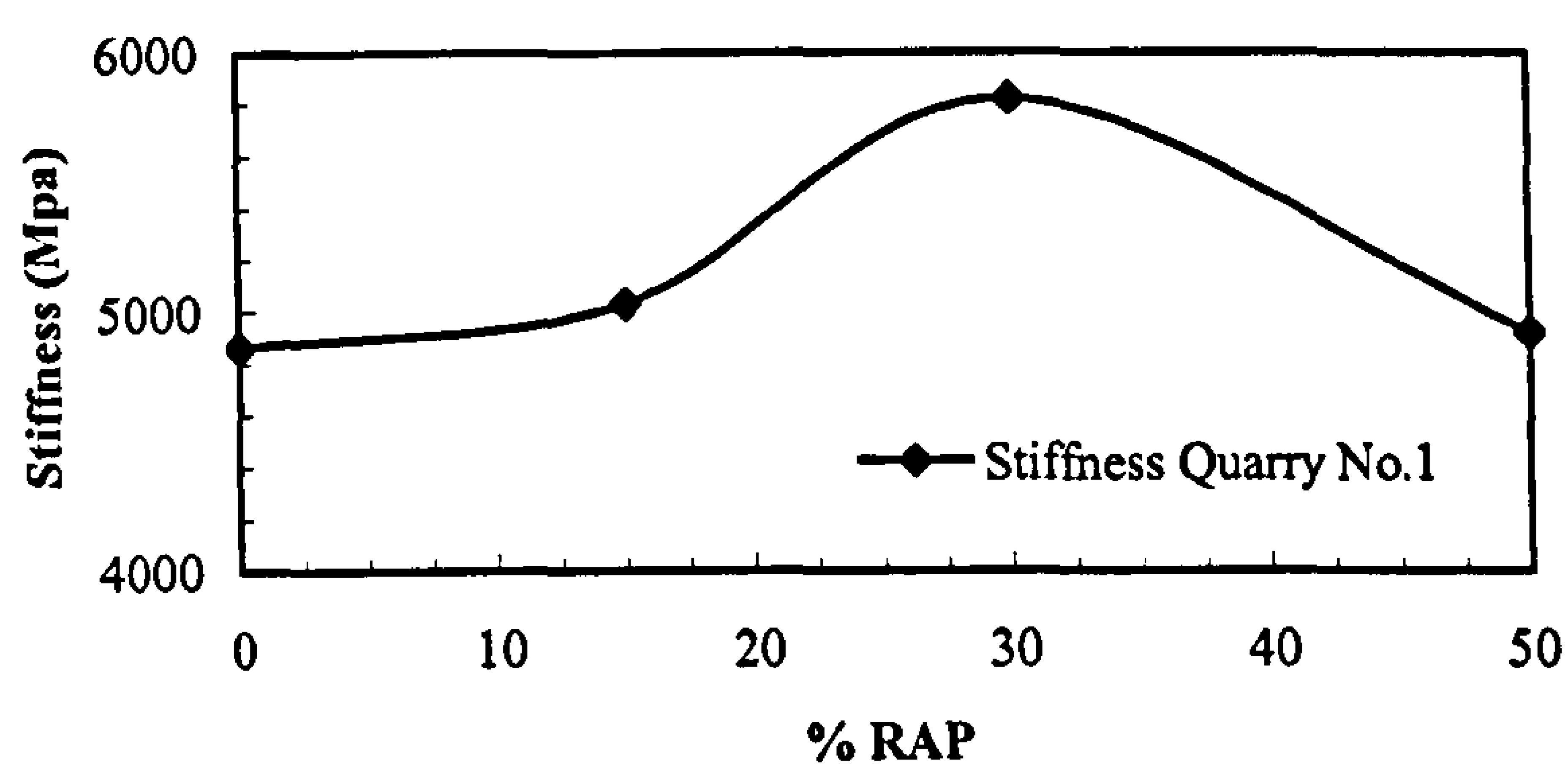


Fig. 4.22 - Stiffness results for basecourse specimen before carrying out the tensile strength

Fig. 4.22 shows that the optimum % RAP is 30%. The figure shows that specimens containing 50% RAP yielded higher stiffness values than the base specimens. These findings coincide with the results obtained in Fig. 4.21. Therefore the conclusion drawn from the ITSM test results suggest that the optimum % RAP that gives the best results in terms of ITSM for the basecourse material is 30% RAP. After completing the ITSM tests the tensile strength test was carried out.

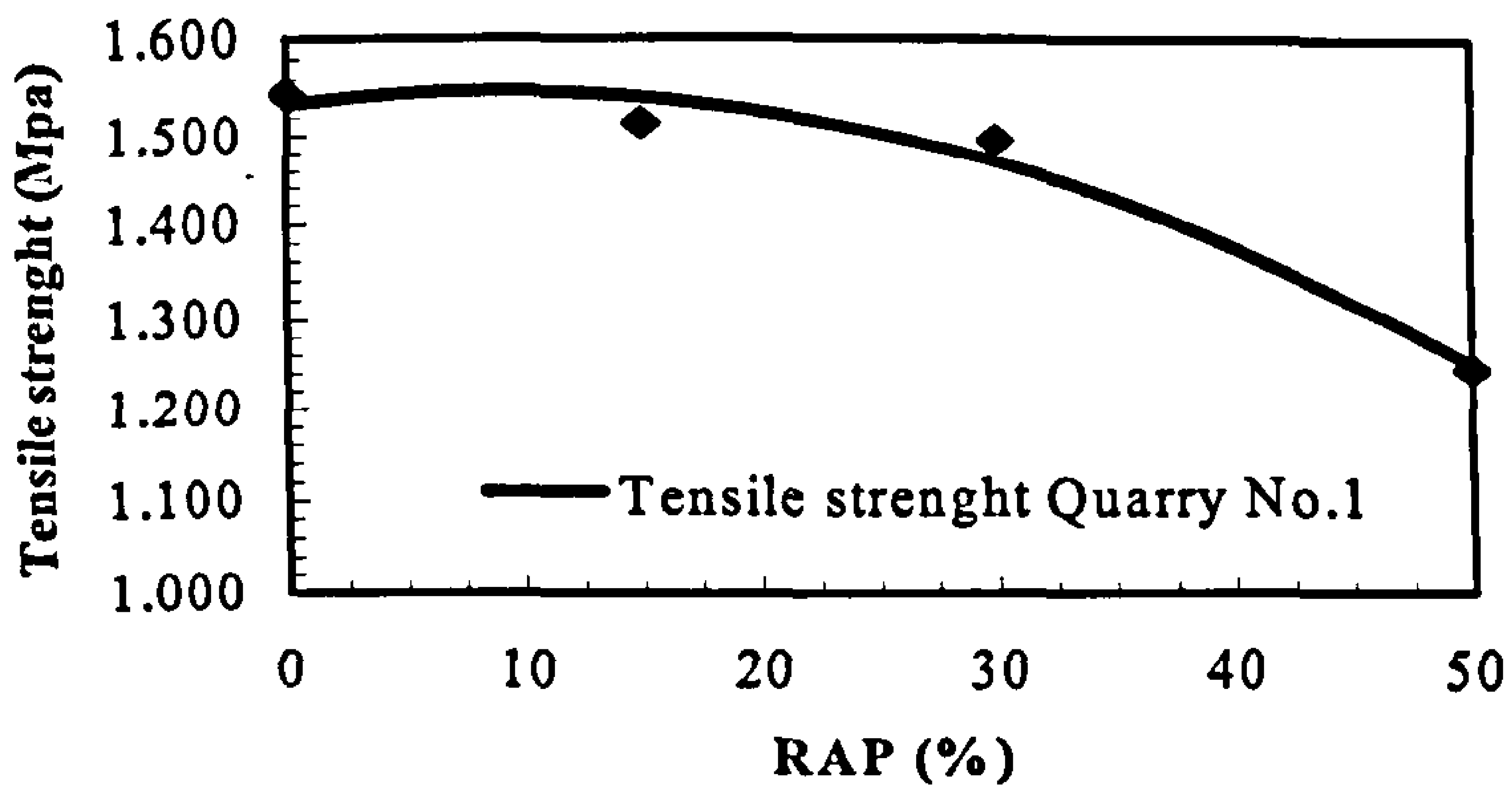


Fig. 4.23 - Tensile strength results at different % RAP

Fig. 4.23 shows that the tensile strength of the basecourse specimens reduces as more RAP added. The reduction in tensile strength decreased rapidly above 30% RAP. This would indicate that the tensile strength reduces rapidly after the optimum % RAP is surpassed.

The research so far has concentrated on RAP material from one location only. In an attempt to further validate the results, RAP material produced from Quarry No. 2 was included in this investigation. The methodology and test programme adopted was similar to that for Quarry No. 1.

4.21 % Air Voids and ITSM Quarry No. 2

Table 4.21 shows the results obtained after carrying out the % air void test calculation using the air/water method and the ITSM results. Figs 4.24 and 4.25 show a graphical representation of these results respectively.

Table 4.21 - Results of air void content calculation and ITSM

RAP %	Air/water voids (%)	S.D.	ITSM (Mpa)	S.D.
0	3.4	0.88	2943	216
10	2.9	1.21	3153	259
20	2.5	1.07	3234	347
30	2.5	0.94	3432	295
40	2.6	0.76	3269	366

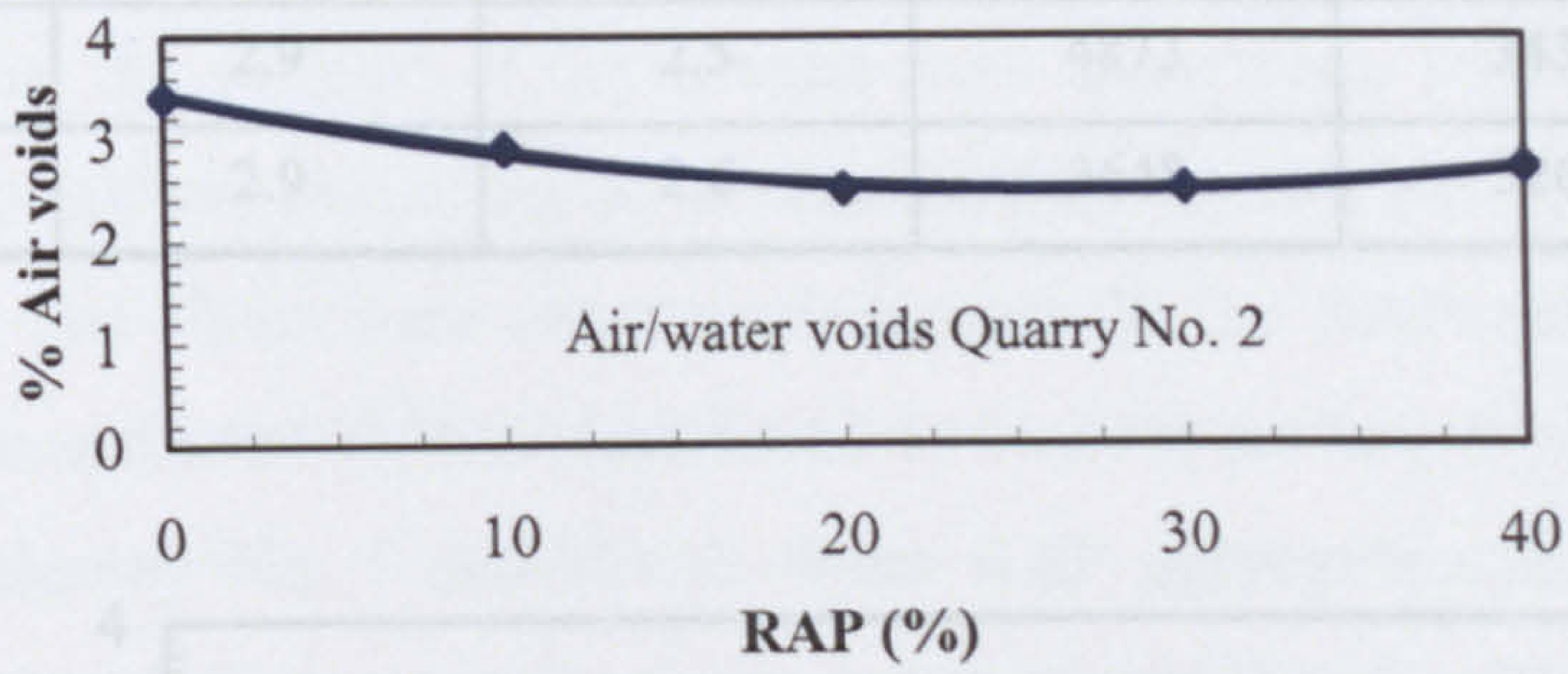


Fig. 4.21 - % air voids vs % RAP

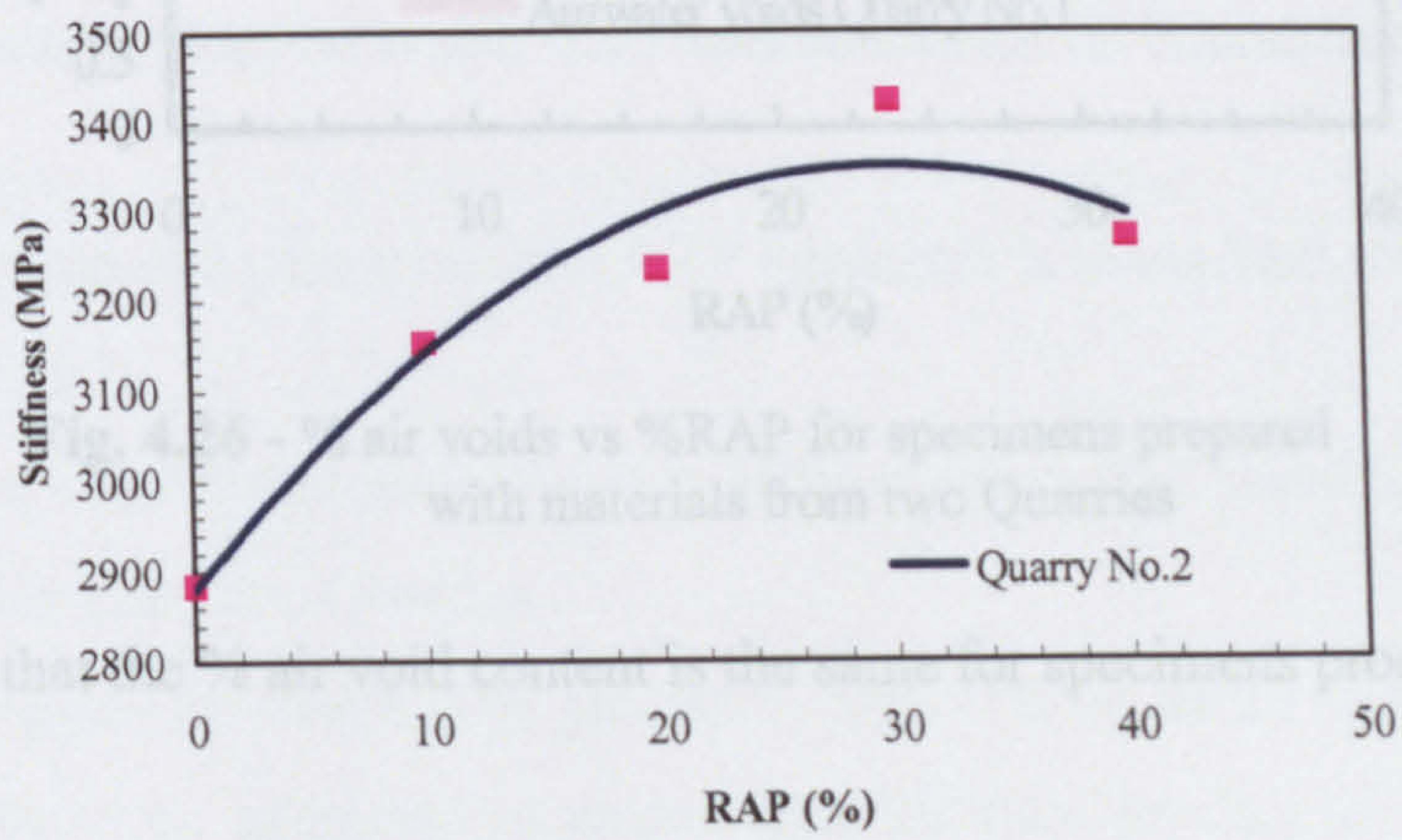


Fig. 4.25 - ITSM results for basecourse from Quarry No. 2

Fig. 4.24 shows that the % air void content reduces with the addition of various amounts of RAP. From Fig. 4.25 it is evident that the stiffness of the material increases as the added RAP material is added. The optimum % RAP that produces the highest stiffness results was found to be 30%. An addition of RAP above 30% resulted in a reduction of the stiffness. These results can be compared to results obtained from Quarry No.1, which were tested using the same parameters. Table 4.22 shows a summary of the result in tabular form. These results are represented in graphical form in Figs 4.26 and 4.27.

Table 4.22 - Comparison of the results from the two Quarries

RAP %	Air voids (%)		ITSM (Mpa)	
	Quarry No. 1	Quarry No. 2	Quarry No. 1	Quarry No .2
0	3.4	3.4	2943	2943
10	2.6	2.9	3236	3153
20	2.8	2.5	3948	3234
30	2.9	2.5	4873	3432
40	2.9	2.6	3648	3269

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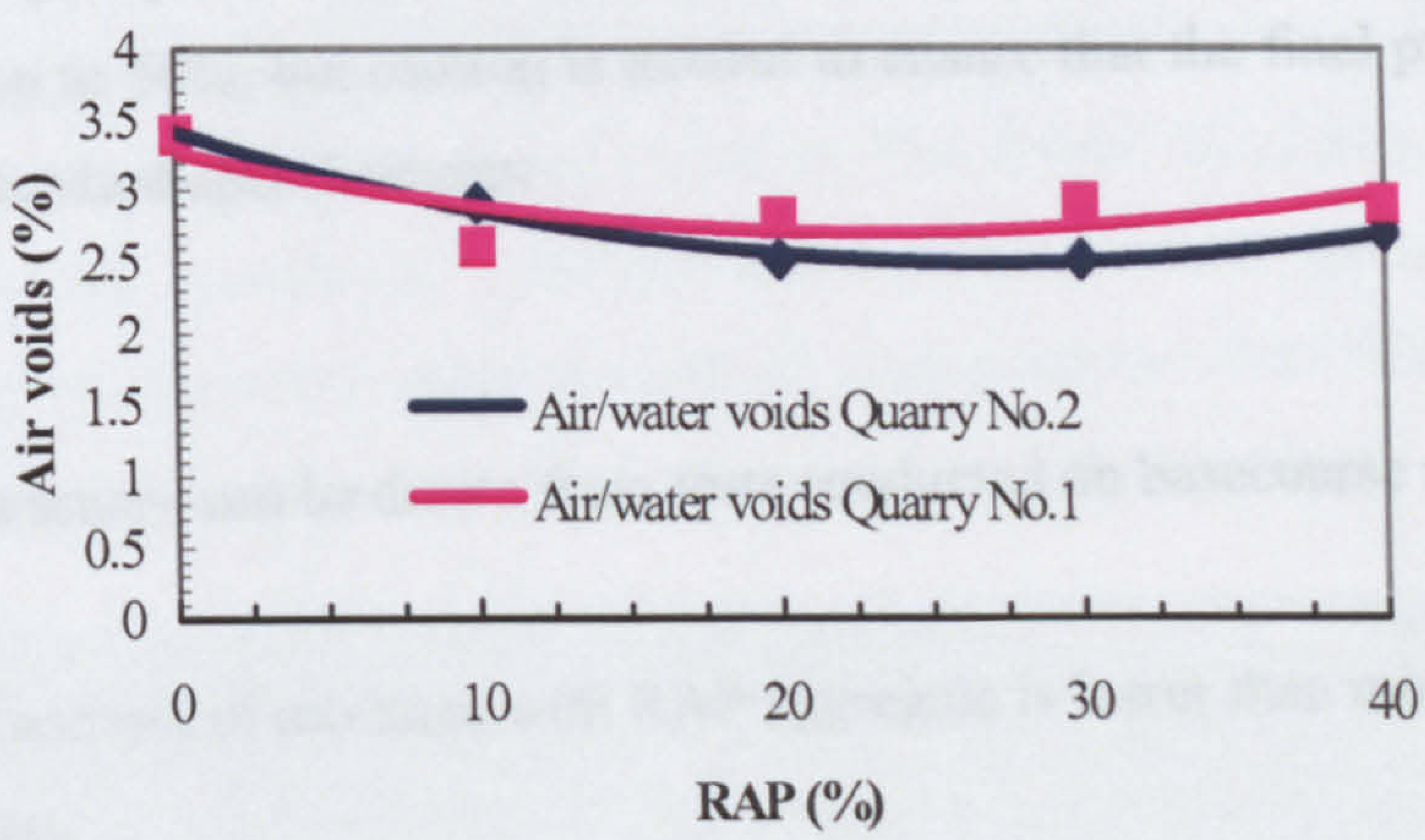
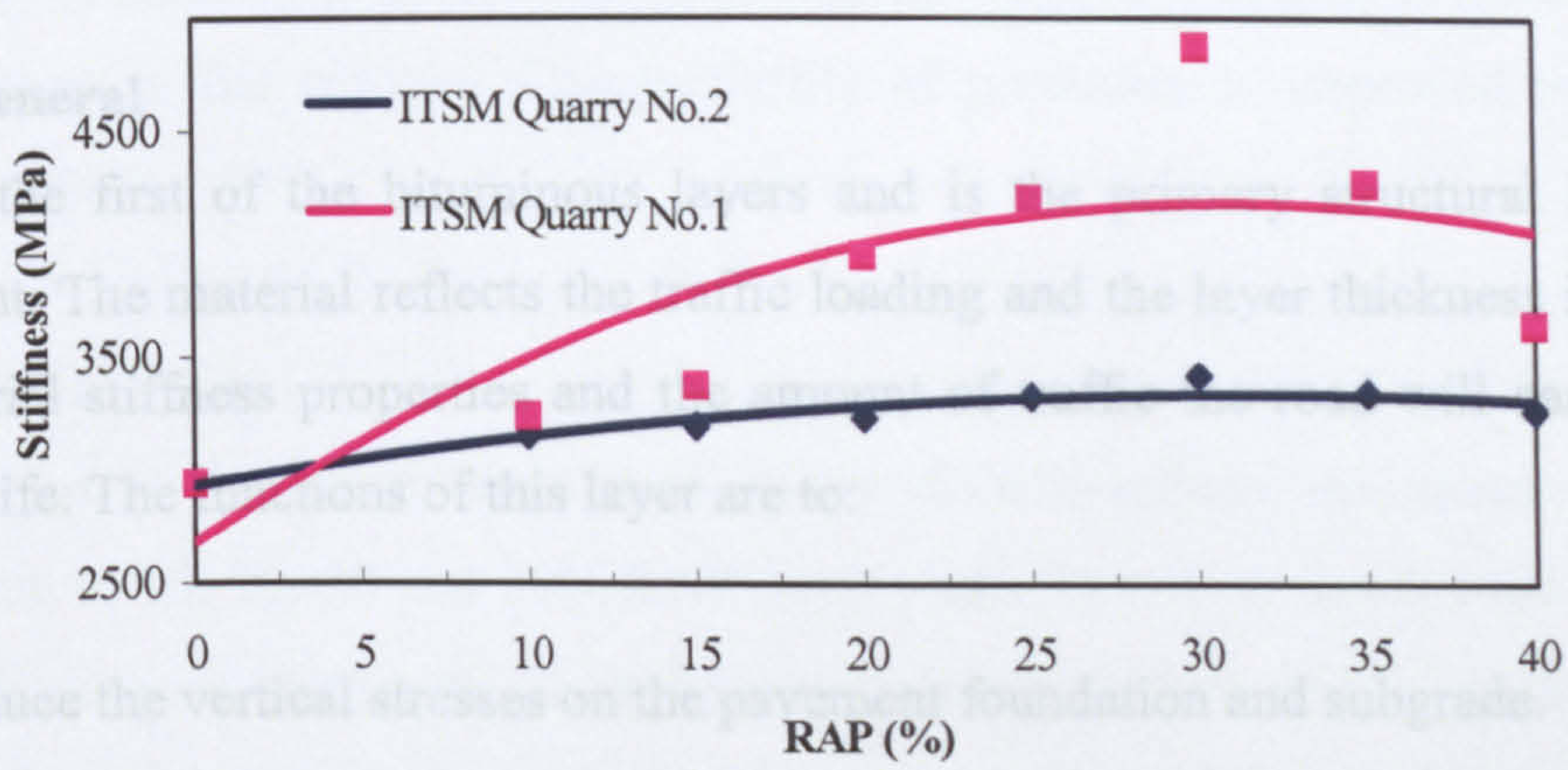


Fig. 4.26 - % air voids vs %RAP for specimens prepared with materials from two Quarries

Fig. 4.26 shows that the % air void content is the same for specimens produced from the two Quarries.



Fig, 4.27 - ITSM results for the mixes containing RAP material from two Quarries

Fig. 4.27 shows that the stiffness of the mixture from Quarry No. 1 increases more rapidly than the specimens containing RAP material from Quarry No. 2. The reason for this could be that there were more contaminants in the RAP obtained from Quarry No. 2. The optimum % RAP was found to be 30% for basecourse prepared with RAP from Quarry No. 1 and No. 2. More RAP aggregate could be added above 30% and up to 50%, but caution is needed to ensure that the final product is compliant with standard specifications.

4.22 Summary

The following summary can be drawn from tests conducted on basecourse material:

- The air void content of mixtures with RAP aggregate is lower than mixtures with no RAP aggregate.
- The optimum % RAP that gives the best stiffness result was found to be 30% using RAP aggregate from two Quarries.
- More RAP aggregate above 30% and up to 50% can be added to basecourse, but caution needs to be exercised to ensure that the final product is compliant with standard specification.
- The use of RAP as a constituent material in the production produces good results in terms of the load bearing capacity of the structure due to the increase in stiffness.

SECTION 5: Roadbase

4.23 General

This is the first of the bituminous layers and is the primary structural layer of the pavement. The material reflects the traffic loading and the layer thickness is a function of material stiffness properties and the amount of traffic the road will carry during it service life. The functions of this layer are to:

- Reduce the vertical stresses on the pavement foundation and subgrade.
- Reduce damage to the surfacing by limiting the amount of deflection under load.
- Provide an accurate surface on which to lay basecourse material.

The above criteria must be met without any premature cracking as well as any internal deformation (Doncaster College (D) 1999). Resistance to permanent deformation depends mainly on aggregate grading and particle characteristics. Mineral aggregates, which are rough and angular when crushed offer better resistance than smooth rounded materials. A low but adequate binder content is needed. Mechanical properties of a roadbase consequently depend on several variables. The objectives of good mix design are to proportion the constituent materials and control the site operation so that satisfactory mechanical properties are realised. A typical composition (Fig. 4.28) of a roadbase is 64% coarse aggregate, 32.2% fine aggregate and 3.8% binder.

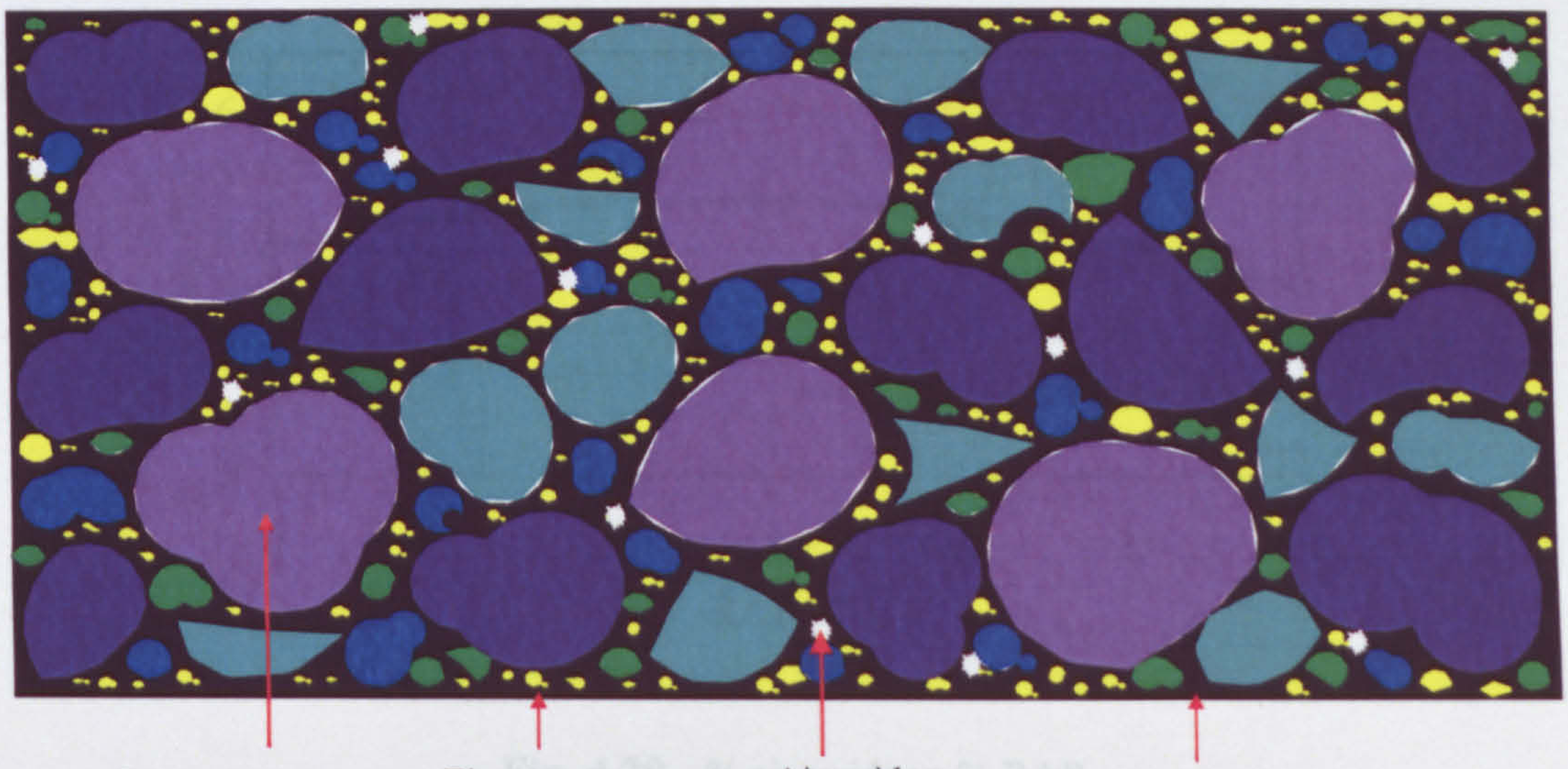


Fig. 4.28 - Typical composition of a roadbase

Durability and compactibility are important properties of a bituminous mixture. Durability is influenced by the binder film thickness on the aggregate relative to the air void content in the mixture. Compactibility of roadbases is improved by the higher binder contents.

4.24 % Air Voids

The gyratory compactor was used to produce roadbase specimens. Specimens containing 0, 10, 15, 20 and 30% RAP were made. The % air voids were obtained for all the specimens using both the weight in air/water method and the parafilm method. Table 4.23 gives the results of air voids, whereas Fig. 4.29 provides these results in graphical form.

Table 4.23 – Air void content results

RAP (%)	Voids (air/water) (%)	S.D.	Voids (parafilm) (%)	S.D.
0	3.7	0.95	5	1.12
10	4.74	1.11	6.07	1.47
15	4.47	0.79	6.09	1.03
20	4.1	1.23	5.9	1.55
30	3.14	0.97	6.3	1.32

Note: Nine specimens were produced for each % RAP

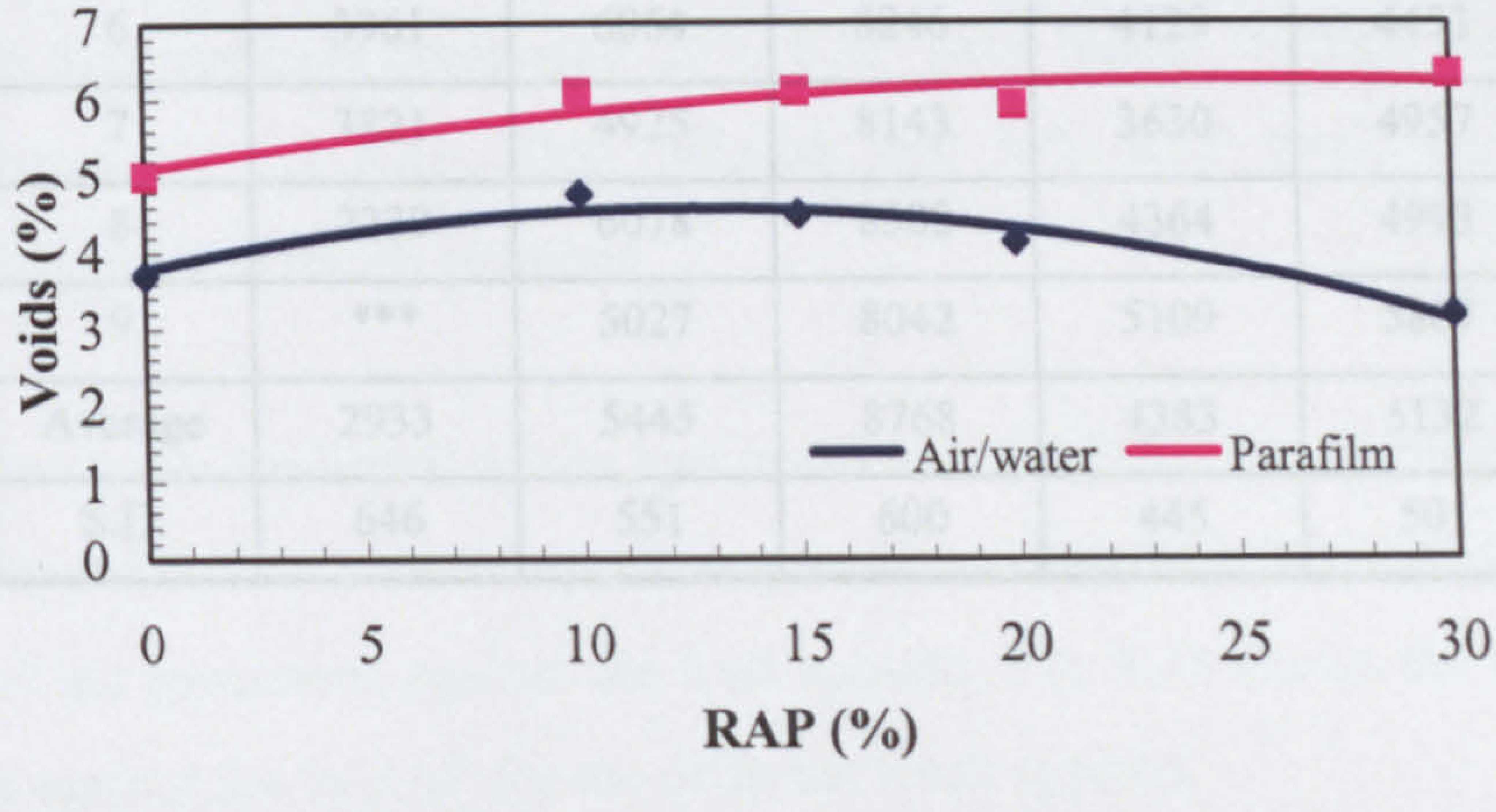


Fig. 4.29 - % air voids vs % RAP

Fig. 4.29 shows that as the % RAP increases the % air voids increase up to the addition of 15% RAP. At 20% RAP the air void content seemed to fall for both methods. At 30% RAP the air voids fell according to air/water method and increased according to the results obtained using the parafilm method. As a result of the fluctuating air void content the optimum % RAP cannot be clearly identified by carrying out a air void content test on its own. ITSM, ITST and ITFT tests were carried out on these specimens in order to determine the optimum % RAP for the roadbase mixture.

4.25 ITSM, ITST and ITFT Test Results

The ITSM test was the first of the three tests to be carried out on the specimens containing different % of RAP. Table 4.24 gives the results obtained for the specimens containing different % RAP. Figs 4.30 and 4.31 show graphical representation of the results obtained from the ITSM test.

Table 4.24 - Results of ITSM test for roadbase containing different % RAP

Specimen No	Stiffness (Mpa)				
	0% RAP	10% RAP	15% RAP	20% RAP	30% RAP
1	2844	6041	9221	4337	5874
2	2917	5058	8743	4818	4962
3	2599	5905	10096	4665	5160
4	2245	4893	9196	4412	4532
5	2841	5019	8323	3986	5455
6	3961	6064	8846	4129	4453
7	3821	4925	8143	3630	4957
8	2239	6078	8302	4364	4993
9	***	5027	8042	5109	5807
Average	2933	5445	8768	4383	5132
S.D.	646	551	600	445	501

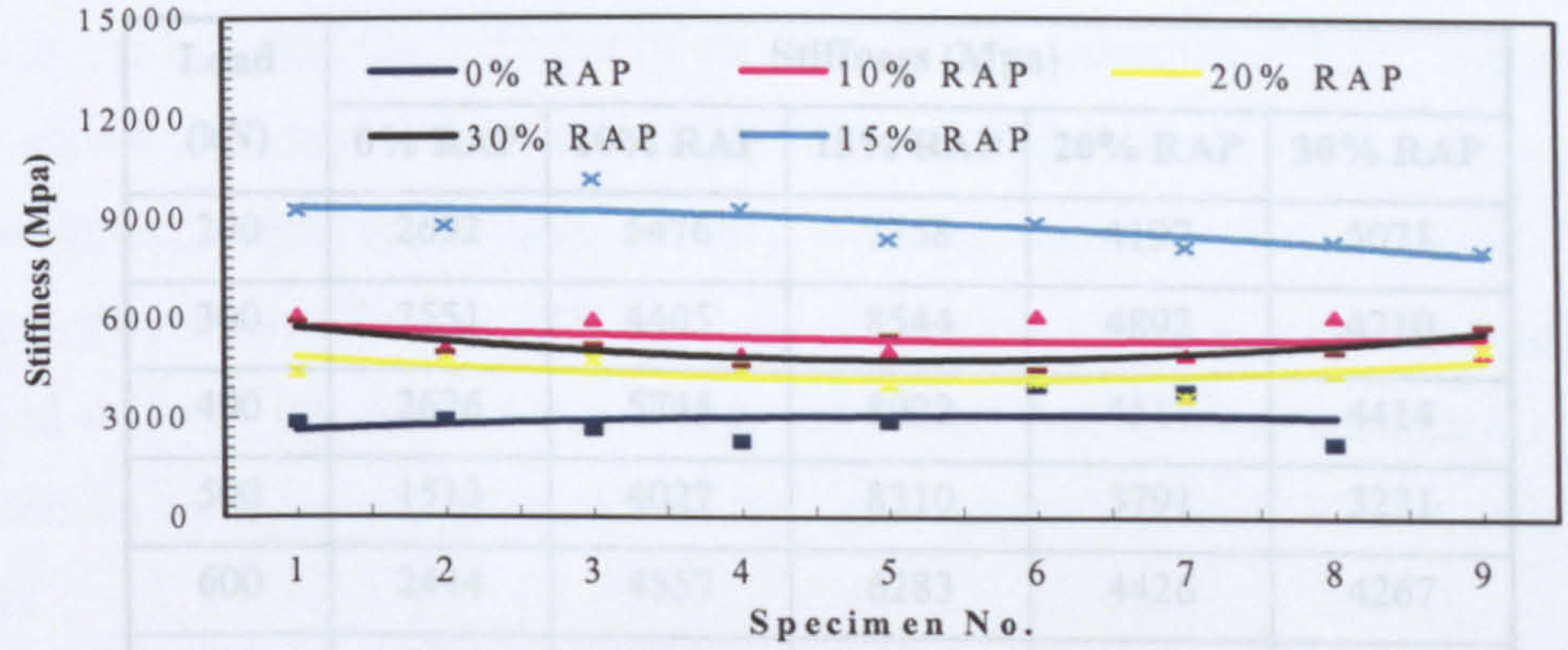


Fig. 4.30 - Results of ITSM at different % RAP

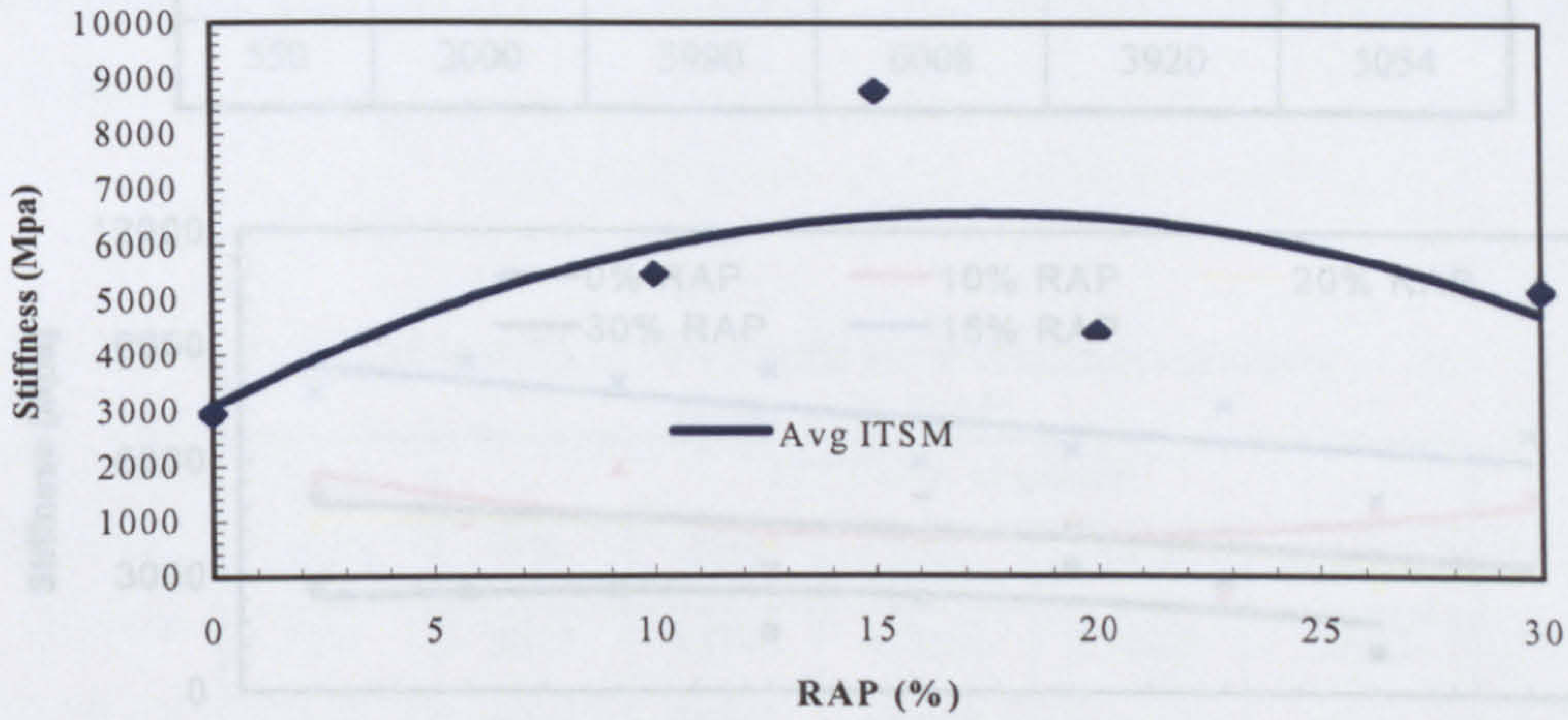


Fig. 4.31 - Average ITSM results at different % RAP

Figs 4.30 and 4.31 shows that material containing 15% RAP yielded the best results. It also shows that roadbase containing 30% RAP gave higher stiffness values than the mixture containing 0% RAP. This indicates that 30% RAP could be added to the mixture with positive results obtained.

The next test to be conducted on the specimens was the ITST test. Table 4.25 shows the results obtained for each set of specimens at the various % RAP. Fig. 4.32 shows the stiffness of the specimens against the load applied. Fig. 4.33 shows the stiffness of the specimens against the % RAP for the different loads applied.

Table 4.25 - ITST results for roadbase with different % RAP

Load (kN)	Stiffness (Mpa) 0% RAP	10% RAP	15% RAP	20% RAP	30% RAP
200	2692	5476	7758	4197	5071
300	2551	4405	8544	4892	4710
400	2626	5745	8022	4517	4414
500	1513	4027	8310	3791	3231
600	2444	4557	6283	4426	4267
700	3238	2854	7422	3739	3536
800	2827	4773	5041	2796	3671
900	1056	5220	6726	3196	3143
550	2000	3990	6008	3920	5054

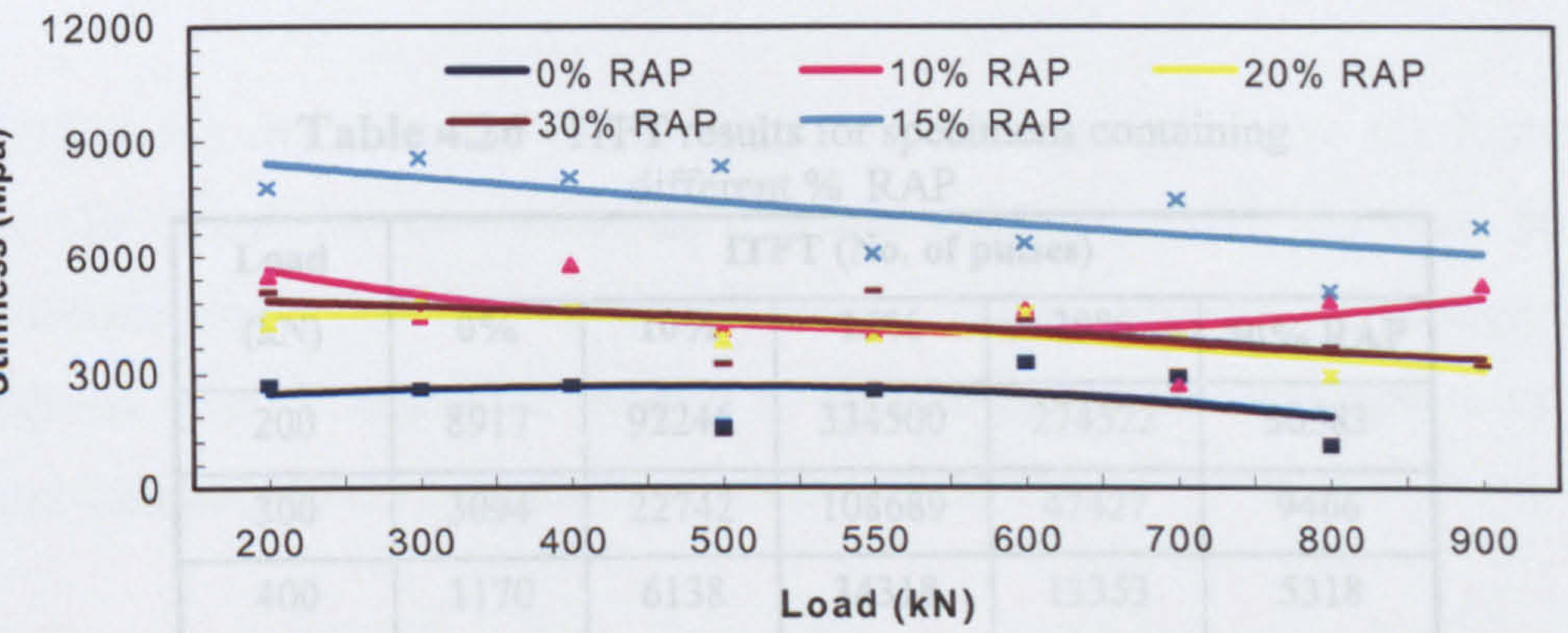


Fig. 4.32 - Load vs ITST stiffness at the different % of RAP

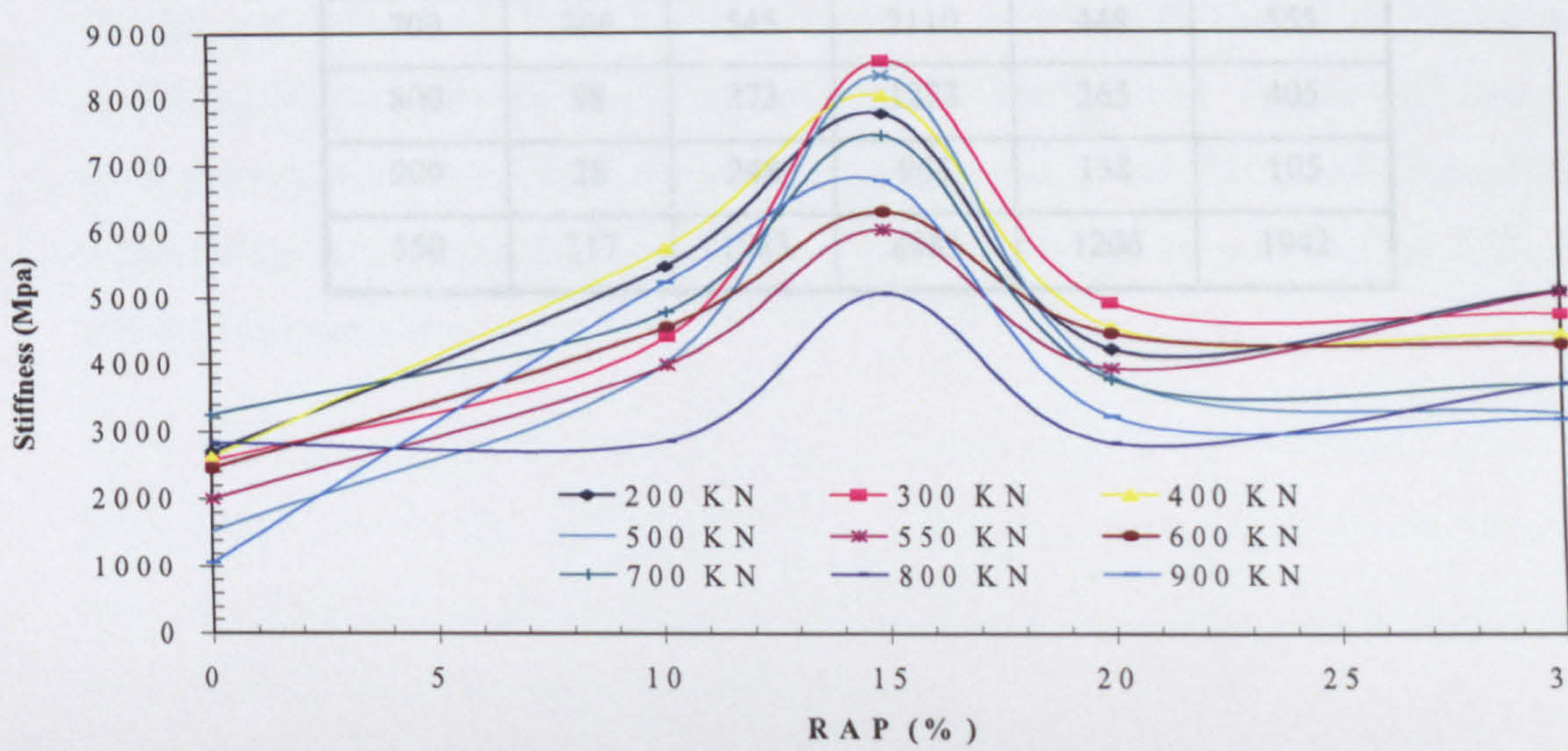


Fig. 4.33 – ITST Stiffness vs % RAP at the different loads applied

Figs 4.32 and 4.33 show that the optimum % RAP for this mixture is 15%. From Fig. 4.30 it is evident that the stiffness of all the mixtures containing RAP aggregate gave higher values than the mixture containing 0% RAP. Fig. 4.33 shows that the stiffness of the specimens seems to increase steadily with the addition of more RAP. With the addition of between 10% and 20% RAP the stiffness of the specimens rises to a peak at 15% and starts to fall beyond that. From the result obtained it is transparent that 30% RAP can be used in this mixture and still give higher stiffness values than the base mixture.

The next test to be carried out on the specimens was the ITFT test. Table 4.26 shows the results obtained from the ITFT. A graphical representation of the results can be seen in Fig. 4.34.

Table 4.26 - ITFT results for specimens containing different % RAP

Load (kN)	ITFT (No. of pulses)				
	0%	10%	15%	20%	30% RAP
200	8917	92246	334500	274522	30583
300	3094	22742	108689	47427	9466
400	1170	6138	34318	13353	5318
500	352	2283	10005	2344	2249
600	282	1186	4175	745	891
700	206	545	2110	448	555
800	98	273	1273	265	405
900	28	248	963	138	105
550	317	1983	8883	1206	1942

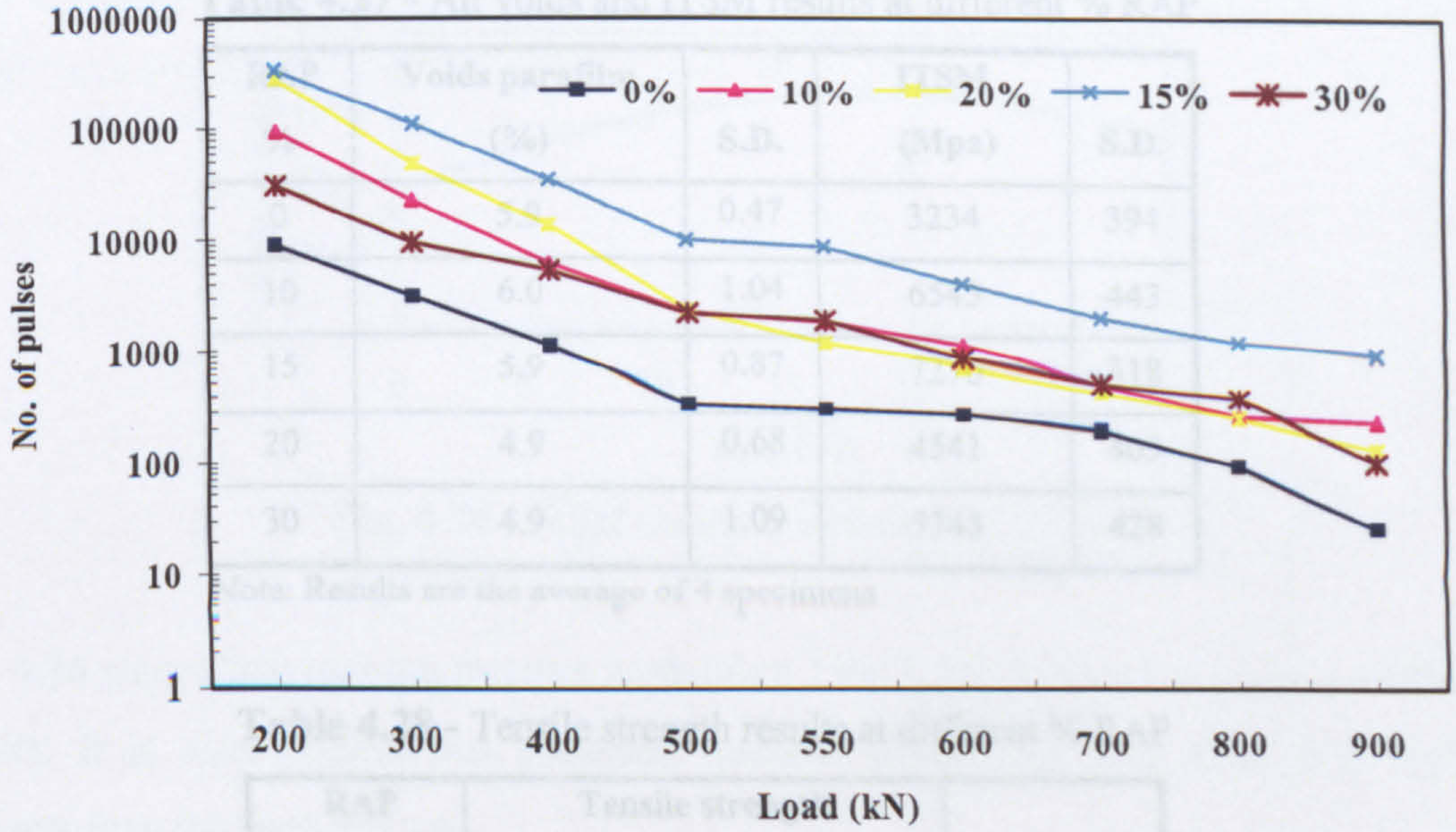


Fig. 4.34 - Load vs log No of pulses

Fig. 4.34 shows that material containing 15% RAP aggregate gave the best fatigue results. The figure also shows that the mixtures containing RAP aggregate produced higher fatigue results than the mixture containing 0% RAP aggregate. This suggests that the addition of RAP aggregate resulted in an increase in the life expectancy of the mixtures compared to the base mixture that has 0% RAP.

4.22 Tensile Strength Test

After carrying out stiffness and fatigue testing, four more specimens containing 0, 10, 15, 20, and 30% RAP were mixed, compacted and tested. The % air voids were calculated followed by the ITSM test and tensile strength test. Tables 4.27 and 4.28 show the results obtained from the air void calculations, ITSM and tensile strength tests respectively. A graphical representation of these results can be seen in Figs 4.35, 4.36 and 4.37 respectively.

Table 4.27 - Air voids and ITSM results at different % RAP

RAP %	Voids parafilm (%)	S.D.	ITSM (Mpa)	S.D.
0	5.9	0.47	3234	394
10	6.0	1.04	6545	443
15	5.9	0.87	7270	318
20	4.9	0.68	4541	403
30	4.9	1.09	5745	428

Note: Results are the average of 4 specimens

Table 4.28 - Tensile strength results at different % RAP

RAP (%)	Tensile strength (kN/mm2)	S.D.
0	9.52	0.66
10	15.61	0.49
15	14.27	1.26
20	10.86	1.42
30	11.77	0.39

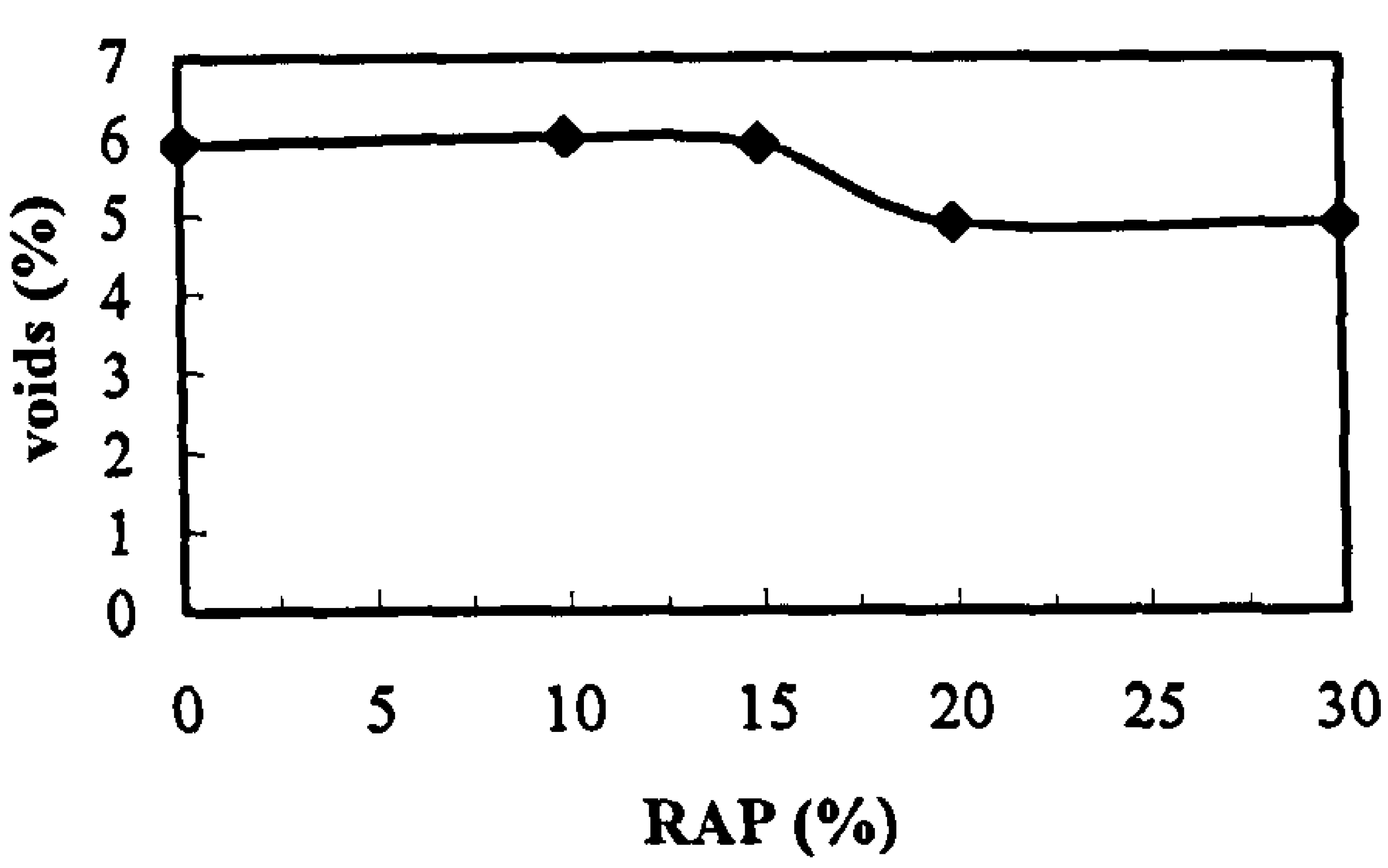


Fig. 4.35 - % air voids vs % RAP

Fig. 4.35 shows that the % air voids remained constant up to the addition of 15% RAP. With the addition of 20% RAP the air voids fell slightly. In order to assess the results fully the ITSM and tensile strength test would have to be considered first to determine which % RAP which gives the best performance.

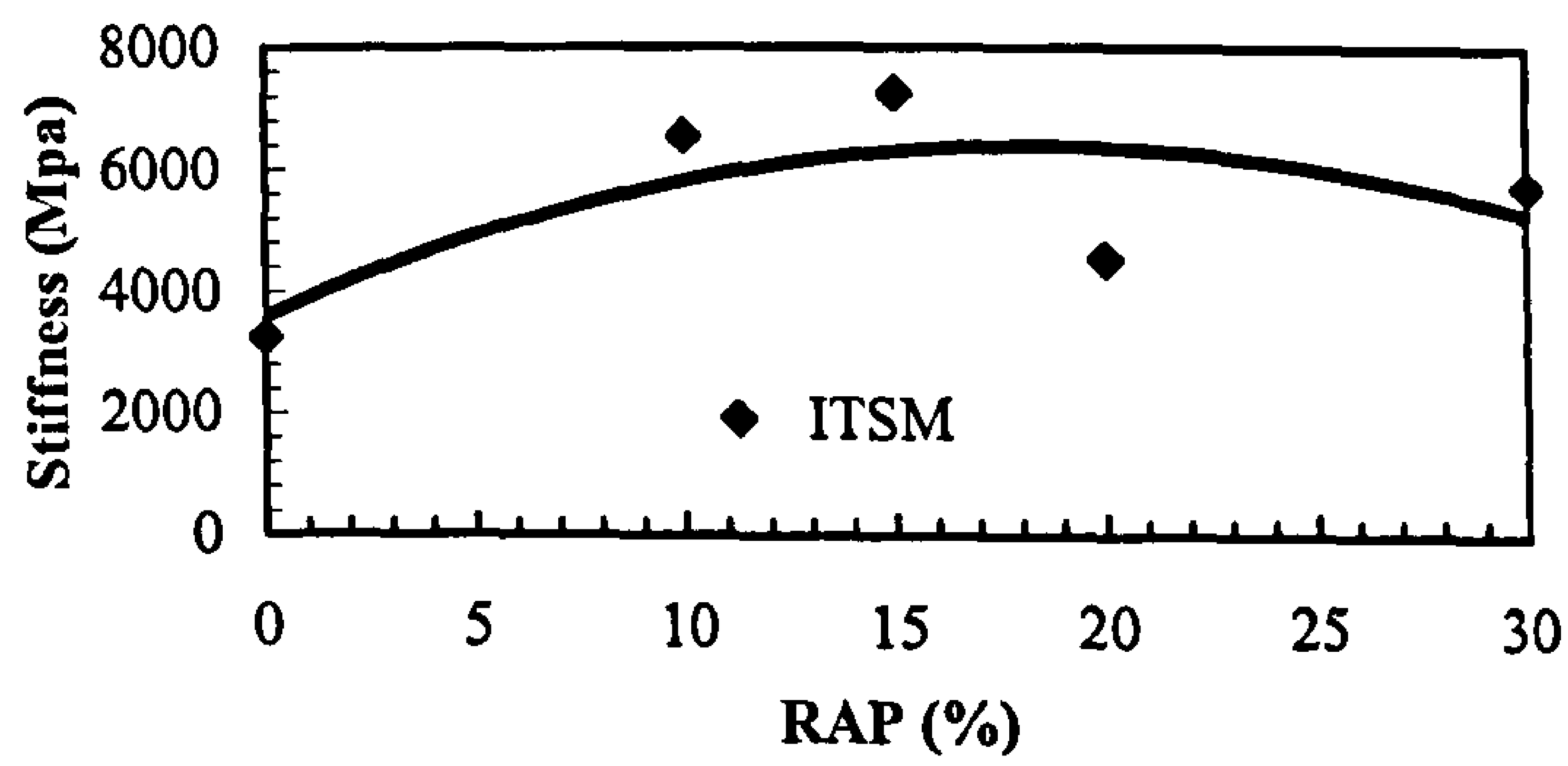


Fig. 4.36 - ITSM results at various % of RAP

Fig. 4.36 shows that that the mixture containing 15% RAP yielded the highest stiffness results. It is also evident that roadbase material containing 30% RAP had higher stiffness than the base mixture.

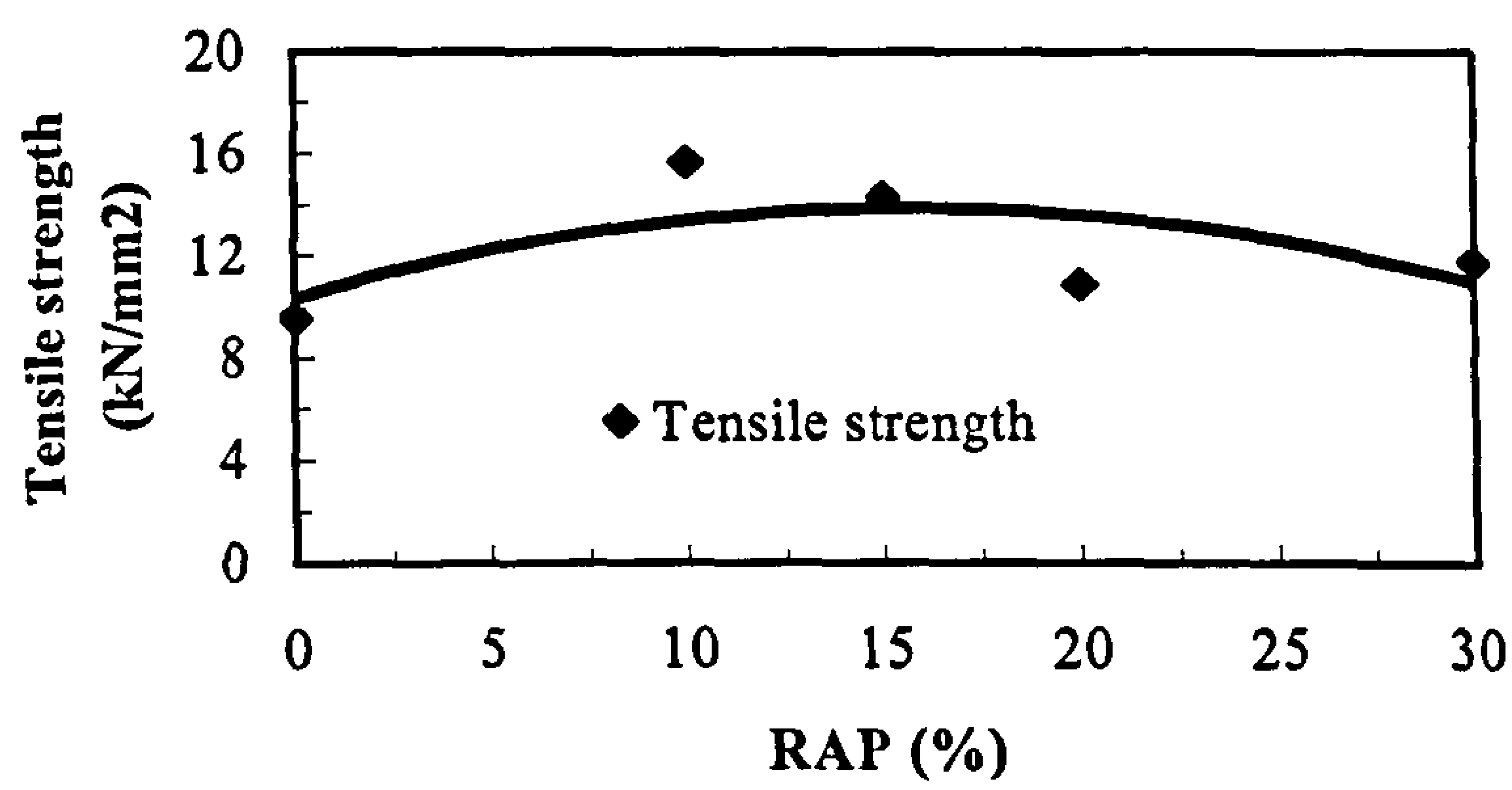


Fig. 4.37 - Tensile strength results at different % RAP

Fig. 4.34 shows that the tensile strength of the mix reached to an optimum value at 15% RAP. The result obtained support all other results from tests carried out on roadbase which give the optimum value of RAP as 15%.

4.27 Cantabro Test

The cantabro test as described in Section 3.9.11 was carried out on specimens compacted in a gyratory compactor. The cantabro test was carried out in accordance with prEN 12697: Part 17:, 1998. Four specimens were mixed and compacted at 0, 10, 15, 20 and 30% RAP. The % air voids was determined and ITST test conducted before the cantabro test was performed. Tables 4.29 and 4.30 shows the results obtained for the

% air voids, ITSM and cantabro tests respectively. A graphical representation of these results can be seen in Figs 4.38, 4.39 and 4.40 respectively.

Table 4.29 - Air voids and ITSM results at different % RAP

RAP (%)	Voids parafilm (%)	S.D.	ITSM (Mpa)	S.D.
0%	6	0.98	4345	280
10%	5.9	1.36	7499	297
15%	6	0.76	8302	529
20%	5	0.82	4979	247
30%	4.5	0.66	6960	302

Table 4.30 - Cantabro results for different % of RAP

% RAP	% Wear	S.D.
0	11.007	2.6
10	10.548	3.81
15	7.8	1.13
20	10.5	1.69
30	11.7	1.71

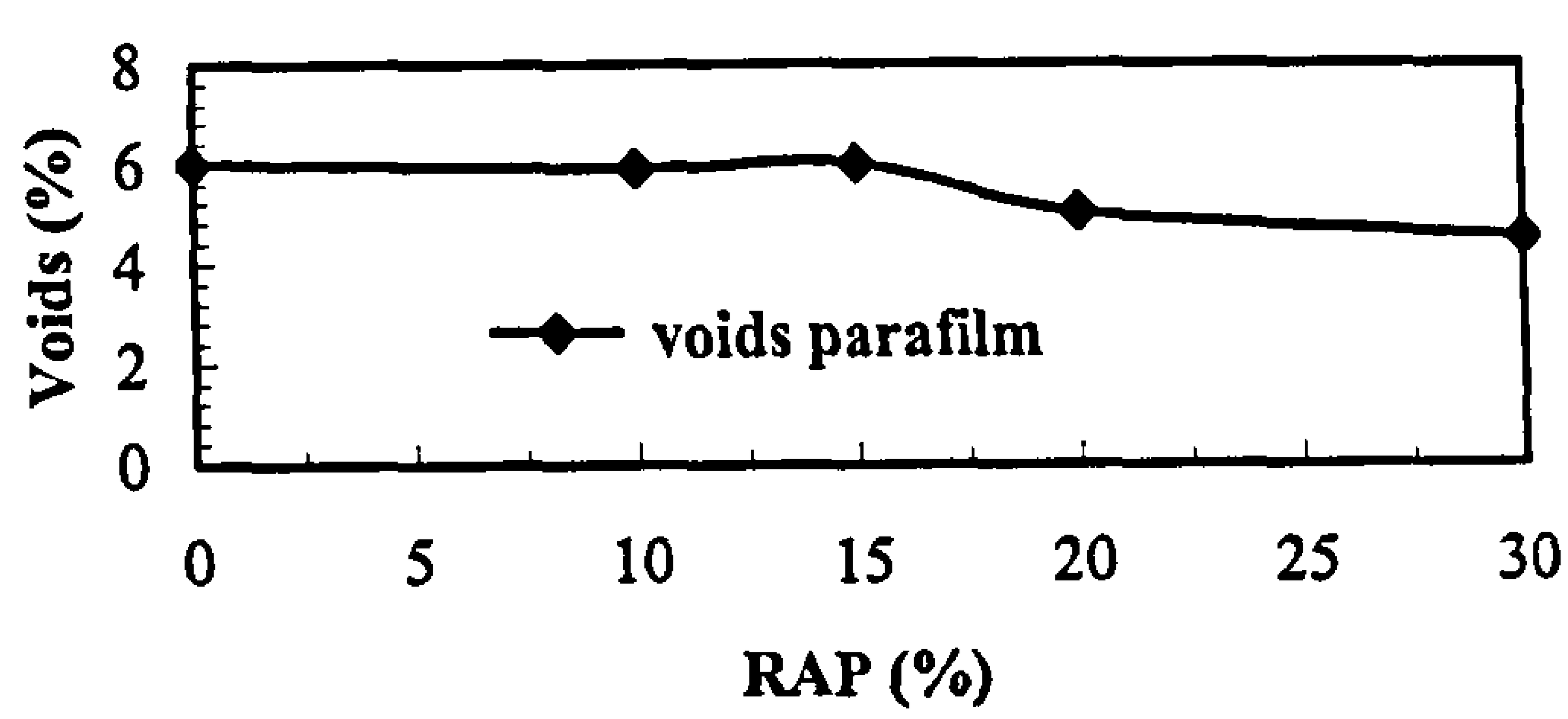


Fig. 4.38 - Air voids at different % RAP

Fig. 4.38 shows that the % air voids decreased with the addition of 20% RAP as was the case with samples made for the tensile test.

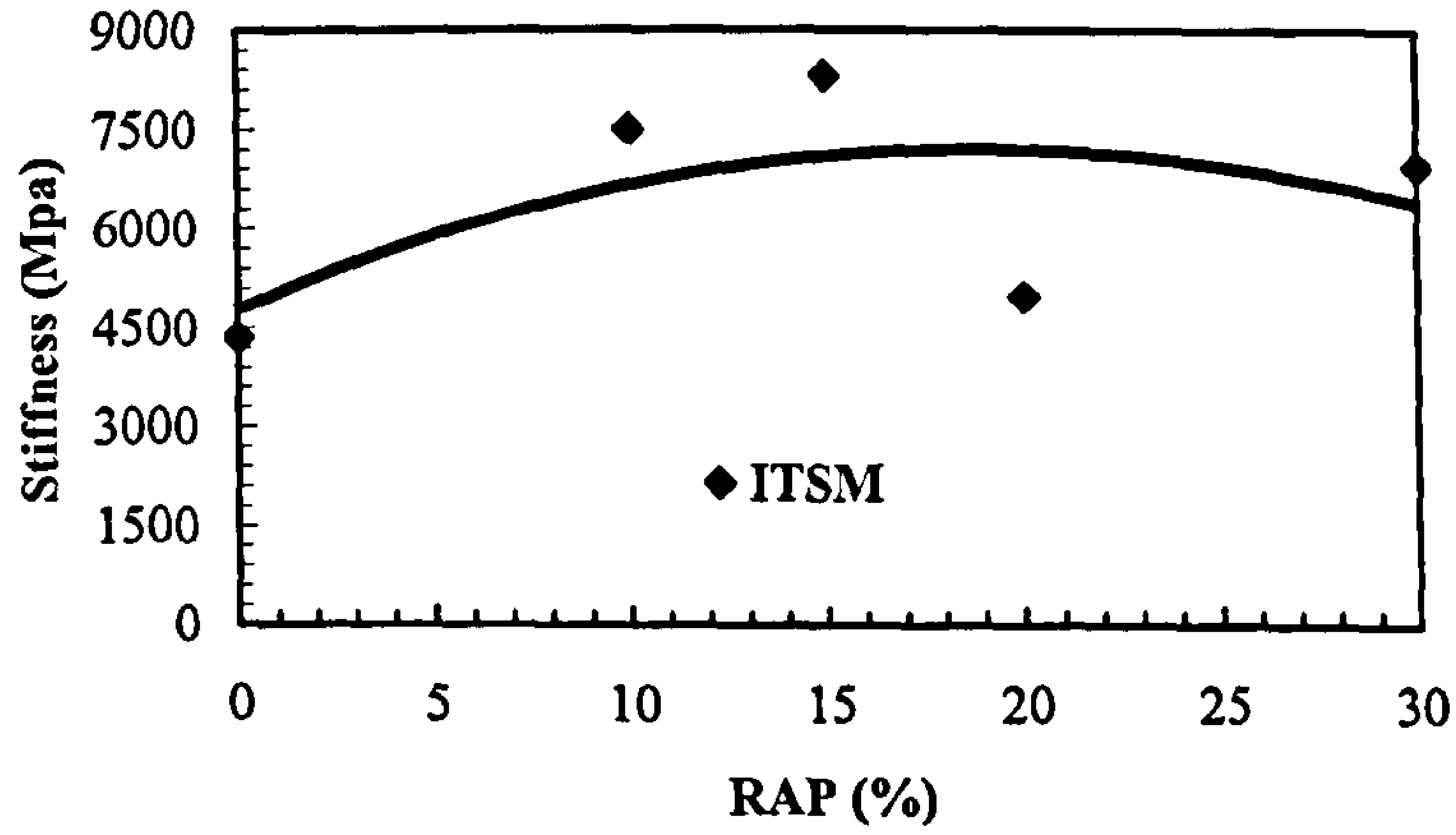


Fig. 4.39 - ITSM results for % RAP

Fig. 4.39 shows that the optimum % RAP for this mixture is 15%. Again these results show that 30% RAP can be added to the mixture and still give higher stiffness values than the specimens containing 0% RAP. After carrying out the ITSM test, the cantabro test was carried out, the results of which can be seen below.

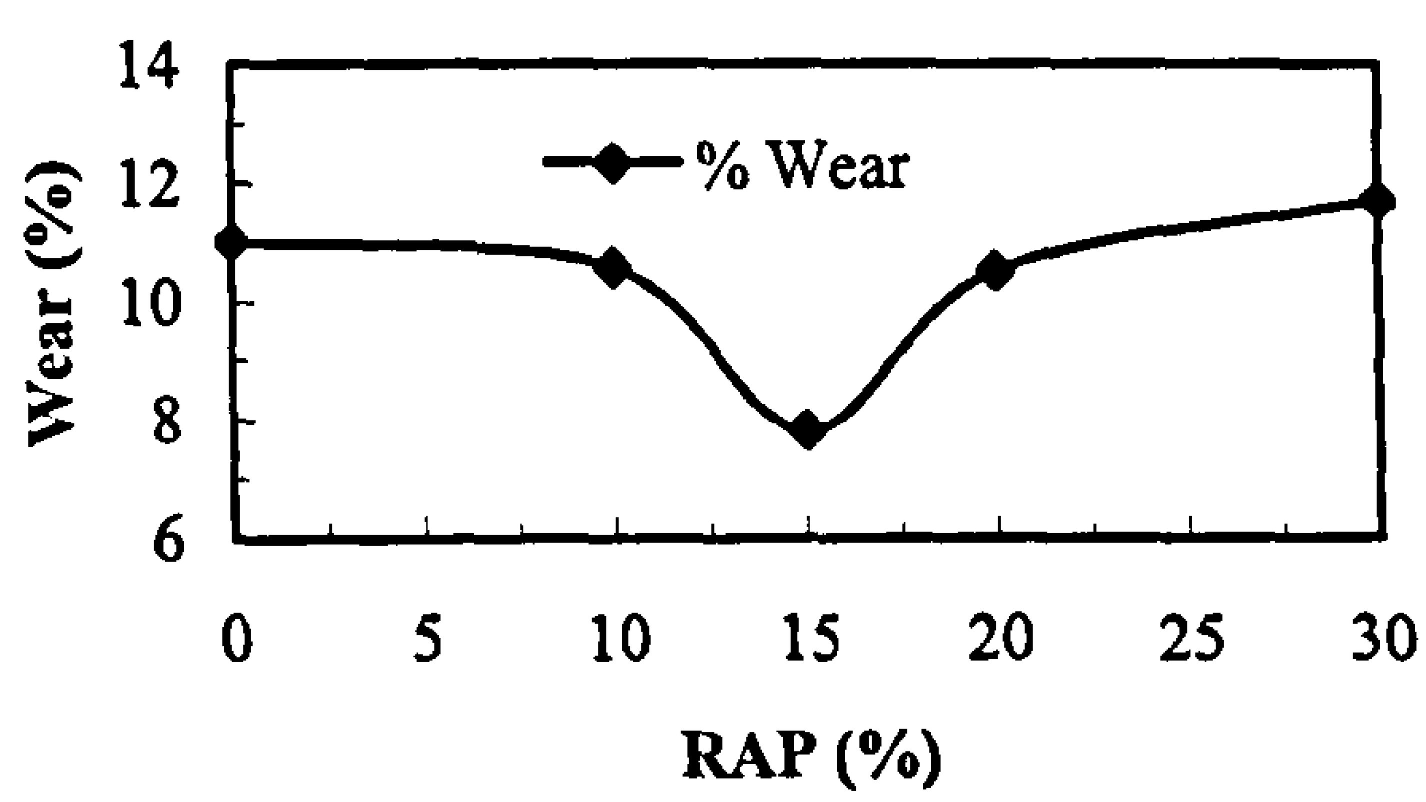


Fig. 4.40 - Cantabro results for different % RAP

Fig. 4.40 shows that the % wear decreased significantly with the addition of 15% RAP. It is also evident from the graph that samples tested containing 30% RAP gave a slightly higher % wear value than the base mix. As this value is less than 1% higher and the standard deviation of the results is lower at 30% (1.71) than the base mix (2.6), it would be realistic to say that 30% RAP can also be used in the production of roadbase and not effect the mix in terms of % wear measurement.

4.28 Summary

From the experimental work carried out on roadbase the results obtained can be expressed in tabular form. These results can be seen in Table 4.31.

Table 4.31 - Summary of results

Test carried out	Optimum %
ITSM	15
ITST	15
ITFT	15
Tensile strength	10
% Wear	15

From the results obtained in this chapter the following conclusions can be made:

- The use of RAP in the production of roadbase increases stiffness, fatigue and tensile strength of the mixture.
- A reduction in the % wear can be expected for mixtures containing RAP up to and including 20%.
- The parafilm method of measuring air voids gave more realistic results.
- The ITSM results showed that the stiffness of the various specimens increased to a maximum at 15% and started to decrease there after. This criterion was found to be the same for the ITST, ITFT and tensile strength tests.
- The % wear decreased to a minimum at 15% and increased there after.
- From the results the optimum percentage RAP for this mix is 15%, while 30% RAP can be added to the mixture and still have higher stiffness, fatigue and tensile strength values than the base mix containing 0% RAP.

SECTION 6: Further testing

4.29 General

After carrying out tests on the wearing course, basecourse and roadbase mixtures, further investigations were conducted to determine:

1. The effects of adding cold RAP aggregate into the mixture.
2. Why the RAP aggregate caused an increase in stiffness, fatigue and tensile strength of the mixture till optimum percentage of added RAP was met, at which point a reduction in stiffness occurred there after.

4.30 Effects of Adding Cold RAP

The problem with heating RAP aggregate in a production plant scenario is that the binder from the heated RAP gets caught on the screens and clogs them over time. on the other hand cold RAP can be added into the mixer as can be seen in Fig. 2.19 thus alleviating the problem. Tests were carried out on roadbase specimens at 10, 20 and 30% RAP. Before mixing commenced the amount of extra heat required to bring the overall temperature of the mixture to 155°C had to be determined. To achieve this the temperature of the mixtures were increased by 10 percent for the addition of 10% RAP, 20% for the addition of 20% RAP and 30% for the addition of 30% RAP. Table 4.32 shows the temperature at which the respective mixtures were mixed.

Table 4.32 - Corrected temperatures for the addition of 0,10, 20 and 30% RAP

RAP (%)	Original temperature	Increase (%)	Mix temperature
0	155°C	0	155°C
10	155°C	10	172°C
20	155°C	20	194°C
30	155°C	30	221°C

After determining the temperature the virgin aggregate had to be heated for the different mixtures, specimens were compacted in the gyratory compactor. The ITSM, ITST and ITFT tests were carried out. Table 4.33 shows the results obtained for the three tests. Graphs of ITSM, ITST and ITFT were plotted. These can be viewed in Figs 4.41, 4.42 and 4.43 respectively. These results were then compared with results obtained from roadbase in which all the material was heated to 155°C. Table 4.34 shows these results. Graphs were drawn to assess the correlation between the two methods of heating the aggregates. Figs 4.44, 4.45 and 4.46 shows the correlation between the methods for heating the aggregate in terms of ITSM, ITST and ITFT respectively.

Table 4.33 - ITSM, ITST AND ITFT results for three different % RAP

	RAP (%)	0	10	20	30
ITSM (Mpa)	(Mpa)	2933	5148	5398	5736
S.D.	-	646	565	377	462
ITST (Mpa)	200kN	2674	4423	4936	5019
	400kN	2626	4519	5294	5104
	600kN	2444	4903	5277	5489
	800kN	2827	3918	4436	4294
ITFT (No. of pulses)	200kN	8917	27737	31613	109372
	400kN	1117	2695	3497	2715
	600kN	282	317	466	562
	800kN	98	299	212	237

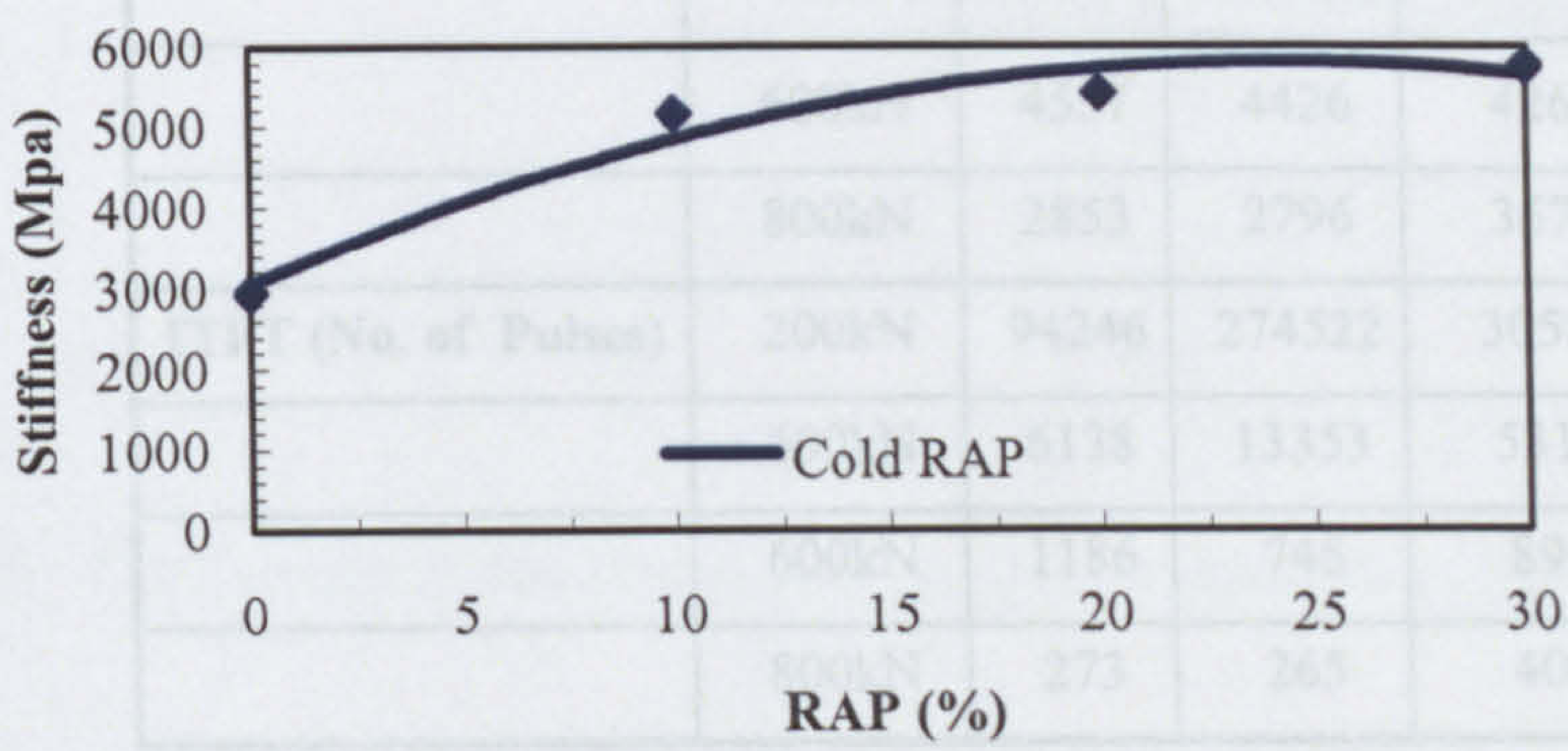


Fig. 4.41 - Average ITSM results at different % RAP

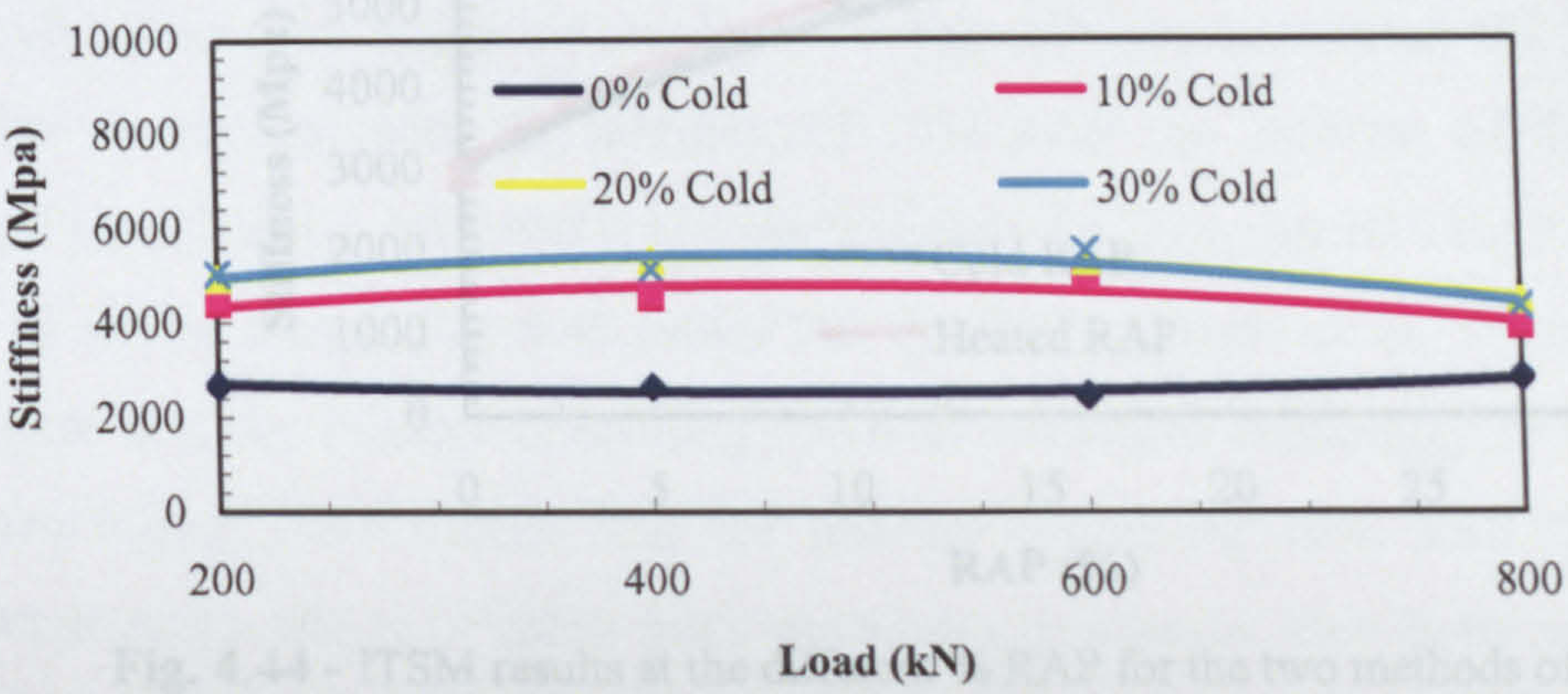


Fig. 4.42 - ITST load vs % RAP

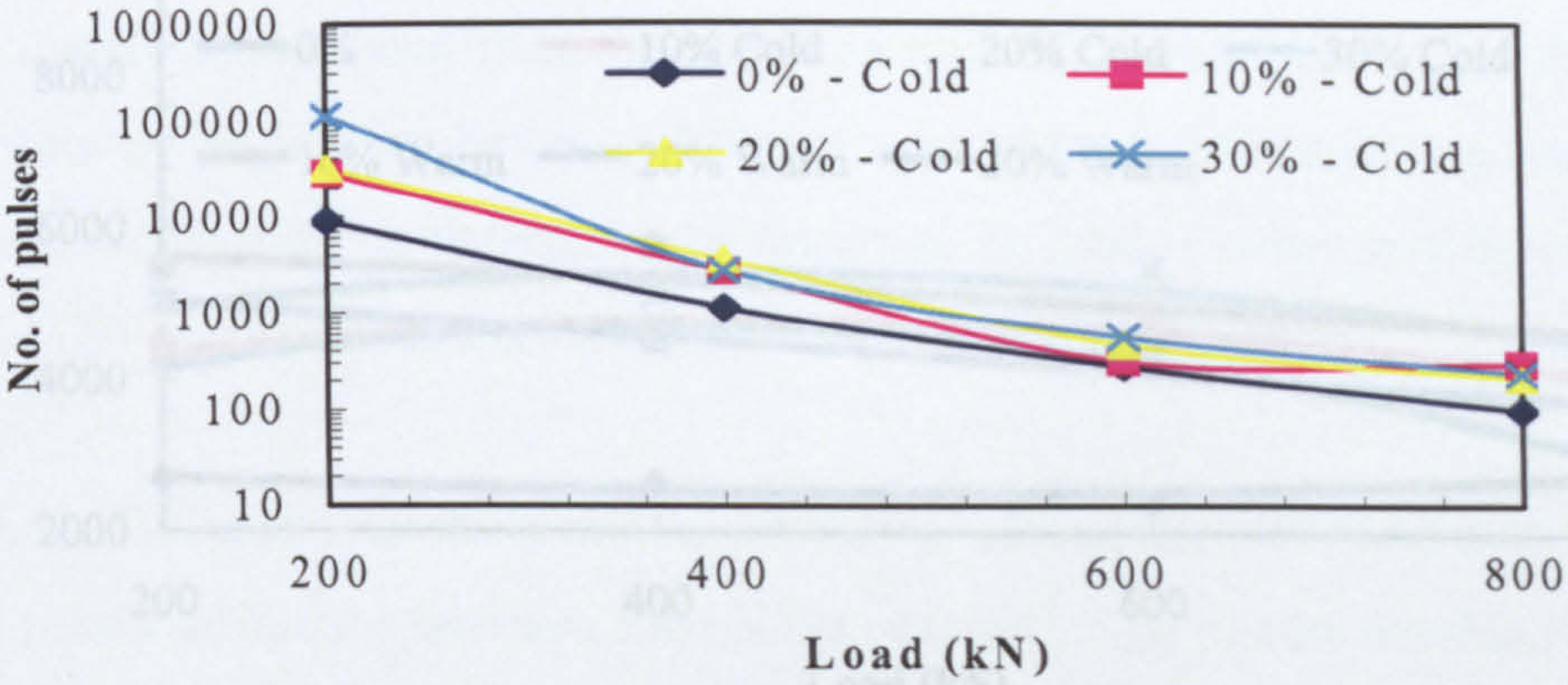


Fig. 4.43 - ITFT load vs No. of pulses

Table 4.34 - ITSM, ITST and ITFT results for three different % RAP in which all the aggregates were heated

	RAP (%)	10	20	30
ITSM (Mpa)	AVG	5445	5383	5132
ITST (Mpa)	200kN	5467	4197	5071
	400kN	5745	4517	4416
	600kN	4557	4426	4267
	800kN	2853	2796	3671
ITFT (No. of Pulses)	200kN	94246	274522	30583
	400kN	6138	13353	5318
	600kN	1186	745	891
	800kN	273	265	405

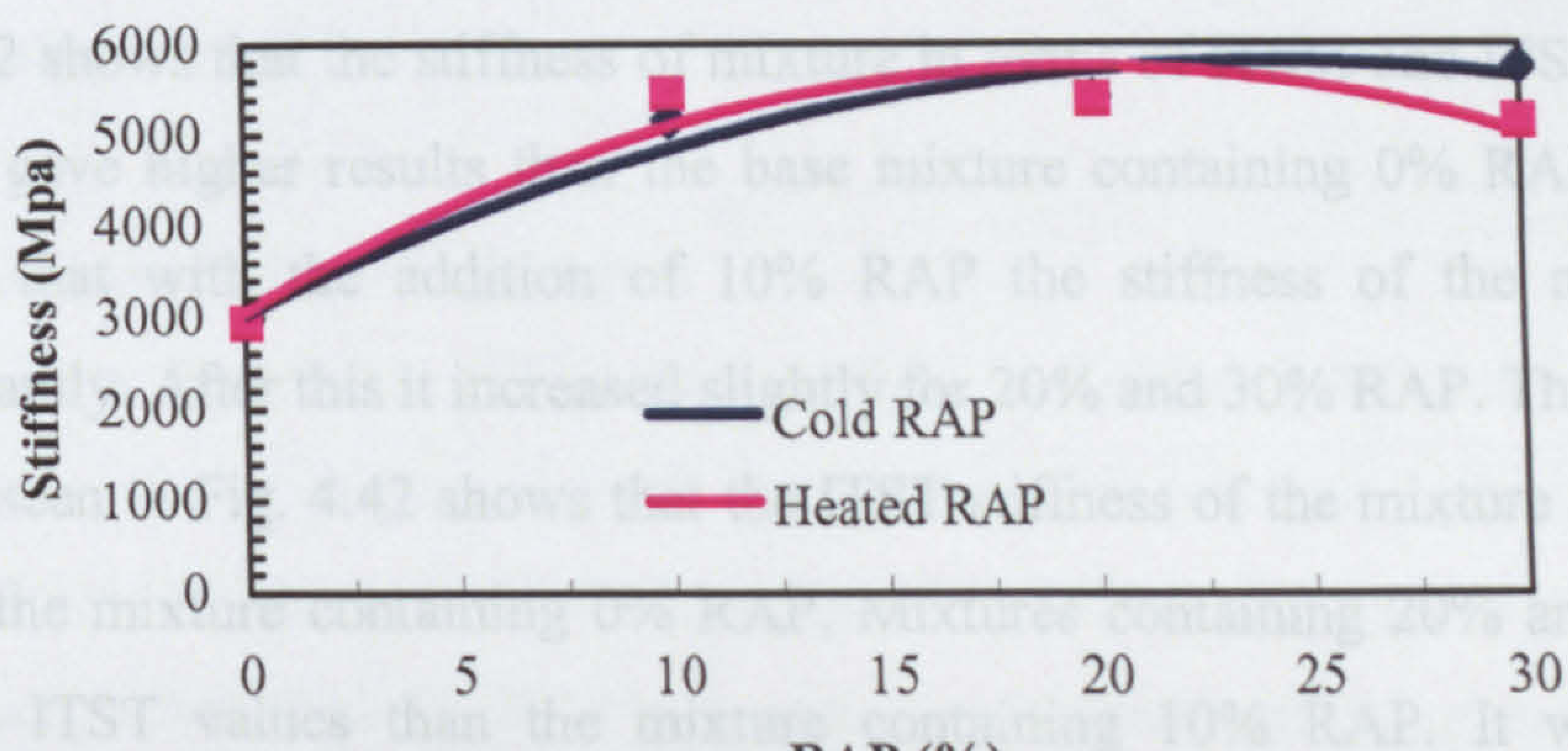


Fig. 4.44 - ITSM results at the different % RAP for the two methods of heating the RAP aggregate

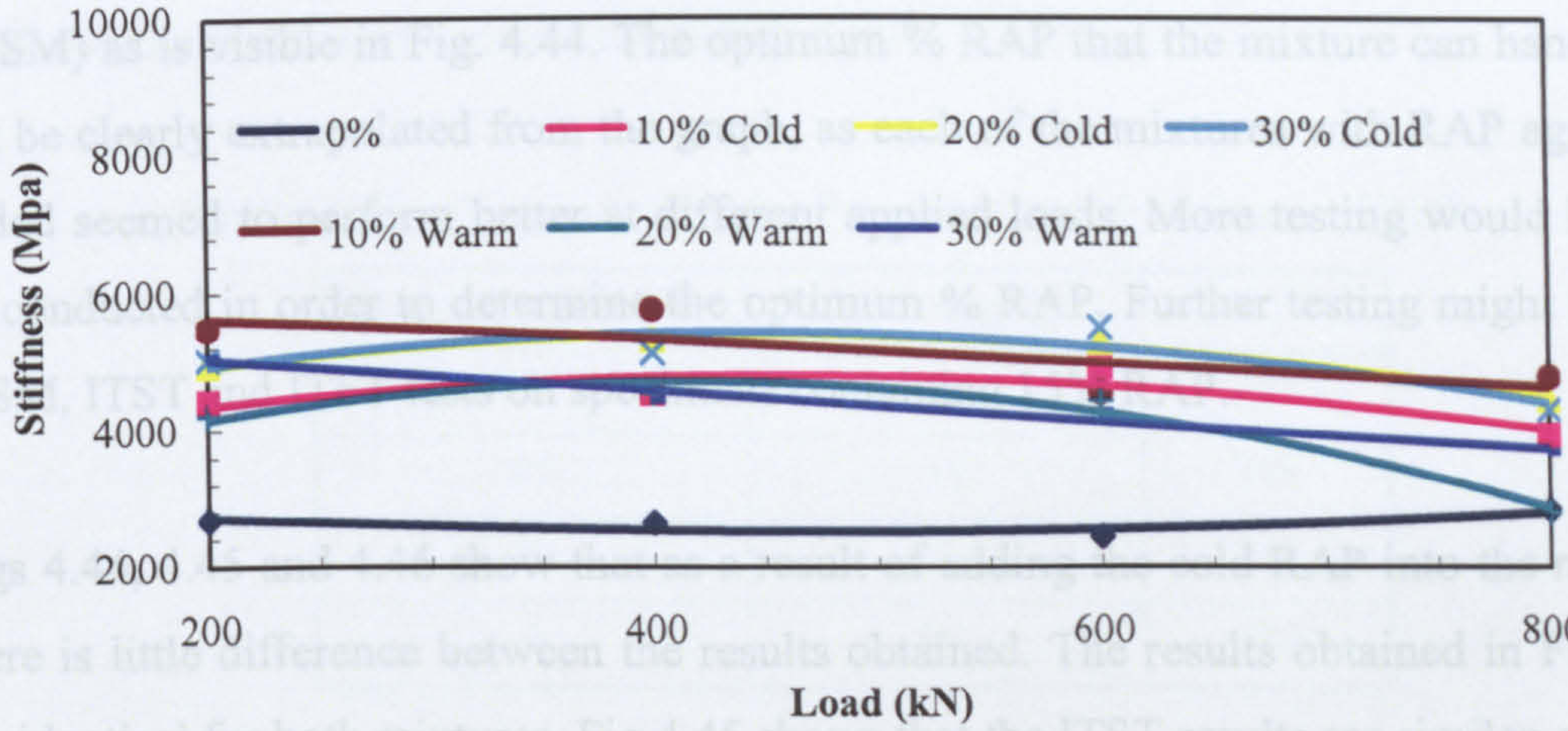


Fig. 4.45 - ITST results at different % RAP for the two methods of heating the aggregate

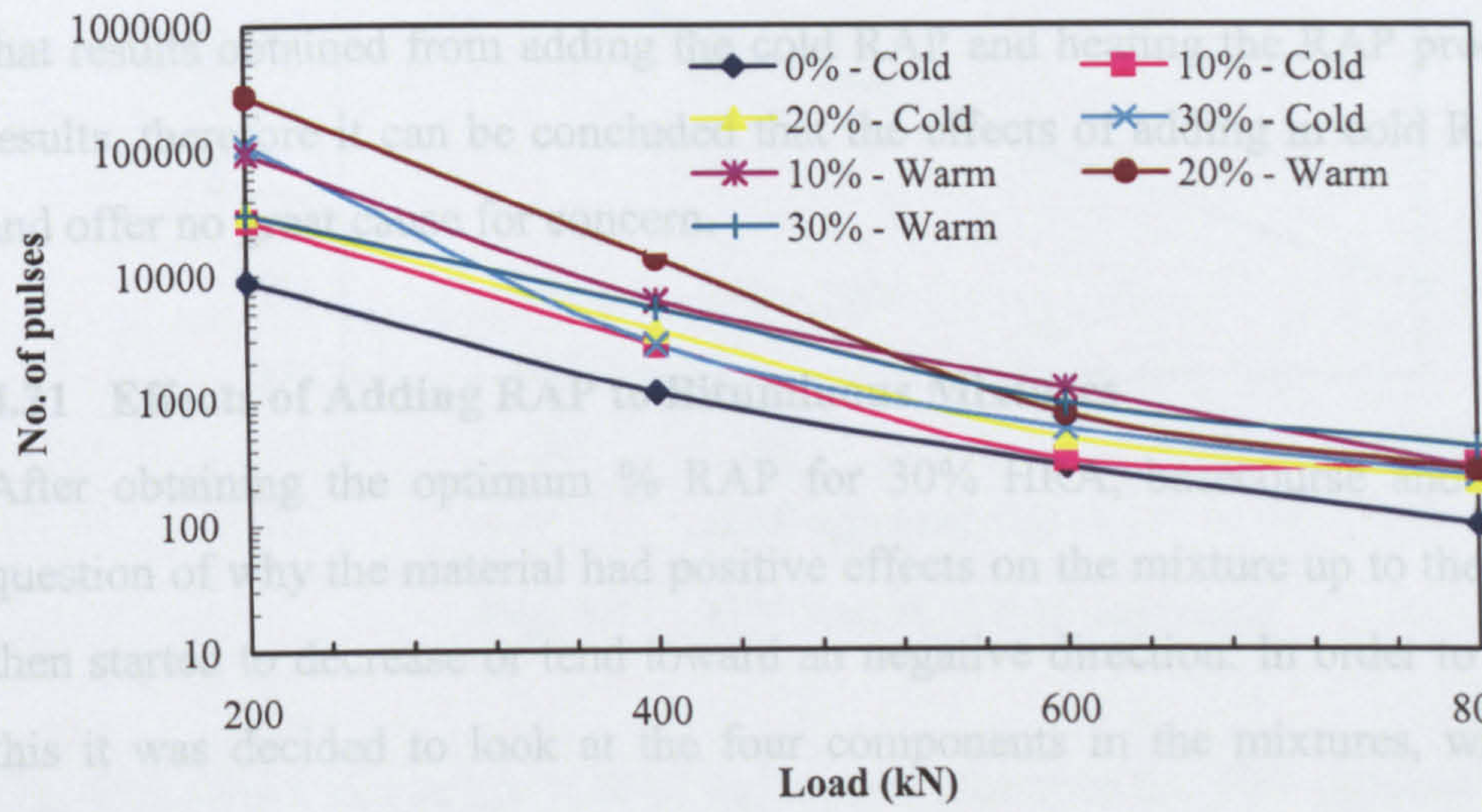


Fig. 4.46 - ITFT results at different % RAP for the two methods of heating the aggregate

Figs 4.41 and 4.42 shows that the stiffness of mixture in terms of ITSM and ITST with cold RAP added gave higher results than the base mixture containing 0% RAP. Fig. 4.41 also shows that with the addition of 10% RAP the stiffness of the mixture increased significantly. After this it increased slightly for 20% and 30% RAP. The ITST results as can be seen in Fig. 4.42 shows that the ITST stiffness of the mixture at 10% was higher than the mixture containing 0% RAP. Mixtures containing 20% and 30% had even higher ITST values than the mixture containing 10% RAP. It was not transparent as to which of these gave the higher results as they both peaked with different loads applied.

The addition of different % of RAP aggregate had positive effects in terms of stiffness (ITSM) as is visible in Fig. 4.44. The optimum % RAP that the mixture can handle can not be clearly extrapolated from the graph, as each of the mixtures with RAP aggregate added seemed to perform better at different applied loads. More testing would have to be conducted in order to determine the optimum % RAP. Further testing might include ITSM, ITST and ITFT tests on specimens containing 15% RAP.

Figs 4.44, 4.45 and 4.46 show that as a result of adding the cold RAP into the mixture, there is little difference between the results obtained. The results obtained in Fig. 4.44 are identical for both mixtures. Fig.4.45 shows that the ITST results are similar, with the cold RAP producing slightly higher values in general. In the ITFT, mixtures produced with the heated RAP gave marginally better fatigue results. Overall it can be concluded that results obtained from adding the cold RAP and heating the RAP produced similar results, therefore it can be concluded that the effects of adding in cold RAP are minor and offer no great cause for concern.

4.31 Effects of Adding RAP to Bituminous Mixtures

After obtaining the optimum % RAP for 30% HRA, basecourse and roadbase, the question of why the material had positive effects on the mixture up to the optimum and then started to decrease or tend toward an negative direction. In order to try to explain this it was decided to look at the four components in the mixtures, which could be broken into two sections:

1. Virgin aggregate and RAP aggregate.
2. Recovered binder and virgin binder.

4.31.1 Virgin aggregate and RAP aggregate

The Ten-percent Fines Value test (TFV) as described in Section 3.4.4 was used to assess the properties of the virgin and RAP aggregates. This test was carried out in accordance with BS 812: Part 111: 1990. Samples of 10/14mm aggregate were used. The binder that was adhered to the RAP aggregate was washed off using methlyene chloride. Samples of 10/14 were then sieved out. Tests were conducted for samples containing various amounts of virgin aggregate and various amounts of RAP aggregate.

Table 4.35 shows the different virgin/RAP aggregate combinations and the TFV. A graphical representation can be seen in Fig. 4.47.

Table 4.35 - Different virgin/RAP aggregate combinations and TFV

% Virgin aggregate	% RAP aggregate	TFV (kN)
100	0	253
85	15	249
70	30	246
0	100	230

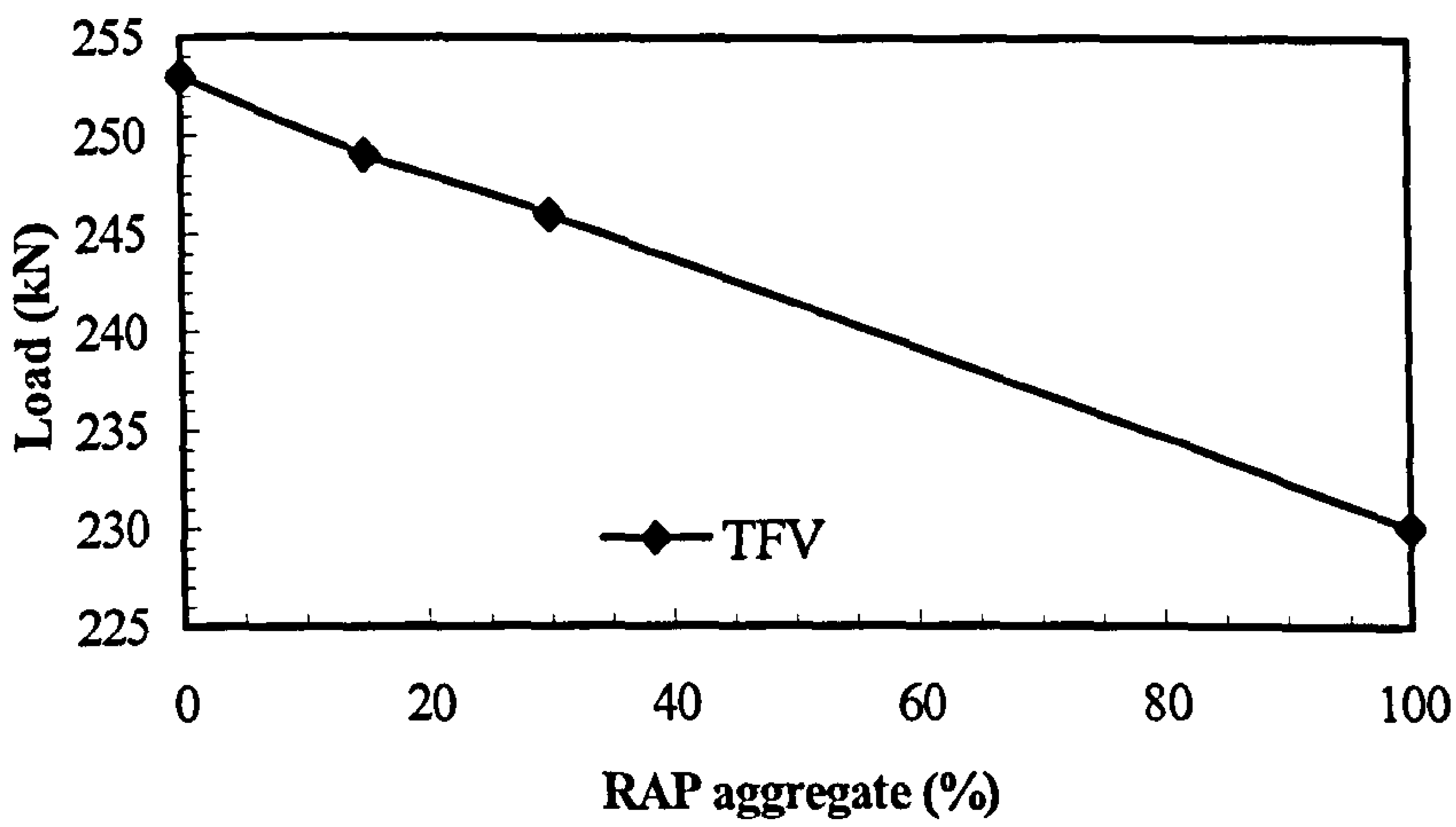


Fig. 4.47 - TFV load vs % RAP aggregate

Fig. 4.47 shows that as the % RAP aggregates increases the TFV reduces to a minimum at an addition of 100% RAP. In order to assess fully the results, an investigation into the effects of binder will have to be addressed firstly.

4.31.2 Recovered binder and virgin binder

This section of the testing had to be broke into two areas in order to assess two different parameters:

- Determination of the actual penetration (pen) of a mixture when reclaimed asphalt is used.
- Use of a stiffer binder in roadbase.

1. Determination of the actual penetration of a mixture: Before the actual penetration of the mixture could be determined, the penetration of the virgin 100pen binder, virgin 50pen binder as discussed in section 3.6.2 and RAP binder were determined. The RAP binder had to be recovered from the RAP aggregate. This was done in accordance with prEN 12697: Part 3: 1998. After the RAP binder was recovered the penetration test as described in section 3.6.2 was carried out on the three binders. Table 4.36 shows the results obtained from the penetration test.

Table 4.36 - Results of penetration test

	Virgin 100pen	**Virgin 50pen**	**Recovered binder**
Penetration @ 25°C	94	63	41

After obtaining these values, the penetration of the binder of a mixture when reclaimed asphalt is used can be calculated in accordance with prEN 13108-4: WD 27: 2002. The following formula is used to calculate the penetration of the binder of a mixture with reclaimed asphalt:

$$a \lg(pen_1) + b \lg(pen_2) = (a+b) \lg(pen_{mix}) \qquad \textbf{Eq. 4.9}$$

Where

pen_{mix} calculated penetration of the binder in the mixture containing reclaimed asphalt

pen_1 penetration of the binder recovered from the reclaimed asphalt

pen_2 penetration of the added binder

a and b portions by mass of the binder from the reclaimed asphalt (a) and from the added binder (b) in the mixture; $a + b = 1$.

Therefore in our situation for a 30% HRA with 10% RAP:

pen_{mix}	= ?
pen_1	= 41
pen_2	= 63
a	= from Appendix A, Table A.17
total binder	= 494g
recovered binder	= 13.6g
percentage saved	= 13.6/494 = 3.0%
b	= 100-3 = 97%

by substitution:

$$0.03\lg(41) + 0.77\lg(63) = \lg(pen_{mix})$$

$$\lg(pen_{mix}) = 1.7943$$

$$=$$

$$pen_{mix}\ 62.33$$

The rest of the calculation were done in the same way for 30% HRA. Table 4.37 shows these results. The results can be seen in graphical form in Fig. 4.48. In the case of basecourse and roadbase the virgin binder used was 100pen with a penetration grade of 94. Tables 4.38 and 4.39 show the results from the calculation for the penetration of the binder in the mixture for basecourse and roadbase respectively. A graphical representation of these can be seen in Fig. 4.49.

Table 4.37 - Calculation of penetration of binder of 30% HRA with various amounts of RAP

Binder (%)	Amount binder (g)	Amount binder reclaimed (g)	binder saved (%)	lg (pen_{mix})	Actual pen of mix
0	507.6	0	0.00	1.7993	63
5	500.8	6.8	0.01	1.7968	62.66
10	494	13.6	0.03	1.7943	62.23
15	487.3	20.3	0.04	1.7919	61.94
100	372	135	0.27	1.7497	56.19

Table 4.38 - Calculation of penetration of binder of basecourse with various amounts of RAP

% Binder	Amount binder (g)	Amount binder reclaimed (g)	binder saved (%)	lg (pen_{mix})	Actual pen of mix
0	289.31	0	0.00	1.973	94
10	280.6	8.71	0.03	1.9622	91.68
15	276.3	13.01	0.04	1.9569	90.55
20	271.9	17.41	0.06	1.9514	89.41
25	267.6	21.71	0.08	1.9461	88.32
30	263.2	26.11	0.09	1.9406	87.21
35	258.9	30.41	0.11	1.9353	86.15
40	254.5	34.81	0.12	1.9298	85.07
50	245.8	43.51	0.15	1.9189	82.96
100	202.3	87.02	0.3	1.8647	73.23

Table 4.39 - Calculation of penetration of binder of roadbase with various amounts of RAP

% Binder	Amount binder (g)	Amount binder reclaimed (g)	binder saved (%)	lg (pen_{mix})	Actual pen of mix
0	240.25	0	0	1.973	94
10	231.3	8.95	0.04	1.960	91.2
15	227.2	13.05	0.05	1.954	89.94
20	222.9	17.35	0.07	1.947	88.51
30	213.4	26.85	0.11	1.933	85.703
100	150.8	89.45	0.37	1839	69.02

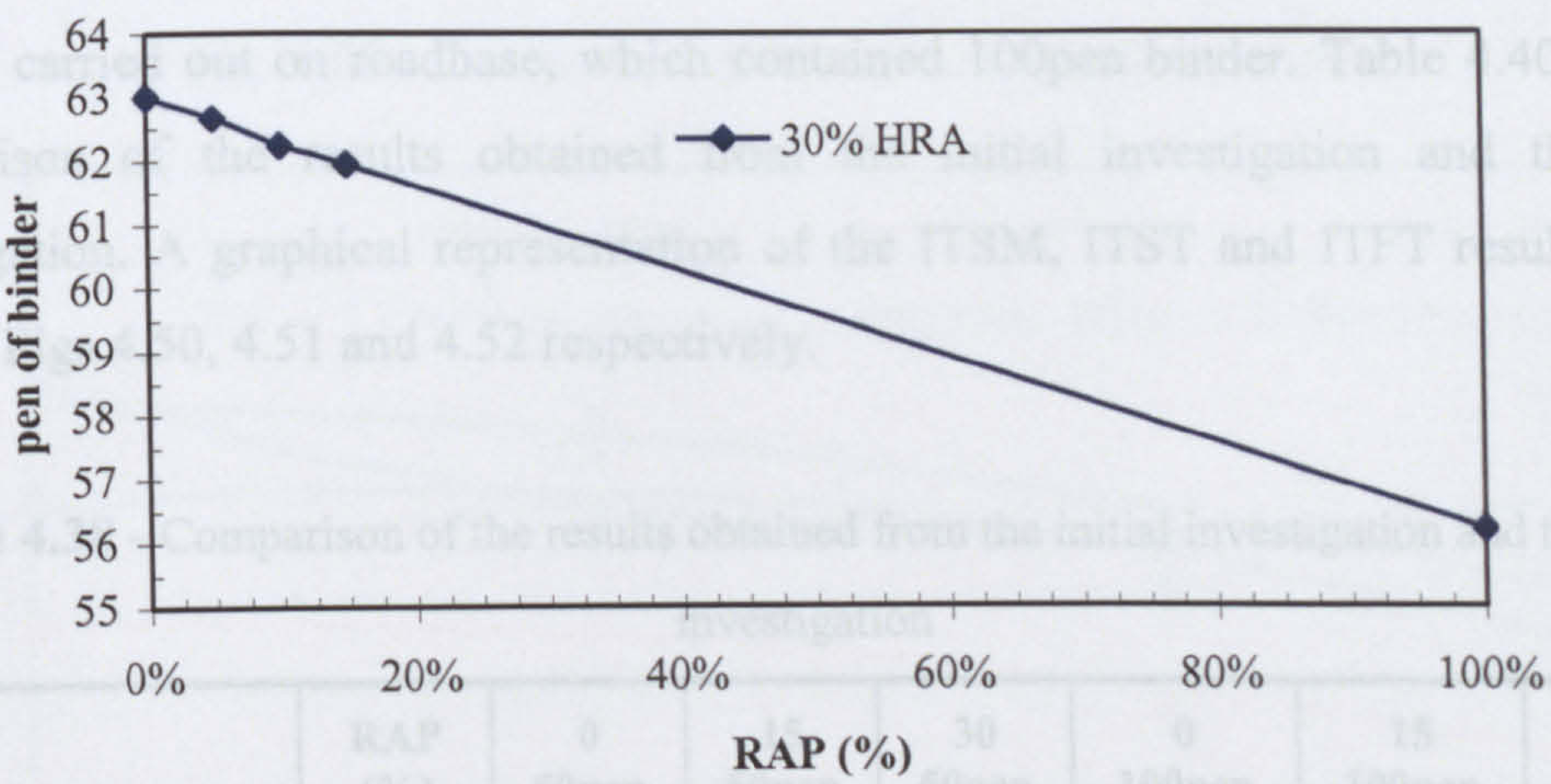


Fig. 4.48 - Graph of % RAP Vs pen of binder for 30% HRA

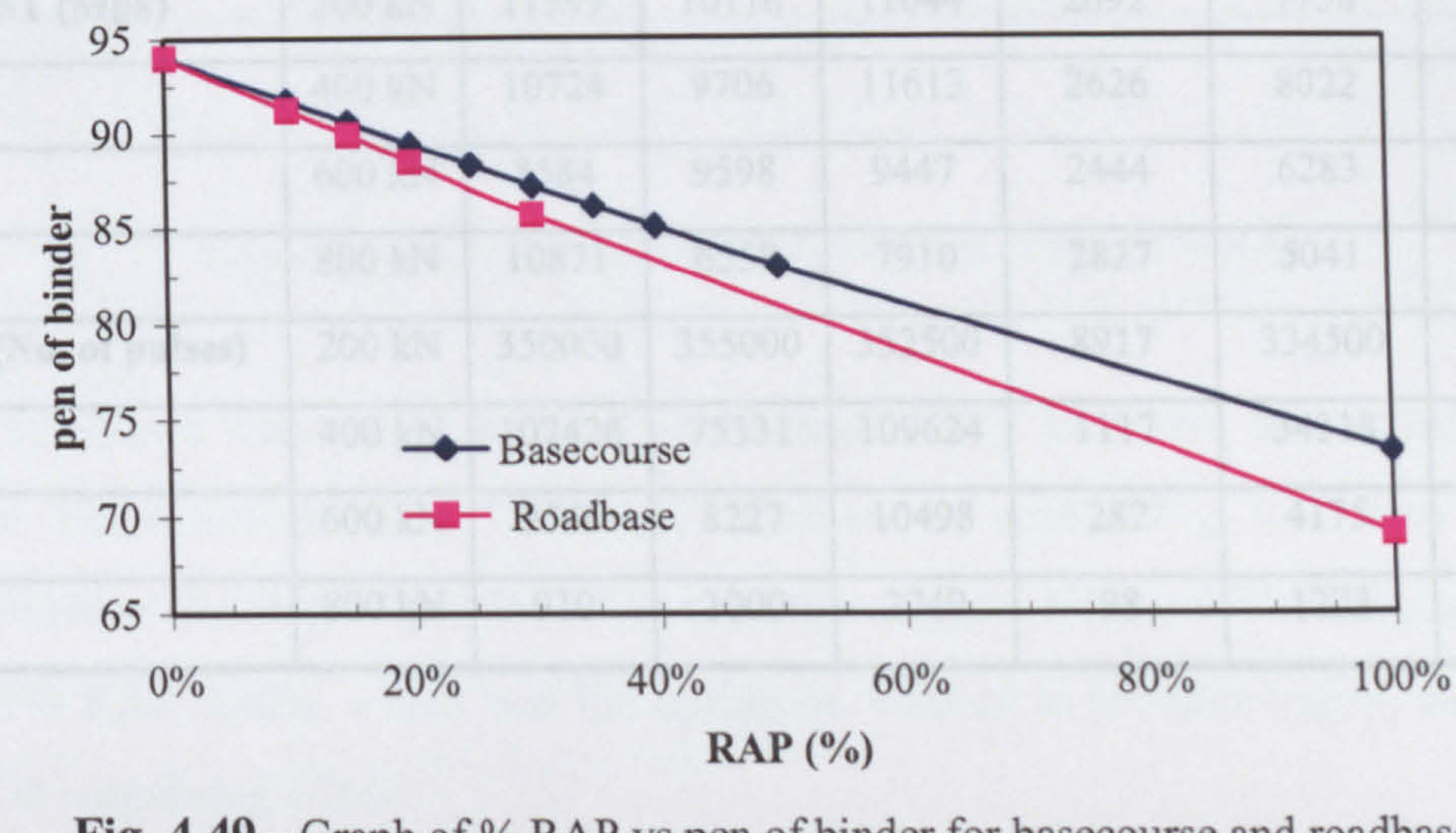


Fig. 4.49 - Graph of % RAP vs pen of binder for basecourse and roadbase

From Figs 4.48 and 4.49 it is transparent that the penetration of the binder in the various mixtures is reduced as the % RAP is increased. This means that the binder is getting stiffer as the RAP content is increased. This would explain why the stiffness of the mixture increased to the optimum and then decreased as the reduced strength of the aggregate outweighed the positive effects of the increased stiffness of the binder.

2. Use of 50pen in roadbase: Stiffness and fatigue tests were carried out on specimens with 50pen binder to determine the effect stiffer binder had on roadbase. Higher values would confirm that the earlier findings in sections 4.12, 4.13 and 4.14 were correct. The ITSM, ITST and ITFT tests were carried out on compacted specimens. The results obtained from these tests would be compared with the results obtained from tests already carried out on roadbase, which contained 100pen binder. Table 4.40 shows a comparison of the results obtained from the initial investigation and the 50pen investigation. A graphical representation of the ITSM, ITST and ITFT results can be seen in Figs 4.50, 4.51 and 4.52 respectively.

Table 4.38 - Comparison of the results obtained from the initial investigation and the 50pen investigation

	RAP (%)	**0 50pen**	**15 50pen**	**30 50pen**	**0 100pen**	**15 100pen**	**30 100pen**
ITSM (Mpa)	Mpa	10647	9399	9347	2933	8768	5132
S.D.	-	1258	804	811	646	505	462
ITST (Mpa)	200 kN	11995	10116	11644	2692	7758	5071
	400 kN	10724	9706	11613	2626	8022	4414
	600 kN	8584	9598	9447	2444	6283	4267
	800 kN	10871	6558	7910	2827	5041	3671
ITFT (No. of pulses)	200 kN	350000	355000	352500	8917	334500	30583
	400 kN	107426	75331	109624	1117	34318	5318
	600 kN	3546	8227	10498	282	4175	891
	800 kN	910	1000	2749	98	1273	405

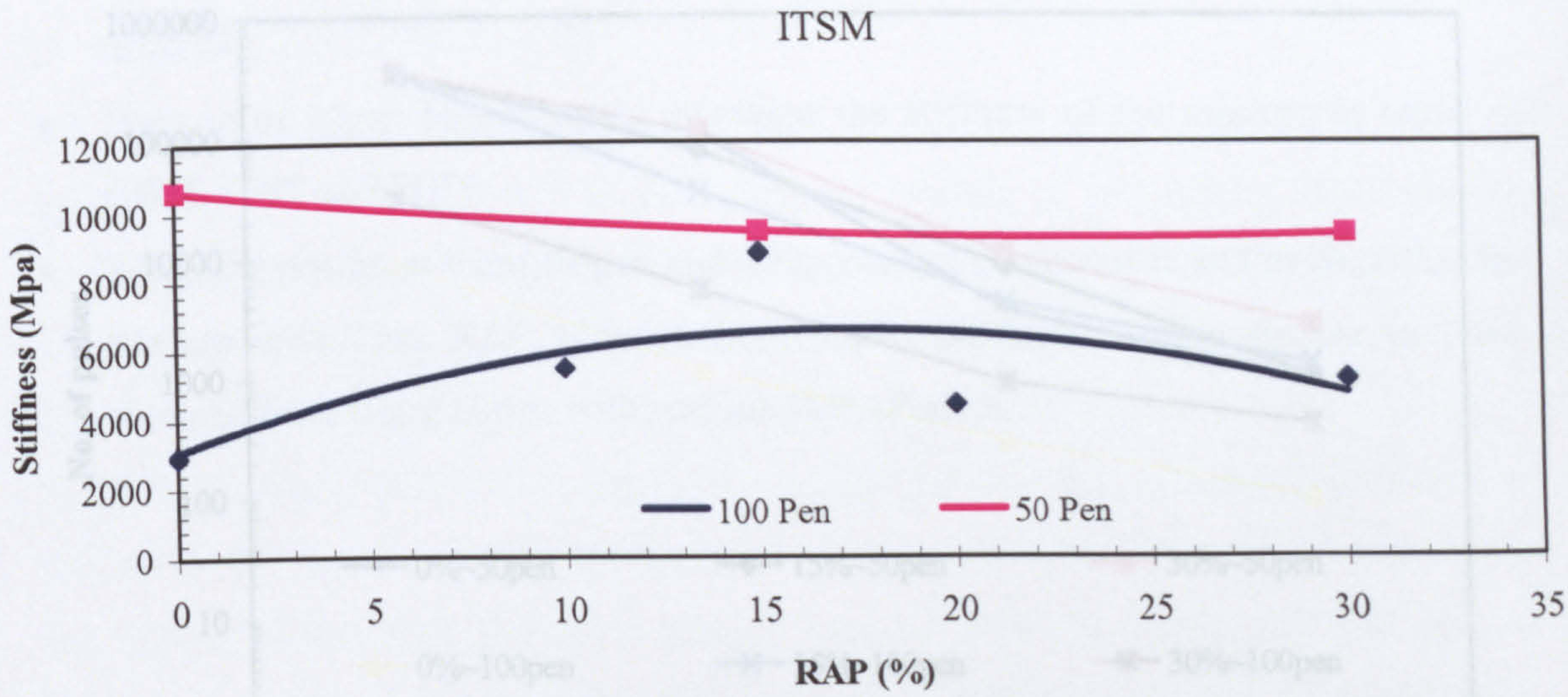


Fig. 4.50 - Comparison of ITSM results at different % RAP

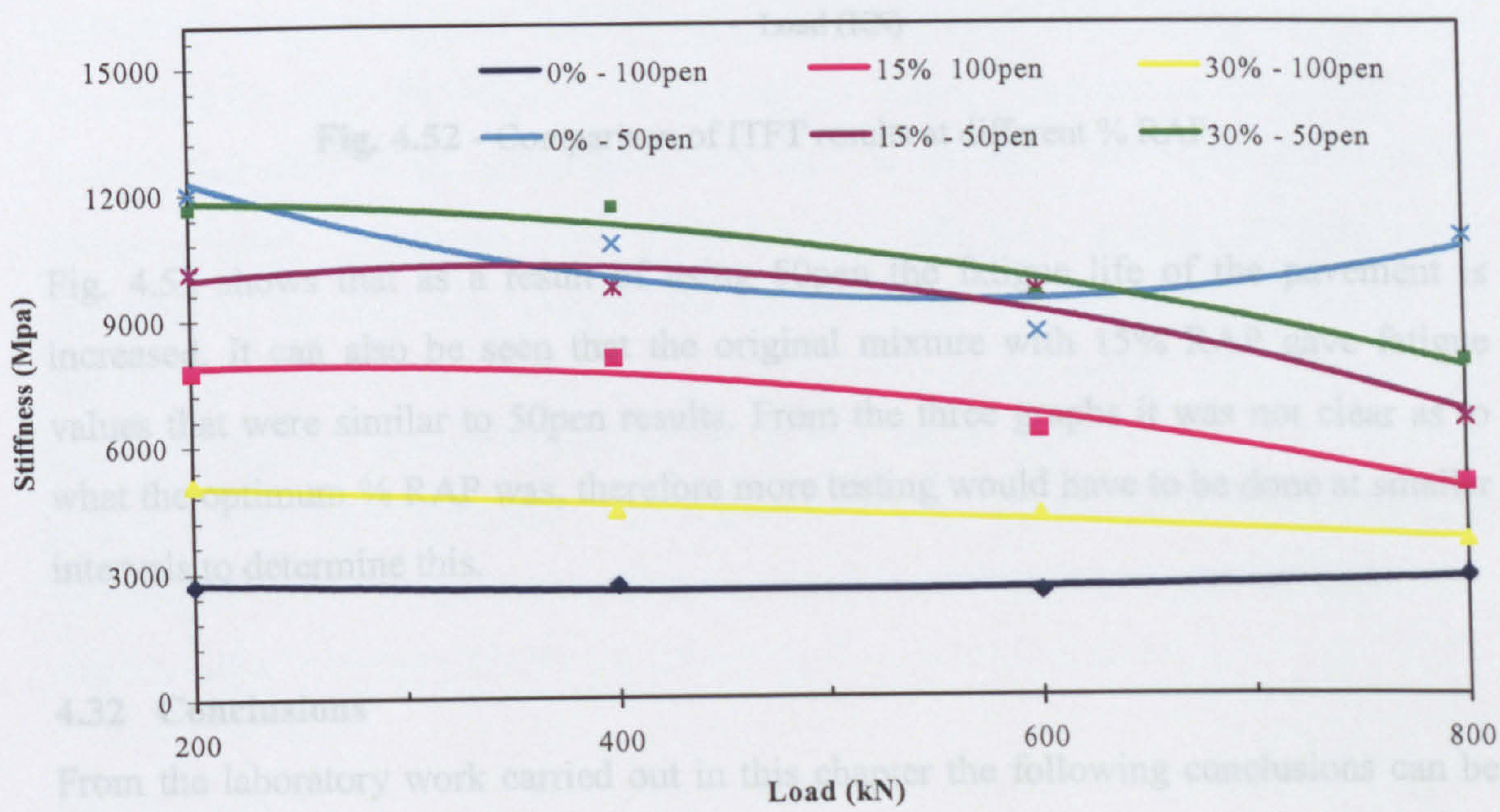


Fig. 4.51 - Comparison of ITST results at different % RAP

Fig. 4.50 shows that using 50pen binder substantially increases the stiffness of the mixture. The results from the ITST test, which can be seen in Fig. 4.51, also show that the mixture containing 50pen is stiffer than the original mixture. The original mixture with 15% RAP added, which was the optimum, seemed to produce values closer to the mixtures containing 50pen.

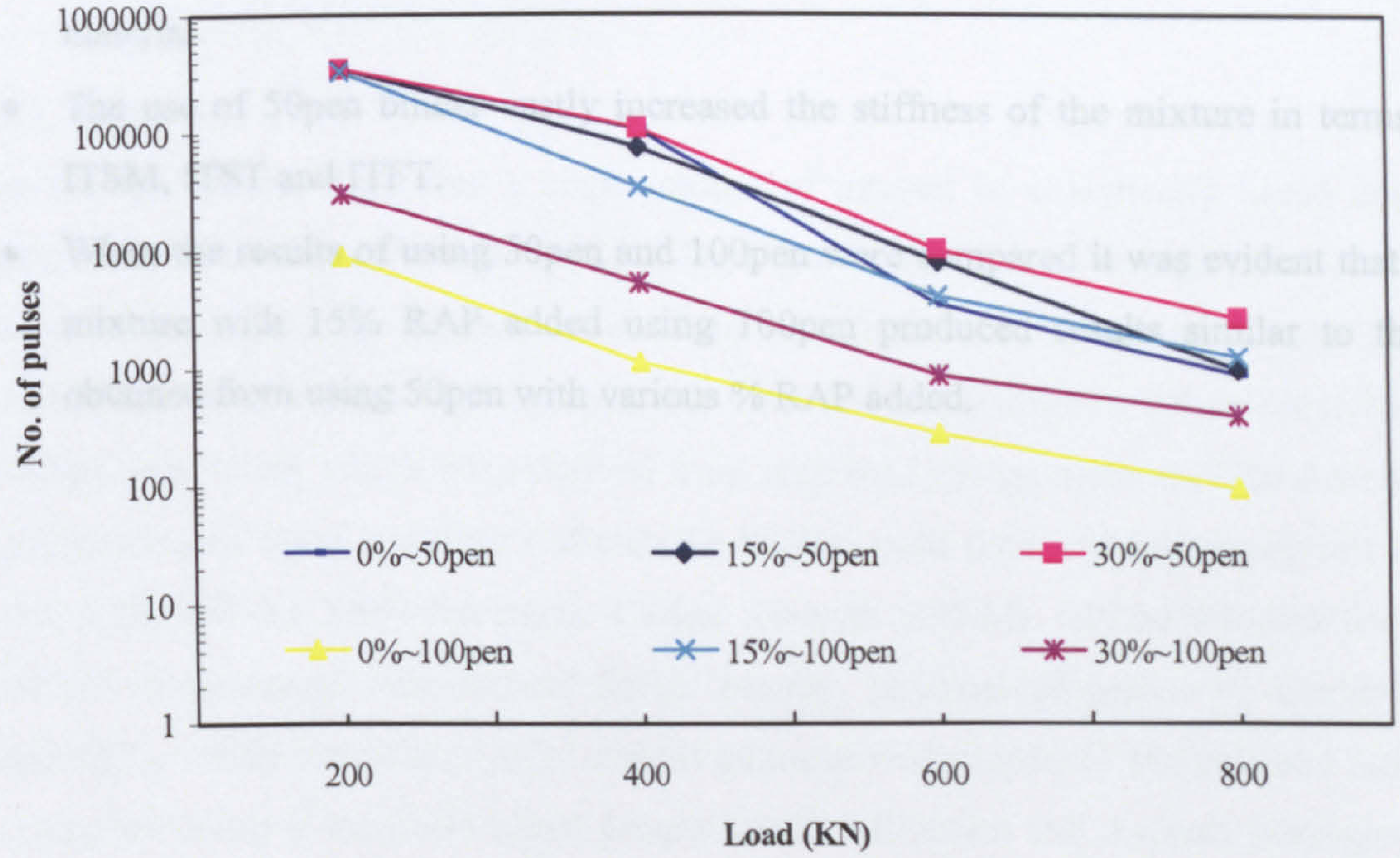


Fig. 4.52 - Comparison of ITFT results at different % RAP

Fig. 4.52 shows that as a result of using 50pen the fatigue life of the pavement is increased. It can also be seen that the original mixture with 15% RAP gave fatigue values that were similar to 50pen results. From the three graphs it was not clear as to what the optimum % RAP was, therefore more testing would have to be done at smaller intervals to determine this.

4.32 Conclusions

From the laboratory work carried out in this chapter the following conclusions can be made:

- The addition of cold RAP into the mixture produced similar results to those experienced from heating the RAP with the aggregate.
- The Ten-percent Fines Value (TFV) tests results showed that as the % RAP increased the load required to produce the TFV reduced.
- The penetration value of the binder decreased as the % RAP increased which in turn increased the stiffness of the mixture.
- The increased stiffness of the binder in the mixture would explain why the stiffness of the mixtures increased till the optimum % RAP was reached. On the other hand the TFV would explain why after the optimum % RAP was surpassed for each

mixture the stiffness started to reduce and tend towards the base mixtures for each material.

- The use of 50pen binder vastly increased the stiffness of the mixture in terms of ITSM, ITST and ITFT.
- When the results of using 50pen and 100pen were compared it was evident that the mixture with 15% RAP added using 100pen produced results similar to those obtained from using 50pen with various % RAP added.

CHAPTER 5 - SHELL PAVEMENT DESIGN

5.1 General

In the late fifties, there was a large amount of interest in analytically based design procedures. Interest grew and in 1963, Shell published a set of design charts, which allowed the calculation of stress and strain in a structure based upon a multi-layer linear elastic analysis. In 1978 this system was extended to incorporate all relevant major design parameters, which were derived from empirical design methods. The American Association of State Highway Officials (AASHO) used road and laboratory test data and published the Shell Pavement Design Manual (SPDM). SPDM allowed for the effects of pavement temperature, traffic density, physical properties of binder and aggregate, whilst standardising the asphalt mixtures with respect to stiffness and fatigue properties using a computer based program called Bitumen and Asphalt Nomographs Developed by Shell (BANDS). The numbers of variable parameters were limited in order to make the charts and tables manageable. In the early 70's, shell developed the BItumen Stress Analysis in Roads (BISAR). This mainframe computer program was designed to calculate and analyse the stress and strains within the layers of the pavement. BISAR 3.0 is a newer version, which can calculate comprehensive strain and stress profiles for virtually any loading pattern, which can be used in refining the designs carried out with SPDM 3.0 or can be used as a separate program for theoretical calculations on elastic multi layer systems (Strickland 2000). The three programs that will be used in this analysis are: BANDS, SPDM 3.0 and BISAR 3.0.

5.2 BANDS

Bands 3.0 allows engineers to estimate the material properties of the bituminous binder and asphalt mixture. It can calculate the stiffness of the bituminous binder at a range of temperatures and loading times from basic binder properties.

The program can calculate the stiffness of the asphalt mixture based on the stiffness of bituminous binder and the volume of air, binder and aggregate in the mixture. This can be done for a range of conditions to calculate the stiffness at different temperatures with varying loading times. This tool allows a comparison of properties of asphalt mixtures that have been measured under different test conditions. The data generated above is presented in a user-friendly format. Output can be copied and pasted into most window-based programs (Strickland 2000).

5.3 SPDM 3.0

SPDM 3.0 program is used to design the basic thickness of the pavement structures based on the fatigue characteristics of the bituminous layer and the permanent deformation characteristics of the subgrade. Few pavements fail due to fatigue damage in the bound material and the move towards thinner pavement design. Because of this, shell has developed a module that will estimate the permanent deformation that accumulates in an asphalt layer. SPDM 3.0 shell pavement design method contains three modules: structural thickness design for new asphalt road pavements, estimation of permanent deformation (rutting) in the asphalt layers and structural thickness design for asphalt overlays on existing asphalt road pavements.

The module used in this investigation was the structural thickness design for new asphalt road pavements.

5.3.1 Structural thickness design

Thickness design of an asphalt layer in a new pavement structure is based on a three-layer system: the asphalt layer, the sub-base layer and the subgrade as can be seen in Fig. 5.1. Calculation of the recommended asphalt thickness is the final result of this design procedure which is carried out in an iterative way. In each iteration, BISAR calculates strains at four critical positions (Fig. 5.1) in the pavement construction: bottom of the asphalt layer, top of the subgrade, under a wheel and between the wheels.

Strains at these points are compared with design strains for subgrade and asphalt. The minimum actual life is compared with the design life. BISAR calculations are used to determine the minimum asphalt layer thickness satisfying critical strain criteria at the top of the subgrade and the bottom of the asphalt. When the actual life and design life differ by 2.5% or less, the corresponding thickness is reported as the final result of the design. The thickness design package requires the input of the following information: climate data, traffic and design life, structural thickness design for new asphalt road pavements, estimation of permanent deformation in the asphalt layers, structural thickness design for new asphalt overlays on existing asphalt pavements, base layers, subgrade strain, asphalt mixture composition and fatigue.

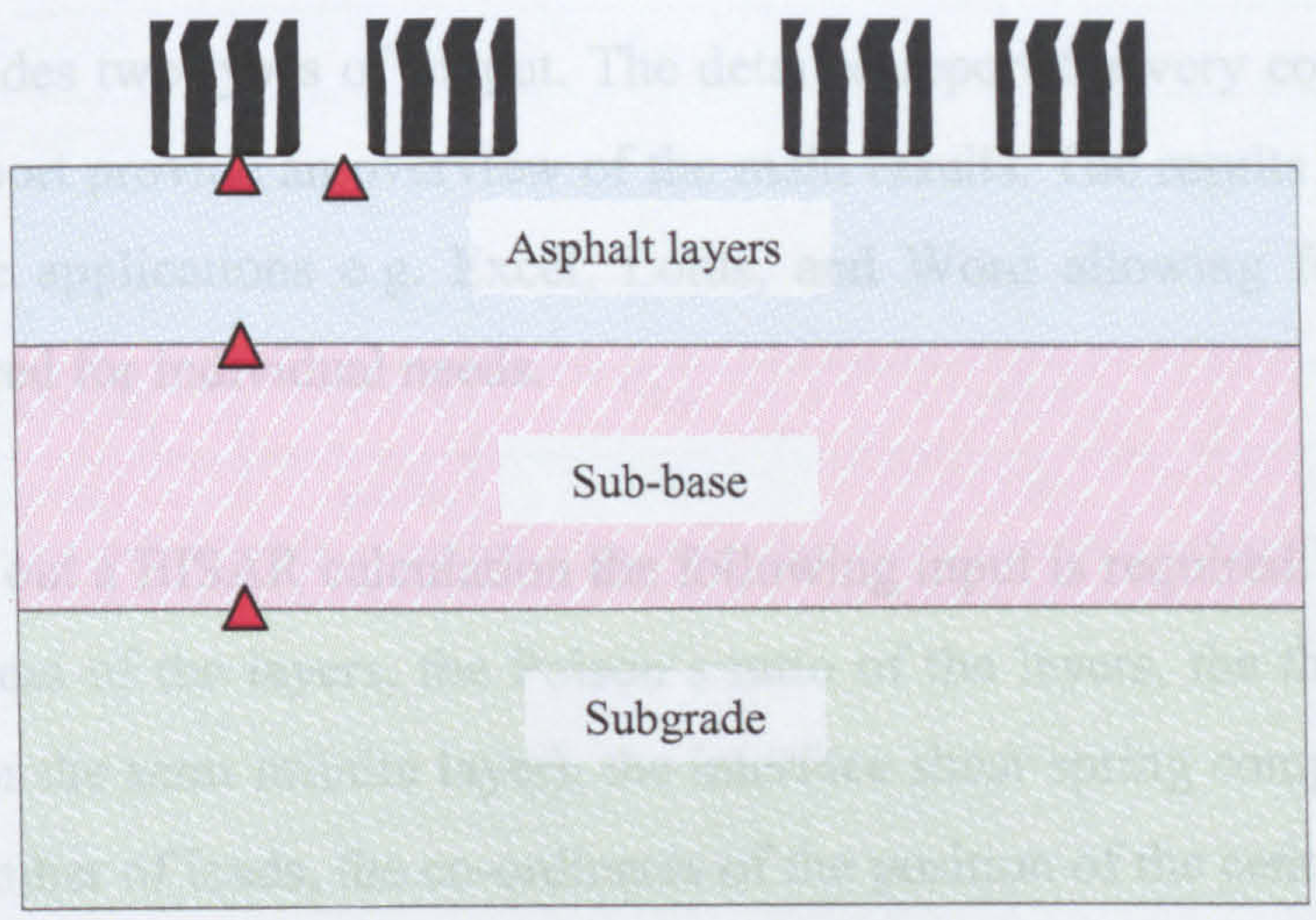


Fig. 5.1 - Various layers in a pavement structure and critical strain positions

5.4 BISAR 3.0

The BISAR 3.0 program can be broken in to three sections loads, layers and positions. The data required for these sections are summarised below.

5.4.1 Loads

This part of the program requires data for the load on the pavement and the radius of the loaded wheel. From this data, the program calculates the area of contact, the force applied and therefore, the stresses and strains in selected positions. The program has standard dual wheel, standard single wheel, and super wheel options that can be selected to ease the amount of initial data preparation required. There is also the option to specify if horizontal and/or shear forces are applied (Shell Pavement Design Manual SPDM 3.0 - 2000).

5.4.2 Layers

Data for the number of layers, the thickness of the layers and Poisson's ratio of the layer are required. The program has been developed to calculate stresses and strains where there is slippage between the layers. Therefore there is also an option to specify if full friction between layers exist.

5.4.3 Positions

The x, y and z co-ordinate position within each layer where the stresses and strains are calculated can be specified or there is a standard option which can be selected.

BISAR 3.0 provides two types of output. The detailed report for very complex studies and the block report provide an overview of the main results. The results can be copied to windows base applications e.g. Excel, Lotus, and Word allowing for graphs and tables to be tailored for individual needs.

In order to carry out a BISAR calculation the following input is required: the number of layers, the stiffness of the layers, the Poison's ratio of the layers, the thickness of the layers (except for the semi infinite layer), the interface shear spring compliance at each interface, the number of loads, the co-ordinates of the position of the centre of the loads, the co-ordinates of the position for which output is required and one of the following combinations to indicate the vertical normal component of the load: stress and load, load and radius or stress and radius.

To facilitate SPDM related calculations the present BISAR package contains options to access with ease the standard wheel configuration and to automatically select important positions in the layer structure under construction (Shell Pavement Design 2000).

5.5 Pavement Design Calculations

Before any design work could be carried using the design packages, information had to be obtained. Information regarding stiffness of the three mixtures had to be calculated using the bands program. The information required to determine the stiffness of three mixtures can be seen in Table 5.1. Using this information the stiffness of the three mixtures could be determined. Table 5.2 shows these results.

Table 5.1 - Input data for BANDS program

Parameter	Unit	30% HRA	basecourse	roadbase
Loading time	Sec	0.02	0.02	0.02
Temperature of binder	°C	14.9	14.9	14.9
Penetration value	0.1mm	63	94	94
Penetration temperature	°C	25	25	25
Softening point	°C	52	52	52
Vol percentage binder	% v/v	19.5	11.5	9.625
Vol percentage agg	% v/v	74.5	82.5	84.375
Fatigue strain	µm/m	500	500	500

Table 5.2 - Stiffness results for the three mixtures calculated using BANDS

Mixture	Stiffness (Mpa)
30% HRA	1960
basecourse	2190
roadbase	2860

Stiffness of mixtures containing RAP aggregate cannot be worked out accurately using BANDS as the penetration value of the reclaimed binder cannot be accounted for. In order to calculate the stiffness of mixtures containing RAP aggregate, stiffness results obtained for mixtures containing RAP aggregate using the ITSM were divided by the base mix stiffness determined in the ITSM to give factors. These factors were applied to the stiffness results obtained using BANDS. Table 5.3 shows the factors obtained from the ITSM results. Table 5.4 shows the stiffness values after the factors have been applied to the BANDS stiffness.

Table 5.3 - Factors obtained from ITSM results

Material	RAP (%)	Stiffness (Mpa)	Factor
30% HRA	0	2316	2316/2316 = 1
basecourse	0	2943	2943/2943 =1
	30	4873	4873/2943 = 1.655
	40	3648	3648/2943 = 1.239
roadbase	0	2933	2933/2933 = 1
	15	8768	8768/2933 = 2.989
	30	5132	5132/2933 = 1.749

Table 5.4 - BANDS stiffness after factors applied

Material	RAP (%)	Factors	BANDS stiffness (Mpa)	Calculated stiffness (Mpa)
30% HRA	0	1	1960	1960
basecourse	0	1	2190	2190
	30	1.655		3624
	40	1.239		2713
roadbase	0	1	2860	2860
	15	2.989		8548
	30	1.749		5002

As the three different asphalt layers are considered as one in SPDM the average stiffness of the whole layer had to be determined. The average stiffness was worked out for both the ITSM results and the Bands results so comparisons could be made. Before the overall stiffness of the asphalt layer could be calculated the number of different combinations of the three mixtures had to be determined. A matrix of these combinations can be seen in Table 5.5.

Table 5.5 - Matrix of the different combinations for the three mixtures

System No.	30% HRA	basecourse	roadbase
1	0 %	0 %	0 %
2	0 %	0 %	Opt %
3	0 %	Opt %	0%
4	0 %	0 %	Max %
5	0 %	Max %	0 %
6	0 %	Opt %	Opt %
7	0 %	Opt %	Max %
8	0 %	Max %	Opt %
9	0 %	Max%	Max %

From the nine combinations the overall stiffness could be determined for each combination. The thickness of the 30% HRA layer and basecourse layer is assumed to be 40mm and 60mm respectively. The roadbase layer is variable and is dependant on the overall thickness of the pavement as calculated in BISAR. With these criteria now established the following formula can be used to determine the overall stiffness of the total asphalt layer:

$$So = \frac{(S_{HRA} \times D_{HRA}) + (S_{20} \times D_{20}) + (S_{28} \times D_{28})}{D_{HRA} + D_{20} + D_{28}}$$ **Eq. 5.1**

Where:

S_O	=	Overall stiffness (Mpa)
S_{HRA}	=	Stiffness of HRA (Mpa)
S_{20}	=	Stiffness of basecourse (Mpa)
S_{28}	=	Stiffness of roadbase (Mpa)
D_{HRA}	=	Depth of HRA (m)
D_{20}	=	Depth of basecourse (m)
D_{28}	=	Depth of roadbase (m)

By substitution, assuming an overall thickness of the asphalt layer of 0.3m and based on stiffness from the ITSM and using system number 1 criteria the overall stiffness of the bituminous layers can be calculated.

$$So = \frac{(2316 \times 0.04) + (2943 \times 0.06) + (2933 \times 0.2)}{0.04 + 0.06 + 0.2}$$

$$So = \frac{855.82}{0.3}$$

$$S_O = 2853 \text{ Mpa}$$

Using the above criteria the overall stiffness of the different systems was calculated in the same manner for various depths of roadbase. Tables 5.6 and 5.7 shows these results for the ITSM and BANDS stiffness.

Table 5.6 - ITSM stiffness results for the different systems at various depths of roadbase

	Depth of roadbase (m)						
System No.	0.08	0.1	0.12	0.14	0.16	0.18	0.2
1	2799	2813	2824	2833	2840	2847	2853
2	5932	5729	6005	6235	6430	6597	6741
3	3443	3392	3350	3315	3286	3261	3239
4	3777	3912	4023	4115	4194	4261	4319
5	3034	3024	3016	3009	3003	2988	2994
6	6035	6309	6533	6719	6877	7012	7129
7	4420	4491	4549	4598	4639	4674	4705
8	5628	5942	6199	6413	6594	6749	6884
9	4012	4124	4215	4292	4356	4412	4460

Table 5.7 - BANDS stiffness results for the different systems at various depths of roadbase

	Depth of roadbase (m)						
System No.	0.08	0.1	0.12	0.14	0.16	0.18	0.2
1	2437	2479	2514	2543	2567	2588	2606
2	4965	5323	5616	5861	6067	6244	6398
3	2915	2909	2905	2901	2898	2895	2893
4	3389	3550	3682	3792	3885	3965	4034
5	2611	2636	2656	2673	2688	2700	2711
6	5443	5753	6007	6219	6398	6552	6685
7	3867	3980	4073	4151	4216	4272	4321
8	5139	5480	5759	5991	6188	6357	6503
9	3563	3707	3825	3923	4006	4077	4139

Once all the information required was gathered for the SPDM, the input of data could commence. The first piece of information that had to be entered was the Mean Monthly Air Temperature (MMAT). Figures for Dublin were extrapolated from the Irish meteorological weather forecast website and can be seen in Table 5.8.

Table 5.8 - Mean monthly air temperatures

Month	January	February	March	April	May	June
MMAT	5	5	6.3	7.9	10.5	13.4
Month	**July**	**August**	**September**	**October**	**November**	**December**
MMAT	15.1	14.9	13.1	10.6	7.0	5.9

From these results the Mean Annual Air Temperature (MAAT) was found to be 10.4°C. Information on traffic and design life had to be entered next. The number of standard axles, rate of traffic growth, number of days with traffic per year and design period were entered. The data entered were common for all of the nine different designs. The lateral distribution factor and healing factor default values were used. After gathering all the above information the design life was calculated. Table 5.9 shows the data entered and the calculated design life.

Table 5.9 - Traffic and design life data and calculated design life

80kN Standard axles per day per lane	2000
Rate of traffic growth per year (%)	3.0
Number of days with traffic per year	365
Design period (Years)	20
Design life (standard axles)	2.02E+07
Lateral distribution factor	2.0
Healing factor	5.0

The next piece of information that had to be inserted was the sub-base thickness and the subgrade layer stiffness. The subgrade layer stiffness was calculated assuming a CBR value of 5% and Poisson's ratio of 0.25 for both layers. Both the sub-base modulus and subgrade criterion were calculated at the 85% confidence level. Table 5.10 shows the data entered.

Table 5.10 - Base layers and subgrade strain

Layer	Thickness (m)	Modulus of elasticity	Poisson's ratio
Sub-base	0.3	63.81	0.25
Subgrade	Infinite	49.00	0.25

The next step involved the input of the asphalt mix composition and fatigue data. On this screen there are three options to enter the asphalt mix composition. The option selected in this project was the entry the volume of binder and aggregate contents. Instead of entering fatigue characteristics the standard fatigue nomograph was used. Table 5.11 shows the data entered for the three mixtures. These values remained constant for the nine different systems.

Table 5.11 - Asphalt mix composition and fatigue

Volume	Depth of roadbase (m)						
	0.08	0.1	0.12	0.14	0.16	0.18	0.2
Binder (%)	12	12	12	12	12	11	11
Aggregate (%)	82	82	82	82	82	83	83
Voids (%)	6	6	6	6	6	6	6

The last screen in the SPDM program is the asphalt stiffness and layer thickness screen. The thickness of the entire asphalt layer, the mix stiffness of the asphalt layer, poisson's ratio, asphalt layer temperature and loading time in seconds had to be entered. The initial thickness of the asphalt layers was taken as 0.3m for each of the nine combinations. ITSM stiffness and BANDS stiffness were extrapolated from Tables 5.6 and 5.7 respectively. These values changed as the thickness of the roadbase layer changed. The asphalt layer temperature and the loading time in seconds were entered but for reference only. Table 5.12 shows the data inputted into the asphalt stiffness and layer thickness module.

Table 5.12 - ITSM stiffness and layer thickness data for System 1

Parameter	Value
Asphalt layer thickness (m)	0.3
Asphalt layer stiffness (Mpa)	2853
Poisson's ratio	0.35
Asphalt layer temperature (°C)	20
Load time in seconds (for reference)	0.12

Once this information had been entered the thickness of the total asphalt layer can be calculated. When the new thickness value is found, the actual stiffness based on the new thickness must be entered along with the new thickness to determine a new thickness so as to ensure that the asphalt layers are not under designed. This is repeated till there is no change in the thickness of the asphalt layer. This procedure was carried out on the nine different systems as discussed in Table 5.5. The ITSM/BANDS stiffness and required thickness of the various asphalt layers for each system can be seen in Table 5.13 and 5.14 respectively.

Table 5.13 - ITSM sStiffness and required thickness of the various asphalt layers

	System No.								
	1	2	3	4	5	6	7	8	9
Stiffness (Mpa)	2833	6741	3239	4319	3009	7129	4705	6884	4460
Thickness (m)	0.24	0.18	0.23	0.21	0.24	0.18	0.21	0.18	0.21
30% HRA	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04
basecourse	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06
roadbase	0.14	0.08	0.13	0.11	0.14	0.08	0.11	0.08	0.11

Table 5.14 - BANDS stiffness and required thickness of the various asphalt layers

	System No.								
	1	2	3	4	5	6	7	8	9
Stiffness (Mpa)	2543	4965	2901	3682	2673	5443	3980	5139	3825
Thickness (m)	0.25	0.2	0.24	0.22	0.245	0.2	0.22	0.2	0.22
30% HRA	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4
basecourse	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
roadbase	0.15	0.1	0.14	0.12	0.14	0.1	0.12	0.1	0.12

From the results obtained it can be seen that the thickness of the pavement decreases using stiffness results obtained from the ITSM. In order to determine the effects of the reduced pavement thickness, the stresses had to be calculated using the BISAR 3.0 program for the nine different systems outlined in Table 5.5.

In the BISAR program there are three modules loads, layers and positions. The first module on loads requires input of mode load. In our case the option used was the standard wheel load option. These stresses and loads were constant throughout the nine systems.

The next screen required the input of the thickness of the layer, the modulus of elasticity of each layer and Poisson's ratio which is 0.35 for bituminous layers and 0.25 for stone layers. As each system had a different combination of layers and stiffness values had to be entered separately. Tables 5.15 and 5.16 show the thickness, stiffness and Poisson's ratio of the different layers for each of the nine systems based on ITSM and BANDS stiffness respectively.

Table 5.15 - ITSM stiffness, thickness and Poisson's ratio for the different layers

Layer No.	1	2	3	4	5	6	7	8	9
30% HRA									
Thickness (m)	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04
Stiffness (Mpa)	2316	2316	2316	2316	2316	2316	2316	2316	2316
Poisson's ratio	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35
basecourse									
Thickness (m)	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06
Stiffness (Mpa)	2943	2943	4873	2943	3648	4873	4873	3648	3648
Poisson's ratio	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35
roadbase									
Thickness (m)	0.14	0.08	0.13	0.11	0.14	0.08	0.11	0.08	0.11
Stiffness (Mpa)	2933	8766	2933	5132	2933	8766	5132	8766	5132
Poisson's ratio	0.35	0.35	0.35	0.35	0.35	035	0.35	0.35	0.35
Sub-base									
Thickness (m)	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3.	0.3
Stiffness (Mpa)	64	64	64	64	64	64	64	64	64
Poisson's ratio	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25
Subgrade									
Thickness (m)	Thickness of subgrade is assumed to be infinite								
Stiffness (Mpa)	49	49	49	49	49	49	49	49	49
Poisson's ratio	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25

Table 5.16 - BANDS stiffness, thickness and Poisson's ratio for the different layers

Layer No.	1	2	3	4	5	6	7	8	9
30% HRA									
Thickness (m)	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04
Stiffness (Mpa)	1960	1960	1960	1960	1960	1960	1960	1960	1960
Poisson's ratio	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35
basecourse									
Thickness (m)	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06
Stiffness (Mpa)	2190	2190	3624	2190	2713	3624	3624	2713	2713
Poisson's ratio	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35
roadbase									
Thickness (m)	0.15	0.1	0.14	0.12	0.14	0.1	0.12	0.1	0.12
Stiffness (Mpa)	2860	8548	2860	5002	2860	8548	5000	8548	5002
Poisson's ratio	0.35	0.35	0.35	0.35	0.35	035	0.35	0.35	0.35
Sub-base									
Thickness (m)	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3.	0.3
Stiffness (Mpa)	64	64	64	64	64	64	64	64	64
Poisson's ratio	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25
Subgrade									
Thickness (m)	Thickness of subgrade is assumed to be infinite								
Stiffness (Mpa)	49	49	49	49	49	49	49	49	49
Poisson's ratio	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25	0.25

After entering all the relevant information for the nine systems the next data entered was the positions in the road pavement where the stresses, strains and displacements were to be measured. There was also an option to determine the stresses, strains and displacements at the bottom of one layer and the top of the next layer. The positions of the standard wheel load were used in this investigation.

Once the data has been entered for the nine different systems the stresses, strains and displacements can be calculated for each of the positions outlined above. In this investigation we are concerned with the stresses, strains and displacements at the top/bottom of the subgrade. Tables 5.17 and 5.18 show the stresses, strains and displacements at the top/bottom of the subgrade for the different ITSM and BANDS stiffness respectively.

Table 5.17 - Stresses, strains and displacements obtained at the top/bottom of the subgrade in each layer based on ITSM stiffness

System number	Stress (Mpa) (E-02)		Strain (μstrain) (E+02)		Displacements (μm) (E+02)	
	Top	Bottom	Top	Bottom	Top	Bottom
1	-3.989	-3.874	-4.669	-4.519	4.799	4.638
2	-1.871	-1.779	-3.618	-3.419	4.140	4.058
3	-1.551	-1.482	-3.091	-2.934	3.722	3.659
4	-1.637	-1.562	-3.198	-3.034	3.855	3.786
5	-1.506	-1.439	-2.994	-2.844	3.667	3.606
6	-1.648	-1.574	-3.173	-3.012	3.89	3.821
7	-1.57	-1.5	-3.066	-2.913	3.775	3.710
8	-1.712	-1.632	-3.295	-3.124	3.963	3.89
9	-1.518	-1.452	-2.960	-2.815	3.712	3.650

Table 5.18 - Stresses, strains and displacements obtained at the top/bottom of the subgrade in each layer based on BANDS stiffness

System number	Stress (Mpa) (E-02)		Strain (μstrain) (E+02)		Displacements (μm) (E+02)	
	Top	Bottom	Top	Bottom	Top	Bottom
1	-1.538	-1.469	-3.052	-2.897	3.708	3.645
2	-1.733	-1.652	-3.338	-3.162	3.986	3.911
3	-1.545	-1.475	-3.074	-2.918	3.715	3.652
4	-1.645	-1.578	-3.226	-3.059	3.875	3.805
5	-1.558	-1.488	-3.100	-2.943	3.734	3.670
6	-1.647	-1.572	-2.791	-2.654	3.886	3.817
7	-1.560	-1.491	-3.044	-2.892	3.763	3.699
8	-1.738	-1.655	-2.929	-2.780	3.985	3.911
9	-1.607	-1.534	-3.134	-2.975	3.820	3.753

Using the strain figures the number of million standard axles (msa) the subgrade can accommodate were calculated using BISAR 3.0. Table 5.19 shows the number of msa that each system can take before deformation of the subgrade occurs.

Table 5.19 - Number of msa for each system based on ITSM and BANDS stiffness

System No.	ITSM (msa)	BANDS (msa)
1	1.65	4.74
2	2.42	3.33
3	4.51	4.61
4	3.94	3.81
5	5.11	4.46
6	4.07	6.75
7	4.66	4.79
8	3.5	5.58
9	5.35	4.27

From the results obtained it evident that the amount of msa that the subgrade can handle before failure was higher using BANDS stiffness. One reason for this could be that BANDS stiffness was calculated at 14.9°C, whereas the ITSM tests were conducted at 20°C. From the ITSM results it is evident that as a result of adding RAP aggregate into the mixtures the number of standard axles that the subgrade can take increased significantly.

5.6 Conclusions

From the pavement design work carried out the following conclusions can be made:

- Thickness of the asphalt layers can be reduced as a result of adding RAP aggregate.
- Number of msa axles that the subgrade can carry increased as a result of adding RAP aggregate into the bituminous mixtures.
- Number of msa based on BANDS stiffness figures as calculated earlier were higher than those calculated using the stiffness determined using the ITSM test.

CHAPTER 6 - ECONOMICS OF RECYCLING

6.1 General

Before any new road is built a cost benefit analysis is carried out to ensure that the anticipated benefits to be gained outweigh the economic costs. The economic factors relating to the materials used in construction mainly derive from the costs of extracting the material, the processing and hauling of it to the site, all of which are closely related to energy consumption (Sherwood 1995).

The technology for processing demolition wastes for re-use is proven and available. The equipment is robust and has been developed to a high degree of efficiency. The capital cost to purchase a plant and to make modifications to existing plants is significant and therefore operators are cautious about investing in plant. The economics of recycling are very specific to the individual projects on which Construction and demolition waste (C&DW) arise. Management of construction and demolition sites can reduce costs in terms of: haulage cost to and from the site, landfill costs as the material is segregated and reduced cost of importing primary material.

In addition there are environmental benefits resulting from less truck movements. Usually these demolition projects are undertaken in urban areas where there is traffic congestion. Local authorities might place restrictions on the number of movements at times of peak traffic congestion. The economic cost however of such a measure would have to be addressed.

On motorway construction projects which include both new motorways and reconstruction of existing ones. The use of recycled material can reduce the overall cost of the project. A good example of this is British Aviation Authority (BAA) pavement team project.

In this project BAA tendered out the reconstruction of three airport runways namely Heathrow, Gatwick and Stansted. This type of tendering is referred to as global costing. This project is still in progress with two of the runways completed. The benefits from this project to date are: 210,000 tonnes of onsite material have been recycled resulting in savings of £1.8 million, reduced offsite landfill from BAA airports, a reduction in imported primary aggregate utilised, saved 42,000 lorry movements and the saving of 1.85 million road miles (Reid 2003).

From the results obtained it is evident that there are benefits to be achieved from recycling of bituminous material. In this chapter a cost benefit analysis is going to be carried out in order to assess economically if the use of RAP aggregate in new asphalt mixes is a viable option.

6.2 Economics of Using RAP in Bituminous Mixes

In order to determine if using RAP in bituminous mixes is economical, information in relation to the following has to be determined: cost to produce each mix, tonnage's produced of each material per annum and the amount of RAP aggregate available for recycling, cost of modifying existing plant to allow recycling to take place and financial benefits of a reduced layer thickness of the pavement.

6.3 Cost to Produce Each Mix

In order to determine the cost to produce each mix costs had to be obtained for the cost of the materials used and the various plant costs associated with producing each material. Table 6.1 shows the cost of the different materials used and Table 6.2 shows the various plant costs associated with producing the mix. It should be noted that factors have been applied to the materials, plant and tonnages. Rounding has also been carried out.

Table 6.1 - Costs for each of the materials used

Material	Cost/t of material (£)
Binder	100
Onsite material	2
Bought material	2.49
RAP	1.254

Table 6.2 - Plant costs associated with making one tonne of bituminous material in a batch mix plant

Item description	Cost/t of mix (£)	Item description	Cost/t of mix (£)
Maintenance	0.2	Power	0.236
Production wages	0.166	Fuel	0.566
Maintenance wages	0.1	Location	0.123
Depreciation	0.0833		
		Total	**1.474**

After ascertaining these costs, the cost of producing one tonne of roadbase and basecourse had to be calculated. From the % of each material in the mix and knowing the plant cost, the cost per tonne for each mix could be calculated. Table 6.3 and 6.4 shows the materials used, the % of each material in the mix and the cost per tonne of mix for roadbase and basecourse respectively.

Table 6.3 - Calculation for the cost of material per tonne of roadbase

Material	Material in mix (%)	Cost/t (£)	Cost/t of mix (£)	
Binder	3.85	100	(100 x 0.0385)	3.85
Stone	86.6	2	(2 x 0.866)	1.732
Bought material	9.6	2.49	(2.49 x 0.096)	0.239
RAP	0	1.254	(1.254 x 0)	0
			Total cost (£)	**5.821**

Table 6.4 - Calculations for cost of material per tonne of basecourse

Material	Material in mix (%)	Cost/t (£)	Cost/t of mix (£)	
Binder	4.6	100	(100 x 0.046)	4.6
Stone	81.1	2	(2 x 0.811)	1.622
Bought material	14.3	2.49	(2.49 x 0.143)	0.356
RAP	0	1.254	(1.254 x 0)	0
			Total cost (£)	**6.578**

From this we can calculate the cost per tonne to produce roadbase by adding the plant cost and total cost of material per tonne. The costs were calculated in the same manner for basecourse. Table 6.5 shows the plant costs, material costs and total cost for the two mixes.

Table 6.5 - Plant costs, material costs and total costs for the two mixes

Mix type	Plant costs (£)	Material costs (£)	Total costs (£)
roadbase	1.474	5.821	7.295
basecourse	1.474	6.578	8.052

The next part of the costing involved adding the RAP aggregate into each mix. This would involve calculating the increased energy required to increase the temperature of the virgin aggregate in order to accommodate the addition of cold RAP aggregate material and the associated cost savings of using the RAP aggregate as a replacement for the onsite aggregate and bought aggregate. The increase in energy required to superheat the aggregate has already been calculated in Table 4.29. The energy required is increased by the same % as the % RAP aggregate added. Therefore the plant costs are calculated as follows:

$$(0.2 + 0.166 + 0.1 + 0.0833 + (0.236x(1 + \%\ RAP)) + (0.566x(1 + \%\ RAP)) + 0.123$$

Therefore for the addition of 10% RAP:

$$(0.2 + 0.166 + 0.1 + 0.0833 + (0.236x1.1) + (1.566x1.1) + 0.123 = 1.5545 \qquad \textbf{Eq. 6.1}$$

Table 6.6 shows the increased plant cost as a result of using different % RAP.

Table 6.6 - Increased plant costs as a result of using RAP

RAP (%)	Plant cost (£)	RAP (%)	Plant cost (£)
0	1.474	5	1.5144
10	1.5545	15	1.5952
20	1.6356	25	1.676
30	1.7164	35	1.7568
40	1.7972	50	1.8376

The savings as a result of recovering binder from the RAP aggregate had to be determined. The % recovered binder had already been determined as can be seen in Appendix A. Using these figures the % of virgin binder could be calculated and the associated savings determined. Tables 6.7 and 6.8 shows the binder cost savings as a result of using different % RAP in roadbase and basecourse respectively.

Table 6.7 - Binder savings as a result of adding different % RAP to roadbase

RAP (%)	Binder in mix (%)	Recovered binder in mix (%)	Virgin binder in mix (%)	Cost binder/t of mix (£)	£ Saved
0	3.85	0	3.85	3.85	0.00
10	3.85	0.1331	3.717	3.717	0.1331
15	3.85	0.2014	3.649	2.649	0.2014
20	3.85	0.2681	3.582	3.582	0.2681
30	3.85	0.4156	3.435	3.435	0.4156

Table 6.8 - Binder savings as a result of adding various % RAP to basecourse

RAP (%)	Binder in mix (%)	Recovered binder in mix (%)	Virgin binder in mix (%)	Cost binder/t of mix (£)	£ Saved
0	4.6	0	4.6	4.6	0.00
10	4.6	0.132	4.468	4.468	0.132
15	4.6	0.198	4.402	4.402	0.198
20	4.6	0.264	4.336	4.336	0.264
25	4.6	0.33	4.27	4.27	0.33
30	4.6	0.396	4.204	4.204	0.396
35	4.6	0.462	4.138	4.138	0.462
40	4.6	0.528	4.072	4.072	0.528
50	4.6	0.66	3.94	3.94	0.66

The next cost savings to be addressed were the savings as a result of replacing the onsite material and bought material with the RAP aggregate. Firstly the % of onsite material and bought material had to determined. From the gradings obtained in Table 4.2 and Appendix A, the ratio of onsite to bought material was found to be 85:15 and 87:13 for roadbase and basecourse respectively. Using these ratios the cost savings could be worked out for both the onsite material and bought material for roadbase and basecourse mixes at different % RAP as can be seen in Tables 6.9, 6.10, 6.11 and 6.12 respectively. The cost of the onsite and bought aggregates were taken from Table 6.1.

Table 6.9 - Onsite material savings as a result of adding different % RAP to roadbase

RAP (%)	Onsite material in mix (%)	RAP material in mix (%)	Virgin material in mix (%)	Cost onsite material /t of mix (£)	Saved (£)
0	86.6	0	86.6	(2x0.866) = 1.732	0.00
10	86.6	8.5	78.1	(2x0.781) = 1.668	0.064
15	86.6	12.75	73.8	(2x0.738) = 1.636	0.096
20	86.6	17	69.6	(2x0.696) = 1.605	0.127
30	86.6	25.5	61.1	(2x0.611) = 1.541	0.191

Table 6.10 - Bought material savings as a result of adding different % RAP to roadbase

RAP (%)	Bought material in mix (%)	RAP material in mix (%)	Virgin material in mix (%)	Cost Bought material/t of mix (£)	Saved (£)
0	9.6	0	9.6	(2.49x0.096) = 0.2390	0.00
10	9.6	1.5	8.1	(2.49x0.081) = 0.2205	0.0185
15	9.6	2.25	7.35	(2.49x0.073) = 0.2112	0.0278
20	9.6	3	6.6	(2.49x0.066) = 0.2019	0.0371
30	9.6	4.5	5.1	(2.49x0.051) = 0.1832	0.0556

Table 6.11 - Onsite material savings as a result of adding different % RAP to basecourse

RAP (%)	Onsite material in mix (%)	RAP binder in mix (%)	Virgin material in mix (%)	Cost onsite material /t of mix (£)	Saved (£)
0	81.1	0	81.1	1.622	0.00
10	81.1	8.70	72.4	1.557	0.065
15	81.1	13.05	68.05	1.524	0.098
20	81.1	17.40	63.70	1.492	0.13
25	81.1	21.75	59.35	1.459	0.163
30	81.1	26.10	55.00	1.427	0.195
35	81.1	30.45	50.65	1.394	0.228
40	81.1	34.80	46.3	1.362	0.26
50	81.1	43.5	37.6	1.297	0.325

Table 6.12 - Bought material savings as a result of adding different % RAP to basecourse

% RAP	Bought material in mix (%)	RAP binder in mix (%)	Virgin material in mix (%)	Cost bought material /t of mix (£)	Saved (£)
0	14.3	0	14.30	0.356	0.00
10	14.3	1.3	13.00	0.340	0.016
15	14.3	1.95	12.35	0.331	0.025
20	14.3	2.6	11.70	0.3239	0.0321
25	14.3	3.25	11.05	0.3159	0.0401
30	14.3	3.9	10.40	0.3078	0.0482
35	14.3	4.55	9.75	0.299	0.057
40	14.3	5.2	9.1	0.2917	0.0643
50	14.3	6.5	7.9	0.2782	0.0778

Once these cost were calculated the total cost savings for each mixture could be calculated. Tables 6.13 and 6.14 summarise the costs obtained for plant, binder, onsite and bought aggregate, and the total cost to produce a tonne of roadbase and basecourse respectively.

Table 6.13 - Total cost to produce one tonne of roadbase material at different % RAP

Costs/t of mix (£)	0% RAP	10% RAP	15% RAP	20% RAP	30% RAP
Binder	3.85	3.717	3.649	3.582	3.435
Onsite aggregate	1.732	1.668	1.636	1.605	1.541
Bought aggregate	0.239	0.2205	0.2112	0.2019	0.184
Plant	1.474	1.5545	1.5952	1.6356	1.7164
Total (£)	**7.295**	**7.16**	**7.09**	**7.02**	**6.87**

Table 6.14 - Total cost to produce one tonne of basecourse material at different % RAP

Costs/t of mix (£)	0% RAP	10% RAP	15% RAP	20% RAP	25% RAP	30% RAP	35% RAP	40% RAP	50% RAP
Binder	4.6	4.468	4.401	4.336	4.27	4.204	4.138	4.072	3.94
Onsite	1.622	1.557	1.524	1.492	1.459	1.427	1.394	1.362	1.297
Bought	0.356	0.340	0.331	0.3239	0.3159	0.3078	0.299	0.2917	0.2782
Plant	1.474	1.5545	1.5952	1.6356	1.676	1.7164	1.7568	1.7972	1.8376
Total (£)	**8.052**	**7.9195**	**7.851**	**7.787**	**7.72**	**7.655**	**7.587**	**7.522**	**7.3528**

It is evident from Tables 6.13 and 6.14 that as a result of using RAP there are considerable cost savings for each material. The cost of buying the additional equipment in order to allow the use of RAP will be investigated later in this section. The next step in relation to costs was to determine the tonnages of roadbase and basecourse produced in each location as well as the tonnages of RAP aggregate available for all three individual locations.

6.4 The Available Volumes of Each Material Per Annum

In order to determine the quantities of each material produced in the three locations, the monthly tonnages had to be obtained first. After obtaining the tonnages for the locations over a number of years, it was evident that a number of plants had been de-commissioned and more modern plants had been commissioned in their place. The logistics of these plants meant that they would be producing greater tonnages of material. These changes could be seen in the monthly reports proceeding the commissioning of the new plants. The average monthly tonnages of each material were based on the monthly tonnage figures obtained for the 12 months proceeding the commissioning of the last plant. Table 6.15 shows the average monthly tonnages for roadbase and basecourse for the three locations.

Table 6.15 - Monthly tonnages of each material for the three locations

.	roadbase (t)	basecourse (t)	Total (t)
Quarry No. 1	1,662	1,000	2,662
Quarry No. 2	3,134	1,825	4,959
Quarry No. 3	1,790	1,512	3,302
Total (t)	6,586	4,337	10,923

After obtaining the tonnages for the different Quarries, the amount of RAP aggregate available in each Quarry had to be determined. Facilities for the recovery of surplus bituminous materials are presently established in two of these Quarries. Table 6.16 show the average quantities of material available for the two Quarries on a monthly basis.

Table 6.16 - Tonnage's of RAP stockpiled, to be crushed and monthly tonnages entering the facilities

	Crushed (t)	To be crushed (t)	Monthly tonnage's entering facilities
Quarry No. 1	3,000	1,250	150
Quarry No. 2	4,000	3,250 – 3750	150

Before any costing could be done, the amount of RAP aggregate required by each location if it were to start recycling had to be determined to ensure that there was ample amounts of RAP aggregate to allow continual recycling based on present quantities and monthly tonnages entering the facilities. The amount of RAP aggregate required to produce roadbase and basecourse based on using the optimum % RAP aggregate and the maximum % RAP aggregate that can be put into the mixes was determined for the two Quarries. Table 6.17 shows the results for the two Quarries.

Table 6.17 - Amount of RAP aggregate required per month to start recycling at the maximum and optimum % of RAP added

	28 mm Roadbase		basecourse	
	15%	30%	30%	50%
Quarry No.1	250t	500t	300t	500t
Quarry No.2	170t	940t	547t	912t

A generic matrix of the different combinations of RAP aggregate added for roadbase and basecourse for each Quarry was obtained in order to determine the different amounts of RAP aggregate per month required based on the different combinations. Table 6.18 shows this matrix.

Table 6.18 - Matrix of the different combinations and amounts of RAP aggregate required per month

	Combination No.	roadbase	basecourse	Tonnage's per month
Quarry No. 1	1	15%	30%	550
	2	15%	50%	750
	3	30%	30%	799
	4	30%	50%	1,000
Quarry No. 2	5	15%	30%	1,018
	6	15%	50%	1,383
	7	30%	30%	1,488
	8	30%	50%	1,853

From the matrix and the data obtained from Table 6.18 a graph of cumulative stocks versus cumulative usage could be plotted to determine how long the stocks would last based on current supplies. Figs 6.1 and 6.2 shows the results for Quarry No. 1 and Quarry No. 2 respectively.

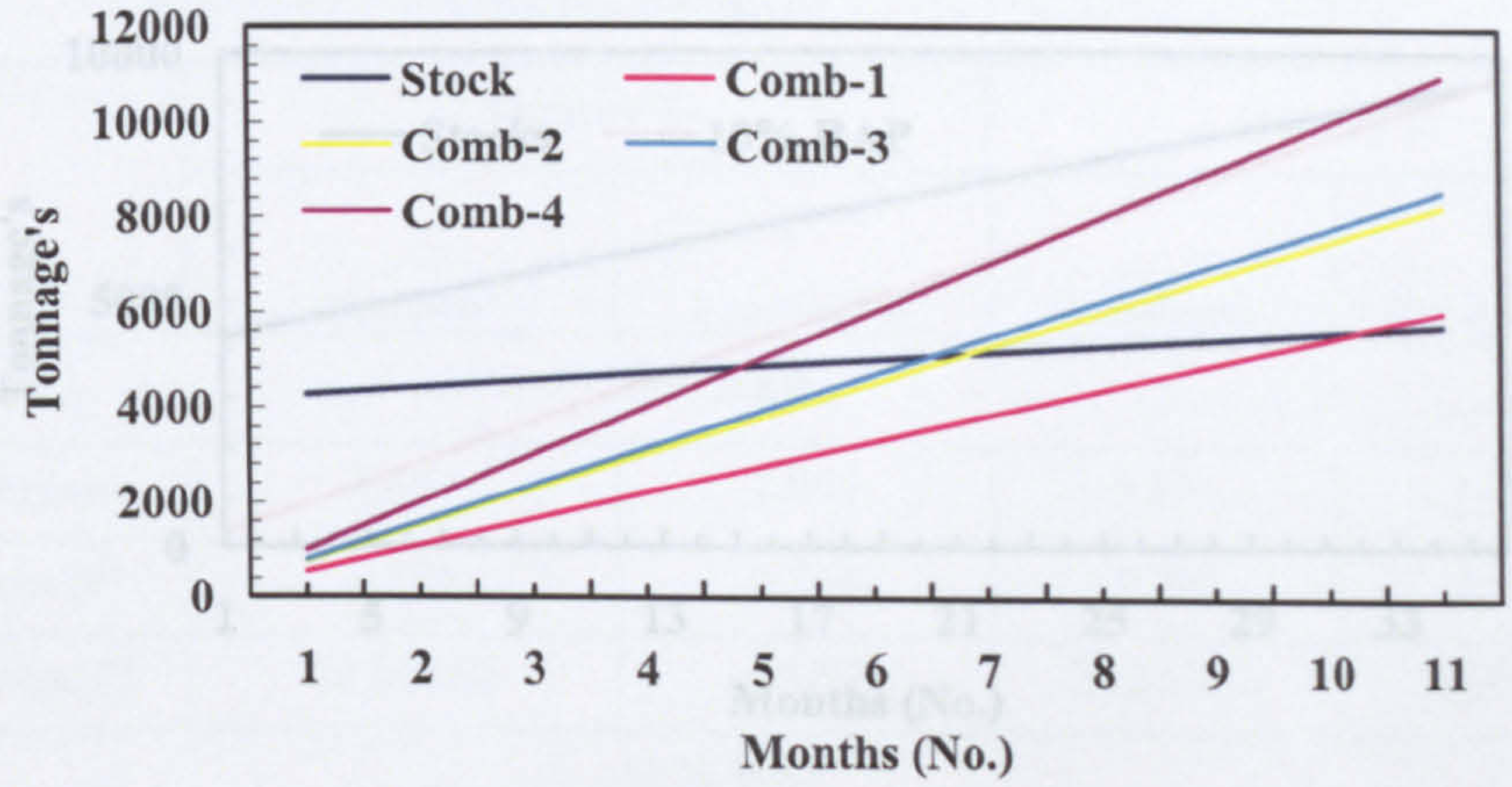


Fig. 6.1 - RAP usage vs Stocks for Quarry No. 1

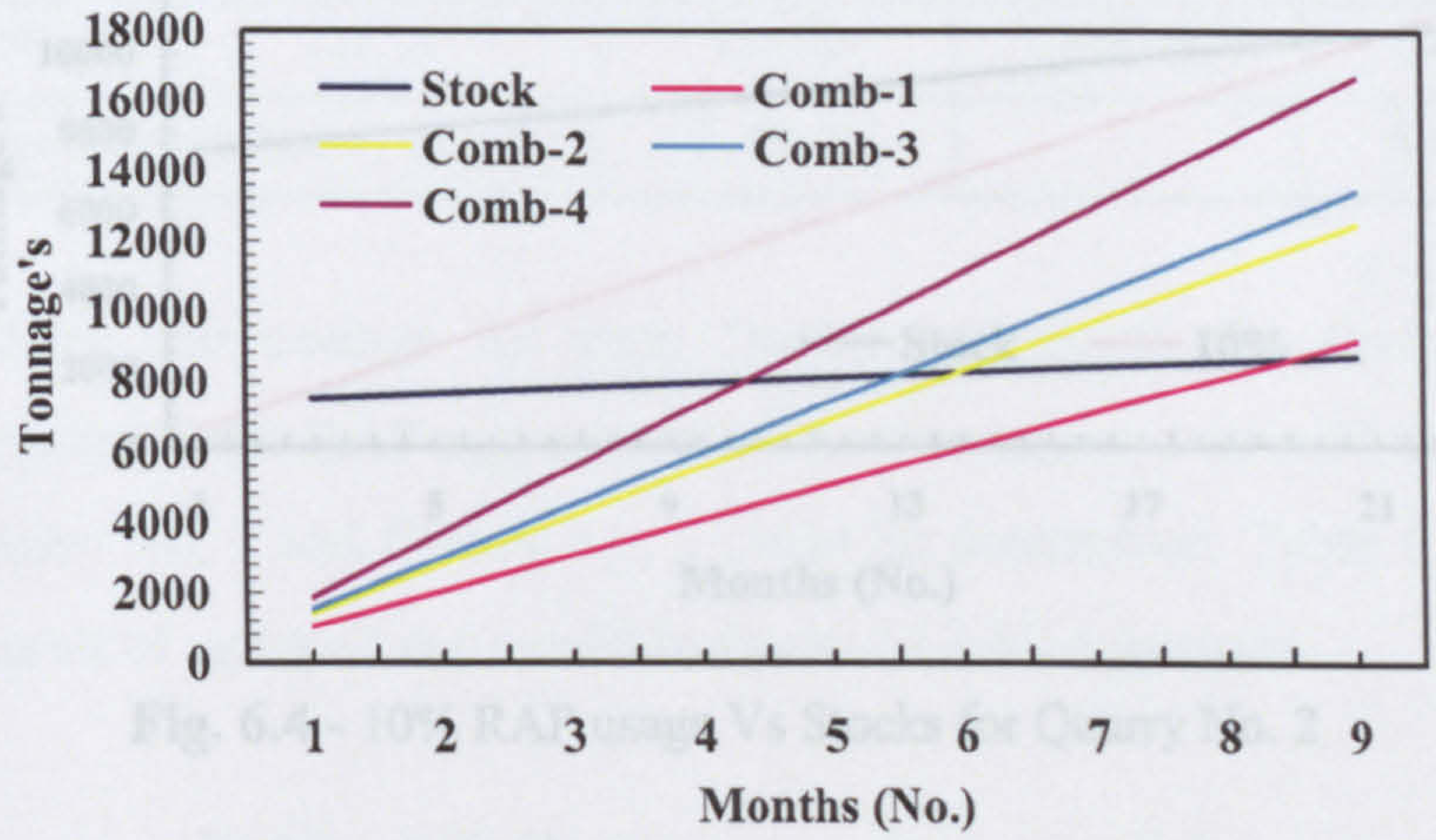


Fig. 6.2 - RAP usage vs Stocks for Quarry No. 2

From these graphs it is evident that there is only enough RAP to recycle at the rates mentioned above for anything between 4–10 months depending on the combination utilised. If one were to exclude the combinations altogether and to recycle at a constant overall rate of 10% RAP for the two mixes this would be more plausible. As the concept of recycling C&DW is only starting, the quantities of RAP aggregate will inevitably increase as the cost of landfill as well as the restrictions on disposal of C&DW increase. Figs 6.3 and 6.4 shows the RAP aggregate usage at 10% for both mixes against stocks for Quarry No. 1 and Quarry No. 2 respectively.

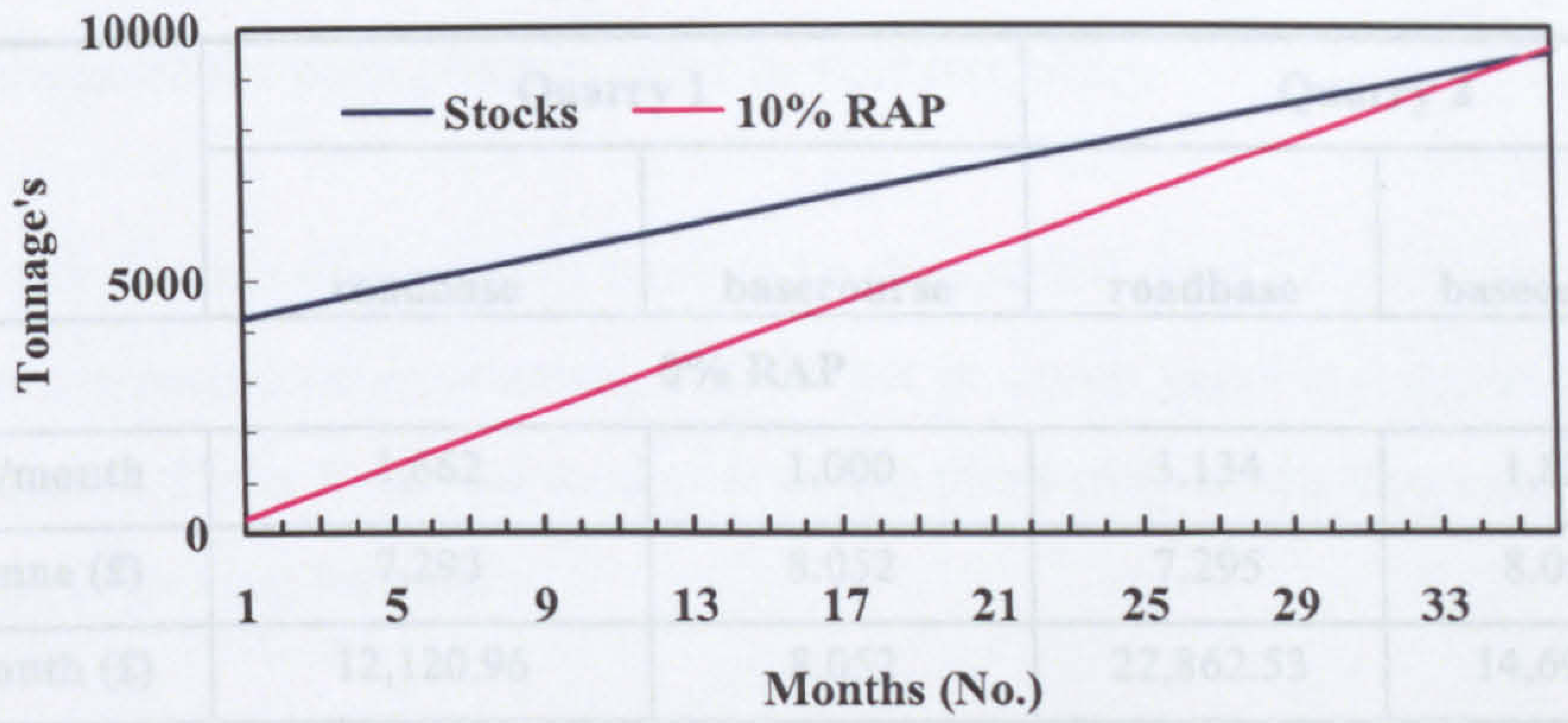


Fig. 6.3 10% - RAP usage Vs Stocks for Quarry No. 1

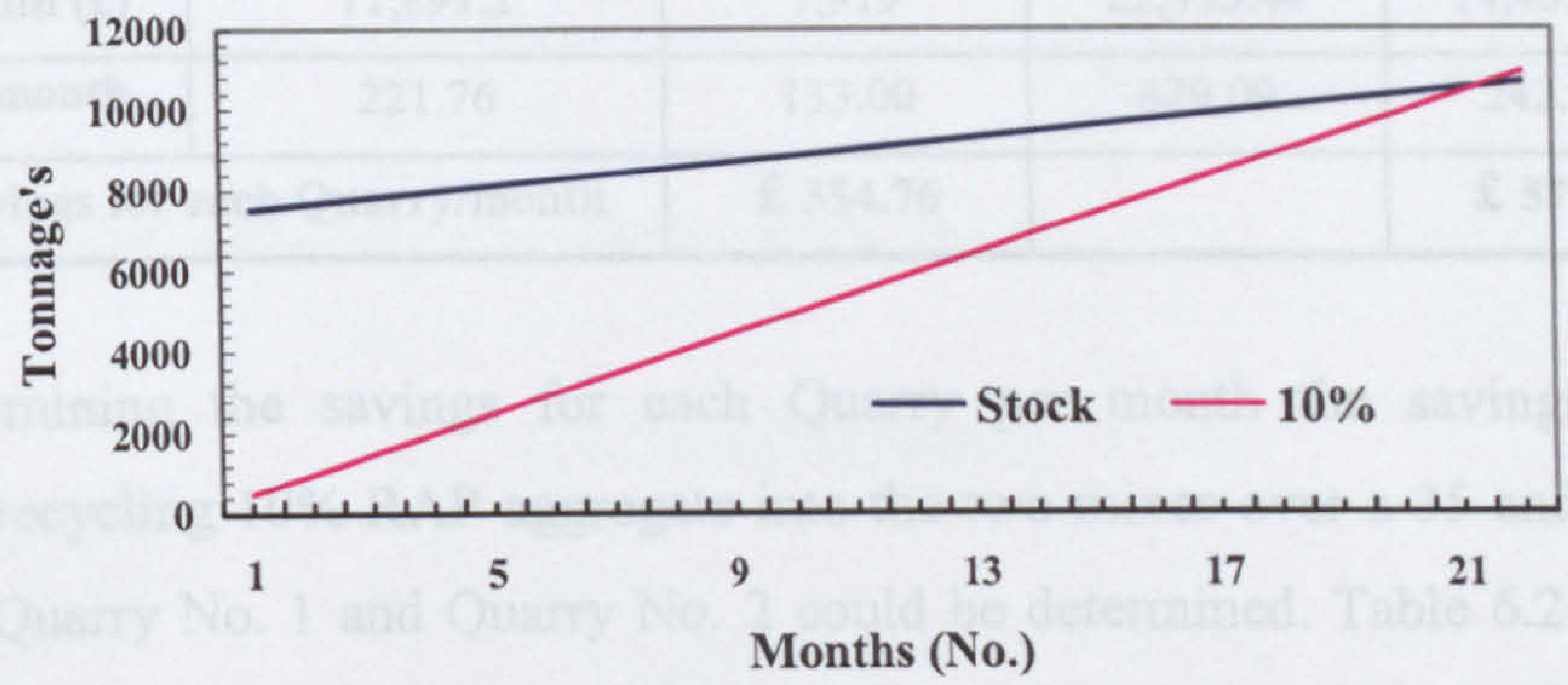


Fig. 6.4 - 10% RAP usage Vs Stocks for Quarry No. 2

From Figs 6.3 and 6.4 it is evident that recycling 10% RAP aggregate into the two mixes would allow recycling to take place for 35 months and 21 months for Quarry No. 1 and Quarry No. 2 respectively based on current stockpiles and current quantities being received each month at the two recycling facilities. The cost savings as a result of using 10% RAP aggregate in the two mixes from each Quarry over a 35 month and 21 month period respectively can be determined. Table 6.19 shows the saving as a result of the above criteria.

Table 6.19 - Saving per month as a result of using 10% RAP

	Quarry 1		Quarry 2	
	roadbase	basecourse	roadbase	basecourse
0% RAP				
Tonnes/month	1,662	1,000	3,134	1,825
Cost/tonne (£)	7.293	8.052	7.295	8.052
Cost/month (£)	12,120.96	8,052	22,862.53	14,694.9
10% RAP				
Tonnes/month	1,662	1,000	3,134	1,825
Cost/tonne (£)	7.16	7.919	7.16	7.919
Cost/month (£)	11,899.2	7,919	22,433.44	14,452.175
Savings/month	221.76	133.00	429.09	242.725
Total savings for each Quarry/month		**£ 354.76**		**£ 571.82**

After determining the savings for each Quarry per month the savings based on constantly recycling 10% RAP aggregate into the two mixes over a 35 and 21 months period for Quarry No. 1 and Quarry No. 2 could be determined. Table 6.20 shows the savings as a result of using all the available stocks of RAP aggregate.

Table - 6.20 Savings as a result of using all the available stocks of RAP aggregate

	Quarry 1	Quarry 2
Savings per month (£)	354.76	671.82
No. of months	35	21
Total savings (£)	12,416.6	14,108.22

There are considerable savings from using RAP aggregate in bituminous mixes. In order to start recycling the present plants would have to be altered to accommodate for the addition of the RAP aggregate into the mixes.

6.5 Cost of Modifying Existing Plant to Allow Recycling to Take Place

The additional equipment would include feed hopper conveyor belts as outlined in Fig. 2.20. Sheds like the one shown in Fig. 2.24 would have to be built to store the RAP. The total cost to modify the plants is in the region of £13,333 and the erection of the sheds would result in an approximate cost of £5,714. Therefore the total expenditure of £19,047 is required for each plant in order to start recycling.

In order to pay for this plant, it will have to be discounted over a certain period of time. In this case the period of discounting will be set at seven years and an interest rate of 3.6% per annum. From these the total cost of the plant over seven years can be worked out using the following formula:

$$(1+n)^y (f) \qquad \textbf{Eq. 6.2}$$

Where: n = Interest rate expressed as a decimal

^{y} = Number of years plant is depreciated over

f = Amount of finance needed to purchase plant

Therefore the total cost of the plant including the shed is:

$$(1+0.036)^7 (£19,047)$$

$$= 1.2809 * 19,047$$

$$= £24,397$$

After calculating the total cost of the plant over seven years, the savings as a result of using the available tonnages of RAP aggregate over seven years based on present stockpiles and monthly tonnages had to be determined in order to see if buying the additional plant to start recycling is viable.

It is assumed that the cost of the material per tonne will remain the same and that the there will be a 50:50 split between the amount of RAP aggregate used in roadbase and basecourse after the existing stockpiles are used. Table 6.21 shows the savings per month based on the third year of using RAP aggregate for Quarry 1. Table 6.22 shows the savings per year and the total savings after seven years for Quarry No. 1 and Quarry No. 2 respectively.

Table 6.21 - Savings per month based on the third year of using RAP for Quarry No. 1

Month	Available RAP tonnage's	Amt RAP agg used in roadbase	Amt RAP Agg used in basecourse	Savings from using RAP in roadbase	Savings from using RAP in basecourse	Total savings per month
1	266	166	100	224	132.5	356.5
2	266	166	100	224	132.5	356.5
3	266	166	100	224	132.5	356.5
4	266	166	100	224	132.5	356.5
5	266	166	100	224	132.5	356.5
6	266	166	100	224	132.5	356.5
7	266	166	100	224	132.5	356.5
8	266	166	100	224	132.5	356.5
9	266	166	100	224	132.5	356.5
10	266	166	100	224	132.5	356.5
11	266	166	100	224	132.5	356.5
12	181	90.5	90.5	122	120	242
				Total savings for year (£)	**4,163**	

Table 6.22 - Annual savings and total savings after seven years for Quarry No. 1 and Quarry No. 2

Year	Savings Quarry No. 1	Savings Quarry No. 2
1	4,279	7,980
2	4,279	6,711
3	4,163	2,408
4	2,408	2,408
5	2,408	2,408
6	2,408	2,408
7	2,408	2,408
Total (£)	**22,353**	**26,731**

From the results obtained it is evident that after seven years the cost of the recycling equipment has been recovered with a profit in the region of £3,306 and £7,684 for Quarry No. 1 and Quarry No. 2 respectively. In order to fully assess the viability of these investments the time value of money needs to be taken into consideration. Two methods of doing this are the: Net Present Value (NPV) and Internal Rate of Return (IRR).

In the NPV all the expenditure and savings are brought back to the base year and when the positives and negatives are netted out the result is the NPV. The decision values for NPV are: NPV $\geq$ 0 Accept and NPV < 0 Reject

The NPV can be calculated using the following formula:

$$NPV = F_0 + (F_1/((1+k)^n)) \qquad \textbf{Eq. 6.3}$$

Where:

F_0 = Cash flow at time zero

F_1 = Cash flow one year after time zero

k = Interest rate at which compounding takes place = 3.6%

n = Number of years over which compounding takes place

From this formula the savings as a result of adding the RAP aggregate can be discounted back to time zero to determine if the project is viable. Tables 6.23 and 6.24 shows the discounted cash flows and the NPV for Quarries No. 1 and No. 2 respectively.

Table 6.23 - Discounted cash flows and NPV for Quarry No. 1

Year No.	Cash flows	Discounted cash flows £	NPV
0	-19,047	-19,047	-19,047
1	4,279	4,130	-14,917
2	4,279	3,986	-10,931
3	4,163	3,743	-7,188
4	2,408	2,090	-5,098
5	2,408	2,017	-3,081
6	2,408	1,947	-1,134
7	2,408	1,879	745

Table 6.24 - Discounted cash flows and NPV for Quarry No. 2

Year No.	Cash flows	Discounted cash flows £	NPV
0	-19,047	-19,047	-19,047
1	7,980	7,702	-11,345
2	6,711	6,252	-5,093
3	2,408	2,165	-2,928
4	2,408	2,090	-838
5	2,408	2,017	1,179
6	2,408	1,947	3,126
7	2,408	1,879	5,005

From the Tables above it is evident that based on the NPV results both plants give a rate of return which is greater than 3.6%. From these results it can be seen that after 7 years and 5 years the NPV is positive for Quarries No. 1 and Quarry No. 2 respectively.

The next method of appraisal to be used is the Internal Rate of Return (IRR). The IRR tells the interest you will receive by putting your money into a particular project. It shows the degree to which the cash inflows exceed the cash outflows on an annualised % basis, taking account of the timing of these cash flows. The following formula was used to calculate the IRR:

$$F_0 + (F_1/(1+r)) + (F_2/(1+r)^2) + \ldots.(F_n/(1+r)^n) = 0 \qquad \textbf{Eq. 6.4}$$

Where:

F_0 = Cash flow at time zero.

F_1 = Cash flow one year after tome zero.

r = Interest rate.

^{n} = Number of years over which compounding takes place.

In this formula it is the value r, which makes the equation hold. By changing the value of (r) a positive or negative NPV can be obtained. By extrapolation the IRR value can be determined. Table 6.25 shows how the NPV was calculated at an interest rate of 5% for Quarry No. 1.

Table 6.25 - NPV calculated at an interest rate of 5% for Quarry No. 1

Year	Costs/savings	$(1+r)^n$	Discounted costs/savings	NPV
0	-19047	1	-19,047	-19,047
1	4,279	1.05	4,075	-14,971
2	4,279	1.1025	3,881	-11,090
3	4,163	1.157	3,596	-7,974
4	2,408	1.215	1,981	-5,513
5	2,408	1.276	1,886	-3,626
6	2,408	1.34	1,797	-1,829
7	2,408	1.407	1,711	-188

From Table 6.25 the NPV is negative which indicates that the IRR is less than 5% for Quarry 1. NPV's were worked out at different interest rates for Quarries No. 1 and No. 2 and the IRR is calculated for same. Table 6.26 shows the NPV results at different interest rates for Quarries No. 1 and No. 2.

Table 6.26 - NPV results at different interest rates for Quarry 1 and 2

Quarry No. 1		Quarry No. 2	
Interest rate	NPV	Interest rate	NPV
4	539	14	159
5	-118	15	-261

From these results the interest rate that produces an NPV of zero can be determined using the following formula:

$$IRR = P + \left(\frac{X \longrightarrow Y}{(X \longrightarrow Z)(P - P_1)} \right) \quad \textbf{Eq. 6.5}$$

Where:

P = Interest rate which gives positive NPV.

X = Positive NPV.

Y = NPV when IRR is determined (always zero).

Z = Negative NPV.

P_1 = Interest rate which gives negative NPV

By substitution

$$IRR = 4 + (((539 - 0)/(539 - (-118))x5-4)$$
$$= 4 + ((0.1019x(1))$$
$$= 4.1019$$

The IRR for Quarry No. 1 is 4.1019%. Using the same formula an IRR of 14.378% was calculated for Quarry No. 2. The reason for higher IRR values for Quarry No.2 is there is a larger amount of RAP aggregate available at this location. These results suggest that depreciating the plant in Quarry 1 over seven years would not be viable. From the high IRR value for Quarry No. 2, the period of depreciation could be reduced as most establishments expect an IRR in the region of 8%. The next part of the costing to be looked at were the savings as a result of the reduce layer thickness of the pavement as calculated using the shell design package.

6.6 Financial Benefits of a Reduced Layer Thickness

Based on the reduced layer thickness (Table 5.15) as calculated using the shell pavement design package the financial benefits can be calculated. The benefits for each of the nine systems can be compared with the base mix. It was decided that the costs would be based on a road of 1km in length and a width of 21m for each of the nine systems. The costs for each individual layer could be determined for the nine systems as outlined in Table 5.5. Tables 6.27, 6.28 and 6.29 show the cost for 30% HRA, basecourse and roadbase respectively based on the above criterion.

Table - 6.27 HRA costs for the nine different systems

System No.	1	2	3	4	5	6	7	8	9
RAP (%)	0	0	0	0	0	0	0	0	0
Length (m)	1,000	1,000	1,000	1,000	1,000	1,000	1,000	1,000	1,000
Width(m)	21	21	21	21	21	21	21	21	21
Depth (m)	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04
Density (kg/m^3)	2238	2238	2238	2238	2238	2238	2238	2238	2238
Tonnage's (t)	375	375	375	375	375	375	375	375	375
Cost/t (£)	11.38	11.38	11.38	11.38	11.38	11.38	11.38	11.38	11.38
Cost/km (£)	4,267	4,267	4,267	4,267	4,267	4,267	4,267	4,267	4,267

Table 6.28 - Basecourse costs for the nine different systems

System No.	1	2	3	4	5	6	7	8	9
RAP (%)	0	0	30	0	40	30	30	40	40
Length (m)	1,000	1,000	1,000	1,000	1,000	1,000	1,000	1,000	1,000
Width (m)	21	21	21	21	21	21	21	21	21
Depth (m)	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06
Density (kg/m^3)	2.363	2.363	2.363	2.363	2.363	2.363	2.363	2.363	2.363
Tonnage's (t)	533	533	533	533	533	533	533	533	533
Cost/t (£)	8.052	8.052	7.655	8.052	7.522	7.655	7.655	7.522	7.522
Cost/km (£)	4,291	4,291	4,080	4,291	4,009	4,080	4,080	4,009	4,009

Table 6.29 - Roadbase costs for the nine different systems

System No.	1	2	3	4	5	6	7	8	9
RAP (%)	0	15	0	30	0	15	30	15	30
Length (m)	1,000	1,000	1,000	1,000	1,000	1,000	1,000	1,000	1,000
Width (m)	21	21	21	21	21	21	21	21	21
Depth (m)	0.14	0.08	0.13	0.11	0.14	0.08	0.11	0.08	0.11
Density (kg/m^3)	2.388	2.388	2.388	2.388	2.388	2.388	2.388	2.388	2.388
Tonnage's (t)	1231	703	1143	967	1231	703	967	703	967
Cost/t (£)	7.295	7.09	7.295	6.87	7.295	7.09	6.87	7.09	6.87
Cost/km (£)	8,980	4,984	8,338	6,643	8,980	4,984	6,643	4,984	6,643

From these cost the overall cost can be calculated for the pavement. Table 6.30 combines these costs to obtain the cost based on 1km of road with an average width of 21m.

Table 6.30 - Cost as a result of reduced layer thickness for the nine systems

System No.	30% HRA (£)	Basecourse (£)	Roadbase (£)	Total (£)
1	4,267	4,291	8,986	17,538
2	4,267	4,291	4,984	13,542
3	4,267	4,080	8,330	16,896
4	4,267	4,291	6,643	15,201
5	4,267	4,009	8,980	17,256
6	4,267	4,080	4,984	13,335
7	4,267	4,080	6,643	14,990
8	4,267	4,009	4,984	13,260
9	4,267	4,009	6,643	14,919

From Table 6.30 it is clear that system 8 cost the least amount to construct. It is also clear that system 1 which represents all the base mixes is more expensive. These results shows that using recycled materials can be cost effective and environmentally friendly as the amount of material that is used in the construction can be reduced thus saving on aggregate extraction, reducing the amount of material to landfill and reducing the cost/km for new roads and reconstruction of old roads.

6.7 Summary

From the cost benefit analysis carried out in this chapter the following conclusions can be made:

- Savings can be achieved as a result of using RAP aggregate.
- The initial cost of setting up the plant to start recycling can be recouped after seven years with an IRR of 4.1019% and 14.378% for Quarries No. 1 and No. 2 respectively.
- There are ample amounts of RAP aggregate to allow recycling to commence.
- There are also financial benefits as a result of the reduced layer thickness as calculated using the shell pavement design package.

- Environmental savings as a result of using RAP in terms of: reduction in quantities of virgin aggregate extracted, a reduction in the placing of RAP aggregate in landfill and noise/dust pollution associated with virgin aggregate extraction.

CHAPTER 7 – SUMMARY AND CONCLUSIONS

7.1 General Summary

This thesis presents a comprehensive study into the possibilities of using crushed Reclaimed Asphalt Pavements (RAP) as an alternative aggregate in new bituminous mixtures. The experimental and theoretical results presented in this thesis prove that bituminous mixtures can be successfully produced using RAP aggregates, from road planings and surplus bituminous materials. Bituminous materials produced with these aggregates performed better than bituminous materials made using virgin aggregate. In some cases an addition of 50% RAP was achievable. The increase in performance of RAP aggregate in bituminous mixtures resulted in a reduction in the overall depth of a road pavement. The addition of RAP aggregate, sometimes up to 50% in new bituminous mixtures generates savings in costs due to the low cost of RAP aggregate.

Chapter 2 presented a thorough literature review on the subject. The history of road construction; the history of recycling construction and demolition wastes (C&DW) and the recycling of RAP. This chapter reviewed recycling prospects, environmental aspects, economic aspects, methods of crushing and sorting of C&DW and road recycling techniques. The emphasis on quality control system in the production of the recycled aggregate was also addressed.

Chapter 3 outlined an extensive literature review and theoretical background on the mechanical and physical tests carried out on aggregate, the penetration test and softening point test carried out on binder as well as empirical, fundamental and simulative tests carried out on bituminous materials.

Chapter 4 examined results of the experimental and theoretical investigation carried out on bituminous materials with and without RAP aggregate. Mix designs and theoretical densities were determined for the various mixtures. Equations were presented to convert standard vehicle wheel loads into loads applied to laboratory specimens. Results presented for: % air void, stiffness, fatigue, stability, flow, wheel tracking, tensile strength, cantabro and ten percent fines. The results for bituminous mixtures containing virgin aggregate and different %RAP aggregate were compared to bituminous mixtures containing purely virgin materials.

Chapter 5 investigated reducing the pavement thickness as a result of the improvement in the stiffness of the new bituminous mixtures containing RAP aggregate. Results of stresses, strains and displacements at various depths in the pavement were compared to those obtained for the bituminous mixtures with 0% RAP aggregate.

Chapter 6 presented the results of the investigation into the economical viability of using RAP aggregate in the production of new bituminous materials. Savings resulting from using RAP aggregate were offset against the cost of modifying the existing plant to allow the addition of RAP aggregate. The Internal Rate of Return (IRR) and Net Present Value (NPV) were obtained to determine the economical viability of the project from a commercial prospective.

7.2 Conclusions

From the work carried out in this thesis the following conclusions can be made:

- Specimens containing 30% RAP aggregate recorded the highest stiffness (ITSM) result for basecourse made with RAP aggregate from both Quarries No. 1 and No. 2. RAP aggregate between 30% and 50% can be added to basecourse, but caution needs to be exercised to ensure that the final product is compliant with the standard specification.

- Specimens containing 15% RAP recorded the highest stiffness (ITSM), (ITST), fatigue (ITFT), tensile strength (ITT) and the lowest % wear results for roadbase. RAP aggregate between 15% and 30% can be added to roadbase and still have higher stiffness, fatigue and tensile strength values than 0% RAP.

- The maximum amount of %RAP added to the 30% HRA mix was 7.5%. Any % RAP above that will cause the specimens to fail due to high rut depth.

- As a result of the increased stiffness (ITSM), (ITST), fatigue (ITFT), tensile strength (ITT) and % wear from adding RAP aggregate to bituminous mixtures the thickness of the asphalt layers can be reduced. The number of million standard axles that the subgrade can carry increased also as a result of the increased stiffness.

- There are financial savings from a reduction in layer thicknesses. These savings will benefit local authorities whose budgets are already tight. Savings resulting from using RAP aggregate which is cheaper than virgin aggregate as well as utilising binder recovered from RAP aggregate benefits the producer. Environmental savings as a result of using RAP aggregate are: reduction in quantities of virgin aggregate extracted, a reduction in the placing of RAP aggregate in landfill and noise/dust pollution associated with virgin aggregate extraction.

- Results from the Ten-percent Fines Value (TFV) tests showed that as the % RAP aggregate increased TFV and the penetration value of the recovered binder decreased. The reduced penetration of the binder by adding RAP aggregate explains why the stiffness of the bituminous mixtures increased till the optimum % RAP. The results showed adding RAP aggregate resulted in the reduction of the TFV. As a result the stiffness of the bituminous mixtures was reduced after the optimum % RAP aggregate (30% roadbase, 15% basecourse) value was reached. The use of 50pen binder vastly increased the stiffness of the mixture in terms of ITSM, ITST and ITFT.

- The addition of cold RAP aggregate to heated virgin materials gave similar results to adding heated RAP aggregate with heated virgin materials, which makes the addition of RAP aggregate into mixing plants easier as heating of RAP aggregate would cause the binder to melt and thus clog the screens.

- The air void content of bituminous mixtures with RAP aggregate was lower than mixtures without RAP aggregate. The results showed that the parafilm method used to determine % air void measurement gives more realistic results than the air/water method.

- Since the completion of this project Roadstone Dublin Ltd. has started to use RAP aggregate in the production of bituminous mixtures at Quarry No. 2. A number of trials are planned in conjunction with Offaly County Council for the summer of 2005. A new plant has been acquired for Quarry No. 1 and is due to be installed in the winter of 2005. This plant will use RAP aggregate in the production of bituminous mixtures.

7.3 Suggestions for Further Research

After carrying out this research the following areas for further research were discovered:

1. More testing would have to be conducted to determine the optimum percentage RAP when the cold RAP aggregate is added.
2. The effects that moisture has on the addition of cold RAP aggregate into the mix in terms of superheating the aggregate to accommodate the fluctuating moisture content.

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APPENDIX A

TABLES OF RESULTS

Table A.1 - Calculation of aggregate required and theoretical density

Roadbase Quarry 1				
Calculation of maximum densities for bituminous mixtures				
	Density (kg/m^3)	**% AGG**	**Total % of mix**	**Amt of Agg reqd**
28mm	2700	12	11.538	720
20mm	2700	12	11.538	720
14mm	2690	14	13.461	840
10mm	2660	16	15.384	960
6mm	2760	13	12.4995	780
CRF	2700	23	22.1145	1380
Sand	2697	10	9.615	600
Binder	1030		3.85	

Total aggregate content (%)	96.15	Binder Content (%)	3.85
Theoretical max density (kg/m^3)	**2540**		
Air voids	6		
Mix density (kg/m^3)	**2388**		

Mass required to achieve density (g)	
density =mass/volume	
Height required (mm)	70
Radius (mm)	50
Volume (mm^3)	549778.3
Mass (g)	1313.103

Total mass required into mould (g)	**1313.1**
Total binder required	
Mass of aggregate batch (g)	6000
Mass of aggregate +binder (g)	6240.25
Total binder (g)	**240.25**

Table A.2 - Calculation of mix proportions

Roadbase Quarry 1				
Batch size (g)	**% Virgin**	**% RAP**	**Amount virgin Agg (g)**	**Amount RAP Agg (g)**
6000	90	10	5400	600
RAP grading	**% RAP**	**Amount RAP (g)**	**Total Amt Agg required (g)**	**Amount virgin (g)**
28mm	1.9	11	720	709
20mm	5.3	32	720	688
14mm	7.4	44	840	796
10mm	13.5	81	960	879
6mm	30.3	182	780	598
Dust	36.7	220	1380	1160
Filler	4.9	29	600	571
Total	100	600		5400
Binder (%)	2.98	18		
Recovered binder (%)	2.98	Theoretical maximum density = 2540 Kg/m³		
		Mix density = 2388 Kg/m³		
Binder content(%)	3.85	Total mass into mould 1313g		
Theoretical binder reqd (g)	222.4			
Actual binder reqd (g)	**231.3**	% Recovered binder in mix = 3.85-((231.3/6231)*100) = 0.13311		

Table A.3 - Calculation of mix proportions

Roadbase Quarry 1				
Batch size (g)	**% Virgin**	**% RAP**	**Amount virgin Agg (g)**	**Amount RAP Agg (g)**
6000	85	15	5100	900
RAP grading	**% RAP**	**Amount RAP (g)**	**Total Amt Agg required (g)**	**Amount virgin (g)**
28mm	1.9	17	720	703
20mm	5.3	46	720	674
14mm	7.4	65	840	775
10mm	13.5	118	960	842
6mm	30.3	265	780	515
Dust	36.7	321	1380	1059
Filler	4.9	43	600	557
Total	100	900		5100
Binder (%)	2.98	26		
Recovered binder (%)	2.98	Theoretical maximum density = 2540 Kg/m^3		
		Mix density = 2388 Kg/m^3		
Binder content (%)	3.85	Total mass into mould 1313g		
Theoretical binder reqd (g)	214.2			
Actual binder reqd (g)	**227.2**	% Recovered binder in mix = 3.85-((227.2/6227.2)*100) = 0.2014		

Table A.4 - Calculation of mix proportions

Roadbase Quarry 1				
Batch size (g)	**% Virgin**	**% RAP**	**Amount virgin Agg (g)**	**Amount RAP Agg (g)**
6000	80	20	4800	1200
RAP grading	**% RAP**	**Amount RAP (g)**	**Total Amt Agg required (g)**	**Amount virgin (g)**
28mm	1.9	22	720	698
20mm	5.3	62	720	658
14mm	7.4	86	840	754
10mm	13.5	157	960	803
6mm	30.3	353	780	427
Dust	36.7	428	1380	952
Filler	4.9	57	600	543
Total	100	1200		4800
Binder (%)	2.98	35		
Recovered binder (%)	2.98	Theoretical maximum density = 2540 Kg/m^3		
		Mix density = 2388 Kg/m^3		
Binder content (%)	3.85	Total mass into mould 1313g		
Theoretical binder reqd (g)	205.5			
Actual binder reqd (g)	**222.9**	% Recovered binder in mix = 3.85-((222.9/6222.9)*100) = 0.2681		

Table A.5 - Calculation of mix proportions

Roadbase Quarry 1				
Batch size (g)	**% Virgin**	**% RAP**	**Amount virgin Agg (g)**	**Amount RAP Agg (g)**
6000	70	30	4200	1800
RAP grading	**% RAP**	**Amount RAP (g)**	**Total Amt Agg required (g)**	**Amount virgin(g)**
28mm	1.9	34	720	686
20mm	5.3	95	720	625
14mm	7.4	133	840	707
10mm	13.5	243	960	717
6mm	30.3	545	780	235
Dust	36.7	661	1380	719
Filler	4.9	88	600	512
Total	100	1800		4200
Binder (%)	2.98	54		
Recovered binder (%)	2.98	Theoretical maximum density = 2540 Kg/m^3		
		Mix density = 2388 Kg/m^3		
Binder content (%)	3.85	Total mass into mould 1313g		
Theoretical binder reqd (g)	186.6			
Actual binder reqd (g)	**213.4**	% Recovered binder in mix = 3.85-((213.4/6213.4)*100) = 0.4155		

Table A.6 - Calculation of aggregate required and theoretical density

Basecourse Quarry 1				
Calculation of maximum densities for bituminous mixtures				
	Density (kg/m^3)	**% AGG**	**Total % of mix**	**Amt of Agg reqd (g)**
28mm	2700	0	0	0
20mm	2700	24	22.848	1440
14mm	2690	15	14.28	900
10mm	2660	13	12.376	780
6mm	2760	10	9.52	600
CRF	2700	23	21.896	1380
Sand	2697	15	14.28	900
Binder	1030		4.6	

Total aggregate content (%)	95.2	Binder Content (%)	4.8
Theoretical max density (kg/m^3)	**2516**		
Air voids (%)	6		
Mix density (kg/m^3)	**2365**		

Mass required to achieve density	
density =mass/volume	
Height required (mm)	70
Radius (mm)	50
Volume (mm^3)	549778.3
Mass (g)	1300.33

Total mass required into mould (g)	**1300.33**
Total binder required (%)	
Mass of aggregate batch (g)	6000
Mass of aggregate + binder (g)	6302.52
Total binder (g)	**302.521**

Table A.7 - Calculation of mix proportions

Basecourse Quarry 1				
Batch size (g)	**% Virgin**	**% RAP**	**Amount virgin Agg (g)**	**Amount RAP Agg (g)**
6000	90	10	5400	600
RAP grading	**% RAP**	**Amount RAP (g)**	**Total Amt Agg required (g)**	**Amount virgin (g)**
28mm	0	0	0	0
20mm	4	24	24	1416
14mm	11	66	15	834
10mm	18.6	112	13	668
6mm	25.3	152	10	448
Dust	22.7	136	23	1244
Sand	18.4	110	15	790
Total	100	600		5400
Binder (%)	17			
Recovered binder (%)	2.9	Theoretical maximum density = 2516 Kg/m^3		
		Mix density = 2365 Kg/m^3		
Binder content (%)	4.6	Total mass into mould 1300.33g		
Theoretical binder reqd (g)	271.9			
Actual binder reqd (g)	**280.6**	% Recovered binder in mix = 4.6-((280.6/6280.6)*100) = 0.132		

Table A.8 - Calculation of mix proportions

Basecourse Quarry 1				
Batch size (g)	**% Virgin**	**% RAP**	**Amount virgin Agg (g)**	**Amount RAP Agg (g)**
6000	85	15	5100	900
RAP grading	**% RAP**	**Amount RAP (g)**	**Total Amt Agg required (g)**	**Amount virgin (g)**
28mm	0	0	0	0
20mm	4	36	24	1404
14mm	11	99	15	801
10mm	18.6	167	13	613
6mm	25.3	228	10	372
Dust	22.7	204	23	1176
Sand	18.4	166	15	734
Total	100	900		
Binder (%)	26			
Recovered binder (%)	2.9	Theoretical maximum density = 2516 Kg/m^3		
		Mix density = 2365 Kg/m^3		
Binder content (%)	4.6	Total mass into mould 1300.33g		
Theoretical binder reqd (g)	263.2			
Actual binder reqd (g)	276.3	% Recovered binder in mix = 4.6-((276.3/6276.3)*100) = 0.197		

Table A.9 - Calculation of mix proportions

Basecourse Quarry 1				
Batch size (g)	**% Virgin**	**% RAP**	**Amount virgin Agg (g)**	**Amount RAP Agg (g)**
6000	80	20	4800	1200
RAP grading	**% RAP**	**Amount RAP (g)**	**Total Amt Agg required (g)**	**Amount virgin (g)**
28mm	0	0	0	0
20mm	4	48	24	1392
14mm	11	132	15	768
10mm	18.6	223	13	557
6mm	25.3	304	10	296
Dust	22.7	272	23	1108
Sand	18.4	221	15	679
Total	100	1200		4800
Binder (%)	35			
Recovered binder (%)	2.9	Theoretical maximum density = 2516 Kg/m^3		
		Mix density = 2365 Kg/m^3		
Binder content (%)	4.6	Total mass into mould 1300.33g		
Theoretical binder reqd (g)	254.5			
Actual binder reqd (g)	**271.9**	% Recovered binder in mix = 4.6-((271.9/6271.9)*100) = 0.264		

Table A.10 - Calculation of mix proportions

Basecourse Quarry 1				
Batch size	**% Virgin**	**% RAP**	**Amount virgin Agg (g)**	**Amount RAP Agg (g)**
6000	75	25	4500	1500
RAP grading	**% RAP**	**Amount RAP (g)**	**Total Amt Agg required (g)**	**Amount virgin (g)**
28mm	0	0	0	0
20mm	4	60	24	1380
14mm	11	165	15	735
10mm	18.6	279	13	501
6mm	25.3	380	10	221
Dust	22.7	341	23	1040
Sand	18.4	276	15	624
Total	100	1500		4500
Binder (%)	44			
Recovered binder (%)	2.9	Theoretical maximum density = 2516 Kg/m^3		
		Mix density = 2365 Kg/m^3		
Binder content (%)	4.6	Total mass into mould 1300.33g		
Theoretical binder reqd (g)	245.8			
Actual binder reqd (g)	**267.6**	% Recovered binder in mix = 4.6-((267.6/6267.6)*100) = 0.330		

Table A.11 - Calculation of mix proportions

Basecourse Quarry 1				
Batch size (g)	**% Virgin**	**% RAP**	**Amount virgin Agg (g)**	**Amount RAP Agg (g)**
6000	70	30	4200	1800
RAP grading	**% RAP**	**Amount RAP (g)**	**Total Amt Agg required (g)**	**Amount virgin (g)**
28mm	0	0	0	0
20mm	4	72	24	1368
14mm	11	198	15	702
10mm	18.6	335	13	445
6mm	25.3	455	10	145
Dust	22.7	409	23	971
Sand	18.4	331	15	569
Total	100	1800		4200
Binder (%)	52			
Recovered binder (%)	2.9	Theoretical maximum density = 2516 Kg/m^3		
		Mix density = 2365 Kg/m^3		
Binder content (%)	4.6	Total mass into mould 1300.33g		
Theoretical binder reqd (g)	237.1			
Actual binder reqd (g)	263.2	% Recovered binder in mix = 4.6-((263.2/6263.2)*100) = 0.397		

Table A.12 - Calculation of mix proportions

Basecourse Quarry 1				
Batch size (g)	**% Virgin**	**% RAP**	**Amount virgin Agg (g)**	**Amount RAP Agg (g)**
6000	65	35	3900	2100
RAP grading	**% RAP**	**Amount RAP (g)**	**Total Amt Agg required (g)**	**Amount virgin (g)**
28mm	0	0	0	0
20mm	4	84	24	1356
14mm	11	231	15	669
10mm	18.6	391	13	389
6mm	25.3	531	10	69
Dust	22.7	477	23	903
Sand	18.4	386	15	514
Total	100	2100		
Binder (%)	61			
Recovered binder (%)	2.9	Theoretical maximum density = 2516 Kg/m^3		
		Mix density = 2365 Kg/m^3		
Binder content (%)	4.6	Total mass into mould 1300.33g		
Theoretical binder reqd (g)	228.4			
Actual binder reqd (g)	**258.9**	% Recovered binder in mix = 4.6-((258.9/6258.9)*100) = 0.463		

Table A.13 - Calculation of mix proportions

Basecourse Quarry 1				
Batch size (g)	**% Virgin**	**% RAP**	**Amount virgin Agg (g)**	**Amount RAP Agg (g)**
6000	60	40	3600	2400
RAP grading	**% RAP**	**Amount RAP (g)**	**Total Amt Agg required (g)**	**Amount virgin (g)**
28mm	0	0	0	0
20mm	4	96	24	1344
14mm	11	264	15	636
10mm	18.6	446	13	334
6mm	25.3	607	10	-7
Dust	22.7	545	23	835
Sand	18.4	442	15	458
Total	100	2400		3500
Binder (%)	70			
Recovered binder (%)	2.9	Theoretical maximum density = 2516 Kg/m^3		
		Mix density = 2365 Kg/m^3		
Binder content (%)	4.6	Total mass into mould 1300.33g		
Theoretical binder reqd (g)	219.7			
Actual binder reqd (g)	**254.5**	% Recovered binder in mix = 4.6-((254.4/6254.5)*100) = 0.53		

Table A.14 - Calculation of mix proportions

Basecourse Quarry 1				
Batch size (g)	**% Virgin**	**% RAP**	**Amount virgin Agg (g)**	**Amount RAP Agg (g)**
6000	50	50	3000	3000
RAP grading	**% RAP**	**Amount RAP (g)**	**Total Amt Agg required (g)**	**Amount virgin (g)**
28mm	0	0	0	0
20mm	4	120	24	1320
14mm	11	330	15	570
10mm	18.6	558	13	222
6mm	25.3	759	10	-159
Dust	22.7	681	23	699
Sand	18.4	552	15	348
Total	100	3000		
Binder (%)	87			
Recovered binder (%)	2.9	Theoretical maximum density = 2516 Kg/m^3		
		Mix density = 2365 Kg/m^3		
Binder content (%)	4.6	Total mass into mould 1300.33g		
Theoretical binder reqd (g)	202.3			
Actual binder reqd (g)	**245.8**	% Recovered binder in mix = 4.6-((245.8/6245.8)*100) = 0.664		

Table A.15 - Calculation of aggregate required and theoretical density

30% HRA Quarry 1				
Calculation of maximum densities for bituminous mixtures				
	Density (kg/m^3)	**% AGG**	**Total % of mix**	**Amt of Agg reqd**
20mm	2700	0	0	0
14mm	2690	23	21.206	1380
10mm	2660	12	11.064	720
6mm	2760	0	0	0
CRF	2700	0	0	0
Filler	2510	60	5.532	360
Sand	2697	659	54.398	3540
Binder	1030		7.8	

Total aggregate content (%)	95.2	Binder Content (%)	7.8
Theoretical max density (kg/m^3)	**2382**		
Air voids %	**6**		
Mix density (kg/m^3)	**2238**		

Mass required to achieve density	
density =mass/volume	
Height required (mm)	70
Radius (mm)	50
Volume (mm^3)	549778.3
Mass (g)	1230.8

Total mass required into mould (g)	**1230.76**
Total binder required (%)	
Mass of aggregate batch (g)	6000
Mass of aggregate +binder (g)	6507.6
Total binder (g)	**507.6**

Table A.16 - Calculation of mix proportions

30% HRA Quarry 1				
Batch size (g)	**% Virgin**	**% RAP**	**Amount virgin Agg (g)**	**Amount RAP Agg (g)**
6000	95	5	5700	300
RAP grading	**% RAP**	**Amount RAP (g)**	**Total Amt Agg required (g)**	**Amount virgin (g)**
28mm	0	0	0	0
20mm	0	0	1380	1380
14mm	0	0	720	720
10mm	0	0	0	0
6mm	0	0	0	0
Filler	4	12	360	348
Sand	96	228	3540	3252
Total	100	300		5700
Binder (%)	14			
Recovered binder (%)	4.52	Theoretical maximum density = 2382 Kg/m^3		
		Mix density = 2238 Kg/m^3		
Binder content (%)	7.8	Total mass into mould 1230g		
Theoretical binder reqd (g)	494			
Actual binder reqd (g)	**500.8**	% Recovered binder in mix = 7.8-((500.8/6500.8)*100) = 0.096		

Table A.17 - Calculation of mix proportions

30% HRA Quarry 1				
Batch size (g)	**% Virgin**	**% RAP**	**Amount virgin Agg (g)**	**Amount RAP Agg (g)**
6000	90	10	5400	600
RAP grading	**% RAP**	**Amount RAP (g)**	**Total Amt Agg required (g)**	**Amount virgin (g)**
28mm	0	0	0	0
20mm	0	0	1380	1380
14mm	0	0	720	720
10mm	0	0	0	0
6mm	0	0	0	0
Filler	4	34	360	336
Sand	96	576	3540	2964
Total	100	600		5400
Binder (%)	27			
Recovered binder (%)	4.52	Theoretical maximum density = 2382 Kg/m^3		
		Mix density = 2238 Kg/m^3		
Binder content (%)	7.8	Total mass into mould 1230g		
Theoretical binder reqd (g)	480.5			
Actual binder reqd (g)	494	% Recovered binder in mix = 7.8-((494/6494)*100) = 0.192		

Table A.18 - Calculation of mix proportions

30% HRA Quarry 1				
Batch size (g)	**% Virgin**	**% RAP**	**Amount virgin Agg (g)**	**Amount RAP Agg (g)**
6000	85	15	5100	900
RAP grading	**% RAP**	**Amount RAP (g)**	**Total Amt Agg required (g)**	**Amount virgin (g)**
28mm	0	0	0	0
20mm	0	0	1380	1380
14mm	0	0	720	720
10mm	0	0	0	0
6mm	0	0	0	0
Filler	4	36	360	624
Sand	96	864	3540	2676
Total	100	900		5100
Binder (%)	41			
Recovered binder (%)	4.52	Theoretical maximum density = 2382 Kg/m^3		
		Mix density = 2238 Kg/m^3		
Binder content (%)	7.8	Total mass into mould 1230g		
Theoretical binder reqd (g)	466.9			
Actual binder reqd (g)	**487.3**	% Recovered binder in mix = 7.8-((487.3/6487.3)*100) = 0.29		

Table A.19 - Calculation of mix proportions

Basecourse Quarry 2				
Batch size (g)	**% Virgin**	**% RAP**	**Amount virgin Agg (g)**	**Amount RAP Agg (g)**
6000	90	10	5400	600
RAP grading	**% RAP**	**Amount RAP (g)**	**Total Amt Agg required (g)**	**Amount virgin (g)**
28mm	0	0	0	0
20mm	11	66	1440	1374
14mm	13.7	82	900	818
10mm	12.5	75	780	705
6mm	2.6	116	600	484
Dust	2.6	156	1380	1244
Sand	17.4	104	900	796
Total	100	600		5400
Binder (%)	13			
Recovered binder (%)	2.2	Theoretical maximum density = 2516 Kg/m^3		
		Mix density = 2365 Kg/m^3		
Binder content (%)	4.6	Total mass into mould 1300.33g		
Theoretical binder reqd (g)	276.1			
Actual binder reqd (g)	**282.8**	% Recovered binder in mix = 4.6-((282.7/6282.7)*100) = 0.10		

Table A.20 - Calculation of mix proportions

Basecourse Quarry 2				
Batch size (g)	**% Virgin**	**% RAP**	**Amount virgin Agg (g)**	**Amount RAP Agg (g)**
6000	85	15	5100	900
RAP grading	**% RAP**	**Amount RAP (g)**	**Total Amt Agg required (g)**	**Amount virgin (g)**
28mm	0	0	0	0
20mm	11	99	1440	1341
14mm	13.7	123	900	777
10mm	12.5	113	780	668
6mm	2.6	175	600	425
Dust	2.6	234	1380	1146
Sand	17.4	157	900	743
Total	100	900		5100
Binder (%)	20			
Recovered binder (%)	2.2	Theoretical maximum density = 2516 Kg/m^3		
		Mix density = 2365 Kg/m^3		
Binder content (%)	4.6	Total mass into mould 1300.33g		
Theoretical binder reqd (g)	269.5			
Actual binder reqd (g)	**279.5**	% Recovered binder in mix = 4.6-((279.5/6279.5)*100) = 0.149		

Table A.21 - Calculation of mix proportions

Basecourse Quarry 2				
Batch size (g)	**% Virgin**	**% RAP**	**Amount virgin Agg (g)**	**Amount RAP Agg (g)**
6000	80	20	4800	1200
RAP grading	**% RAP**	**Amount RAP (g)**	**Total Amt Agg required (g)**	**Amount virgin (g)**
28mm	0	0	0	0
20mm	11	132	1440	1308
14mm	13.7	164	900	736
10mm	12.5	150	780	630
6mm	2.6	233	600	367
Dust	2.6	312	1380	1068
Sand	17.4	209	900	691
Total	100	1200		4800
Binder (%)	26			
Recovered binder (%)	2.2	Theoretical maximum density = 2516 Kg/m^3		
		Mix density = 2365 Kg/m^3		
Binder content (%)	4.6	Total mass into mould 1300.33g		
Theoretical binder reqd (g)	2629			
Actual binder reqd (g)	**276.1**	% Recovered binder in mix = 4.6-((276.1/6276.1)*100) = 0.20		

Table A.22 - Calculation of mix proportions

Basecourse Quarry 2				
Batch size (g)	**% Virgin**	**% RAP**	**Amount virgin Agg (g)**	**Amount RAP Agg (g)**
6000	75	25	4500	1500
RAP grading	**% RAP**	**Amount RAP (g)**	**Total Amt Agg required (g)**	**Amount virgin (g)**
28mm	0	0	0	0
20mm	11	165	1440	1275
14mm	13.7	206	900	695
10mm	12.5	188	780	593
6mm	2.6	291	600	309
Dust	2.6	390	1380	990
Sand	17.4	261	900	639
Total	100	1500		4500
Binder (%)	33			
Recovered binder (%)	2.2	Theoretical maximum density = 2516 Kg/m^3		
		Mix density = 2365 Kg/m^3		
Binder content (%)	4.6	Total mass into mould 1300.33g		
Theoretical binder reqd (g)	256.3			
Actual binder reqd (g)	**272.8**	% Recovered binder in mix = 4.6-((272.8/6272.8)*100) = 0.251		

Table A.23 - Calculation of mix proportions

Basecourse Quarry 2				
Batch size (g)	**% Virgin**	**% RAP**	**Amount virgin Agg (g)**	**Amount RAP Agg (g)**
6000	70	30	4200	1800
RAP grading	**% RAP**	**Amount RAP (g)**	**Total Amt Agg required (g)**	**Amount virgin (g)**
28mm	0	0	0	0
20mm	11	198	1440	1242
14mm	13.7	247	900	653
10mm	12.5	225	780	555
6mm	2.6	349	600	251
Dust	2.6	468	1380	912
Sand	17.4	313	900	587
Total	100	1800		4200
Binder (%)	40			
Recovered binder (%)	2.2	Theoretical maximum density = 2516 Kg/m^3		
		Mix density = 2365 Kg/m^3		
Binder content (%)	4.6	Total mass into mould 1300.33g		
Theoretical binder reqd (g)	249.7			
Actual binder reqd (g)	**269.5**	% Recovered binder in mix = 4.6-((269.5/6269.5)*100) = 0.301		

Table A.24 - Calculation of mix proportions

Basecourse Quarry 2				
Batch size (g)	**% Virgin**	**% RAP**	**Amount virgin Agg (g)**	**Amount RAP Agg (g)**
6000	65	35	3900	2100
RAP grading	**% RAP**	**Amount RAP (g)**	**Total Amt Agg required (g)**	**Amount virgin (g)**
28mm	0	0	0	0
20mm	11	231	1440	1209
14mm	13.7	288	900	612
10mm	12.5	263	780	518
6mm	2.6	407	600	193
Dust	2.6	546	1380	834
Sand	17.4	365	900	535
Total	100	2100		
Binder (%)	46			
Recovered binder (%)	2.2	Theoretical maximum density = 2516 Kg/m^3		
		Mix density = 2365 Kg/m^3		
Binder content (%)	4.6	Total mass into mould 1300.33g		
Theoretical binder reqd (g)	243.1			
Actual binder reqd (g)	**266.2**	% Recovered binder in mix = 4.6-((266.2/6266.2)*100) = 0.351		

Table A.25 - Calculation of mix proportions

Basecourse Quarry 2				
Batch size (g)	**% Virgin**	**% RAP**	**Amount virgin Agg (g)**	**Amount RAP Agg (g)**
6000	60	40	3600	2400
RAP grading	**% RAP**	**Amount RAP (g)**	**Total Amt Agg required (g)**	**Amount virgin (g)**
28mm	0	0	0	0
20mm	11	264	1440	1176
14mm	13.7	329	900	571
10mm	12.5	300	780	480
6mm	2.6	466	600	134
Dust	2.6	624	1380	756
Sand	17.4	418	900	482
Total	100	2400		3600
Binder (%)	53			
Recovered binder (%)	2.2	Theoretical maximum density = 2516 Kg/m^3		
		Mix density = 2365 Kg/m^3		
Binder content (%)	4.6	Total mass into mould 1300.33g		
Theoretical binder reqd (g)	236.5			
Actual binder reqd (g)	**262.9**	% Recovered binder in mix = 4.6-((262.9/6262.9)*100) = 0.402		

Table A.26 - Calculation of mix proportions

Basecourse Quarry 2				
Batch size (g)	**% Virgin**	**% RAP**	**Amount virgin Agg (g)**	**Amount RAP Agg (g)**
6000	50	50	3000	3000
RAP grading	**% RAP**	**Amount RAP (g)**	**Total Amt Agg required (g)**	**Amount virgin (g)**
28mm	0	0	0	0
20mm	11	330	1440	1110
14mm	13.7	411	900	489
10mm	12.5	375	780	405
6mm	2.6	582	600	18
Dust	2.6	780	1380	600
Sand	17.4	522	900	378
Total	100	3000		3000
Binder (%)	66			
Recovered binder (%)	2.2	Theoretical maximum density = 2516 Kg/m^3		
		Mix density = 2365 Kg/m^3		
Binder content (%)	4.6	Total mass into mould 1300.33g		
Theoretical binder reqd (g)	223.3			
Actual binder reqd (g)	**256.3**	% Recovered binder in mix = 4.6-((256.3/6256.3)*100) = 0.503		