The University of Nottingham

Department of Civil Engineering

AN INVESTIGATION INTO COLD IN-PLACE RECYCLING OF ASPHALT PAVEMENTS

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Thesis submitted to the University of Nottingham
for the degree of Doctor of Philosophy

January 2009

ABSTRACT

Cold in-place recycling of existing pavement materials has become an increasingly important activity for the maintenance of highways around the world. The technique uses a specialist machine to pulverise the existing deteriorated road and to inject a stabilising agent to improve the structural properties of the recycled pavement materials before being re-compacted into a new stabilised road base or subbase course. It is strongly believed that this technique will play an important role in pavement maintenance and rehabilitation in the near future.

As the use of cold in-place recycling techniques increases, research into material characterisation of these cold recycled mixtures is required. The research described in this thesis attempts to contribute toward this field by conducting an investigation into the mechanical properties and performance of cold recycled asphalt pavement materials for the purpose of utilisation as road base courses.

In this research, recycled asphalt pavement materials stabilised by two common types of binder, foamed bitumen and cement, were studied. Recycled asphalt pavement materials were manufactured in the laboratory by mixing reclaimed asphalt pavement with new crushed limestone and treated with stabilising agents.

Foamed bitumen bound recycled pavement materials were evaluated for their deformation behaviour in a repeated load triaxial apparatus. Cement stabilised mixtures were subjected to flexural testing to assess their flexural characteristics under repeated loading. The Nottingham Asphalt Tester was used to evaluate the effect of cementitious additives on the mechanical properties of foamed bitumen stabilised mixtures. A pilot-scale trial of recycled asphalt pavement base course was constructed in the laboratory. The Nottingham Pavement Test Facility was used to more realistically traffic the various trial recycled pavement sections using a loaded wheel.

The results from the research will contribute to a better understanding of the deterioration behaviour of recycled asphalt pavement materials and may be useful for applying to the design of pavements incorporating this type of material.

ACKNOWLEDGEMENTS

The author received a scholarship from the Royal Thai Government and was given permission from the Department of Highways to pursue this research study at the University of Nottingham. The completion of the research would not have been possible without generous contributions from several people and organisations, to whom the author wishes to present his obligation.

First of all, the author owes a great debt to Dr Nicholas Thom for his excellent supervision of this project. The author is also indebted to Prof Stephen Brown for his valuable advice at the beginning of the research.

Most of the study is based on laboratory experiments. The author would like to extend his appreciation to Dr Salah Zoorob for his useful suggestions on experimental works and to Barry Brodrick for his expertise in instrumentation and equipment related issues. The author also appreciates the effort from technicians, particularly, Mick Winfield, Rich Blakemore, Neil Parkes, Kevin Gilbert, Chris Fox, Jon Watson, Richard Meehan, Nancy Hodge, and Rob Cragg for their assistance with the laboratory tests.

Manfield Asphalt, Dene Quarry and Shell Bitumen provided the materials used in this project. Their support is kindly acknowledged.

During the course of the three-year research, the author has been aided and had many enjoyable times given by other researchers. The author would like to thank them for their kindness and friendship, in alphabetical order: Phil Boothroyd, Lelio Brito, Dr James Grenfell, Dr Riccardo Isola, Dr Joe Kwan, Dr Ted Liao, Davide LoPresti, Muhammad Mubaraki, Dr Joel Oliveira, Elisa Salighini, Xiaoyi Shi, Muslich Sutanto, Pierpaolo Viola, Jed Wu, and Junwei Wu. Special thanks are due to Dr Nono Sunarjono for fruitful discussions on common research interests.

In addition, the author is particularly grateful to his family, relatives and friends, who have always provided the author with a lot of encouragement.

Finally, the author would like to express his extreme gratitude to his mother for her unending support, unstinting sacrifice, and unconditional love throughout the duration of this study.

DECLARATION

The work described in this thesis was conducted at the Nottingham Transportation Engineering Centre, Department of Civil Engineering, University of Nottingham between October 2004 and March 2008. I declare that the work is my own and has not been submitted for a degree of another university.

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Nottingham

January 2009

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1 INTRODUCTION

1.1 Preamble

Roads are valuable assets as they are the primary means of communication and transportation. The major structural load-carrying elements of roads are the pavements. Once roads are open to traffic, pavements gradually deteriorate with time for a number of reasons. Without appropriate maintenance, these pavements inevitably reach the end of their useful lives. After that time, they are typically rutted, cracked, contain numerous other distress types and are rough to drive on. Because the worse the pavement condition the greater the cost of remedial measures, it is important to take action to maintain the road at as high a condition as possible. Certainly, pavements are not inexpensive parts of the transportation infrastructure. With the relatively large investment involved in pavements, even marginal improvement in their condition may effect a large saving in expenditure, both directly in capital and maintenance costs and indirectly to the road users. In the last few decades, therefore, pavement maintenance and rehabilitation have become a major problem facing highway agencies around the world and is becoming an increasingly crucial activity. In fact, nowadays in many countries, road authorities are paying more attention to maintenance and rehabilitation than to developing new roads.

When a pavement is significantly out of shape and showing signs of structural weakness, there are many maintenance, rehabilitation, and reconstruction alternatives available for upgrading it. Overlaying an asphalt layer on the existing surface is one of the simplest solutions. Because it is very easy, fast and relatively cheap and usually rides well and looks good, a hot mix asphalt overlay is commonly used to rehabilitate almost everything, regardless of the pavement condition or types of failure. However, it is only suitable for problems that are limited to the upper part of the pavement such as surface cracking in the asphalt layer.

In principle, a pavement has to be strong and/or thick enough to spread the repeated heavy loads from vehicle axles to a weak subgrade. Therefore, in order to design or rehabilitate a road, the available options would involve strengthening the pavement structure or increasing the total depth of the pavement. An increase in layer thickness is a preferred option for construction of a new road or rehabilitation projects where the upgrading exercise requires considerable changes to the geometry of the road. Otherwise, repeated increase in pavement elevation, such as a thick asphalt overlay, normally reduces the width of the road and an increase in pavement elevation often creates drainage and access problems. In this case, increasing the strength of existing layers by stabilisation using a Cold In–place Recycling technique may provide a solution.

In-place stabilisation is a process whereby the load-carrying capacity and/or stability of the existing pavement materials are improved to enhance the performance of the pavement and increase its useful life. Stabilisation comes in many forms but usually involves the inclusion of an additive to the existing pavement. Cold in–place recycling stabilised with cement and/or bitumen will strengthen the pavement structure. The additional thickness of the new pavement will be only 50 – 100 mm of surface overlay to provide a smooth surface. Not only does pavement recycling save natural resources but it also keeps road geometry the same as much as possible so that drainage and access problems are minimised.

Cold in-place recycling has been established practice in the UK for more than 30 years on minor roads, housing estate roads etc. [Whiteoak, 2000]. It is now an accepted road rehabilitation process in many countries worldwide. The process can be used to improve and rehabilitate various road pavements ranging from low volume unsurfaced roads to main highways carrying heavy traffic.

1.2 Recycling of asphalt pavements.

The concept of recycling or reuse of existing bituminous pavement materials into a new pavement layer has long been recognised because of economic, energy and environmental conservation reasons. The recycling of bituminous pavements can be divided into two broad categories namely, hot recycling and cold recycling.

Hot recycling. Hot recycling is a process to rehabilitate asphalt pavement surface distress by softening the existing surface with heat, mechanically removing the pavement surface, mixing with a modifier agent, adding new asphalt or aggregate if required, and replacing it on the pavement [Button et al, 1994]. Hot recycling can be carried out in-place or in-plant. The in-plant process typially uses up to 30 % of recycled asphalt pavement from any layer. For hot in-place recycling, the process normally treats existing pavement to a depth of about 25 – 50 mm but can be up to 80 mm. The hot in-place recycling process is suitable for highways with slight to moderate non-structural surface distresses.

Cold recycling. Cold recycling is a process that consists of combining, without heating existing pavement materials, stabilising agents, and new aggregate (if required) to produce a new material that is expected to satisfy the specification for its use. Cold recycled materials are most commonly used for road base although they can be employed as subbase and surface courses as well [Epps, 1990]. The process can be performed either in-place or in-plant. The cold in-place recycling process may need a new asphalt surface course to be applied. This project focuses on cold in-place recycling only.

1.3 Cold in-place recycling

With regard to structural maintenance of highway pavements, the term 'cold in-place recycling' refers to the procedures using specialist plant to pulverise and stabilise existing road materials, in-place, at ambient temperature, with the addition of hydraulic cement and/or bitumen binders [Milton and Earland, 1999].

1.3.1 Cold in-place recycling process

The cold in-place recycling process consists of a series of procedures. In principle, the existing asphalt layer is milled and mixed with a predetermined amount of underlying base or subbase. An additive or additives together with water are blended with the pulverised materials. After mixing with additive and water, the pulverised material is then shaped and compacted. The end product is an improved road base. Once the recycled layer has been completed, it should be paved with a bituminous layer. The thickness of surface layer ranges from thin surface treatment for lightly traffic roads, to one or more layers of hot mixed asphalt courses if the road carries heavy traffic.

In the United States, portland cement and bitumen emulsion and the combination of these two additives are used to produce reclaimed mixtures [Mallick et al, 2002a]. In Europe and Canada, portland cement and bitumen emulsion are also used in combination to produce reclaimed mixtures with high early strength and increased resistance to water damage [Needham and Brown, 1996]. Recently, foamed asphalt has gained popularity as another type of bituminous stabilising agent [Moore, 2004]. Other types of additives can be utilised such as hydrated lime [Cross, 1999], fly ash [Cross and Fager, 1995] and calcium chloride [Pickett, 1991].

1.3.2 Cold in-place recycling machine

In order to accomplish the pulverisation and mixing tasks, it is necessary to employ a powerful road reclaiming machine. The cold recycling process is carried out by a specially designed machine capable of recycling a thick pavement layer in a single pass. At the centre of these machines is a milling and mixing drum equipped with a large number of hardened steel cutting bits. The drum rotates, milling the materials in the existing road pavement, as shown diagrammatically in Figure 1.1 [Wirtgen, 2001]

During the milling process, water from a tanker attached to the recycler is delivered through a hose and is sprayed into the recycler’s mixing chamber. The water, which is measured accurately using a microprocessor controlled feeding system, is mixed thoroughly together with the milled materials to achieve optimum moisture content for compaction.

Fluid stabilising additive such as cement-water slurry or bitumen emulsion, either individually or in combination, can be introduced directly into the mixing chamber in a similar manner to the water. Cement can also be applied dry by spreading onto the existing road surface ahead of the milling machine. The machine passes over the dry cement mixing it, together with water, into the milled pavement. Foamed bitumen can be injected into the mixing chamber through a separate specially designed spray bar.

After recycling and mixing, the material receives an initial pass from a roller to consolidate it. Then it is graded with a motor grader before being compacted using a vibratory roller.

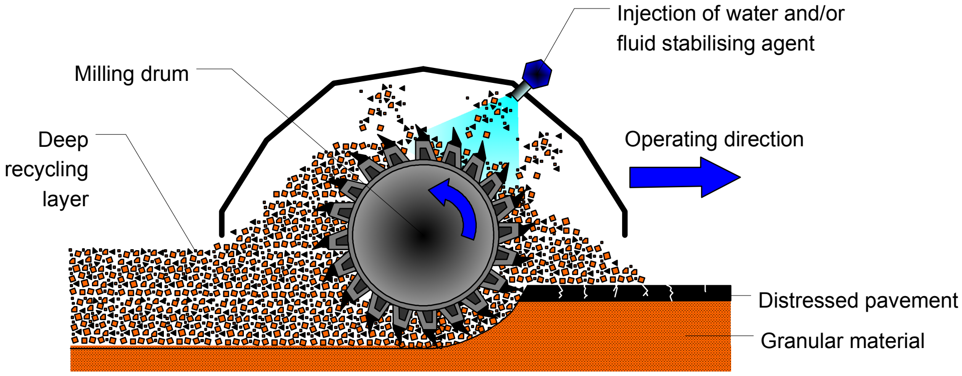


Figure 1.1 Milling/mixing drum and spray bar systems [Wirtgen, 2001]

Recycling trains may be configured differently, depending on the recycling application and the type of stabilising agent that is used. Some of them have the capacity for emulsion and cement stabilisation and others have the added capacity for in-place foamed bitumen stabilisation. Typical recycling trains are shown in Figure 1.2 [Lewis and Collings, 1999].

The train in Figure 1.2(a) is used when carrying out cold recycling with cement alone. The recycler is attached to the slurry mixer ahead of it. The slurry in the mixer is transferred through a hose and then injected into the recycler mixing chamber. When bitumen emulsion is used alone the configuration of the train is as shown in Figure 1.2(b). Figure 1.2(c) shows the typical trains for use with foamed bitumen. When only foamed bitumen is used, the recycler is attached to two tankers, one filled with hot bitumen and another one with water. When foamed bitumen is used with cement, there will be a tanker filled with bitumen and a water tanker ahead of the slurry mixer as shown in Figure 1.2(d).

There are many cold recycling machines available on the market from many manufacturers such as Caterpillar RM-350, CMI RS-500B or RS-650, Wirtgen WR-2500 and Hamm Raco 550 etc. [Kearney and Huffman, 1999]. Figure 1.3 shows a photograph of a recycling train in operation.

1.3.3 Applications of cold in-place recycling

Three common applications of cold in-place recycling are as follows. [Lewis and Collings, 1999, Wirtgen , 2001]

Recycling of asphalt layers. This pavement recycling technique can rehabilitate pavement distress caused by aging of asphalt layers such as minor cracking and surface degradation that occurs only in the asphalt surface layer without severe rutting. This application of the recycling technique is usually performed in asphalt surface course to depths ranging from 80 mm to 150 mm. This technique is so called thin layer recycling and is normally carried out as a means of improving surface riding quality rather than structural capacity.

Structural strengthening of pavements. Modern recycling machines are capable of recycling through a flexible or composite pavement consisting of thick layers from 150 mm to more than 300 mm in a single pass. With this ability, the full depth of an existing deteriorated asphalt layer together with underlying unbound materials can be recycled and stabilised. Thus pavements whose weakness occurs in deep layers, which would normally be associated with the presence of severe deformation and cracking, can be strengthened. This application is known as deep recycling. The use of heavy vibratory rollers is necessary to ensure proper compaction of thick recycled layers. The structural capacity of the pavement will be significantly improved. Once the recycling has been carried out, it is normal to pave one or more layers of hot-mixed asphalt on top of the recycled layer to provide a smooth surface and/or further strengthen the pavement.

Upgrading unsurfaced roads. The cold in-place recycling process can be used to upgrade unsurfaced roads by recycling existing granular roads with cement, bitumen emulsion or foamed bitumen and applying a thin surfacing such as a chip seal or slurry seal. This type of recycling is normally carried out to a depth of between 100 to 150 mm.

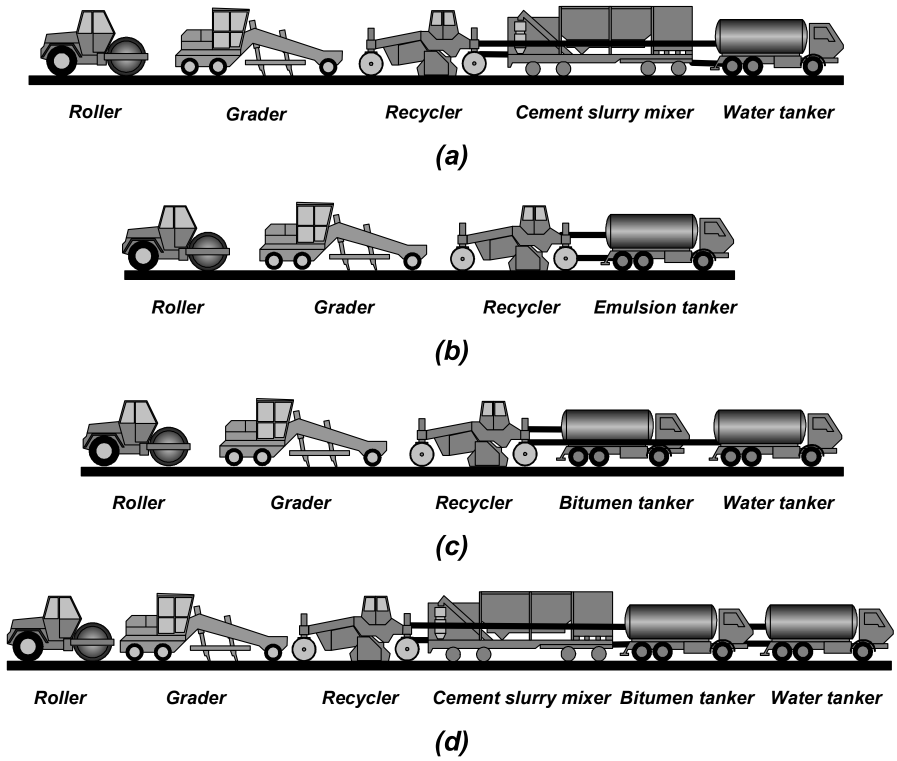


Figure 1.2 Typical recycling trains [Lewis and Collings, 1999] (a) cement (b) bitumen emulsion (c) foamed bitumen (d) combination of foamed bitumen and cement

Figure 1.3 A recycler and a slurry mixer in operation.

1.4 Advantages and disadvantages of cold in-place recycling technique

Cold in-place recycled pavement exhibits several advantages. Some benefits which are often cited in the literature [e.g., Epps, 1990, Kearney and Huffman, 1999, Forsberg et al, 2002, Mallick et al, 2002a, 2002b] include:

1. Cold in–place recycling conserves resources. As with other recycled materials, recycled pavement reuses the existing pavement materials (aggregate and asphalt). The new imported materials in the process, if any, are very little. Thus the limited resources are conserved.

2. Cold in–place recycling conserves energy. As its name implies, cold in-place recycling is a cold process and no heating is needed. Thus, the required energy is minimised. It is completely in place and on grade, so trucking and other material hauling are eliminated or greatly reduced.

3. The use of cold recycling is environmentally friendly. Disposal of old pavement materials is greatly reduced. There is less air pollution because there is no heating or material hauling.

4. Cold recycling produces thick, bound layers that are of much higher stiffness and strength than a conventional granular base. Recycled material is also more durable. It has been reported [e.g. Mallick et al, 2002a, 2002b] that the addition of cement, lime or asphalt to the reclaimed materials appears to improve the durability of the mix.

5. Cold in-place recycling appears effective in mitigating cracking. Unlike base layers of high plasticity soil stabilised with cement where the cracking problem is one of the major disadvantages, it has been reported that roads with a recycled base layer are in good condition after about 10 years in service with little cracking of the asphalt surface [Jahren et al, 1998].

6. Cold in-place recycling is less susceptible to shrinkage cracking compared with typical soil–cement material.

7. The geometry of the pavement is maintained. Crown and cross slope can be restored so that a uniform overlay can be applied.

8. Cold in-place recycling is a relatively quick rehabilitation process. Nowadays, recycling machines are capable of high production rates that significantly shorten construction times compared to alternative rehabilitation methods.

9. A high level of traffic safety can be achieved during the recycling process. Because only single lane closure is required, the road users' inconvenience is therefore minimised.

10. Cold In–place recycling is long–term cost effective. In addition maintenance costs are reduced.

There are also a few disadvantages to the cold in-place recycling technique reported in the literature. Some of them are:

1. This technique must be conducted in warm, dry weather and thus construction is limited to the summer season [Kazmierowski et al, 1992].

2. Cold recycled materials stabilised with some types of additives are susceptible to stripping and ravelling if they are open to direct traffic [Mallick et al, 2002b]. The surface of recycled layer also lacks smoothness. It is recommended to pave a wearing surface such as a thin hot mix overlay or surface treatment on the recycled base course.

3. In practice, the in-place recycling process may be subject to potential problems including: variability of aggregate grading, non-uniformity of distribution and mixing of binding agent, variability of moisture content, variable layer thickness and difficult compaction at the bottom of thick layer construction [Milton and Earland, 1999]. Therefore, the process requires careful inspection to avoid non-uniformity of the materials.

4. It is commonly known that all cement treated materials tend to crack due to shrinkage. However, cracking can be minimised by careful mix design, keeping cement content and moisture content as low as possible.

However, these disadvantages are not major drawbacks. The advantages combine to make cold recycling an attractive process for pavement rehabilitation compared with other conventional rehabilitation strategies.

1.5 Materials in cold in-place recycling

The materials when recycling existing asphalt pavements normally include three main components. They are: (1) reclaimed asphalt pavement, (2) aggregate substructure, and (3) stabilising agent or binder.

Reclaimed Asphalt Pavement Reclaimed Asphalt Pavement (RAP) is an asphalt pavement that has been recovered, usually by milling, and can be used in part or as a whole in a recycled pavement by mixing it with aggregate or bitumen, cement, lime or other materials. In the cold recycling process, RAP is produced by pulverising existing asphalt surface course by a powerful milling machine. The use of RAP as an alternative pavement material is gaining attention throughout the world because of conservation of limited resources as well as a reduction in funds available for highway construction.

Aggregate substructure In many occasions in cold in-place recycling, the entire asphalt pavement layer is pulverised together with a predetermined amount of underlying material and then treated to produce a new stabilised road base. Therefore, the aggregate in recycled pavement material is typically existing base material. However, if the in-place material is not sufficient to provide the desired depth of the treated layer, new aggregate can be spread over the road surface and included in the process. The other reasons for adding aggregate include correcting gradation and providing for acceptance of additional binder. [Epps, 1990]

Stabilising agents Portland cement or hydrated lime have been used as a primary stabilising agent in the majority of cold in-place recycling works. More recently, a variety of alternative binding agents have been used, including bitumen emulsion and foamed bitumen. Portland cement and lime can also be used as additives in combination with bituminous primary binders.

1.6 Stabilising agents

Nowadays, stabilising agents are used in pavement construction worldwide to overcome the shortage of natural resources. By adding a small amount of stabilising agent at a relatively low cost, the required strength can be obtained from a local marginal material [Williams, 1986]. Besides providing an increase in strength, stabilising agents are often able to improve durability as well as the water resistance of the pavement [Mallick et al 2002a, 2002b, Tarefder et al, 2005]. Similarly, stabilising agent can be utilised to improve the materials recovered from existing pavements. Therefore in many cases there is no need to import new materials to produce stronger layers in the rehabilitated pavement structure.

Stabilisation agents can be categorised broadly into two types – chemical, and bituminous [Kearney and Huffman, 1999]. Chemical stabilisation uses additives such as Portland cement, fly ash, or hydrated lime as stabilising agents. In the case of bituminous stabilisation, foamed bitumen and asphalt emulsion are usually used as the bituminous stabilising agents.

A wide range of stabilising agents is available with different advantages and disadvantages. However, all stabilising agents have the same objectives of binding material particles together and increasing strength and/or improving durability.

1.6.1 Cementitious stabilising agent

Cement, lime, and a combination of these materials with fly ash, ground granulated blast furnace slag and other such materials are the most common cementitious stabilised materials.

Cement, which is usually a type I Portland cement, and water are mixed together in precise and predetermined quantities to form cement slurry. The slurry is mixed and stored in a slurry mixer in the recycling train and is injected into the recycler’s mixing chamber. Alternatively, dry cement can be spread as a powder ahead of the recycling machine although this may cause some dust problems at the construction site.

The amount of cement required in a stabilising process differs from one road to another and from country to country. These differences are governed by standards, specifications and desired functions of the roads. Normally, strength is the criteria for cement content determination. A cement content is normally specified that results in an unconfined compressive strength (UCS), after some curing period, of a certain value.

Lime ($Ca(OH)_2$) and quick lime (CaO) are also effective economical stabilising agents for fine grained soil with high plasticity [Williams, 1986]. Like cement, lime and quick lime can be applied as dry powder or slurry.

1.6.2 Bituminous stabilising agent

The use of bitumen as a stabilising agent is popular due to advancement in the technology. The bituminous stabilising agents are normally applied in the forms of emulsion or foam. Bitumen bound materials produce a flexible layer with superior fatigue properties compared to those treated with cementitious stabilising agent [Kearney and Huffman, 1999].

Bitumen emulsion is sprayed into the mixing chamber where it is blended with the milled materials. An emulsion stabilised base course is flexible, fatigue resistant and not likely to crack. However, it is usually more expensive than cement and it takes time to cure and develop its full strength [Mallick et al, 2002b]. When in-place moisture content is high, addition of asphalt emulsion can increase moisture content above optimum, resulting in reduced layer strength [Kearney and Huffman, 1999].

Foamed bitumen is produced by injecting air and water droplets under a high pressure (5 bars) into hot (160 – 180 $^{\circ}C$) liquid bitumen, resulting in bitumen taking the form of foam as schematically illustrated in Figure 1.4 [Wirtgen, 2002]. The volume of bitumen will increase while viscosity considerably reduces. As to with other fluid stabilising agents, foamed bitumen is then transferred through nozzles and mixed with pulverised materials in the mixing chamber.

Foamed asphalt stabilisation is currently attractive because equipment manufacturers now provide improved foaming systems [Moore, 2004]. It is also usually less expensive than bitumen emulsion [Moore, 2004].

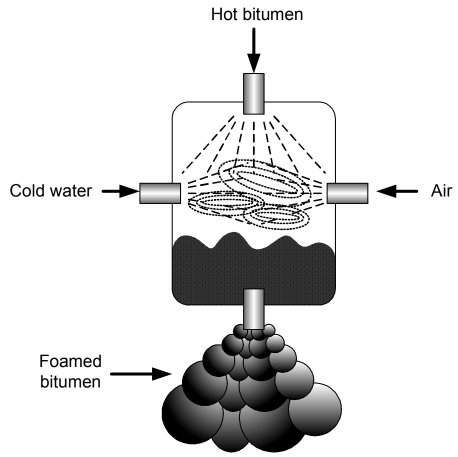


Figure 1.4 Foamed bitumen production [Wirtgen, 2002]

1.6.3 Combination of cementitious and bituminous stabilising agents

In many cases a combination of bituminous and cementitious stabilising agents is found to be effective [e.g., Tarefder et al, 2005, Mallick et al, 2002a, 2002b]. Foamed bitumen and bitumen emulsion will benefit from adding a small percentage of cementitious agent such as cement or lime. Although it is likely to be more expensive than using bituminous or cementitious agent alone, the combination of bituminous and cementitious agents has advantages as follows:

- Adhesion between bitumen and aggregate is improved [Mallick et al, 2002b]
- The strength of the recycled layer increases more rapidly, enabling the road to be opened to traffic sooner. [Forsberg et al, 2002, Mallick et al, 2002b]
- Strength of the recycled material is improved. [Cross and Fager, 1995]
- Performance of recycled materials (resistance to deformation, rutting and moisture) is improved. [Cross, 1999, Mallick et al, 2002a, 2002b, Tarefder et al, 2005]

Cementitious agent can easily be introduced into the recycled mixture together with bituminous binder by using the methods described earlier.

1.6.4 Selection of stabilising agents

The choice of most effective stabilising agent for a particular application depends on several factors, the principal of which are:

- Price. The relative costs of the stabilising agent;
- Availability. The ease with which the stabilising agent can be resourced;
- Acceptability. Certain stabilising agents are often more acceptable locally;
- Material type. Different stabilising agents are more effective with certain types of aggregate than others.

Table 1.1 summarises the advantages and disadvantages of the three most commonly used stabilising agents in the cold in-place recycling process.

Table 1.1 Advantages and disadvantages of three types of stabilising agent.

Cement	
Advantages	**Disadvantages**
- Normally relatively inexpensive compared to bitumen. - Cement is available worldwide. - Cement is well known in the construction industry. - Cement can be spread by hand in the absence of spreaders and slurry mixers. - Improves compressive strength properties of most materials. - Improves materials' resistance to water damage	- Shrinkage cracking is unavoidable, but can be controlled. - Increase in rigidity but decrease in fatigue resistance. - Requires proper curing before being open to traffic.
Bitumen emulsion	
Advantages	**Disadvantages**
- Stabilising with bitumen creates a visco-elastic material with good fatigue characteristics - The recycler is coupled to the bulk tanker for application through a spray bar. - Bitumen emulsion is relatively well known in the construction industry.	- Bitumen emulsion is not normally produced on site. Production requires strict quality control. Emulsifying agents are expensive. Water also needs transportation not only bitumen. - Moisture content of material in existing pavement is sometimes too high and becomes saturated when emulsion is added. - Curing can take long time. Strength development is dictated by moisture loss. The required formulation for recycling work may not be obtainable.
Foamed bitumen	
Advantages	**Disadvantages**
- Like bitumen emulsion, foamed bitumen can be applied through a special spray bar after coupling the bulk supply tanker. - Foamed bitumen stabilised material has a flexible mortar bonding the coarser aggregates. Therefore, it exhibits good resistance to fatigue. - Foamed bitumen uses standard penetration grade bitumen. There is no manufacturing cost. - Construction cost can be reduced since foamed bitumen allows use of waste and local material. - Binder can be applied in relatively low amounts. - Material can be trafficked immediately after placing.	- Foaming process requires heating of the bitumen, which requires special heating facilities and safety precautions. - The quality of the stabilised material is determined by the foaming characteristics which are primarily dependent on the quality of bitumen. - Saturated material and material deficient in fines cannot be treated with foamed bitumen.

1.7 Summary

The main purpose of pavement recycling is to reuse all or a portion of the existing pavement material for pavement rehabilitation. Hot recycling and cold recycling are viable alternatives for rehabilitation of asphalt pavements. Cold recycling may be more attractive than hot recycling as the latter involves more energy. The cold in-place recycling technique as a pavement rehabilitation measure has become increasingly popular over the past decades mainly because of its significant financial and environmental benefits which have been mentioned earlier in this chapter. In the process, the pavement layers are milled to the required depth, mixed with selected stabilising agents, and compacted. Asphalt pavements with moderately deformed profile and with moderate to severely cracked surface are generally suitable for applying this technique.

There would appear to be much promise in the in-place cold recycling process due to its significant financial and environmental benefits. It is believed that this type of technique will play a key role in pavement maintenance and rehabilitation in the future.

1.8 Objectives and scope of this research

The increased use of the cold in-place recycling technique prompts research into the materials characteristics and performance of these cold recycled materials for an accepted analytical design and analysis method of pavements incorporating this type of material. However, there are many factors that influence the performance of cold recycled mixtures including aggregate origin, aggregate properties, type of binder, binder content, moisture content, density, among others [Jenkins, 2000]. Because of the diversity of the materials, an effort to develop a methodology for predicting the performance of such mixtures analytically is, therefore, highly challenging.

This study did not attempt to develop any complicated performance prediction model for recycled pavement base course materials because of the reason indicated above. The main objective of the research is to assess the mechanical properties of recycled asphalt pavement materials for use as base course materials. It is also believed that the results from the research will provide the information for better understanding of the deterioration behaviour of recycled asphalt pavement materials. In the end, the study attempts to show that the findings gained on the behaviour of the recycled pavement materials may be successfully applied to the design of pavements incorporating this type of material.

To achieve the above-mentioned goals, the research is divided into three main tasks, including:

- Investigating the mechanical properties of recycled asphalt pavement materials. Addressing the effect of proportion of granular and recycled asphalt materials and different binders.
- Addressing the behaviour of large scale recycled pavements.
- Implementing the findings into a pavement design and management system.

Throughout this research, the materials were fabricated by mixing and compacting recycled pavement materials in the laboratory to simulate the recycling process in the field. The recycled pavement materials consist of reclaimed asphalt pavement (RAP) and limestone aggregate. The binding agents include cement, foamed bitumen and foamed bitumen plus a small addition of cement or lime.

All experiments and material preparation were performed in the laboratory. A Hobart mixer was used for mixing the mixtures. Wirtgen WLB-10 laboratory foaming machine was used to produce foamed bitumen. Compaction of the mixtures was achieved by a gyratory compactor, roller compactor, Marshall hammer or Wacker vibrating plate compactor. The main test equipments for mechanical properties investigation were, Nottingham Asphalt Tester (NAT), Repeated load triaxial apparatus, Four-point bending apparatus, and the Nottingham Pavement Test Facility (PTF). The first three apparatuses were used to investigate the material properties at an element scale while the last equipment was employed to study the material behaviour more realistically at a larger scale.

1.9 Thesis outline

This thesis is organised into nine chapters. Chapter 1 forms an introduction, giving the general background of the cold in-place recycle technique, stating objectives and scope of the research.

Chapter 2 reviews the literature concerning cold recycled asphalt pavements particularly in three main areas including, (1) mix design and properties of cold recycled materials, (2) structural design of cold recycled pavements and (3) performance of cold in-place recycled roads.

Chapter 3 is used to introduce all the materials used in this investigation, namely reclaimed asphalt pavement, limestone aggregate, foamed bitumen and cement. The basic characteristics of each material are presented in this chapter. The mix design results that optimise the properties and content of binder for the mixtures are also provided.

An investigation into some technical aspects regarding the deformation of foamed bitumen stabilised recycled asphalt pavements is the main topic of Chapter 4. This chapter reports an experimental study of foamed bitumen stabilised materials subjected to a form of loading by the repeated load triaxial test, which simulates traffic loading.

Chapter 5 presents an investigation of mechanical properties, with emphasis on stiffness modulus and fatigue characteristic, of cement bound recycled asphalt pavement materials. A four-point bending apparatus manufactured at the University of Nottingham was used as the investigating tool.

The performance of a range of stabilised recycled asphalt pavement mixes under realistic trafficking conditions with moving wheel load was examined. The Nottingham Pavement Test Facility was used as the main investigating tool. Chapter 6 reports this investigation.

Chapter 7 presents a study of the effects of cementitious additives on the mechanical properties of foamed bitumen stabilised asphalt pavement materials, as it was found, from the previous chapter, that cement has a beneficial effect on foamed bitumen bound mixtures.

Chapter 8 describes the suitability of cold recycled asphalt pavement materials to be utilised as pavement materials. This chapter also shows how to apply the investigation results from previous chapters to the problems of pavement analysis.

Lastly, conclusions derived from this research and recommendations for future work in this field are presented in Chapter 9.

2 COLD IN-PLACE RECYCLED ASPHALT PAVEMENTS

2.1 Introduction

Cold in-place recycling technique is a promising option of rehabilitating roads. The developing interest in pavement recycling has been growing worldwide. Nowadays, recycling of existing pavement materials has become an increasingly important activity of the maintenance of highways. The increased use of cold in-place recycling technique prompts research into the materials characteristic and performance of these materials. Many researchers have investigated cold in-place asphalt recycling technology, in three main areas, namely: (1) Mix design and properties of cold recycled materials, (2) Structural design of cold in-place recycled pavements, and (3) Performance of cold in-place recycled roads. Some of the research conducted in these field as well as other related topics are reviewed in this chapter.

2.2 Mix design and properties of cold in-place recycled materials

So far, the mix design procedures of recycled material have been developed by a number of agencies for their own purposes. There is no standard method for mix design available. However, the basic steps typically for the mix design process can be outlined as follow: [Epps, 1990]

- Obtaining representative samples from field or from stockpiles of reclaimed materials for use in mix design.
- Processing of samples for use in mix design.
- Evaluation of recycled pavement,
    - Asphalt content
    - Asphalt physical properties (penetration, viscosity)
    - Aggregate gradation
    - Recycled pavement gradation
- Selection of amount and gradation of new aggregate.
- Selection of asphalt required.
- Selection of type and amount of stabilising agent.

- Mixture, compaction, and testing of trial mixture
- Establishment of job mix formula.
- Adjustment in the field.

2.2.1 Sample processing

The materials involved in a mix design of cold in-place recycled mixes include: (1) RAP, (2) aggregate, and (3) stabilising agent or binder. Test samples for the mix design should be representative, for both grading and shape, of the pulverised pavement [Milton and Earland, 1999]. Ideally, pulverised field samples should be obtained. However, actual pulverised material is not usually available during the pre-construction investigation; the mix design can rely on pavement cores or bulk samples which are crushed in the laboratory. After obtaining the representative samples from the field, the reclaimed asphalt aggregate is blended with reclaimed aggregate materials and/or virgin aggregate.

2.2.2 Evaluation of RAP

Emery (1993) suggested that moisture content, gradation of RAP, asphalt binder content including penetration should be determined. According to the Asphalt Institute cold-mix recycling design method [AI, 1983] this information is used for determining the quantity of new bitumen to be added to the bituminous bound recycled mixtures.

2.2.3 Material gradation

Milton and Earland (1999) recommended two grading envelopes for pulverised materials used for cold in-place recycled mixes, namely: Zone A and Zone B, as shown in table 2.1. Particle size distribution of pulverised materials should preferably conform to Zone A grading envelope or, in certain circumstances, within Zone B provided that results of the mix design show that acceptable recycled materials can be produced [Milton and Earland, 1999].

Table 2.1 Recommended particle size distribution of granular material for cold in-place recycling [Milton and Earland, 1999].

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

Normally, cement bound recycled materials are not affected by aggregate gradation. On the other hand, bitumen bound materials are highly sensitive to reclaimed material grading, especially the proportion of fine particles [Milton and Earland, 1999, Wirtgen , 2002]. Additional new aggregate may be needed to accommodate the binding agent as well as to correct gradation or to increase the thickness of the recycled pavement [Wood et al, 1988].

2.2.4 Binder selection

A variety of alternative binding agents can be used in a cold in-place recycling project. In the United States, Portland cement and bitumen emulsion and the combination of these two additives are used to produce reclaimed mixtures [Mallick et al, 2002a]. In Europe and Canada, Portland cement and bitumen emulsion are also used in combination to produce reclaimed mixtures [Needham and Brown, 1996]. In the United Kingdom, the primary binder agents are Portland cement and foamed bitumen [Milton and Earland, 1999]. A flow chart outlining a process of selection between these two stabilising agents is shown in Figure 2.1 [Milton and Earland, 1999].

2.2.5 Job mix formula

Normally, selection of the final binder content for cold in-place recycled mixes is based on the determination of compressive strength for cement treated materials and on Marshall testing or indirect tensile strength for bituminous bound materials.

Because of the similarity between the materials, the mix design procedure for cement bound materials is normally applied for cement treated recycled materials. The trial mixes are compacted at optimum moisture content. Then, the unconfined compressive strength at a specific curing period (typically 7 days) is used to determine the cement content [Kennedy and Clark, 1990].

In the case of bitumen bound mixes, Wood et al (1988) conducted a survey regarding the mix design procedure for cold in-place recycling treated with bitumen. The survey revealed that many highway agencies in the United States have used the Marshall procedure to compact samples for cold in-place recycling mix design. The compactive effort and curing procedure are different from agency to agency. Strength and plastic flow values are measured by Marshall testing. Density, stability and air voids are frequently used to establish the optimum binder content. The Hveem resilient modulus and indirect tension test were also used by other agencies [Wood et al, 1988]. The detailed descriptions of Marshall, Hveem and indirect tensile testing procedures are described elsewhere [e.g., Hunter, 2000].

Recently, some researchers [e.g., Mallick et al, 2002a, Cross, 2003] utilised a gyratory compactor as a compaction tool for cold in-place recycling mix design.

The mix design procedures for foamed bitumen stabilised mixtures are available in the following documents.

- *Foamed Asphalt Mixes: Mix Design Procedure* [Muthen, 1999],
- *Wirtgen Cold Recycling Manual* [Wirtgen, 2001], and
- *Developing of a Mix Design Process for Cold-in-place Rehabilitation Using Foamed Asphalt* [Lee and Kim, 2003]

All of which select the optimum binder content based on the indirect tensile strength of compacted foamed bitumen samples over a range of binder contents under soaked condition. However, Muthen (1999) suggested other tests such as resilient modulus and dynamic creep tests in order to verify the selected optimum mix and to ensure adequate performance of the mix.

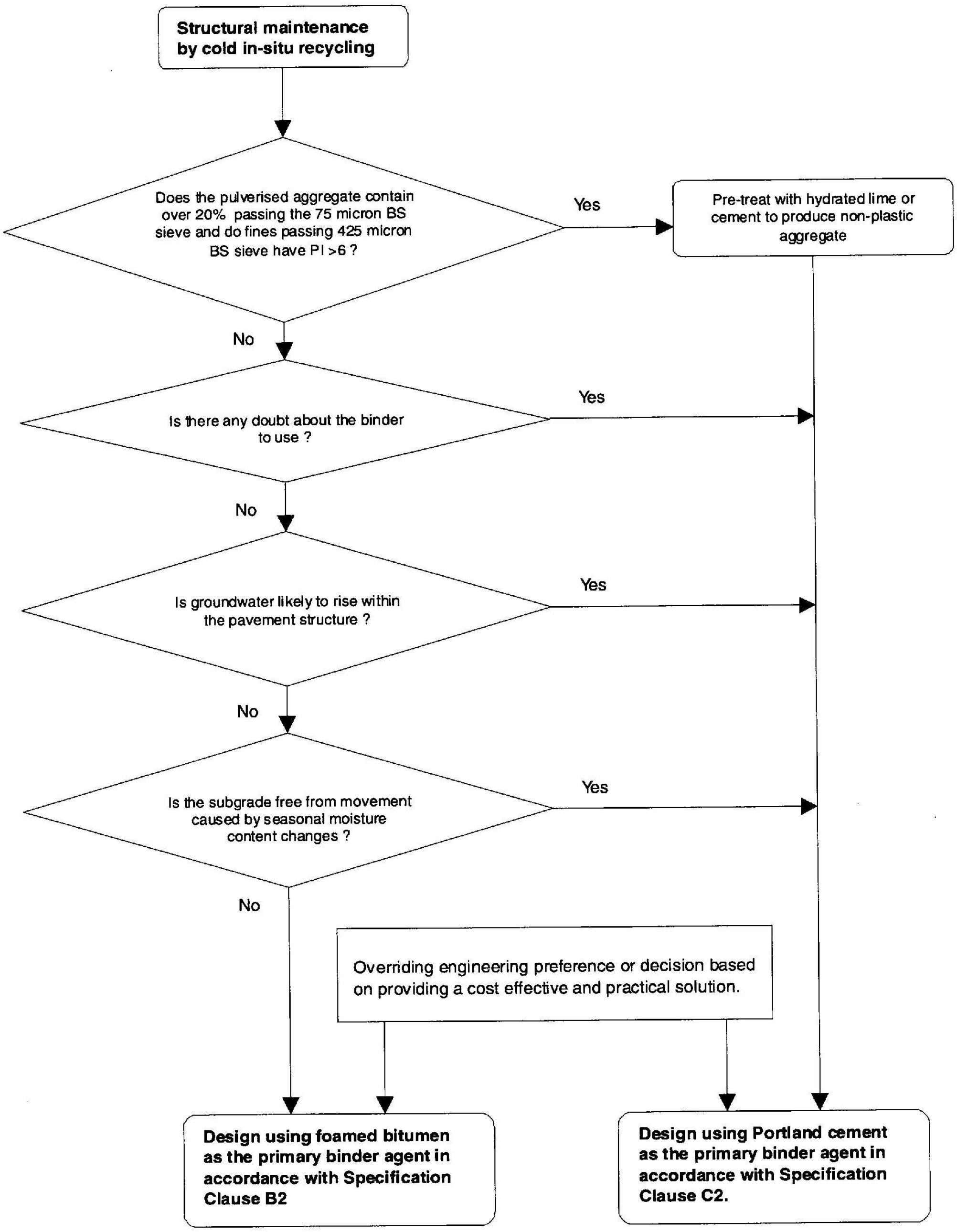


Figure 2.1 Binder agent selection [Milton and Earland, 1999]

The mix design procedure for foam bitumen bound mixtures suggested in the *Wirtgen Cold Recycling Manual* [Wirtgen, 2001] is briefly described as follows. The first step in the mix design procedure is to optimise the asphalt-to-water ratio of the binder. Foam bitumen expands to its maximum volume and then completely dissipates. This character can be defined in terms of expansion ratio and half-life. Expansion ratio is the maximum foam volume divided by the volume of bitumen after the foam has completely dissipated. Another term is half-life which is the time that foam takes to collapse to half of its maximum volume. Large expansion ratios are achieved by sacrificing the half-life and vice versa. At a given temperature, there is an optimum amount of added water that maximises the expansion ratio and half-life of the foam bitumen. Figure 2.2 shows a theoretical plot of moisture content versus expansion ratio and half-life for the optimisation of moisture at a given temperature. The optimum foam bitumen content in the mixes can be determined by standard Marshall or indirect tensile test at the optimum moisture content.

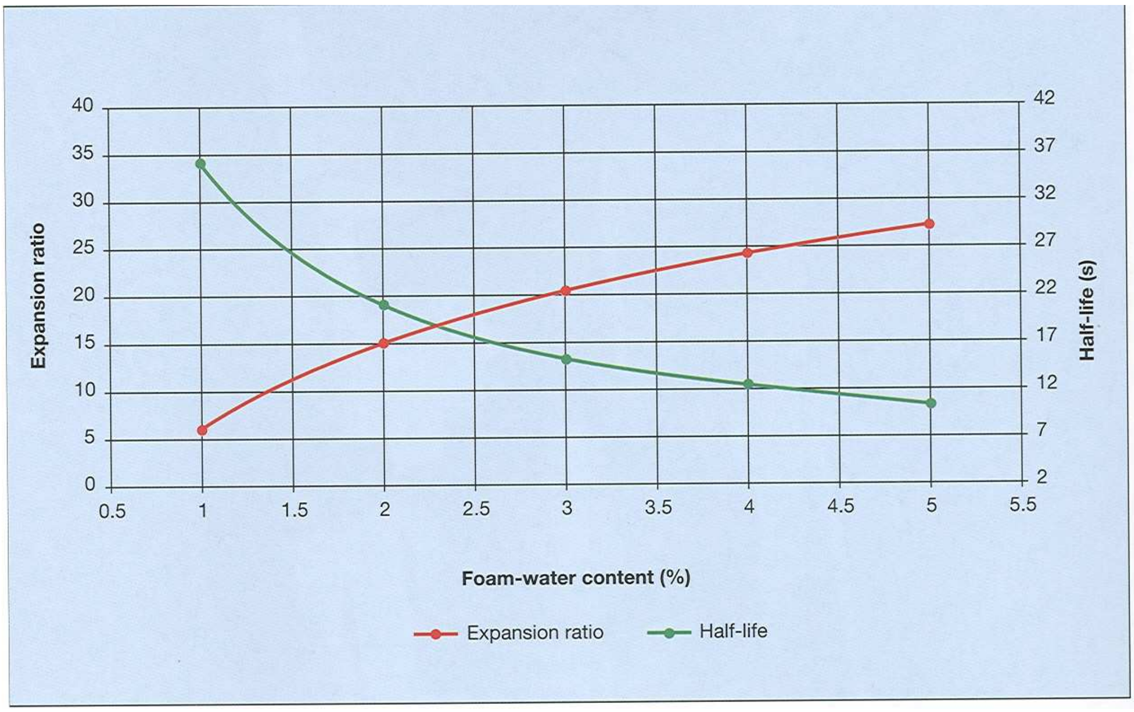


Figure 2.2 Half-life and expansion ratio versus foam-water content [Wirtgen, 2002]

2.3 Properties of cold recycled mixtures

The most important engineering properties in the mix design of cold in-place recycled materials include: Compressive strength – for cement bound mixes, and indirect tensile strength or Marshall properties – for bitumen bound mixes. Additional properties, such as resilient modulus, fatigue resistance and water resistance are also important for both cementitious and bituminous bound materials.

The basic engineering property data on cold recycled materials from selected studies are presented below.

2.3.1 Cement stabilised recycled asphalt pavement materials

<u>Compressive strength</u>

Taha et al (1999) investigated the possibility of using RAP in road construction. They reported data on laboratory CBR values of RAP and virgin aggregate blends. The results are shown in Figure 2.3. A low CBR value was found for high percentage of RAP aggregate. As the amount of virgin aggregate in the mix increases, the CBR value increases. This can be attributed to the better load transfer mechanism and better interlocking between virgin aggregate and the slip surface of the bitumen-coated particles of the RAP. Later, Taha et al (2002) introduced cement into the RAP and virgin aggregate blends. The mixtures were subjected to an unconfined compressive strength (UCS) test. Figure 2.4 shows the relationship between compressive strength of various mixes and curing time. It can be seen that the unconfined compressive strength typically increases as the curing periods and/or cement content increase. Increasing virgin aggregate also increases strength.

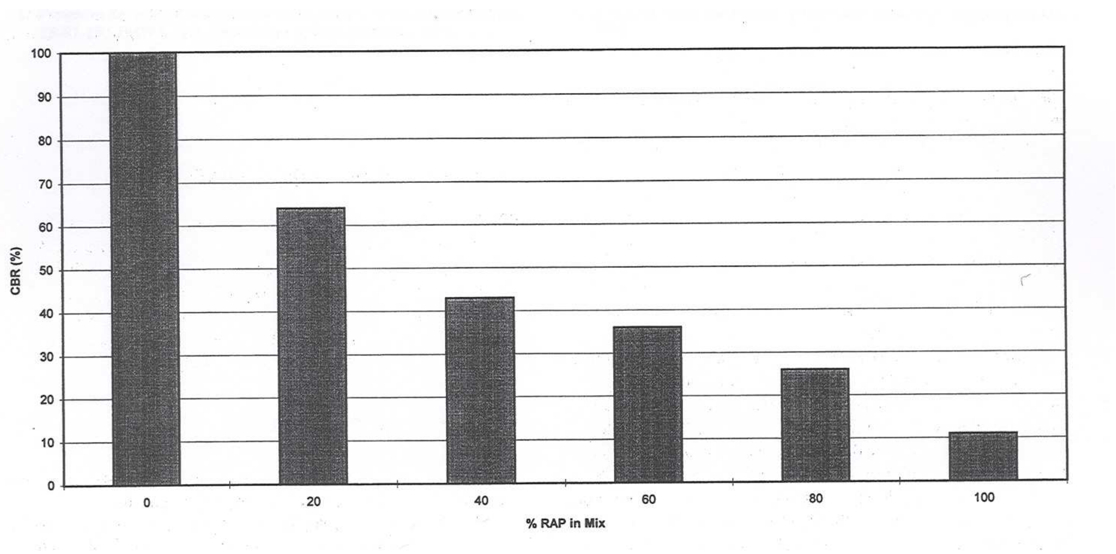


Figure 2.3 CBR values of various RAP and virgin aggregate blends [Taha et al, 1999]

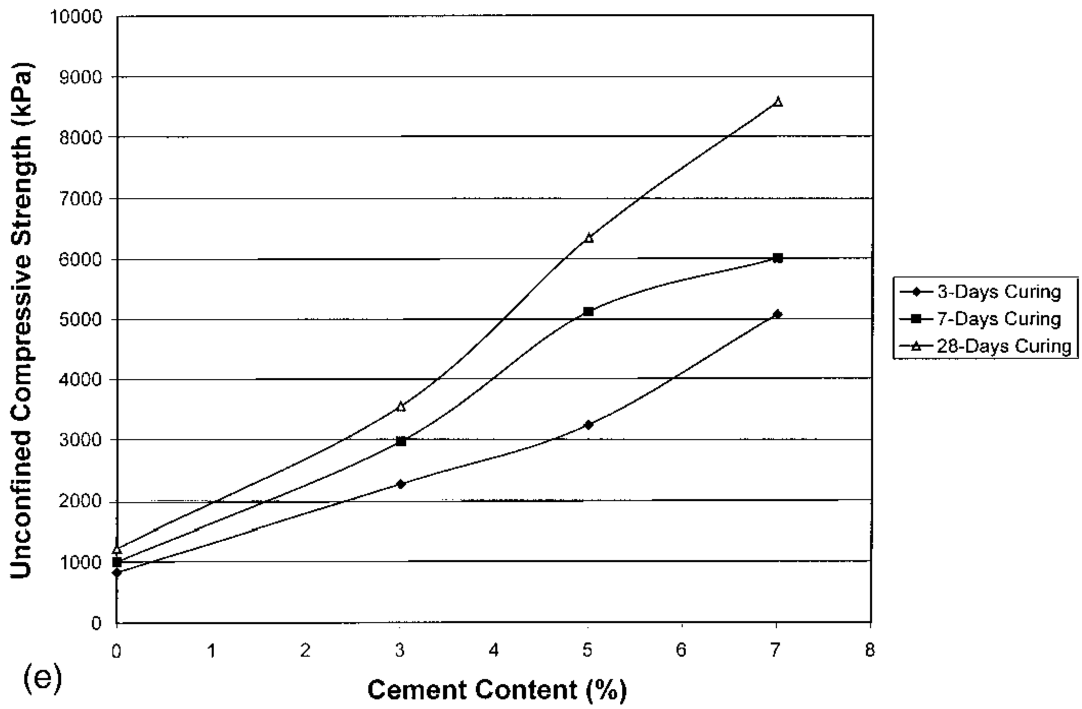


Figure 2.4 Typical unconfined compressive strength results of cold in-place cement bound material [Taha et al, 2002]

2.3.2 Bituminous stabilised recycled asphalt pavement materials

Marshall stability and flow

DeFoe (1977)† reported the relationships among Marshall properties, curing time, soaking, binder type, and binder content of bituminous bound cold recycled samples as shown in figure 2.5. Generally, stability increases with curing time. Soaking the mixes for 24 hours causes a loss in strength until sufficient curing time is obtained. Usually, a minimum of one week curing time is required for proper strength gain and water resistance [Epps, 1990].

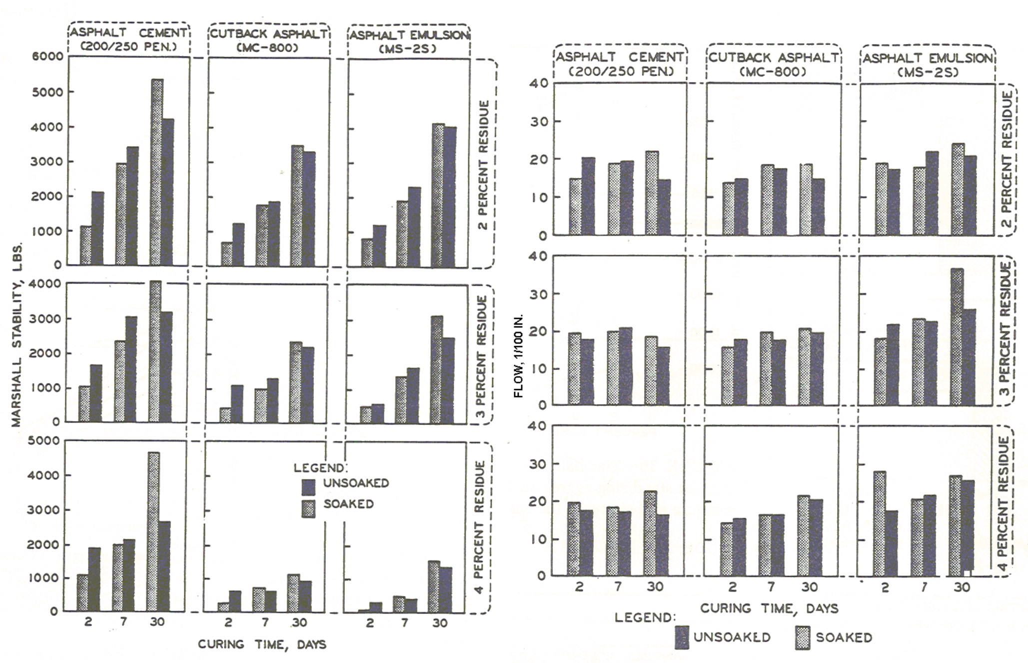


Figure 2.5 Relationship of Marshall stability and flow versus curing time for soaked and unsoaked bitumen emulsion bound cold recycled samples [DeFoe, 1977]†

Cohen et al (1989) conducted the Marshall stability test on cores taken from bitumen emulsion treated recycled roads. The relation between the field Marshall stability and the design value (retained Marshall stability) is plotted with time as presented in Figure 2.6. The results show that initial retained Marshall stability is relatively low but increases rapidly after 100 days. After that the values become steady with time. The rapid gain in strength is attributed to the loss of water and solvent from the emulsified

† See Epps (1990)

agent. In this case, the recycled layer passed from the "breaking and fluxing" to the "fluxing" condition after approximately 100 days [Cohen et al, 1989].

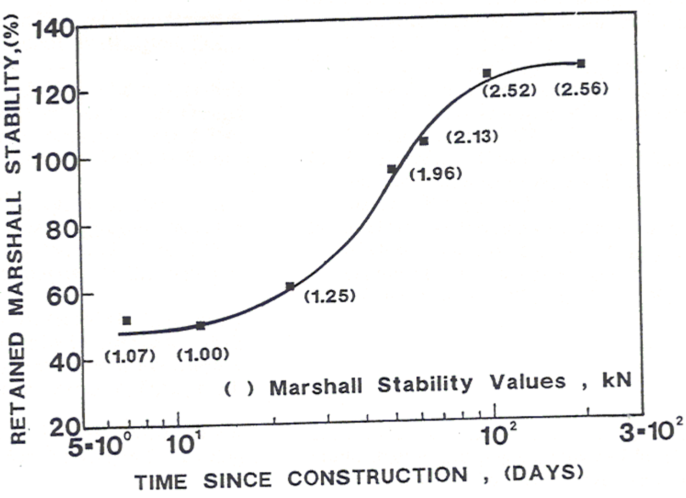


Figure 2.6 Retained Marshall stability versus time since construction [Cohen et al, 1989]

Indirect tensile strength

Table 2.2 shows the mix design results of RAP and virgin crushed stone in various proportions, treated with foam bitumen [Ruenkrairergsa et al, 2004]. The results indicate that both dry and soaked indirect tensile strengths decrease as the amount of RAP in the mixes increases. However, as the proportion of RAP increases, the moisture resistance of the mix is improved as indicated by retained ITS.

Table 2.2 Properties of foam bitumen mixtures during mix design [Ruenkrairergsa et al, 2004]

Mix proportion (Crushed stone : RAP)	20 : 80	50 : 50	100 : 0
Optimum foam content (%)	2.1	2.6	3.4
Unsoaked ITS (kPa)	453	465	540
Soaked ITS (kPa)	366	348	390
Retained ITS (%)	80	75	72
Dry density (Mg/m^3)	2.188	2.190	2.210

Muhammad et al (2003) presented the indirect tensile strength results of dry and soaked samples treated with foam bitumen in various proportions and an addition of 1.5% Portland cement as shown in Figure 2.7. The strength of soaked samples is generally higher than that of dry samples. They explained that the increase in wet strength is due to the addition of Portland cement.

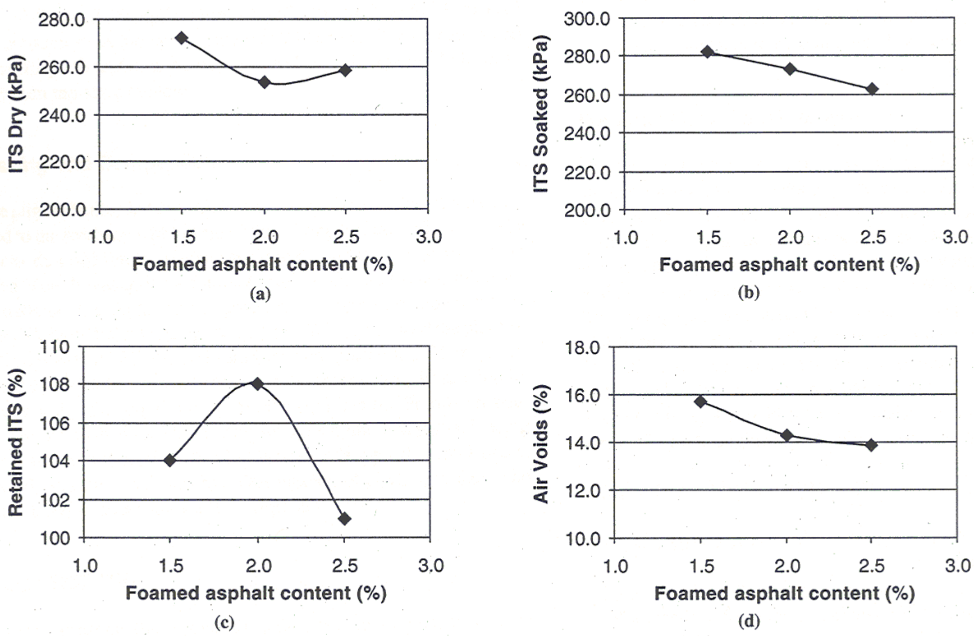


Figure 2.7 Indirect tensile strength (ITS) test results: Foam bitumen versus (a) ITS dry, (b) ITS soaked, (c) retained ITS, and (d) air void [Mohammad et al, 2003]

Stiffness modulus

Cross and Fager (1995) determined the stiffness modulus of cold in-place recycled mixes stabilised with bitumen emulsion. The resilient modulus tests were conducted on laboratory prepared cold recycled RAP with increasing bitumen emulsion (HFMS-1). The results are shown in Figure 2.8. It can be seen that as the amount of bitumen emulsion increases the resilient modulus of the mixes decreases.

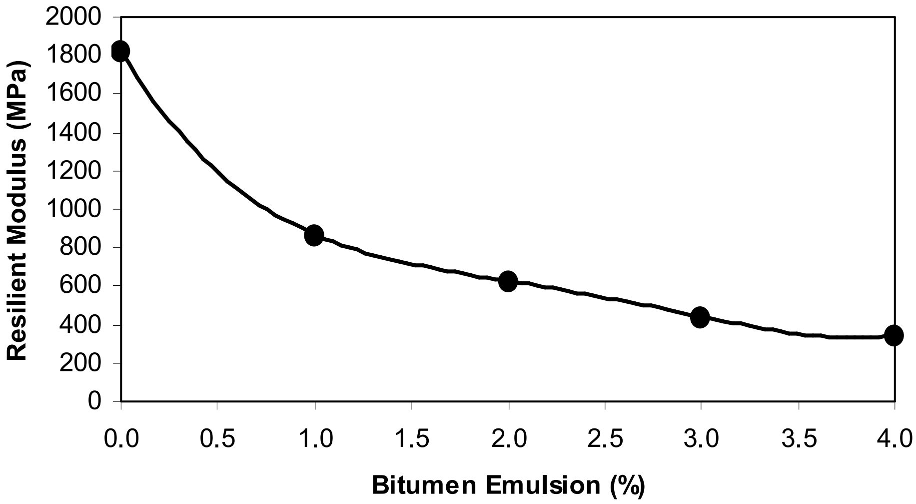


Figure 2.8 Relationship of resilient modulus and percentage of bitumen emulsion of cold recycled mixes [Cross and Fager, 1995]

Ruenkrairergsa et al (2004) found quite a surprising result that foam bitumen mixtures containing 50% RAP and 50% virgin crushed stone have higher resilient modulus than the mixes containing 0% and 80% RAP. This result is shown in Figure 2.9. The figure also shows the effect of temperature on the resilient modulus of the mixes. The resilient modulus of the mixes decreases as the temperature increases. The results also indicate that the mix containing 80% RAP shows marginally the largest decrease in resilient modulus when temperature increases.

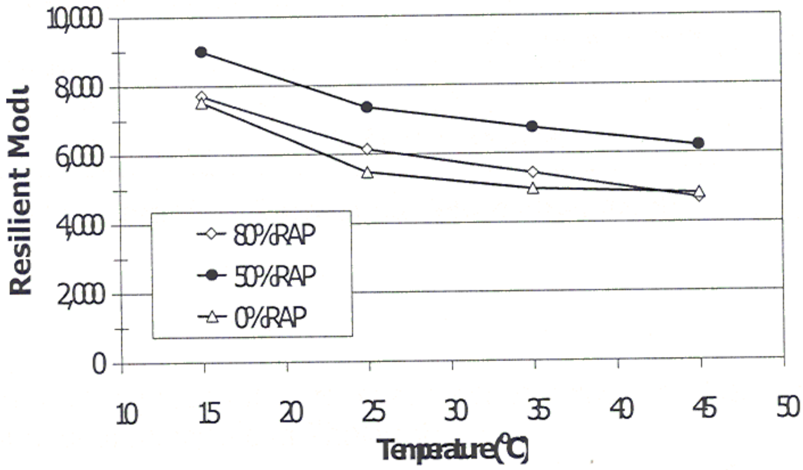


Figure 2.9 Resilient modulus of foam bitumen mixes at various temperatures [Ruenkrairergsa et al, 2004]

Figure 2.10 shows a relationship between resilient modulus from triaxial test and indirect tensile strength of cold recycled mixtures treated with foam bitumen [Rodrigues et al, 2003].

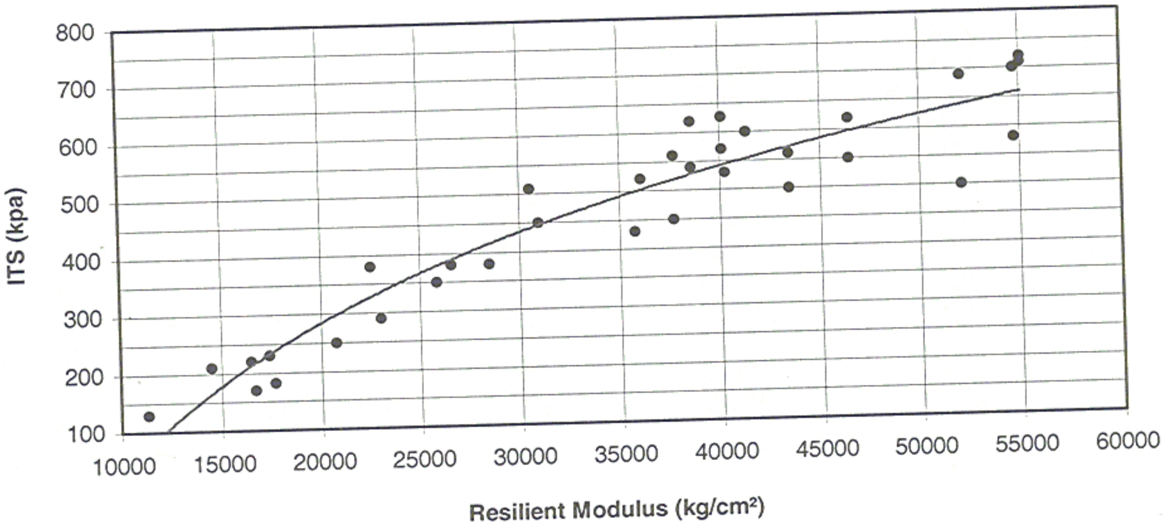


Figure 2.10 Indirect tensile strength versus resilient modulus of foam bitumen cold recycled mixes [Rodrigues et al, 2003]

Fatigue cracking resistance

Fatigue resistance data of foam bitumen mixes prepared in the laboratory has been reported by Ruenkrairergsa et al (2004). Three blends of RAP and virgin crushed stone at various proportions were treated with foam bitumen. The mixtures were subjected to indirect tensile fatigue test at a tensile stress of 250 kPa at 25 °C until failure (resilient modulus = 50% of initial resilient modulus). The results are shown in Figure 2.11. The mixes containing 80% RAP and 20% virgin crushed stone show somewhat poorer fatigue resistance compared with other mixes which have a lower RAP proportion. However, the fatigue properties of the mixes containing 50% and 0% RAP are similar. This indicates that RAP can replace virgin aggregate up to approximately 50% of the mix proportion without loss of fatigue properties.

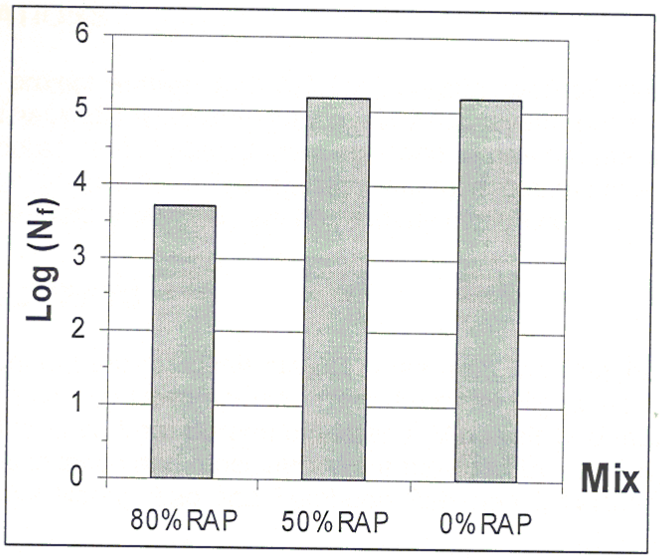


Figure 2.11 Fatigue resistance of foam bitumen cold recycled materials [Ruenkrairergsa et al, 2004]

Moisture resistance

Mallick et al (2002a) evaluated the effect of moisture susceptibility of cold in-place recycled mixes. In their study, the specimens of cold recycled mixes treated with bitumen emulsion, emulsion with lime, emulsion with cement and cement were subjected to indirect tensile strength tests under dry and wet conditions and the tensile strength ratios between wet and dry were determined. The results are shown in Table 2.3.

Table 2.3 Result of moisture susceptibility test for cold recycled mixes [Mallick et al, 2002]

Binder	Condition	Tensile strength (kPa)	Tensile strength ratio
Emulsion	Wet	56.2	0.20
	Dry	274.3	
Emulsion + lime	Wet	189.1	0.72
	Dry	263.0	
Emulsion + cement	Wet	79.9	0.39
	Dry	199.9	
Cement	Wet	61.7	0.97
	Dry	63.3	

The higher the strength ratio indicates the better moisture resistance of that mix. In this case, the cement samples show the highest strength ratio followed by the sample of emulsion with lime. It can be seen that addition of cementitious stabilising agent into bituminous agent improves water resistance of the mixtures.

2.4 Structural design of cold in-place recycled pavement

As with the mix design, there is still no widely accepted method for structural design of cold recycled pavement available. In practice, the required thickness of recycled layers is often determined by conventional methods for ordinary pavements. Some pavement design procedures that can be applied to the pavements with cold recycled layers are reviewed in the following sections.

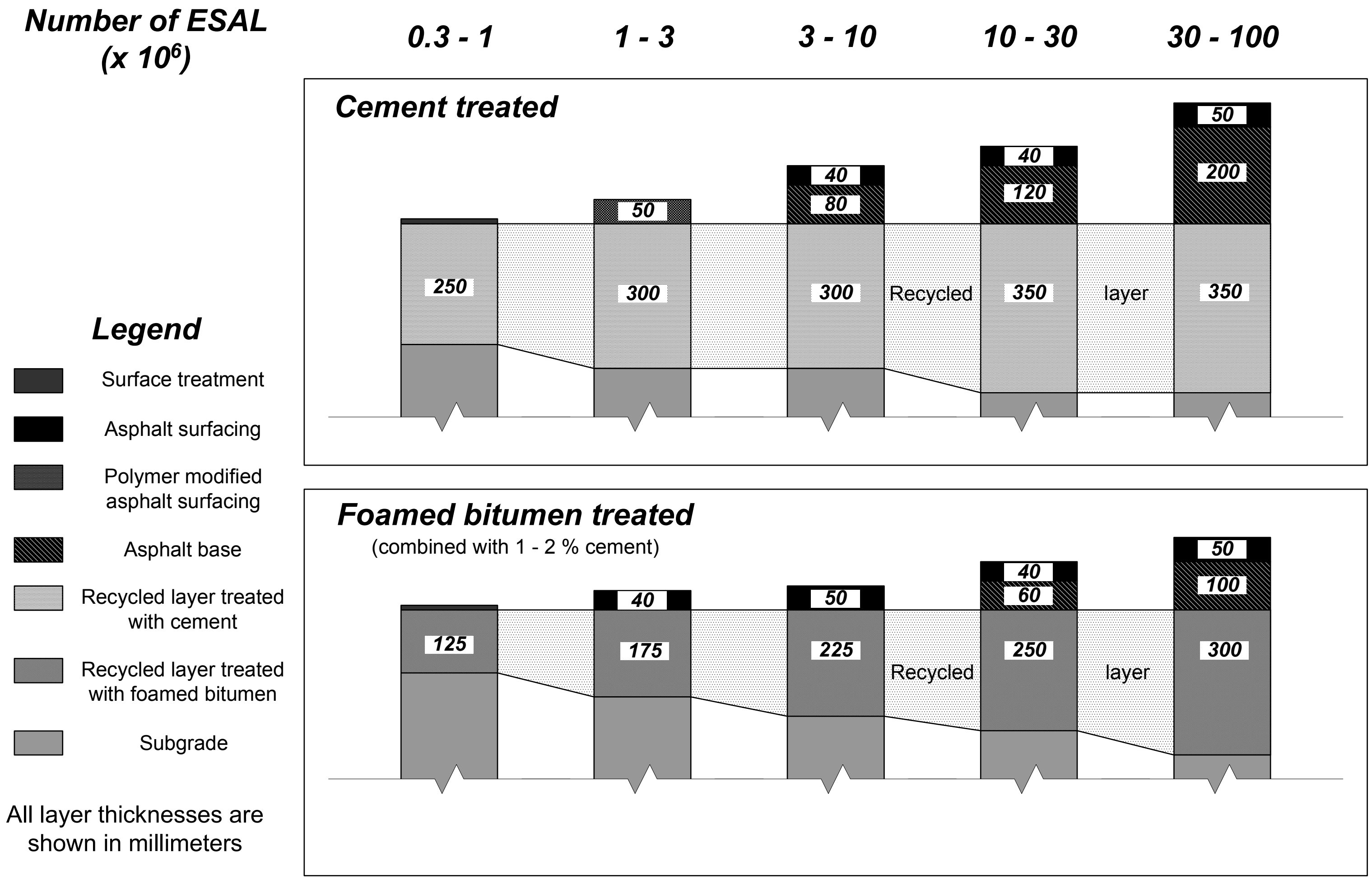
2.4.1 Catalogue design

An example of the catalogue design for a typical recycled pavement structure that includes both cementitious and bituminious treated recycled layers is shown in Figure 2.12 [Wirtgen , 2001]. The catalogue was produced based on assumed properties of the materials. Therefore, it can only be used as a guide for preliminary discussion and cost comparison purposes.

2.4.2 CBR design method

This method is based on soaked California Bearing Ratio (CBR) value of pavement materials. An example of CBR method is given by the Asphalt Institute [AI, 1970]. In this method, the full-depth asphalt thickness (T_A) of desired pavement is determined empirically from subgrade CBR and traffic data. The T_A is then converted to equivalent thicknesses of various pavement materials based on Substitution Ratio (S_r). For instance, the substitution ratio of asphalt concrete : crushed stone base equals 1 : 2 [AI, 1970]. That is 1 mm of asphalt thickness can be substituted by 2 mm of crushed stone base.

In the past, many agencies considered the structural capacity of cold recycled materials to be equal to that of conventional materials [Wood et al, 1988]. From the design by CBR method, the conventional unbound base materials were then simply replaced with an equal thickness of recycled materials without further formal structural design.



Note: the pavement design based on a subgrade support consisting of material with CBR > 10 (Elastic modulus > 100 MPa)

Figure 2.12 Typical recycled pavement structure [Wirtgen , 2001]

Empirical in nature, the CBR method is considered one of the simplest and quickest design methods which should be used only as a guide and should be checked using more theoretical methods.

2.4.3 Asphalt Institute method

The Asphalt Institute (1983) introduced a thickness design procedure for cold recycled mixtures made with bitumen emulsion. The designed thickness of a pavement structure is based on traffic, classified by Equivalent Standard Axle Load (ESAL), and subgrade support, classified by resilient modulus or CBR or R-value, and gradation of the mixes. A typical thickness design chart is illustrated in Figure 2.13. The charts give the combined thickness of a cold recycled base and an asphalt surface course. Recommended minimum thicknesses of the surface course on top of the recycled base are listed in Table 2.4 [AI, 1983].

Table 2.4 Minimum thickness of asphalt surface course over cold recycled base [AI, 1983]

Traffic Level (ESAL)	Minimum surface course thickness (mm)
10^4	50
10^5	50
10^6	75
10^7	100
10^8	130

The remaining portion of existing granular base below a recycled layer is also considered in the thickness design.

Table 2.5 shows the multiplying factors to convert the thickness of existing structural layers to equivalent thickness of cold recycled layer [AI, 1983].

Table 2.5 Factors to convert the thickness of existing structural layers to equivalent thickness of recycled layer [AI, 1983]

Classification of Material	Description of Material	Conversion factors
A	Native subgrade in all cases	0.0
B	Improved subgrade – predominantly granular materials – may contain some silt and clay but have P.I. of 10 or less	0.0
C	Granular subbase or base – reasonably well-graded, hard aggregates with some plastic fines and CBR not less than 20. Use upper part of range if P.I. is 6 or less; lower part of range it P.I. is more than 6.	0.1 – 0.2

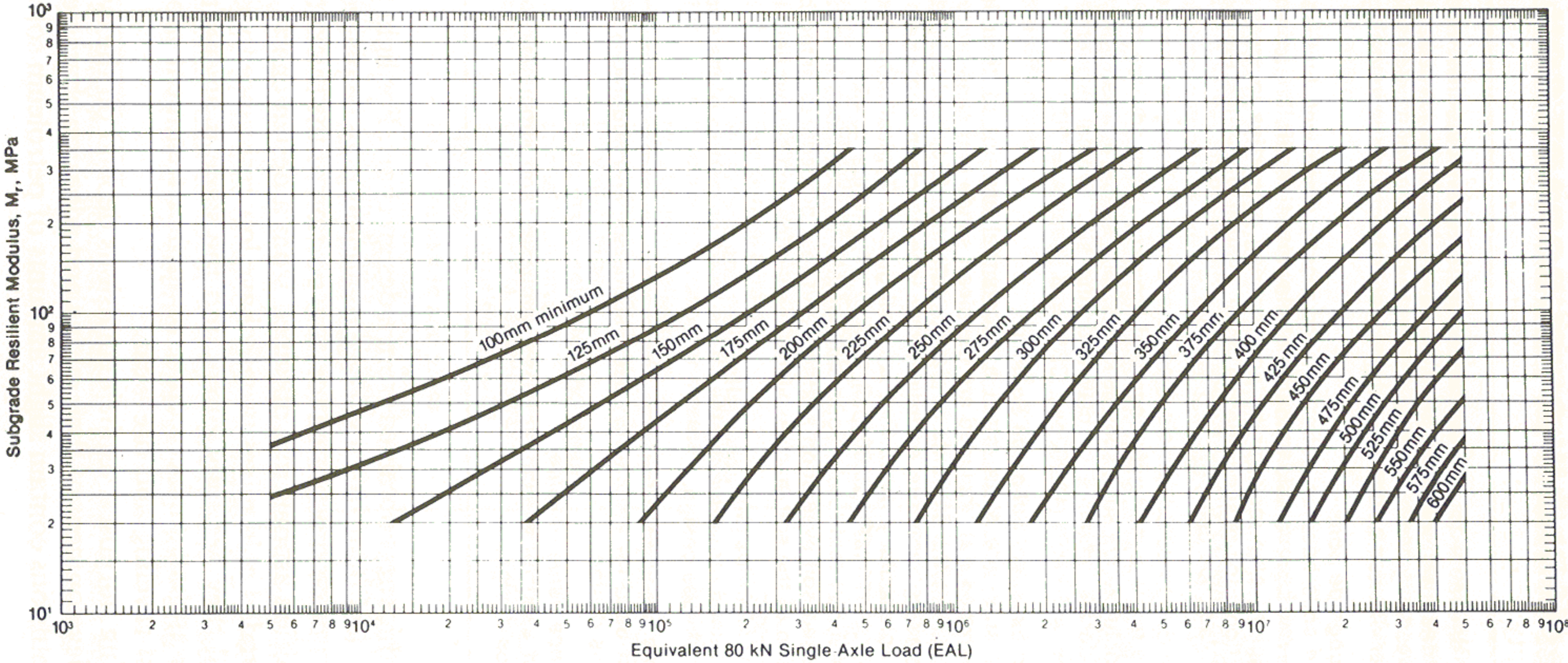


Figure 2.13 A thickness design chart for cold-mix recycled layers [AI, 1983]

2.4.4 AASHTO method

The AASHTO pavement design method as described in the *AASHTO Guide for Design of Pavement Structures* [AASHTO, 1993] can be used to design cold in-place recycled layers if the appropriate layer coefficients are available [Epps, 1990]. Table 2.6 lists some of the AASHTO layer coefficients of cold in-place materials reported in the literature.

Table 2.6 AASHTO layer coefficients of cold in-place recycled layer

References	Binder	AASHTO Layer coefficient
Moore (1985)*	Emulsion	0.30
Kandhal and Koehler (1987)*	Emulsion	0.25 – 0.40
Tia and Wood (1983)	Emulsion	0.22 – 0.49
Wood et al (1988)	Emulsion	0.14 – 0.44
Sebaaly et al (2000)**	(No information)	0.26
Mallick et al (2002)	Emulsion 3.4% + lime 2% Cement 5% Emulsion 2.2%	0.37 0.28 0.24
Wen et al (2003)	Class C fly ash 8%	0.16
New Mexico State Highway and Transportation Department (NMSHD) **	(No information)	0.30

* *See Epps (1990) ** See Sebaaly et al (2004)*

2.4.5 Structural design method of cold in-place recycled pavements in the United Kingdom.

In the United Kingdom, Milton and Earland (1999) introduced a design method for cold in-place recycled pavements. The procedure is briefly cited as follows.

Figure 2.14 shows a flow chart for cement bound recycled pavement design and Figure 2.15 for foamed bitumen bound mixes. The traffic data used in the design is the number of million standard axles that the road is to carry in one lane over its design life. The design criteria and material specifications are also stated in the flow charts. The road to be recycled is categorised into four types according to amount of traffic loading, as specified in **Table 2.7** [HAUC, 1992][†].

[†] See Milton and Earland (1999)

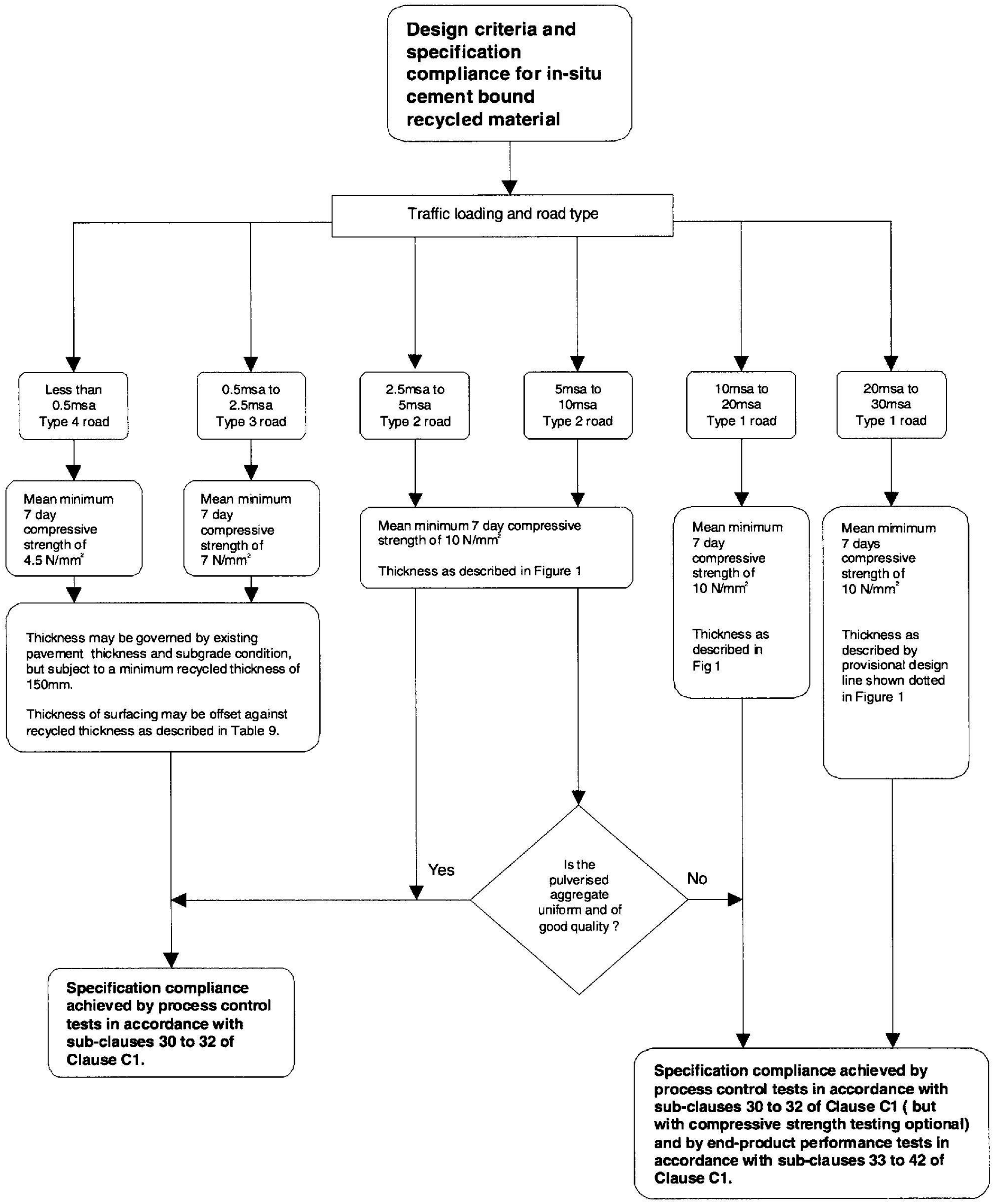


Figure 2.14 Design criteria and specification compliance programme for in-place cement bound recycled material [Milton and Earland, 1999]

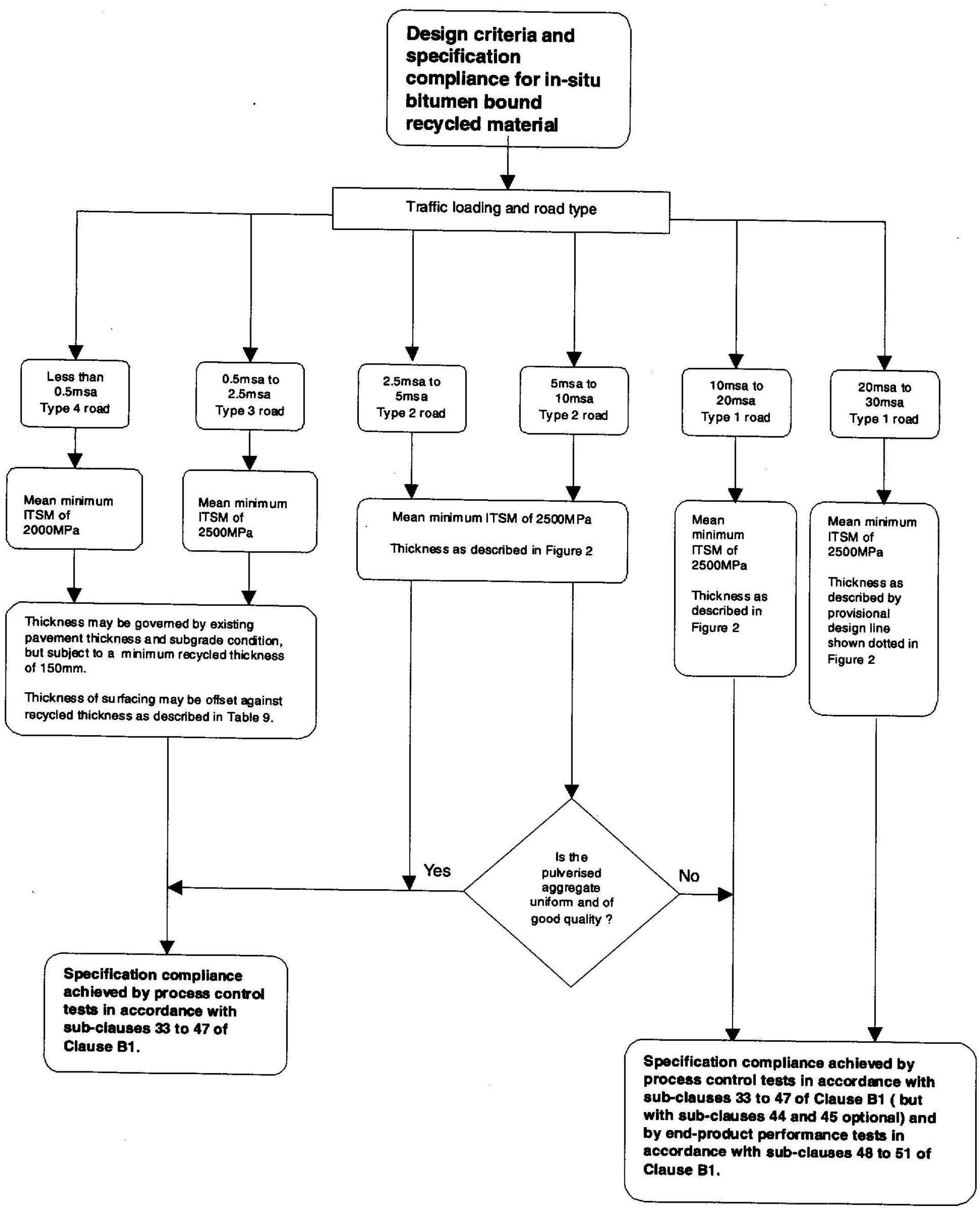


Figure 2.15 Design criteria and specification compliance programme for in-place bitumen bound recycled material [Milton and Earland, 1999]

Table 2.7 Road type category [HAUC, 1992][†]

Road type category	Traffic design standard (msa)
1	More than 10 up to 30
2	More than 2.5 up to 10
3	More than 0.5 up to 2.5
4	Up to 0.5

For relatively heavily traffic roads, i.e. Type 1 and Type 2 roads, the thicknesses of surface course and in-place recycled layer can be found using the design charts given in Figure 2.16 and Figure 2.17 for cement bound and foamed bitumen bound materials respectively. The remaining pavement is assumed to contain a foundation platform equivalent to those given in Table 2.8.

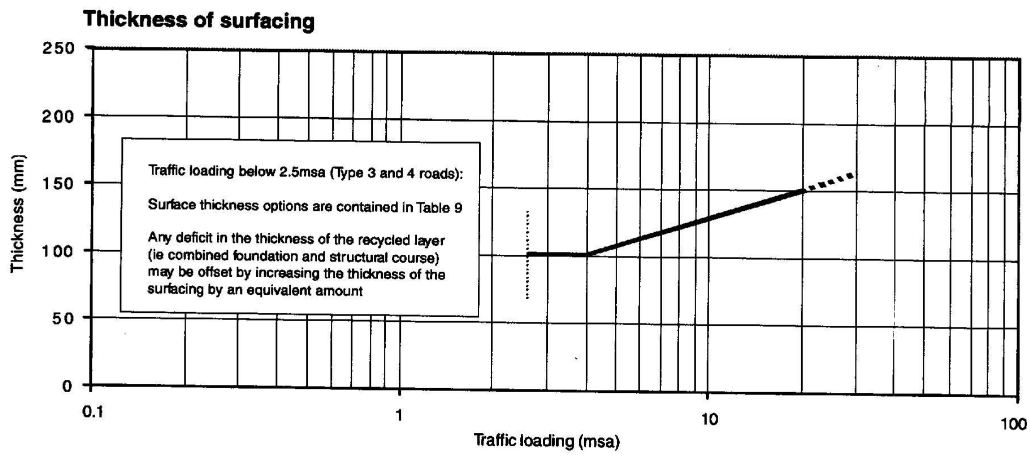
Table 2.8 Assumed foundation platform thickness of existing pavement of Type 1 and Type 2 roads [Milton and Earland, 1999]

Foundation platform thickness (mm)							
Subgrade CBR 2 – 4		**Subgrade CBR 5 – 7**		**Subgrade CBR 8 – 14**		**Subgrade CBR > 15**	
Granular	Remaining bound construction	Granular	Remaining bound construction	Granular	Remaining bound construction	Granular	Remaining bound construction
300	85	250	70	200	60	150	40

If the thickness of the remaining foundation is below the requirements in Table 2.8, the thickness of recycled layer should be increased to compensate for the thickness of foundation platform that is deficient. The equivalent factors are given in Table 2.9. However, the increase in thickness of the recycled layer will cause the thickness of foundation platform to be decreased. Therefore, in the design, any increase in thickness of the treated layer will be factored by the thickness deficit factors also given in Table 2.9.

In the case of lower traffic roads, i.e. Type 3 and Type 4 roads, the thickness of the recycled layer depends on the surfacing thickness options and subgrade CBR as listed in Table 2.10.

[†] See Milton and Earland (1999)



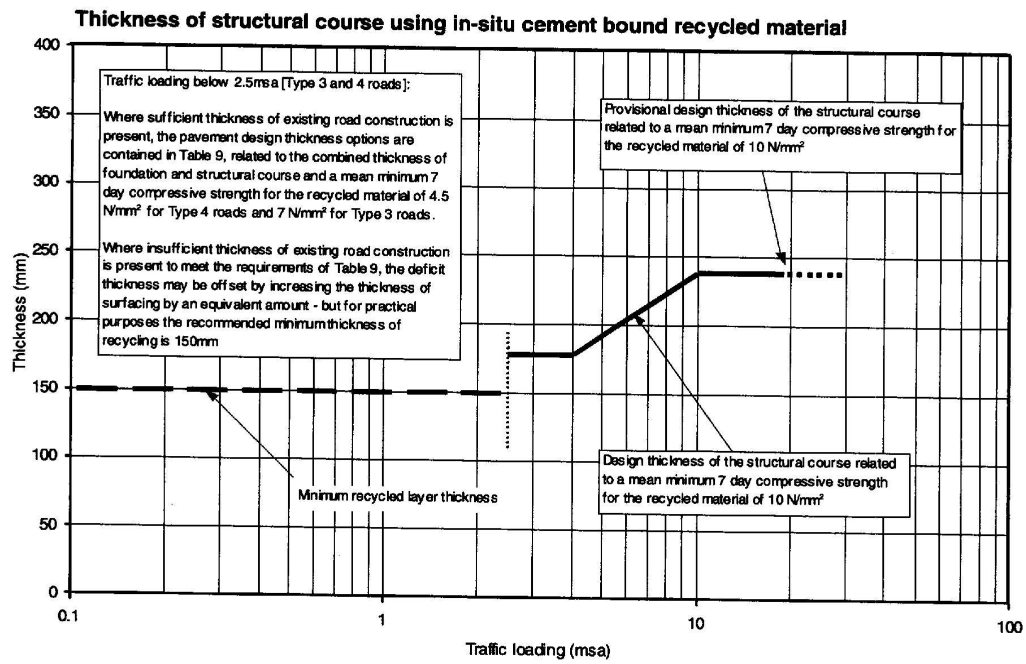


Figure 2.16 Pavement layer thicknesses using in-place cement bound recycling [Milton and Earland, 1999]

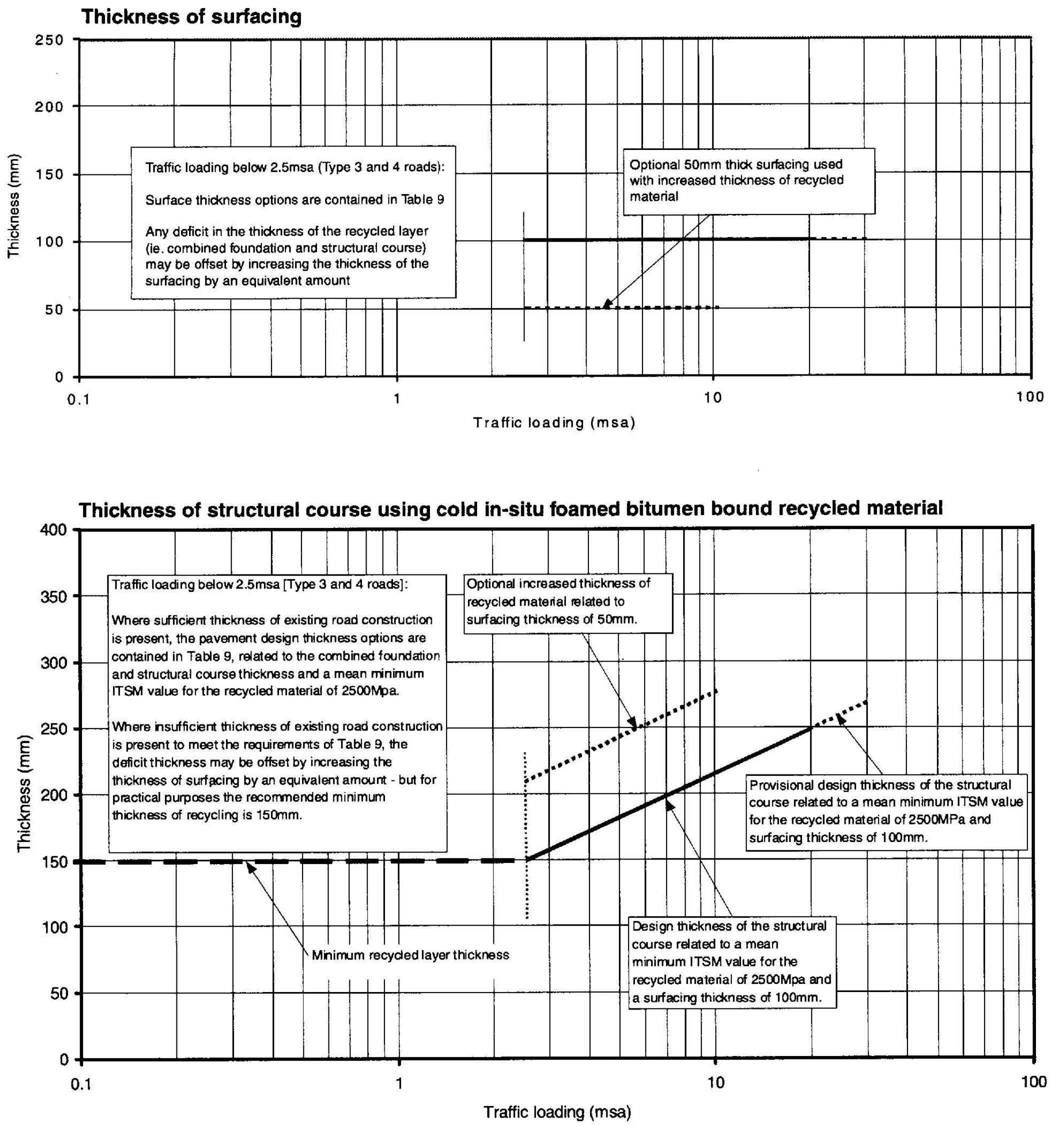


Figure 2.17 Pavement layer thicknesses using cold in-place foamed bitumen bound recycling [Milton and Earland, 1999]

Table 2.9 Thickness of conversion factors relative to granular foundation material [Milton and Earland, 1999]

In-place recycled end product	Equivalent factor	Thickness deficit factor
Cement stabilised material with 7 day compressive strength of 10N/mm2	0.3	0.45
Cement stabilised material with 7 day compressive strength of 7 N/mm2	0.4	0.65
Cement stabilised material with 7 day compressive strength of 4.5 N/mm2	0.5	1.00
Foamed bitumen stabilised material with ITSM of 2500 MPa	0.4	0.65

Table 2.10 Thickness of pavement layers using cold in-place recycled materials in Type 3 and Type 4 roads. [Milton and Earland, 1999]

	Thickness of cold in-place recycled material (mm)					
	Type 3 road			**Type 4 road**		
Binder type	**Cement bound in-place recycled material**					
Surface Thickness (mm)	**Surface dressing**	**40**	**100**	**Surface dressing**	**40**	**100**
Subgrade CBR (%)						
<2	n/r	n/r	n/r	n/r	n/r	n/r
2 – 4	280	240	180	240	200	150
5 – 7	260	220	160	220	180	150
8 – 14	240	200	150	200	160	150
> 15	220	180	150	190	150	150
Binder type	**Foamed bitumen bound in-place recycled material**					
Surface Thickness (mm)	**Surface dressing**	**40**	**100**	**Surface dressing**	**40**	**100**
<2	n/r	n/r	n/r	n/r	n/r	n/r
2 – 4	n/r	310 (n/r)	250	320 (n/r)	280	195
5 – 7	330 (n/r)	290	230	300	260	185
8 – 14	315 (n/r)	275	215	285	245	160
> 15	285	245	185	255	215	150

n/r = not recommended

The practical thickness of the recycled layer should not be in excess of 300 mm, otherwise proper compaction of a single thick lift would be difficult. It is also recommended, especially for Type 1 and Type 2 roads, to avoid mixing subgrade material into the recycled mix. [Milton and Earland, 1999].

It should be noted that research into in-place recycling at the time of publishing this procedure had been for traffic levels up to 20 msa [Milton and Earland, 1999]. Therefore, where the traffic loadings are in excess of this value, the design should be performed with caution. Where subgrade CBR is less than 2 percent, the cold in-place recycling technique is not recommended for any traffic loading [Milton and Earland, 1999].

2.5 Performance of cold in-place recycled pavement

Research conducted in the United States [McDaniel, 1988, Jahren et al, 1999, Thomas et al, 2000, Sebaaly et al, 2004, Wen et al, 2004], Canada [Kazmierowski et al, 1992], Australia [Moffatt and Sharp, 1999] and other countries [Cohen et al, 1989, Hoff et al, 2005] has provided basic information on performance of cold in-place recycled pavements. Results from these studies are summarised as follows.

In a road widening project, Indiana Department of Highways monitored the short-term performance of a cold in-place recycled road treated with bitumen emulsion, compared with a conventional overlay technique [McDaniel, 1988]. It was reported that after one year of service, the recycled pavement seemed to perform better than the conventional resurfaced pavement in terms of reflective cracking and rutting appearance. There was no significant difference between the two pavement types in terms of roughness and deflections.

Jahren et al (1999) systematically evaluated the performance of 18 cold in-place recycled roads in Iowa by using PCI and PSI indexes to rate the recycled roads' performance both quantitatively and qualitatively as shown in Table 2.11. It was found that in general the roads had been performing well. Only a few rutting problems had occurred. It was also apparent that cold in-place recycled pavement mitigated reflective cracks. Based on limited performance data, the service life of these recycled roads was predicted to be 15 to 26 years under low volume traffic conditions.

Table 2.11 shows the performance of the roads up to 10 years after construction [Jahren et al, 1999]. The deduction values for two distress types that are of particular interest for cold in-place recycling, rutting and transverse cracking, are shown in the table. These values are used as components of the total deduction values in the calculation of PCI. Definitions related to PSI rating are listed in Table 2.12. From Table 2.11, about one third of the projects have no deduction for transverse cracking and about two thirds have no deduction for rutting. Poor performance has been noted on Boone County's 198th street and IA-144 which are the only roads that obtained deduction for alligator cracking and the highest rutting deduction of 17. It was reported that, in Boone County's 198th street construction, there was an occurrence of equipment breakdown that resulted in extra water being added into the recycled materials while the IA-144 was constructed during a rainy period.

An experimental partial-depth cold in-place recycling project was performed in Kansas [Thomas et al, 2000]. Two road sections of approximately equal length were recycled, one with Class C fly ash and the other with an emulsion with lime slurry. For the fly ash section, 10 percent by weight of Class C fly ash was added while in the bitumen emulsion section, 1.5 percent of hydrated lime was added. A 40 mm asphalt surface course was overlaid over the recycled layer. Both field observation and laboratory results indicated that the recycled materials were not susceptible to rutting. However, some minor cracks were observed on asphalt surface. The fly ash section had about twice the amount of cracking as the bitumen emulsion with lime section.

Sebaaly et al (2004) monitored the field performance of three cold in-place recycling projects in Nevada. The construction consisted of cold in-place recycling with bitumen emulsion of the top 50 – 75 mm of an existing asphalt layer. The 50 – 75 mm of asphalt surface course was overlaid after recycling. The results indicated that cold in-place recycling reduced reflective cracking and significantly reduced rutting as shown in Figure 2.18.

A performance evaluation of a deep cold in-place recycled pavement stabilised with class C fly ash has been conducted in Wisconsin [Wen et al, 2004]. The road has an average daily traffic of 5,050 vehicles with 5% heavy vehicles. The original pavement structure consisted of 127 mm asphalt surface and a 178 mm granular base course. The new pavement consists of 127 mm asphalt surface and a 305 mm recycled base layer. After 2 years of construction, no cracking or rutting were observed on the pavement.

Table 2.11 Cold in-place recycling projects in Iowa [Jahren et al, 1999]

County	Road	Age	AADT	Existing Asphalt (mm)	Base (mm)	CIR (mm)	CIR Milled (%)	Emulsion	2.5.1.1.1 Hot mix overlay					Rutting	T-crack /mile	PSI	PCI
									Thickness (mm)	Asphalt	AC (%)	Void (%)	Size (mm)				
Muscatine	G-28	5	940	200	150	100	50	CSS-1	50	AC-5/10	4.9	4.5	19	0	2	73	98
Tama	V-18	5	550	0	150	100	67	HF-300RP CSS-1	50	AC-5	6.1	6.5	13	0	0	70	100
Boone	E-52	5	290	200	150	100	50	CSS-1	50	AC-10	5.1	4	13	0	2	73	95
Clinton	Z-30	7	850	125	250	100	80	CSS-1	50	AC-10	6.5	6	13	0	1	64	93
Muscatine	F-70	3	950	100	200	100	100	CSS-1	75	AC-5	5.7	7	13	0	0	82	100
Butler	T-16	3	470	150	150	100	67	CSS-1	50	AC-5	5.5	7	13	0	0	81	100
Tama	E-66	6	1080	100	200(pcc)	100	100	HF-300RP	50	AC-5	5.9	7	13	0	6	61	94
Clinton	E-50	10	520	137.5	162.5	100	73	CSS-1	50	AC-10	5.1	4	13	10	15	51	81
Winnebago	R-34	6	620	150	150	100	67	HF-300RP	50	AC-5	5.7	4.1	13	0	10	63	90
Cerro-Gordo	B-43	7	570	150	150	100	67	CSS-1	50	AC-10	5.2	5.6	13	8	3	68	77
Boone	198^{th}	8	300	150	150	100	67	CSS-1	50	AC-10	5.7	3.6	13	17	0	59	71
Hardin	D-35	4	665	162.5	150	75	46	CSS-1	50	AC-10	5.5	6.5	13	3	8	65	85
Cerro-Gordo	S.S	6	600	200	150	100	50	CSS-1	50	AC-10	6.2	5	13	1	14	61	81
Winebago	R-60	6	340	125	150	100	80	HF-300RP	50	AC-5	5.5	4.3	13	0	0	63	72
Muscatine	Y-14	9	990	150	150	100	67	HFE-150S CSS-1 HFMS	62.5	AC-10	6	5.75	19	9	7	61	52
Calhoun	IA-175	3	1920	200	200	75	38	CSS-1	112.5	AC-10	5.9	5.2	13	0	0	81	100
Greene	IA-144	7	1110	100-150	150	100	67-100	CSS-1 CMS-2P	50	AC-10	5.2	5.5	13	17	7	58	60
Guthrie	IA-4	3	820	150-200	n/a	100	50-67	CSS-1H	75	AC-10	5.6	6.2	13	0	0	90	100

Table 2.12 PSI rating scale [Jahren et al, 1999]

PSI	Ride rating	Appearance rating
100	New – no roughness or cracks	Looks dark and smooth
80	Minor roughness. Cracks cannot be felt while driving	Minor cracks are barely visible
60	Noticeably rough. Crack can be felt while driving	Cracks are clearly visible. Weathered surface.
40	Very rough.	Transverse cracks at short intervals. Longitudinal cracks present.
20	Very rough. Difficult to maintain speed.	Heavily distressed. Transverse, longitudinal and block cracking present.

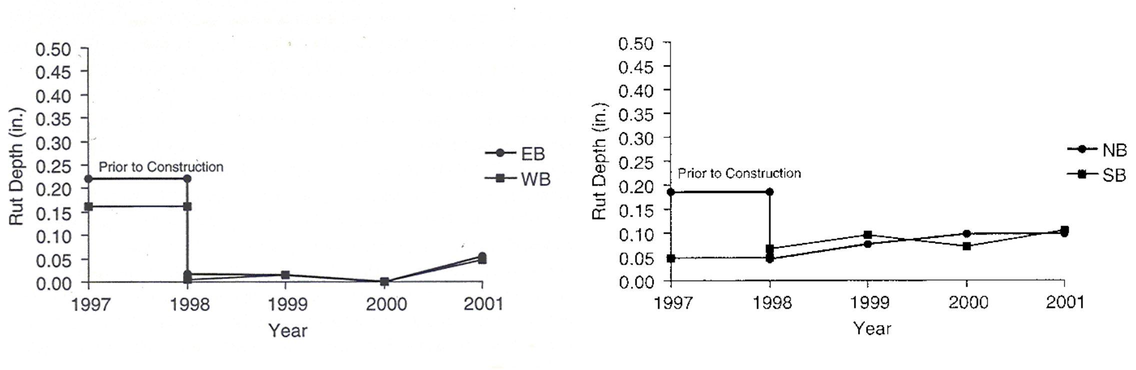
In Canada, Kazmierowski et al (1992) evaluated the performance of a demonstration road in Ontario. The existing pavement structure of this road consisted of 150 mm asphalt surface, 150 mm granular road base and 150 mm granular subbase. The traffic on this test section had an average annual daily traffic (AADT) of 4,000 with 10 to 15 percent heavy vehicles. The pavement was recycled with bitumen emulsion to the depth of 50 to 75 mm and then resurfaced with 50 mm hot mix asphalt. The researchers concluded that the pavement was performing successfully and economically. The riding quality improved significantly. The FWD test indicated that the deflection of the pavement decreased with time and was less than its original deflection before construction as shown in Figure 2.19.

A series of accelerated loading facility (ALF) experiments were conducted on 200 mm thick in-place stabilised, with bitumen/cement and slag/lime materials, test pavements in Melbourne, Australia [Moffatt et al, 1998][†]. Moffatt and Sharp (1999) stated that, in general, the performance of the test sections was satisfactory with no fatigue cracking or subgrade deformation observed.

In Israel, a trial road with a cold recycled bitumen emulsion was observed for field performance [Cohen et al, 1989]. The pavement performed well after one year of service and no sign of fatigue distress or rutting was found visually. Also in Norway, an old gravel road was recycled and stabilised with foamed bitumen. The test sections were established to investigate the pavement performance [Hoff et al, 2005]. An overall

[†] See Moffatt and Sharp (1999)

performance after one year of service was reported "very good" and no visible distress has appeared.



(a) (b)

Figure 2.18 Rutting performance of cold in-place recycling projects in Nevada (a) US-50, (b) US-95 [Sebaaly et al, 2004]

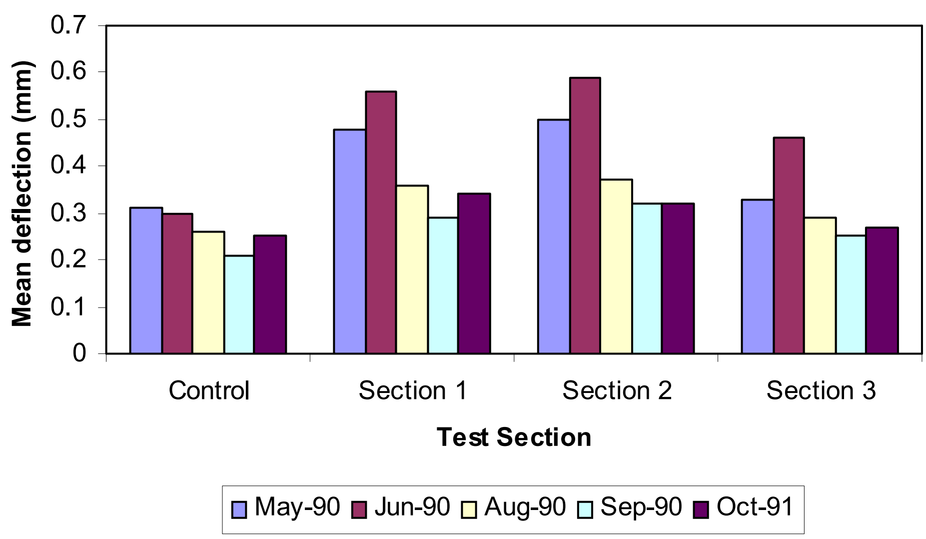


Figure 2.19 FWD testing on a cold recycled road in Ontario [reproduced from Kazmierowski et al, 1992]

2.6 Criteria for cold in-place recycled materials

The properties of materials often used in recycled pavements stabilised with cement and bitumen are listed in the Table 2.13. [Wirtgen , 2001].

Table 2.13 Engineering properties of cold recycled materials [Wirtgen , 2001]

RAP/Crushed rock (50/50 blend)			
Test parameter	Cement Stabilised 2–2½% cement	Bitumen stabilised: 1–1½% cement plus 2½-5% emulsion	Bitumen stabilised: 1% cement plus 1½ - 3% foam
Density % modified AASHTO	96 to 98	98 to 100	98 to 102
Unconfined Compressive Strength MPa	1.5 to 3	N/A	N/A
Indirect Tensile Strength kPa	N/A	350 to 750	350 to 800
Retained strength %	N/A	> 75	> 75
Resilient modulus MPa	~5000*	2500 to 5000	2500 to 5000
Crushed rock			
Test parameter	Cement Stabilised 2–2½% cement	Bitumen stabilised: 1–1½% cement plus 2½-5% emulsion	Bitumen stabilised: 1% cement plus 1½ - 3% foam
Density % modified AASHTO	96 to 98	98 to 100	98 to 102
Unconfined Compressive Strength MPa	1.5 to 3	N/A	N/A
Indirect Tensile Strength kPa	N/A	400 to 800	400 to 900
Retained strength %	N/A	> 60	> 60
Resilient modulus MPa	~5000*	3000 to 6000	3000 to 6000
Gravel			
Test parameter	Cement Stabilised 3–4% cement	Bitumen stabilised: 1–1½% cement plus 4-7% emulsion	Bitumen stabilised: 1% cement plus 1½ - 3% foam
Density % modified AASHTO	95 to 97	97 to 100	98 to 100
Unconfined Compressive Strength MPa	1.5 to 3	N/A	N/A
Indirect Tensile Strength kPa	N/A	250 to 500	250 to 500
Retained strength %	N/A	> 50	> 50
Resilient modulus MPa	~4000*	2000 to 4000	2000 to 4000

* *Before any cracking has occurred*

2.7 Summary

The studies into cold recycling of asphalt pavements have been reviewed in this chapter. Researches have shown that generally cold recycled roads have performed well. The research to characterise these cold mixture for use in design and predicting its performance is therefore required. However, many studies on engineering properties of cold recycled materials in the literature were performed using methods which may not relate to the performance of the materials under traffic load such as indirect tensile strength on bituminous bound mixes and unconfined compressive strength of cement stabilised mixes. The present study, as presented in the following chapters, will attempt to investigate more fundamental mechanical properties of the cold recycled pavement mixtures which can be applied in pavement engineering applications.

3 MATERIALS USED IN THE INVESTIGATION

3.1 Introduction

Throughout this research, the recycled asphalt pavement specimens used in the experiments were manufactured in the laboratory. The materials selected for the investigation are those typically encountered in the actual recycling of existing asphalt pavements. The primary materials used in this study included: (1) reclaimed asphalt pavement (RAP), (2) limestone aggregate, and (3) stabilising agent or binder. RAP is the milled deteriorated asphalt layer in pavements. Limestone aggregate represented granular base materials underneath an asphalt surface. Stabilising agents included foamed bitumen, cement and lime and combinations of foamed bitumen and cementitious additives. All of the specimens of recycled asphalt pavement in this study were fabricated by blending and compacting the above materials in the laboratory to simulate the recycling process in the field. In the first stage of the investigation mix design tests were carried out to optimise the properties and/or content of binder for each mixture type. The characteristics of these materials as well as mix design results are provided in this chapter.

3.2 Materials

3.2.1 Reclaimed asphalt pavement

Reclaimed asphalt pavement (RAP) was obtained from Ratcher Hill quarry, an in-plant recycling site in Mansfield, UK. The RAP was originally milled from various asphalt roads and brought together into one stockpile. A visual inspection indicated that the RAP was somewhat contaminated by rubbish from the roads such as broken glass, tree roots, small pieces of wood and plastic etc. as shown in Figure 3.1. The physical properties of the RAP were assessed in terms of gradation, shape, particle density, and water absorption.

Figure 3.1 RAP sample

RAP aggregate from the quarry was brought to the laboratory and air dried at room temperature (20 ± 5 °C) for 7-10 days. It was found that moisture contents of air-dried RAP were less than 0.5 % indicating the presence of negligible moisture. A sieve analysis in accordance with BSEN933-1:1997 was performed on air-dried RAP aggregate. Three series of tests were performed. The gradation analysis results are shown numerically in Table 3.1. Figure 3.2 presents the particle size distribution curve of RAP graphically. The uniformity coefficient (C_u) was 18 and the coefficient of curvature (C_c) was 1.40. These parameters represent the shape of the grading curve and denote whether the material is well-grade or poorly grade [Barnes, 1995]. RAP can be classified as well-graded gravel (C_u >5, C_c = 1 to 3) with little or no fines (GW) with a very small amount of fines passing 0.075 mm sieve.

The flakiness and elongation index tests were performed in accordance with BS 812-105.1:1989 and BS812-105.2:1990. Particle density and water absorption were tested by a pycnometer and gas jar technique in accordance with BSEN 1097-6:2000. RAP aggregates in each of the following size ranges were tested individually: 10-20 mm, 5-10 mm, and < 5 mm.

A composition analysis was also performed to determine the properties of RAP and its extracted components. The results are shown in Table 3.2

Table 3.1 Particle size distribution and physical properties of RAP

Sieve size	Percent passing			
	Test 1	Test 2	Test 3	Average
37.5 mm	100	100	100	100.0
28 mm	100	98.3	98.7	99.0
20 mm	96.8	91.6	95.8	94.7
14 mm	86.0	78.2	82.9	82.4
10 mm	75.9	62.2	67.3	68.5
6.3 mm	53.6	41.9	47.7	47.7
5.0 mm	33.8	27.2	28.4	29.8
3.35 mm	30.3	24.3	24.9	26.5
2.36 mm	17.1	14.4	14.1	15.2
1.18 mm	10.0	8.5	8.3	8.9
0.600 mm	7.2	6.1	6.0	6.4
0.300 mm	4.5	3.6	3.5	3.9
0.212 mm	2.6	2.0	2.0	2.2
0.150 mm	1.5	1.1	1.1	1.2
0.075 mm	0.6	0.3	0.4	0.4
0.063 mm	0.4	0.3	0.3	0.3
Particle density				
- Particle size 10 – 20 mm	2.56 Mg/m^3			
- Particle size 5 – 10 mm	2.52 Mg/m^3			
- Particle size < 5 mm	2.39 Mg/m^3			
Water absorption				
- Particle size 10 – 20 mm	0.93 %			
- Particle size 5 – 10 mm	1.01 %			
- Particle size < 5 mm	3.83 %			
Elongation Index	12			
Flakiness Index	13			
Shape Index	8			

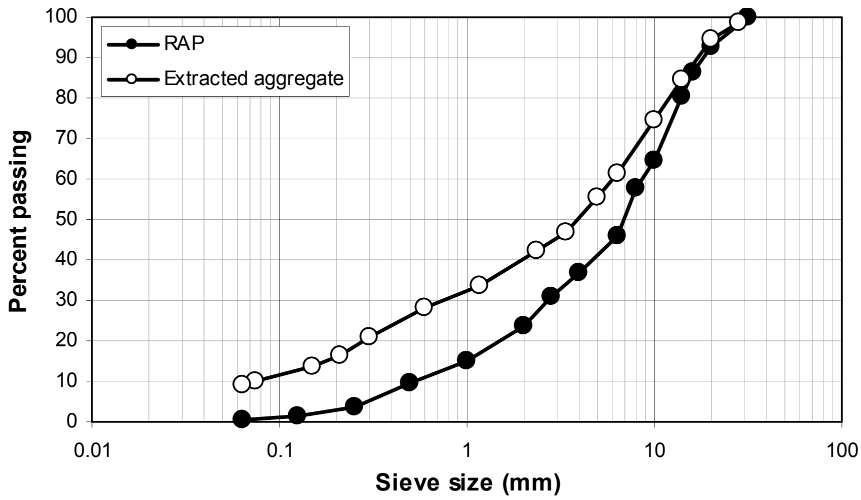


Figure 3.2 Gradations of RAP and its extracted aggregate

Table 3.2 Composition analysis of RAP

Recovered bitumen			
Properties	**Test 1**	**Test 2**	**Test 3**
Binder content (%)	2.9	2.8	2.8
Penetration (25 °C) (0.01 mm)	50	65	130
Softening point (Ring & Ball) (°C)	64.6	62.2	63.2
Viscosity at 135 °C (mPa s)	603.5	385.9	241.7

Gradation of extracted aggregate				
Sieve size	**Percent passing**			
	Test 1	**Test 2**	**Test 3**	**Average**
37.5 mm	100	100	100	100
28 mm	100	100	96.32	98.77
20 mm	98.39	98.37	87.19	94.65
14 mm	85.81	88.71	78.57	84.36
10 mm	75.56	78.52	68.92	74.33
6.3 mm	60.71	64.58	58.27	61.19
5.0 mm	54.61	57.92	54.33	55.62
3.35 mm	45.88	45.11	49.09	46.69
2.36 mm	39.04	43.05	45.14	42.41
1.18 mm	29.62	33.92	37.97	33.84
0.600 mm	23.57	28.15	32.58	28.10
0.300 mm	17.82	21.01	23.62	20.82
0.212 mm	14.64	16.69	18.33	16.55
0.150 mm	12.25	13.48	14.54	13.42
0.075 mm	8.95	9.84	10.54	9.78
0.063 mm	8.34	9.19	9.98	9.17

3.2.2 Limestone aggregate

The mineral aggregate used in this study was limestone, which was obtained from Dene quarry, Derbyshire, UK. The limestone was supplied separately in six nominal sizes according to the following sieves: 20 mm, 14 mm, 10 mm, 6 mm, passing 6 mm (dust) and passing 0.075 mm (filler). The addition of filler was necessary as the RAP was found to be deficient in fine particles passing 0.075 mm sieve. Physical properties of limestone aggregate were determined individually. Particle size distribution was determined according to BSEN933-1:1997. Particle density and water absorption were determined according to BSEN 1097-6:2000 for particles and BSEN1097-7:1999 for filler. Figure 3.3 shows gradations of the aggregate. Table 3.3 summarises the characteristics of the limestone aggregate.

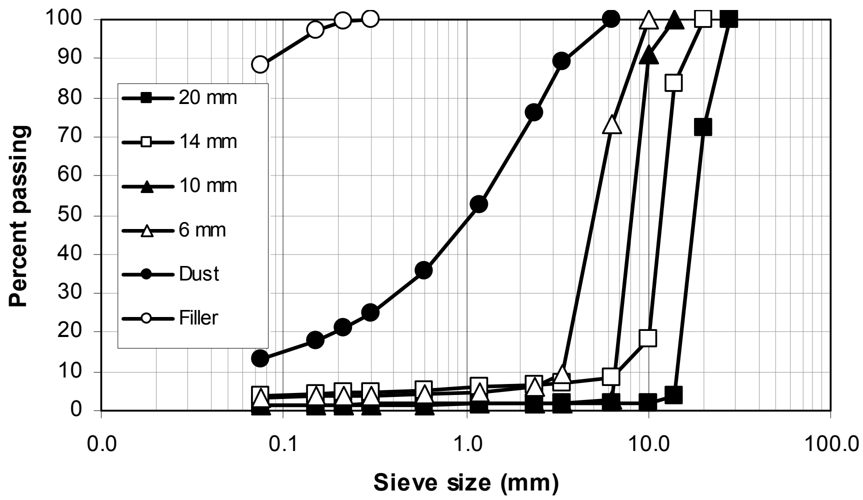


Figure 3.3 Particle size distribution of the nominal single sized limestone

Table 3.3 Particle size distribution and physical properties of limestone aggregate

Sieve size	Nominal size					
	20 mm	14 mm	10 mm	6.3 mm	Dust	Filler
	Percent passing					
28.0 mm	100	100	100	100	100	100
20.0 mm	72.45	100	100	100	100	100
14.0 mm	3.56	83.57	100	100	100	100
10.0 mm	1.81	18.18	91.23	100	100	100
6.3 mm	1.81	8.43	2.91	73.46	100	100
3.35 mm	1.80	7.23	1.72	9.25	89.25	100
2.36 mm	1.80	6.70	1.69	5.96	75.96	100
1.18 mm	1.76	5.95	1.65	4.81	52.58	100
0.6 mm	1.72	5.36	1.64	4.28	35.88	100
0.3 mm	1.65	4.83	1.63	3.94	24.94	100
0.212 mm	1.60	4.59	1.59	3.82	21.17	99.40
0.15 mm	1.55	4.32	1.53	3.63	17.87	97.30
0.075 mm	1.41	3.80	1.45	3.29	13.37	88.30
Particle density (Mg/m^3)**						
oven dried	2.633	2.607	2.608	2.427	2.668	2.623
saturated and surface dried	2.653	2.634	2.640	2.526	2.674	N/A
apparent	2.685	2.679	2.693	2.696	2.686	N/A
Water absorption (%)**	0.74	1.03	1.20	4.10	0.26	N/A

** From Sunarjono (2008)

3.2.3 Foamed bitumen

In this study, the Wirtgen WLB-10 foaming machine was used to generate foamed bitumen from hot penetration grade bitumen. The Wirtgen WLB-10 was designed to produce foamed bitumen in the laboratory. Foamed bitumen produced by this unit is similar to that produced by foamed bitumen systems mounted on actual recycling machines. WLB-10 has a thermostatically controlled container capable of maintaining 10 kg of bitumen at a temperature between 100 °C and 200 °C. A compressed air tank and a water storage tank are included in the apparatus. The air pressure and water pressure are controlled separately by pressure gauges. The plant is also equipped with a flow meter to control the amount of cold water added into hot bitumen in terms of percentage by mass of the bitumen.

Foamed bitumen develops when air and water are injected into hot bitumen. It has been reported that an air pressure of less than 100 kPa adversely affects the foaming ability [Wirtgen, 2001, He and Wong, 2005]. On the other hand, variation in air pressure from 300 – 700 kPa has little effect on the foaming characteristics [He and Wong, 2005]. In addition, the water pressure has to be higher than the air pressure in order to allow water to spray in. Based on this information, air and water pressure were set at 500 kPa and 600 kPa respectively.

To create foamed bitumen in the WLB-10, hot bitumen flows in a circulation pipe at a pressure of 150 kPa, and is released into to an expansion chamber into which both air and water under the above mentioned pressures are simultaneously injected. The amount of foam produced is controlled by the time of discharge of the foam. The WLB-10 is capable of producing foamed bitumen at a rate of 100 g per second. The discharge timer was set to the required time in seconds corresponding to the amount of foamed bitumen required and the foamed bitumen was generated accordingly. It was then sprayed out through a nozzle. Figure 3.4 schematically illustrates the laboratory foamed bitumen production process. Figure 3.5 shows a photograph of the WLB-10 unit.

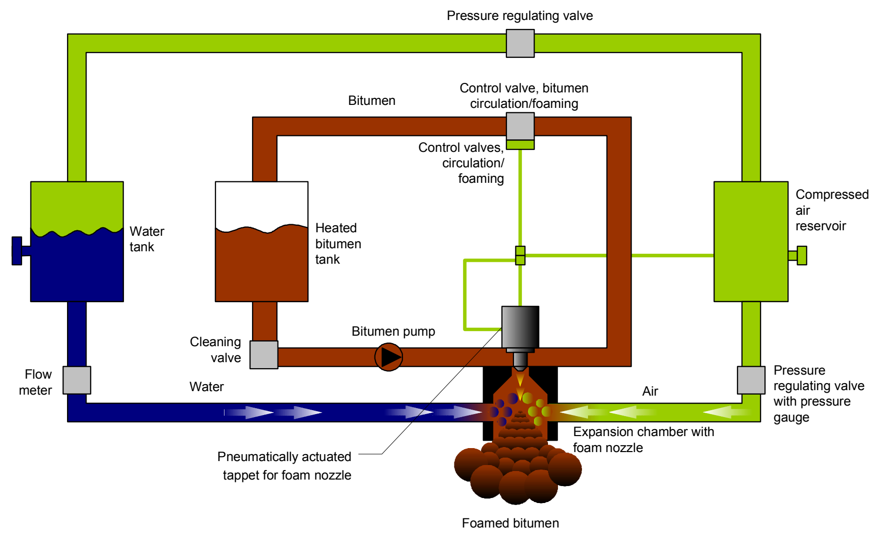


Figure 3.4 Schematic diagram of a foamed bitumen production by WLB-10 unit [Wirtgen, 2001]

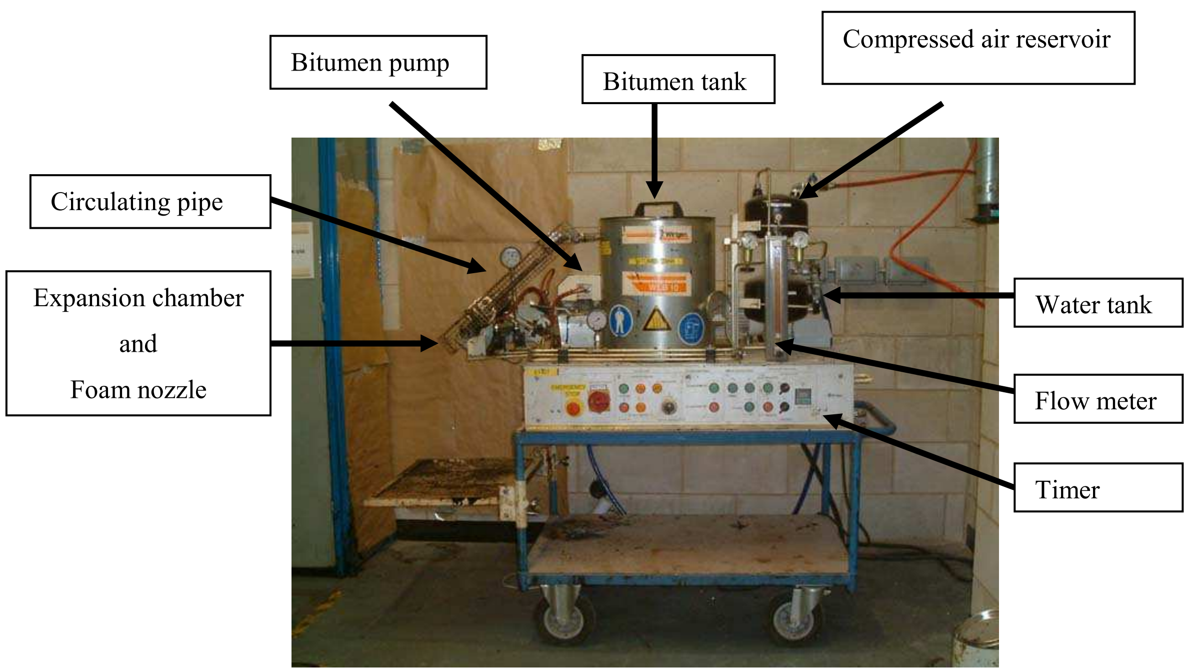


Figure 3.5 Wirtgen WLB-10 laboratory foaming plant

During the bitumen foaming process, the foamed bitumen expands to a maximum volume and then the bubbles collapse completely reverting the bitumen back to its liquid state. Foamed bitumen is commonly characterised by two terms namely, maximum expansion ratio (ER_{max}) and half-life (HL). Maximum expansion ratio is defined as the ratio between the maximum volume achieved in the foamed state and the volume of original bitumen. Half-life is the time taken, normally in seconds, for the volume of the foam to collapse to half of its maximum volume. Figure 3.6 illustrates the definition of ER_{max} and HL. The foaming characteristics of a specific bitumen are influenced by many factors such as bitumen temperature, bitumen type, amount of added water, and the presence of anti-foaming agents [Jenkins, 2000]. However, in this study, only bitumen type, bitumen temperature, and water content were varied in the investigation of foaming characteristics.

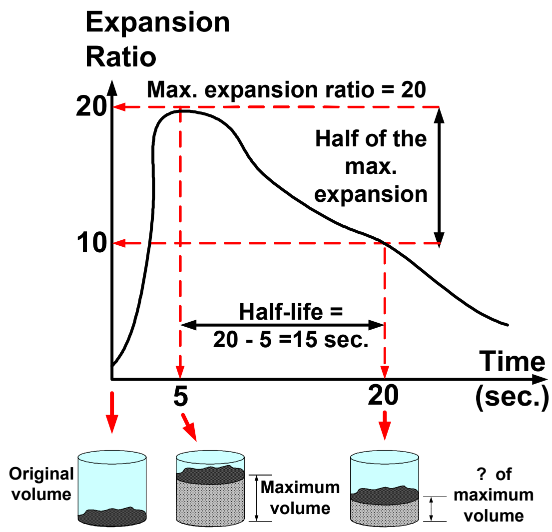


Figure 3.6 Illustration of maximum expansion ratio and half-life

Three penetration grade bitumens, provided by Shell bitumen, PG50/70, PG70/100 and PG160/220, were selected for the production of foamed bitumen. Laboratory tests including the penetration test, the ring and ball softening point test, and the Brookfield viscosity test were conducted to determine the basic properties of the bitumen. The test methods are described elsewhere [e.g., Read and Whiteoak, 2003]. All tests were performed in accordance with British standard specifications [BSEN1426:2000, BSEN1427:2000 and BSEN13302:2003] and the results are shown in Table 3.4.

Table 3.4 Basic properties of bitumen PG50/70, PG70/100 and PG160/220

Properties of bitumen	Standard	Penetration grade		
		50/70	70/100	160/220
Penetration (25 °C, 100 g, 5 s) (0.01 mm)	BSEN1426:2000	56	76	172
Softening point (°C)	BSEN1427:2000	53.1	49.6	38.8
Viscosity (mPa s)	BSEN13302:2003			
140 °C		401	262	164
160 °C		162	114	74
180 °C		83	57	40

The bitumen samples were subjected to foam production at three levels of temperature, 140 °C, 160 °C and 180 °C, and at five levels of water content, 1%, 2%, 3%, 4%, and 5% by mass of bitumen. Two duplicated tests were conducted for each test condition. In each foaming test, 500 g of bitumen was injected into an expansion chamber to produce foamed bitumen. The foamed bitumen was then sprayed from the nozzle into a measuring cylinder. ER_{max} and HL of the foamed bitumen were recorded based on visual observation. Because foamed bitumen expanded and dissipated quickly, it was difficult to observe the values of ER_{max} and HL accurately. A digital video camera was, therefore, used to record the whole foaming process from the time when bitumen spraying started until the time when the foamed bitumen had almost completely dissipated. By watching the recorded video, ER_{max} was noted and HL was timed using a stopwatch.

Figure 3.7 shows ER_{max} and HL for the three bitumen grades at various temperatures and water contents. The test results indicated that all grades of bitumen have lowest ER_{max} and longest HL at 140 °C. As the temperature increases, ER_{max} tends to increase while HL decreases. It can also be observed that at temperatures of 160 °C and 180 °C, the foaming characteristics of bitumens are similar although the HL of the foam is slightly shorter at a temperature of 180 °C.

Generally, the most effective foam should have a large ER_{max} and a long HL to ensure adequate dispersion of bitumen within the material during mixing. Nevertheless, there are still no absolute criteria for these characteristics. CSIR Transportek [Muthen, 1999] recommended that an effective foam is one having an ER_{max} of at least 10 and a HL of at least 12 seconds. Minimum ER_{max} and HL of 8 and 6 seconds are recommended in the *Wirtgen Manual* [Wirtgen , 2001]. Chiu and Huang (2002) adopted 11 and 9 seconds as suitable minimum ER_{max} and HL respectively. Ramanujam and Jones (2007) recommended 10 times ER_{max} and 30 seconds HL.

Based on the results of this study, a temperature of 180 °C and a water content of 2 % by mass of the bitumen were selected to create a suitable foamed bitumen for all bitumen grades. The corresponding ER_{max} and HL values were 15 times and 9 seconds for PG50/70, 17.5 times and 16.5 seconds for PG70/100 and 19.5 times and 5 seconds for PG160/220.

3.2.4 Additives

The addition of additives aims to achieve the main objectives of binding particles together to increase strength and/or make materials more water resistant. In this work, hydrated lime ($Ca(OH)_2$) and Portland cement were considered since they are two of the most commonly used cementitious stabilising agents in cold in-place recycling works.

Because of a countrywide shortage of ordinary Portland cement Type I, it was decided to use Portland limestone cement in this investigation. The cement, which was manufactured by Lafarge Cement in UK, conforms to the British Standard [BSEN197-1:2000] for CEM II/A-LL 42.5. Portland limestone cement contains a controlled level of high purity limestone and can be used as an alternative to Portland cement in civil engineering works [Larfarge Cement, 2008]. However, limestone cement concrete can have a lower 28 day compressive strength than concrete made with ordinary Portland cement. Consequently, higher cement contents may be required to achieve the same strength [Larfarge Cement, 2008]. Typical properties of Portland limestone cement used in this investigation are given in Table 3.5.

Table 3.5 Typical properties and mineral compound of CEM II/A-LL 42.5 [Lafarge Cement, 2008]

Surface area (m^2/kg)	360 – 470
Setting time (minute)	95 – 150
Mortar compressive strength (N/mm^2)	
2 day	21 – 30
7 day	36 – 50
28 day	45 – 65
Apparent particle density (kg/m^3)	3080 – 3180
Sulphate (SO_3) (%)	2.5 – 3.5
Chloride (Cl) (%)	< 0.06
Alkali (Eq Na_2O) (%)	0.35 – 0.65
Tricalcium Silicate (C_3S) (%)	40 – 60
Dicalcium Silicate (C_2S) (%)	6 – 25
Tricalcium Aluminate (C_3A) (%)	6 – 12
Tetracalcium Aluminoferrite (C_4AF) (%)	5 – 10
Limestone content ($CaCO_3$) (%)	6 – 20

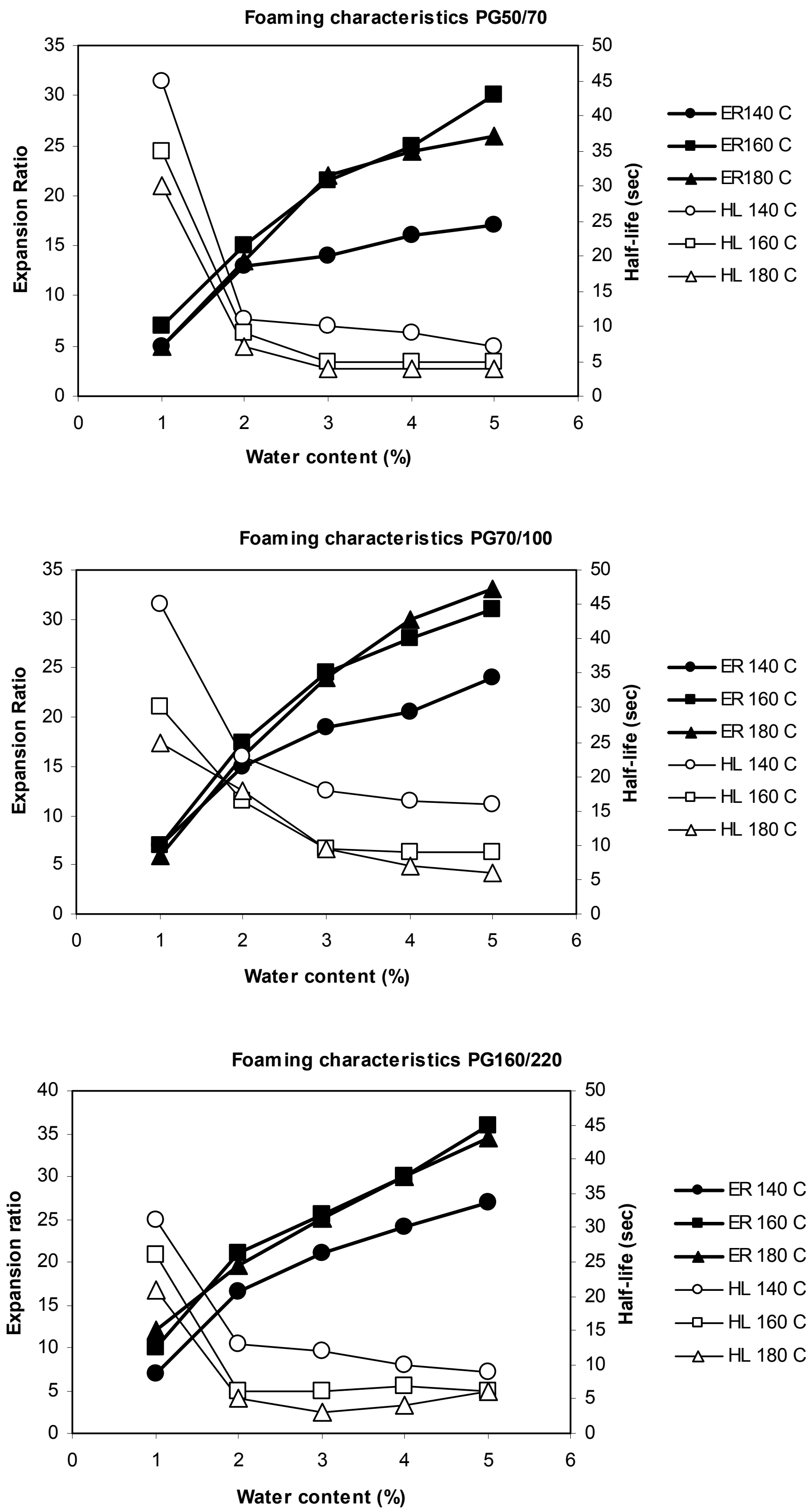


Figure 3.7 Foaming bitumen characteristics

3.3 Mix design

3.3.1 Selection of aggregate gradation

Three proportions of RAP to limestone aggregate were considered in the investigation, namely:

Proportion 1: 75%RAP (75 % RAP and 25 % limestone aggregate by mass)
Proportion 2: 50%RAP (50 % RAP and 50 % limestone aggregate by mass)
Proportion 3: 0%RAP (no RAP, 100 % limestone aggregate)

Normally, bitumen bound recycled material is sensitive to aggregate grading, particularly the fines component. In cold recycled mixtures the bitumen has a tendency to mix and conglomerate with the fines while only partially coating the coarse aggregate component [Jenkins et al, 1999]. It has been suggested that the amount passing the 0.075 mm sieve should be at least 5 percent [Ruckel et al, 1982] but not more than 20 percent. Cement bound recycled material, on the other hand, is relatively insensitive to aggregate grading. However, there is a recommended upper limit of 35 percent passing the 0.075 mm sieve.

The composition of limestone aggregate was designed based on Fuller's equation[1] [Fuller and Thomson, 1907], ensuring that the gradation of the aggregate fell into the recommended grading envelopes for recycled pavement aggregate [Milton and Earland, 1999]. The composition of the aggregate used in this study is presented in Table 3.6.

[1] Fuller's equation :
Grading curve with a minimum of voids determined by Fuller and Thomson's experiment

$$p = (d/D)^{0.5} \times 100$$

where, p = total percentage passing the given size
d = size of sieve opening
D = largest size in the gradation

Table 3.6 Mixture composition of limestone aggregate

Nominal size	Percentage on aggregate mix by mass
20 mm	26
14 mm	15
10 mm	9
6 mm	11
Dust	36
Filler	3

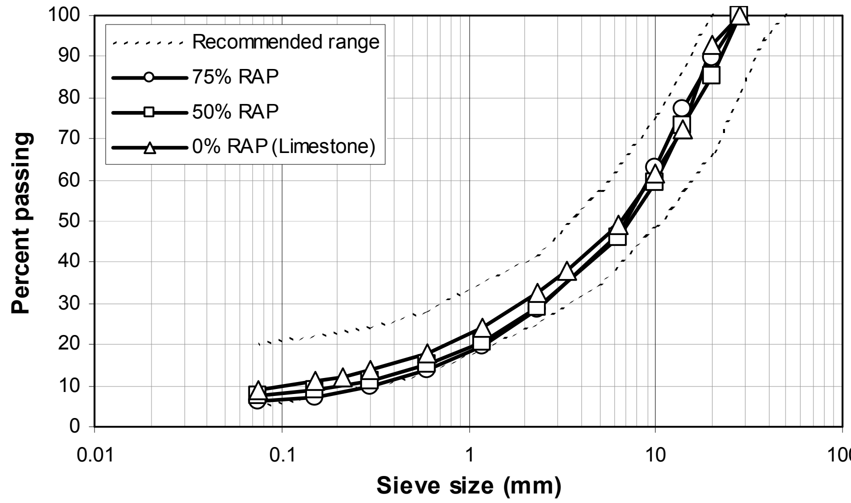


Figure 3.8 Particle size distribution of studied blends

3.3.2 Compaction Test

The modified Proctor test [BSEN 13286-2:2004] was performed to determine the maximum dry density (MDD) and the optimum moisture content (OMC) of all studied blends.

<u>RAP and limestone aggregate</u>

The compaction test results for RAP and limestone blends are shown in Figure 3.9 and listed in Table 3.7. At higher RAP content, the mixture did not show a distinct peak for optimum moisture content. This could be partially attributed to the open graded nature of the RAP aggregate. The water-holding ability of RAP is low and the water was draining out at the bottom of the mould when compacted at high moisture content. The addition of more limestone aggregate made compaction and handling much easier. It was also observed that during compaction, the large RAP particles were broken.

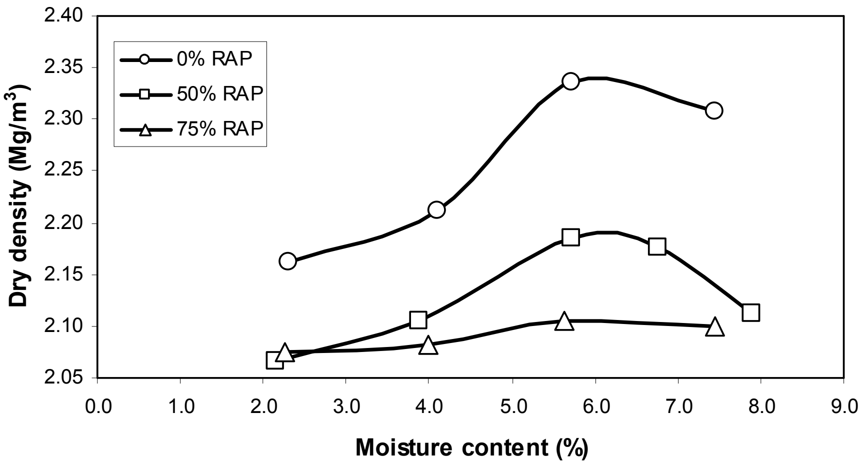


Figure 3.9 Relationships between moisture content and dry density for all studied blends

Table 3.7 Optimum moisture content and maximum dry density of studied blends

Mixture	Optimum moisture content (%)	Maximum Dry density (Mg/m^3)
0% RAP	6.1	2.35
50% RAP	6.1	2.18
75% RAP	6.0	2.11

RAP and limestone aggregate with cement

Various blends were prepared using 0, 4, 6, and 8 % cement by weight of dry aggregate with 0% RAP, 50% RAP and 75% RAP. It was necessary to include the cement in the determination of the OMC because it constitutes part of the aggregate grading. It also affects the moisture demand and hence the density achieved in the test. A summary of the optimum moisture content and maximum dry density values is presented in Table 3.8.

Table 3.8 Optimum moisture content and maximum dry density

Percent cement	0% RAP		50% RAP		75% RAP	
	OMC (%)	MDD (Mg/m^3)	OMC (%)	MDD (Mg/m^3)	OMC (%)	MDD (Mg/m^3)
0	5.6	2.278	6.1	2.180	6.0	2.110
4	5.8	2.315	5.0	2.250	6.4	2.171
6	5.5	2.325	5.8	2.310	6.0	2.190
8	5.0	2.330	6.0	2.320	6.1	2.220

3.3.3 Mix design of foamed bitumen stabilised mixture

Mix design was performed to establish the optimum foamed bitumen content of each mixture. In order to optimise the foamed bitumen content, various foamed bitumen contents were added to the blended aggregate. Bitumen PG70/100 was used to produce foamed bitumen for mix design purposes. A mechanical mixer (Hobart mixer) was attached to the WLB-10 so that the foam could be discharged directly into the mixing bowl of the mixer as shown in Figure 3.10.

Figure 3.10 Hobart mixer

Prior to mixing with the foamed bitumen, water was added to bring the material to a suitable moisture content for compaction. The optimum moisture content for foamed stabilised materials is different from that of soil or granular material because both water and bitumen within the mix affect the densification process. Various proposals on the most suitable moisture content to ensure the best mixing and compaction results have been reported [e.g., Lee, 1981, Bissada, 1987, Ruckel et al, 1983, Castedo and Wood, 1983]. Generally, the most suitable moisture content is reported to lie in the range of 65 – 85 % of OMC. In this study, the added water content was calculated as the following empirical formula [Wirtgen, 2001].

$$W_{add} = 0.7 \times W_{OMC} - W_{moist} + 0.6 \tag{3.1}$$

where,

W_{add}	=	Water content to be added to sample (percent by mass)
W_{OMC}	=	Optimum moisture content (percent by mass)
W_{moist}	=	Moisture content already in the sample (percent by mass)

Dry RAP and limestone aggregate were added, together with a pre-determined amount of water, into the mixing bowl of the mixer. They were mixed for 30 seconds and the foamed bitumen was sprayed directly into the bowl while the mixing was still in progress. The mixing continued for a further 60 seconds.

At each bitumen content, mixtures were prepared in sufficient quantity to allow 6 1200 g cylindrical specimens to be produced. The specimens 100 mm in diameter and 63.5 ± 1.5 mm high were manufactured by applying 75 blows on each side with a Marshall compaction hammer. After compaction, the specimens were cured for 12 hours in the mould at room temperature before extruding by means of an extruding jack. The specimens were then subjected to accelerated curing, which involved oven drying at 40 °C for 72 hours.

Standard indirect tensile strength (ITS) tests were conducted on the specimens in both dry and soaked conditions. The ITS tests were conducted according to BS EN 12697-23:2003 at room temperature (20 ± 5 °C). Test specimens were loaded to failure at a deformation rate of 50.8 mm/minute. The load to failure was recorded and used to compute ITS. The optimum binder content was determined on the basis of the bitumen content that produced the maximum soaked ITS while maintaining a minimum retained ITS value, defined as the ratio between soaked and dry ITS, of typically around 75% [Wirtgen, 2001]. Figure 3.11 shows a typical result of the mix design. Table 3.9 summarises the optimum foamed bitumen contents of the mixtures adopted throughout this study.

Table 3.9 Mix design results of foamed bitumen stabilised materials

	75% RAP	50% RAP	0% RAP
Optimum binder content	2.5 %	2.7 %	4.0 %
Optimum moisture content	6.0 %	6.0 %	6.0 %
Maximum dry density	2.11 Mg/m^3	2.20 Mg/m^3	2.35 Mg/m^3

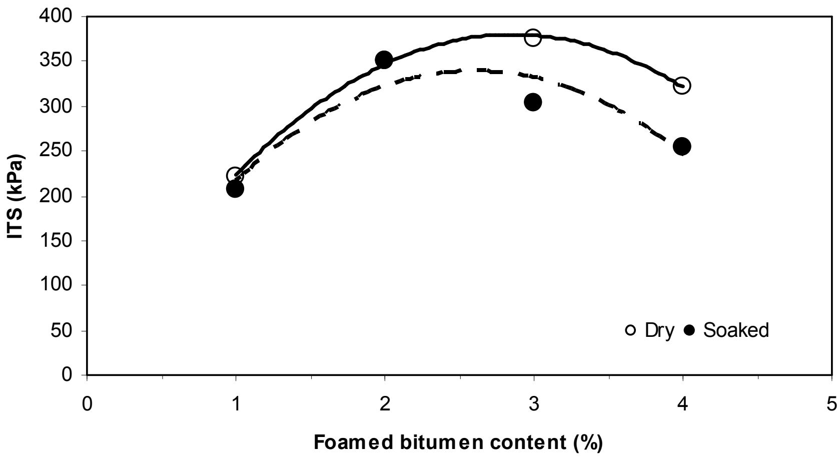


Figure 3.11 Typical mix design results for foamed bitumen stabilised mixtures

3.3.4 Mix design of cement stabilised mixtures

For determination of the cement content, specimens for strength testing were prepared with a range of cement contents. Using the value of OMC and MDD already determined, two 150 mm cubes were made at each cement content using a vibrating hammer in accordance with methods described in BS EN 13286-51:2004. The cubes were then cured at 95% humidity for 7 days and tested in compression in accordance with BS EN 13286-41:2003. The average compressive strength was plotted against cement content as shown in Figure 3.12. The cement content required to achieve the specified strength was selected from this plot. Milton and Earland (1999) recommended a minimum mean cube compressive strength of 4.5 N/mm^2 for low volume roads with traffic of less than 0.5 million standard axles. Based on this criterion, the mix design adopted for all cement stabilised mixtures in this study consisted of 6% cement and 6% moisture content by weight of dry aggregate.

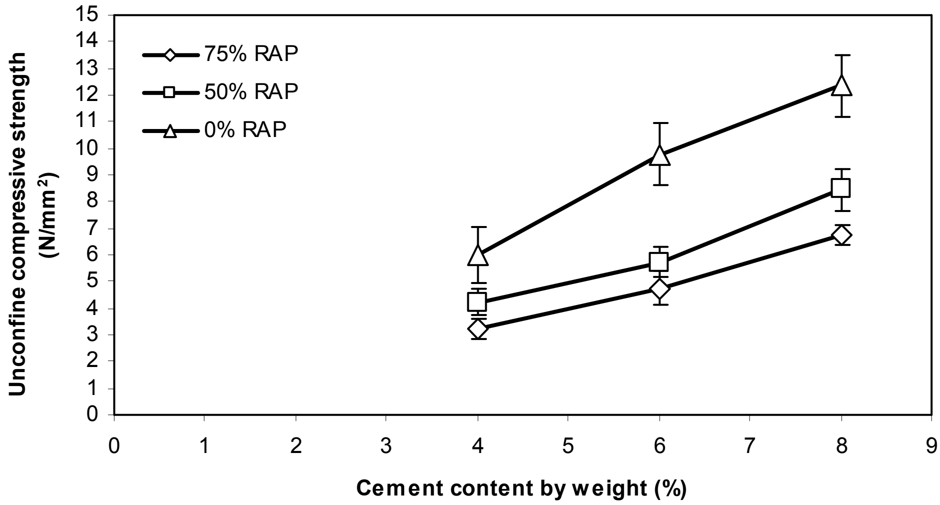


Figure 3.12 Relationship between 7-day cube strength and cement content for specimens compacted at OMC.

3.4 Summary

Recycled asphalt pavement materials in this study were simulated in the laboratory by mixing RAP, limestone aggregate and binder. The main components of simulated recycled asphalt pavement materials were characterised to determine their basic properties. The procedures used to characterise those materials are summarised as follows.

1. The physical properties of the aggregate materials to be used in the investigation, including RAP and limestone, were characterised by gradation, shape, particle density and water absorption. Composition analysis was also conducted for RAP.

2. Foamed bitumen was characterised by its maximum expansion ratio (ER_{max}) and half-life (HL). The temperature and foaming water content to produce the most suitable foam based on ER_{max} and HL were selected.

3. Compaction characteristics of various RAP/limestone blends were determined according to the modified Proctor test procedure.

4. The optimum foamed bitumen content for foamed bitumen stabilised mixtures was judged from the indirect tensile strength (ITS) test results in both dry and soaked conditions.

5. The optimum cement content for cement stabilised recycled asphalt pavement materials was chosen on the basis of cube unconfined compressive strength at 7 days.

The materials described in this chapter were used throughout the investigation unless stated otherwise.

4 DEFORMATION BEHAVIOUR OF FOAMED BITUMEN STABILISED RECYCLED ASPHALT PAVEMENT MATERIALS

4.1 Introduction

In the cold in-place recycling process, the use of foamed bitumen as a stabilising agent has gained popularity over the last decade due to the advance in the technology of cold recycling machines, as well as some perceived advantages of foamed bitumen over other alternative stabilising agents. The benefits of foamed bitumen as a stabilising agent were mentioned in Chapter 1 and will not be repeated again. Instead, an investigation into some technical aspects of foamed bitumen mixtures is the main topic here.

This chapter reports a laboratory study of foamed bitumen stabilised materials subjected to a form of loading which provides an approximate simulation of the traffic loading experienced in actual pavements. The repeated load triaxial mode of testing was employed in this study to mimic the trafficking load. Limestone aggregate and reclaimed asphalt pavement (RAP) were stabilised with foamed bitumen in the laboratory to simulate recycled asphalt pavement materials. The information determined from these triaxial tests consists of permanent and resilient deformations. The results gained from the deformation behavior of this material may allow an approximate preliminary method for design and prediction of the rutting in foamed bitumen stabilised recycled pavements.

4.2 Response of foamed bitumen stabilised materials to traffic load

The first task of the investigation is to address the behaviour of foamed bitumen stabilised materials under traffic loading. Figure 4.1 shows the composition of foamed bitumen stabilised mixtures diagrammatically and their behavioural characteristics and performance [Asphalt Academy, 2002].

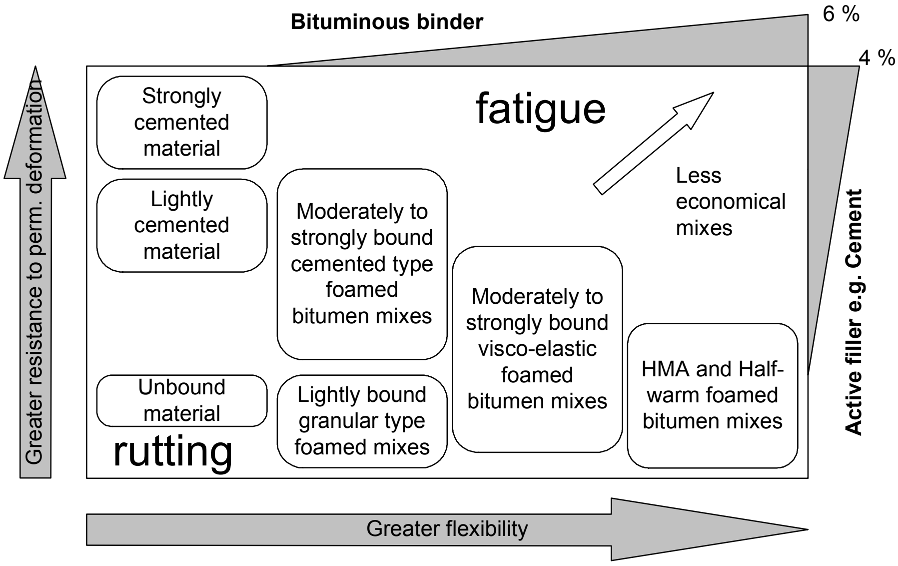


Figure 4.1 Types of stabilised mixtures [Asphalt Academy, 2002]

Usually, the application rate of bitumen in foamed bitumen stabilised materials is in the range of 2.5 to 4.0 % and 1.5 to 3.0 % for crushed stone and RAP/crushed stone blends respectively [Wirtgen, 2001]. As indicated in Figure 4.1, at these amounts of bituminous binder with little or no cementitious additives, foamed bitumen stabilised mixtures tend to fail in rutting. This is probably due to the predominantly granular nature of the mixes. Two essential properties that relate to the performance of this type of material are permanent deformation resistance and resilient modulus. Resistance to permanent deformation is required to prevent rutting. Resilient modulus is required to provide support to the upper layer and also spread load, protecting underlying layers.

When bituminous pavement materials are subjected to a traffic load, which is small compared to their failure strength, a compressive stress pulse is induced in the pavement materials. The stress pulse causes strains. Most of the strains will be recovered when the load is removed, as the materials "spring" back, but a small part will not be recovered. The pavement materials will experience some permanent deformation after each load application. This deformation is of little consequence for a single wheel load but under a large number of load cycles, these small non-recovered strains accumulate to form a measurable permanent strain. The permanent strain induced by traffic load may manifest itself as rutting on the pavement surface.

Resilient modulus is analogous to the elastic modulus, “Young’s modulus”, used in elastic theory. In pavement engineering, resilient modulus is used to describe a material’s transient response to the repeated loading-unloading condition caused by traffic. Resilient modulus is defined as the ratio of the maximum repeated stress to the recoverable elastic (resilient) strain. The elastic modulus based on the recoverable strain under repeated loads is called the resilient modulus.

In this study, the permanent deformation behaviour and resilient modulus of foamed bitumen stabilised materials were experimentally determined by applying a compressive repeated axial load on a material sample mounted inside a triaxial cell.

4.3 The repeated load triaxial deformation test

This experimental investigation of foamed bitumen stabilised materials was aimed at examining the manner in which foamed bitumen stabilised recycled pavement materials respond to stress conditions similar to those imposed by traffic. The permanent deformation behaviour of pavement materials can be evaluated in the laboratory using a number of different methods. The most common tests are the unconfined static uniaxial creep compression test, the repeated load triaxial test and the wheel tracking test. In this study, the repeated load triaxial test was selected, as it is the laboratory test that closely reproduces the stress conditions occurring in a pavement [Brown and Cooper, 1984]. It has the advantage that both vertical and horizontal stresses can be applied at the levels expected in the pavement as shown in Figure 4.2. The repeated load triaxial test can be used to provide an estimate of the permanent deformation potential of soils [Puppala et al., 2005] as well as the elastic properties of pavement materials.

When pavement materials are subjected to repeated axial loading, the deformation strain of the pavement materials can be divided into two components, resilient and permanent. The resilient strain is defined as the peak to peak fluctuating strain component, and permanent strain is defined as the non-recoverable component. The two components of strain are illustrated diagrammatically in Figure 4.3. The total strain is expressed as follows:

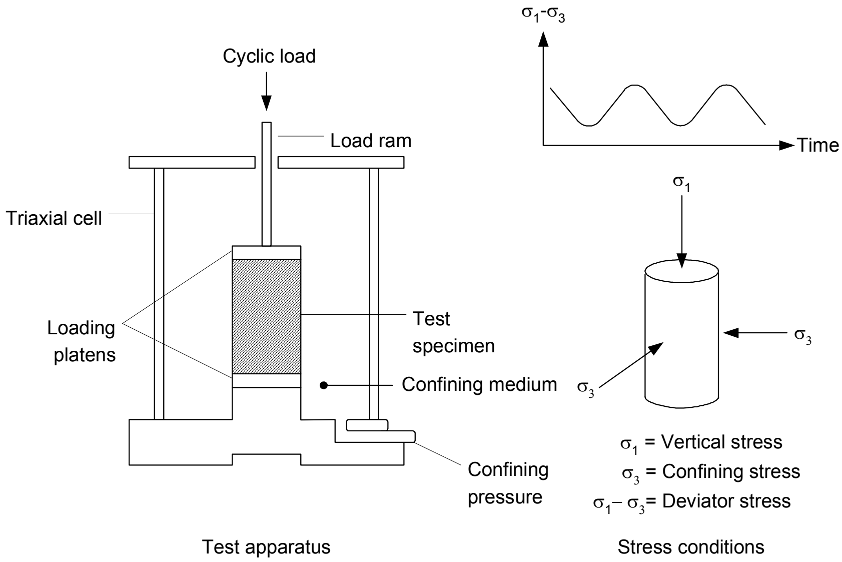


Figure 4.2 The repeated load triaxial test

$$\varepsilon_t = \varepsilon_p + \varepsilon_r \tag{4.1}$$

where,

ε_t = total strain
ε_p = permanent strain
ε_r = resilient strain

The resilient modulus in this study was estimated from the permanent deformation test. The resilient modulus in a repeated load test is defined as the ratio of the maximum cyclic stress to the resilient strain as follows:

$$M_r = \frac{\sigma_c}{\varepsilon_r} \tag{4.2}$$

where,

M_r = resilient modulus (kPa)
σ_c = cyclic stress (kPa)
ε_r = resilient strain

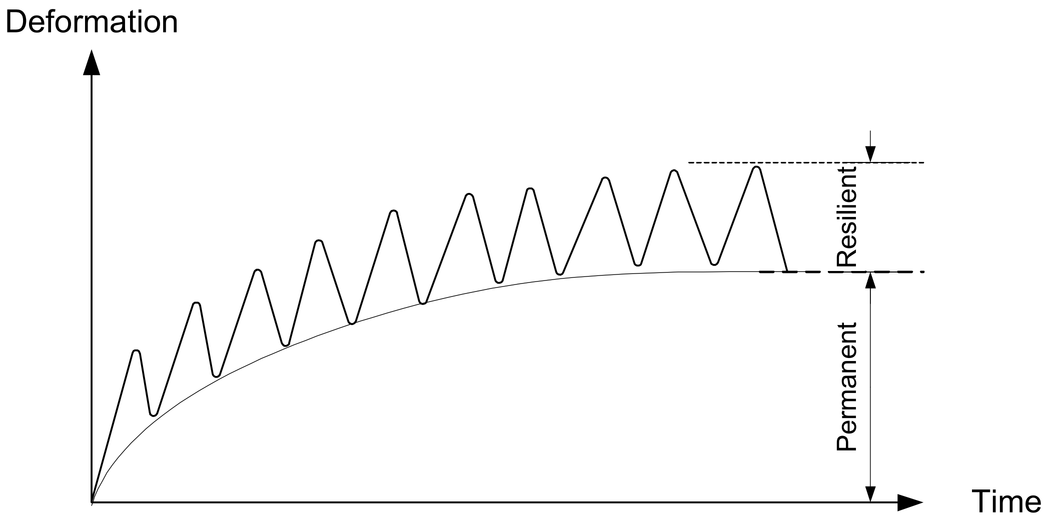


Figure 4.3 Permanent and resilient components of axial strain

4.4 Experimental programme

4.4.1 Description of laboratory tests

Laboratory tests included repeated load triaxial permanent deformation and resilient modulus. Cylinder specimens were prepared in the laboratory and tested in the repeated load triaxial condition to simulate traffic loading. Two independent parameters were selected for examination namely, penetration grade of bitumen generating the foam and proportion of RAP and limestone in the aggregate. Three penetration grade bitumens, PG50/70, PG70/100 and PG160/220, were chosen to stabilise crushed limestone aggregate. RAP was blended with limestone in two proportions, 75%RAP (75% RAP: 25% Aggregate) and 50%RAP (50% RAP: 50% Aggregate), and each was stabilised using PG70/100 grade bitumen. The permanent deformation properties and resilient modulus properties of the various stabilised mixtures were compared.

4.4.2 Test equipment

Figure 4.4 shows a photograph of the triaxial equipment used in this study. The triaxial cell, which is commonly used in soil mechanics, was designed for testing specimens with 100 mm diameter. The cell consists of an aluminum base, a Perspex chamber, a submersible piston and a valve that connects with an air pressure supply. The repeated axial load was applied to the specimen by the use of the piston connected to a servo-hydraulic actuator. Two calibrated LVDTs were mounted on the actuator. Each LVDT's stem has a travel distance of 10 mm and its toe rested on top of the triaxial cell. The axial deformation of the specimen was measured from the displacement of the actuator relative to the cell chamber. A calibrated load cell was used to monitor the applied axial load and provided the feedback signal for controlling the hydraulic actuator which moved vertically until the target load was achieved.

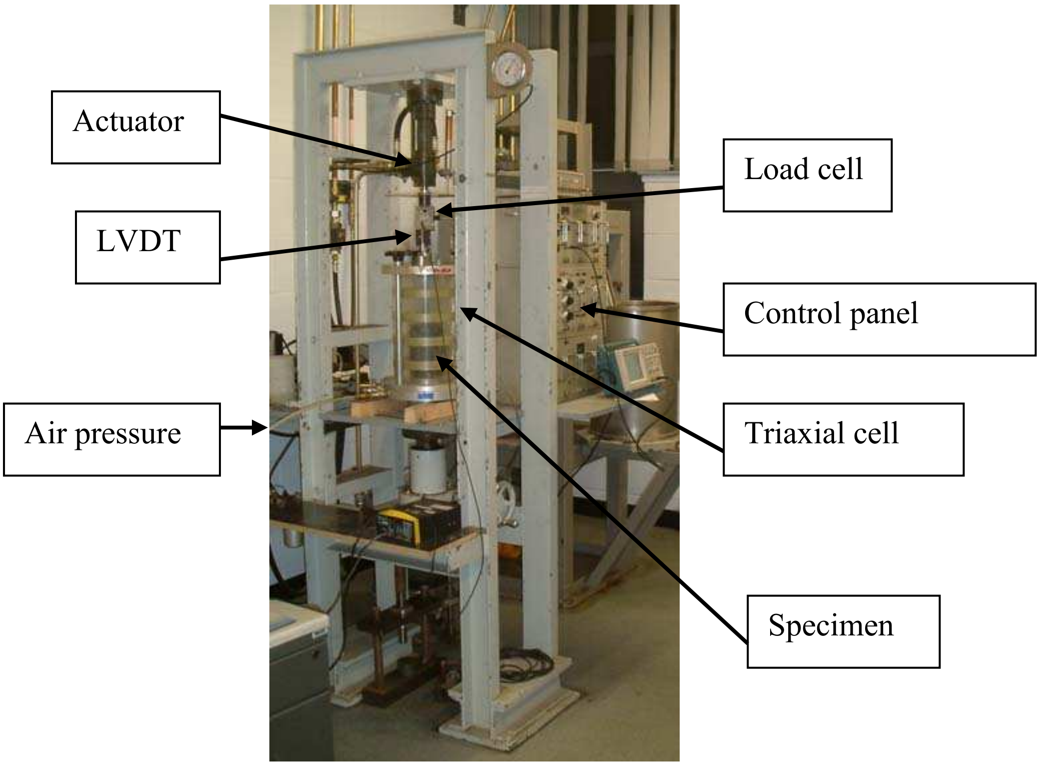


Figure 4.4 The repeated load triaxial test apparatus

4.4.3 Specimen preparation

Cylinder specimens were used for triaxial testing. The diameter of the specimens was 100 mm. The materials were mixed and treated with foamed bitumen as described in Chapter 3. With this foamed bitumen treated material, test specimens were manufactured in a gyratory compactor. Gyratory compaction was chosen because it was believed that this method would yield aggregate structures similar to those set up during the construction of actual pavements. Due to the limitation of the mould's volume, the maximum mass of the mixture to be compacted each time was limited to 1890 g.

Therefore, mixtures were prepared in sufficient quantity to allow eight 1890 g specimens to be produced for each mixture. The gyratory compactor was used with a 100 mm diameter specimen mould. The compaction pressure was 600 kPa and the base rotation rate was 30 revolutions per minute with the mould positioned at an angle of 1.25 degrees (SUPERPAVE). The number of gyrations were selected by trial and error on the basis of bringing the dry density of the specimens close to their corresponding modified Proctor dry density. Different blends were compacted with different numbers of gyrations as given in Table 4.1. The resultant height of the compacted specimens was approximately 100 mm. The compaction was done at ambient temperature. To avoid any effects of long storage period, all specimens were compacted within 2 hours of mixing. Specimens were extracted from their moulds after 12-16 hours and then subjected to accelerated curing by drying in an oven at 40 °C for 72 hours. The density of specimens varied slightly as given in Table 4.2.

Table 4.1 Number of gyrations for compacting foamed bitumen stabilised specimens

Mixture	Number of gyrations
75% RAP	30
50% RAP	70
0% RAP	300

Table 4.2 Variations in dry density of specimens for the triaxial test

Mixture	No. of specimens	Average dry density (Mg/m^3)	Range of dry density (Mg/m^3)	Standard Deviation (Mg/m^3)
Limestone PG50/70	8	2.132	2.067-2.184	0.047
Limestone PG70/100	8	2.116	2.065-2.160	0.050
Limestone PG160/220	8	2.207	2.150-2.265	0.041
50%RAP PG70/100	8	2.092	2.043-2.145	0.043
75%RAP PG70/100	8	2.043	2.003-2.120	0.043

4.4.4 Test procedures

Test set-up

At the beginning, the height of the specimen was measured. The specimen was then placed between two circular platens. The interfaces between the specimen and the platens were lubricated with layers of grease and graphite powder to eliminate friction.

An impervious latex membrane, 100 mm in diameter, was used to seal the specimens from the pressurised air inside the cell. The membrane surrounding the specimen was fastened to the top and bottom platens by rubber O-rings. The specimen, together with platens and rubber membrane, was located on the cell base. The Perspex chamber was placed onto the base and screws were fastened to prevent air leaking out of the cell. The entire cell was then placed under the actuator ready for the test. Figure 4.5 shows the test set-up schematically.

The target confining pressure was applied by an air compressor through a regulating valve and the pressure was read through a pressure gauge. The confining pressure was maintained at a constant value (i.e. no cyclic confining load) for each test condition. After the target confining pressure was achieved, the specimens were conditioned before the test by the application of 1000 cyclic loads at a stress level of 30 kPa and a frequency of 2 Hz. The conditioning was intended to remove irregularities on the top and bottom surfaces of the test specimens. The preloading stress was for seating the platens on the specimen and was immediately followed by the target axial stress. All tests were carried out at room temperature. The axial deformation was measured by the LVDTs and the axial load was measured by the load cell. All data was logged by the computer connected to the equipment.

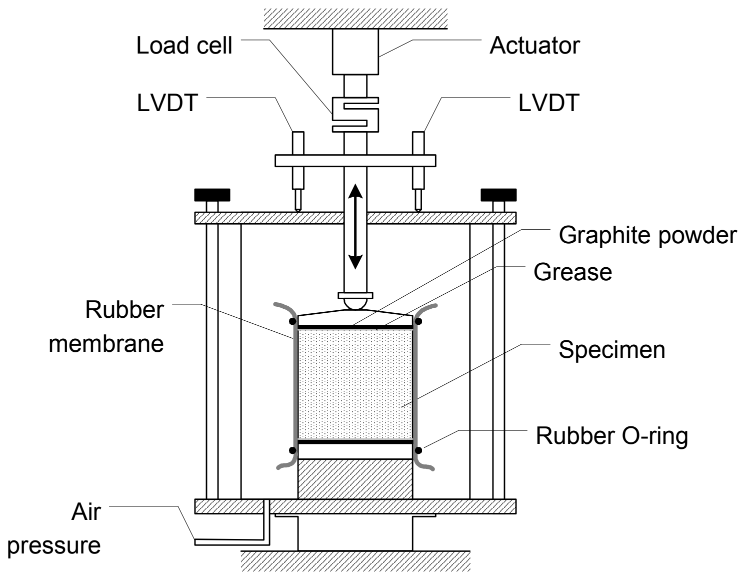


Figure 4.5 Schematic diagram of the triaxial set-up

Permanent deformation test

The permanent deformation test of foamed bitumen stabilised base materials was performed by using a repeated load triaxial compression test. A continuously sinusoidal cyclic load was applied at a frequency of 2 Hz. The term 'cyclic stress' is used to describe the dynamic axial stress pulse applied by the vertical actuator to the specimen. This is in addition to the vertical stress applied to the top of the specimen due to the confining pressure as illustrated in Figure 4.6. In each test, it was intended to apply the load for 70,000 cycles as this can be conducted in a day's testing and it was considered unsafe to operate the hydraulic machine unattended through the night. However, some tests did not reach this target due to problems during the tests. The test was carried out at four vertical cyclic stress levels namely 100, 200, 400 and 800 kPa, and each cyclic stress level was applied on a different specimen. A confining stress level of 50 kPa was maintained. The confining pressure value was selected on the basis of a stress analysis conducted to compute a field representative stress condition in the base layer. During the test, the two LVDTs measured the displacement and a load cell measured the applied loads continuously throughout the 70,000 cycles. At appropriate intervals, readings were manually initiated and recorded. Generally, the readings were conducted at 10, 100, 200, 300, …, 900, 1000, 2000, 3000, …, 9000, 10000, 20000, …, and 70000 load repetitions. At each reading, measurements were recorded for 5 seconds, equivalent to 10 load cycles. The representative values of each reading were taken to be the average of those 10 cycles. During the test, if excessive deformation was observed, the test was terminated.

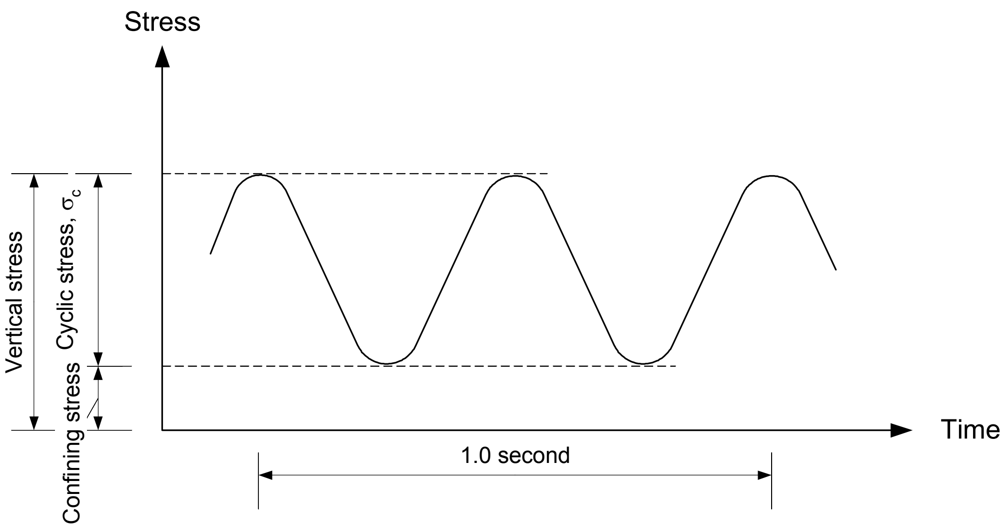


Figure 4.6 Vertical stress pattern

<u>Resilient modulus determination</u>

The resilient modulus in this study was estimated from the permanent deformation test. A separate set of resilient modulus tests was also performed in order to investigate the effect of stress level on the resilient modulus of foamed bitumen mixes. To enable the resilient modulus to be determined as a function of the stress condition, the tests were performed over a range of confining and cyclic stresses as given in Table 4.3. In a single test, a specimen was subjected to all stress conditions and 100 cycles of each cyclic stress level was applied to the same specimen. The calculated resilient modulus value of each test condition was the average over a range of the last 10 cycles.

Table 4.3 Stress combination for resilient deformation test

Confining stress (kPa)	Cyclic stress (kPa)		
100		100 (Conditioning)	
25	25	50	75
50	50	100	150
75	75	100	150
100	75	150	200
150	100	150	250

4.5 Results

4.5.1 Permanent deformation

The results of permanent longitudinal strains were plotted as a function of number of load cycles (*N*) on a log-log scale for all foamed bitumen stabilised materials under different test conditions (stress levels, bitumen types and mix proportions). The increase in the permanent strain with the number of load cycles for various loading conditions is shown in Figure 4.8 to Figure 4.12. The influence of the magnitude of the vertical cyclic stress on permanent axial strain at any stage in the test is shown in these figures. It is obvious from the figures that, as expected, an increase in the stress level increases the permanent deformation substantially. The accumulation of permanent deformation is generally proportional to logarithmic number of repeated load cycles. However, in the initial phase of some specimens, permanent deformation took place at a rapid rate and can be attributed to seating of loading plates and initial permanent strain due to bedding-in, seating of loading plates and initial permanent strain due to aggregate rearrangement and consolidation. Afterwards, the rate of deformation started to stabilise under repeated loading and is characterised by a constant rate of deformation. The relationship between accumulation of permanent deformation and load repetitions is approximately linear. An accelerated accumulation of permanent deformation after the steady stage was also observed. This latter phase, however, may not always be reached, particularly at the lower cyclic stresses. If this phase occurs, the material may be considered to be failing.

In this study, specimen failure was identified as an accelerated rate of increase in the axial deformation. All materials failed when subjected to a cyclic stress of 800 kPa. The failures were sudden and the specimens crumbled apart. For some specimens subjected to a cyclic stress of 400 kPa, an increase in the axial deformation was observed but the specimens did not collapse. In this case, vertical cracks were visible on the surface of the failed specimens. The appearance of specimens at failure is shown in Figure 4.7. It is noted, from the figure, that the approximately uniform deformation of the specimen at failure indicated that grease and graphite layers effectively reduced the friction between specimen and load platens.

The effect of confining pressure on the permanent deformation strain is shown in Figure 4.13. It is clear from the figure that confining pressure significantly enhanced deformation resistance. There was, however, little difference in the result for the two specimens tested at the 50 and 100 kPa. It is therefore not clear that increasing the confining stress from 50 to 100 kPa affects the deformation resistance of the mixture.

The effect of the bitumen type generating the foam on the permanent axial strain is shown in Figure 4.15. Three penetration grade bitumens, PG50/70, PG70/100 and PG160/220, were used for production of foamed bitumen. The results show that the softer bitumen generating the foam yielded the greater permanent deformation. At lower cyclic stress, the permanent deformation in specimens stabilised with foamed bitumen PG50/70 and PG70/100 giving similar strain, while the deformation of the specimen stabilised with foamed bitumen PG160/220 was still the highest. The result indicates that harder bitumen may help the foamed bitumen stabilised mixtures to lower their susceptibility to permanent deformation.

The effect of the proportion of RAP in the mix can be seen in Figure 4.14. At low cyclic stress, the permanent axial strains of all specimens were approximately comparable. The mix containing 75% RAP showed the poorest resistance to permanent deformation under higher cyclic load. However, the permanent axial strain of the mixes containing 50 % and 0 % RAP were still shown to be similar at the higher cyclic stress. The result implies that a RAP content of up to 50%, mixed with crushed limestone in foamed bitumen mixes, does not significantly affect the permanent deformation susceptibility of the mixes.

(a) (b)

Figure 4.7 Specimen failure (a) at cyclic stress = 400 kPa (b) at cyclic stress = 800 kPa

<u>A model for permanent deformation</u>

The permanent strain can be fitted to a model that assumes the accumulated permanent strain (ε_p) is proportional to a power of the number of repeated load cycles (N) as shown in Equation 4.3. This relationship was originally proposed for predicting permanent deformation of granular materials [Sweere, 1990].

$$\varepsilon_p = aN^b \tag{4.3}$$

where,

ε_p	=	cumulative permanent strain (%)
N	=	number of load cycles
a and b	=	material parameters for a given set of cyclic and confining stresses

The permanent deformation test data was fitted to Equation 4.3 and the parameters *a* and *b* and the correlation value (R^2) were calculated from a regression analysis. The results of the analysis for the model are presented in Table 4.4. In most cases, a high correlation ($R^2 > 0.9$) was found.

Table 4.4 Results of permanent deformation test

Materials	Cyclic stress	*N*	*a*	*b*	R^2
Limestone + PG50/70	100 kPa	40000	0.1019	0.0835	0.838
	200 kPa	70000	0.2509	0.0480	0.845
	400 kPa	50000	0.1321	0.1050	0.927
	800 kPa	70000	Specimen failed		
Limestone + PG70/100	200 kPa	70000	0.1956	0.0815	0.980
	400 kPa	30000	0.2270	0.1082	0.955
	800 kPa	30000	Specimen failed		
Limestone + PG160/220	100 kPa	50000	0.0892	0.1660	0.889
	200 kPa	50000	0.3575	0.0852	0.887
	400 kPa	70000	Specimen failed		
	800 kPa	10000	Specimen failed		
50% RAP + PG50/70	100 kPa	70000	0.0805	0.1003	0.908
	200 kPa	70000	0.2930	0.0697	0.995
	400 kPa	70000	0.2876	0.0817	0.986
	800 kPa	10000	Specimen failed		
75% RAP + PG50/70	100 kPa	70000	0.1012	0.0796	0.954
	200 kPa	70000	0.1452	0.1284	0.978
	400 kPa	70000	Specimen failed		
	800 kPa	3000	Specimen failed		

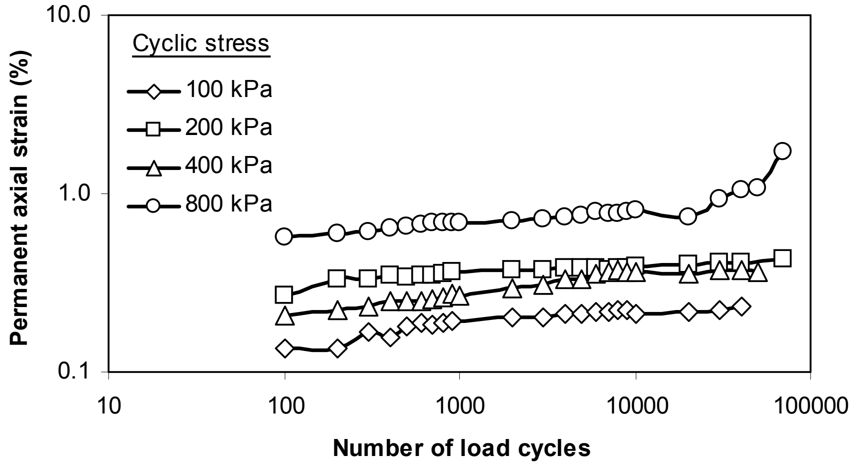


Figure 4.8 Permanent strains of foamed bitumen (PG50/70) stabilised crushed limestone

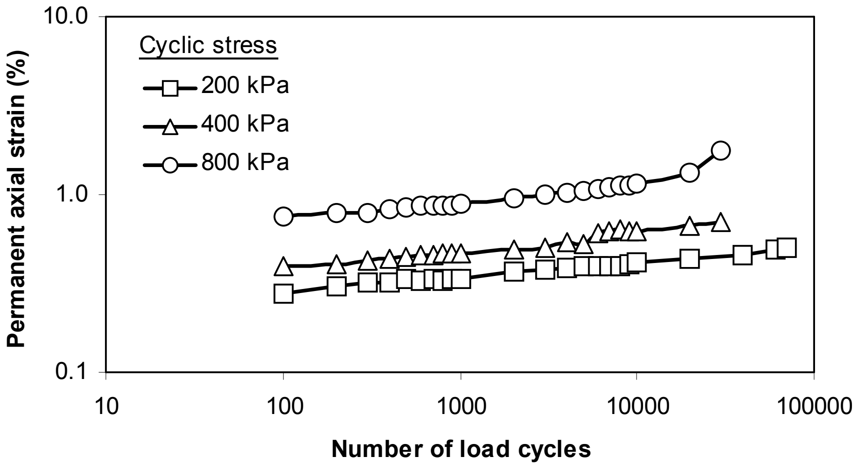


Figure 4.9 Permanent strains of foamed bitumen (PG70/100) stabilised crushed limestone

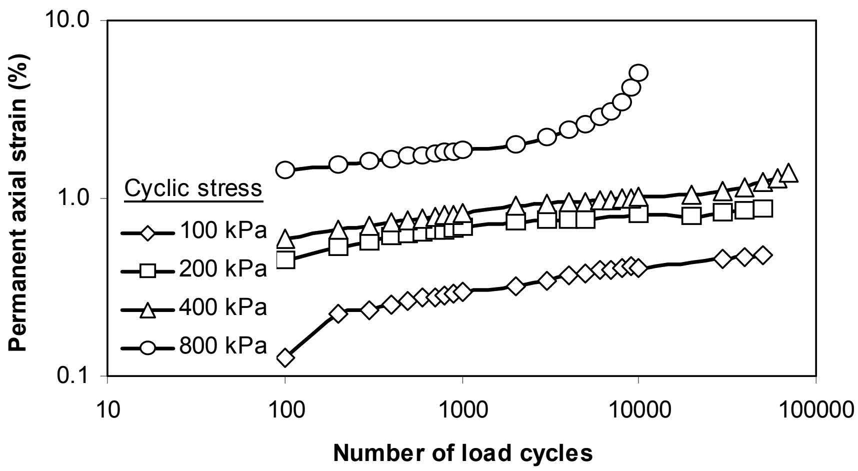


Figure 4.10 Permanent strains of foamed bitumen (PG160/200) stabilised crushed limestone

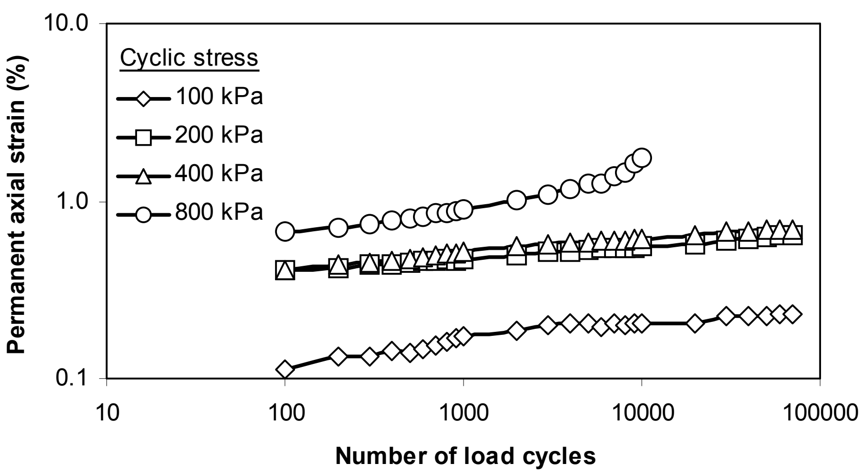


Figure 4.11 Permanent strains of foamed bitumen (PG50/70) stabilised 50%RAP mixtures

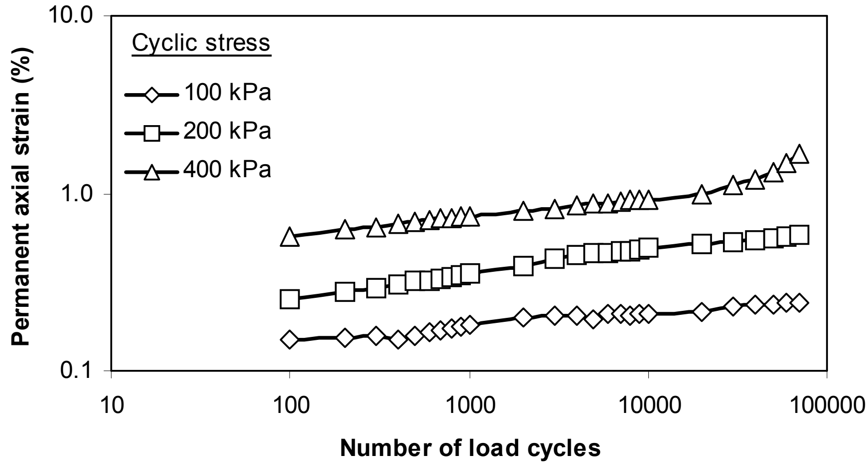


Figure 4.12 Permanent strains of foamed bitumen (PG50/70) stabilised 75%RAP mixture

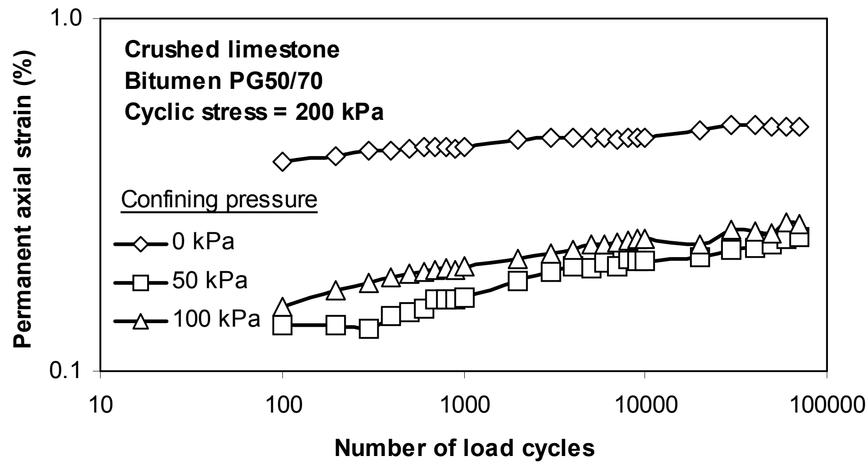
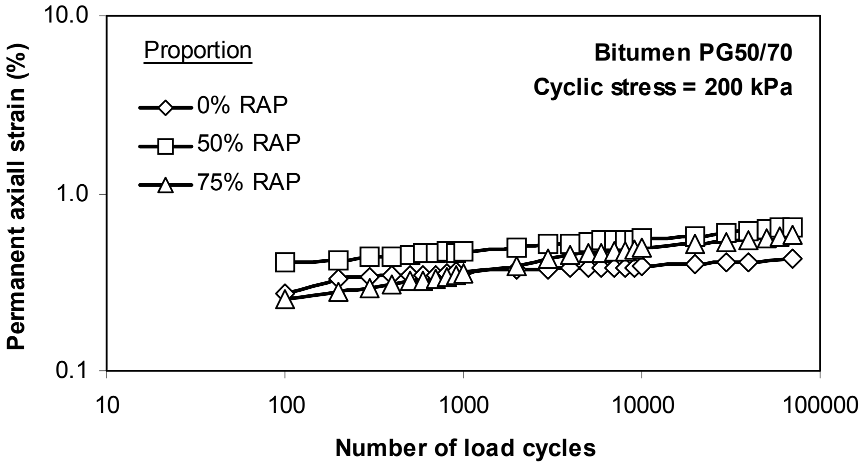
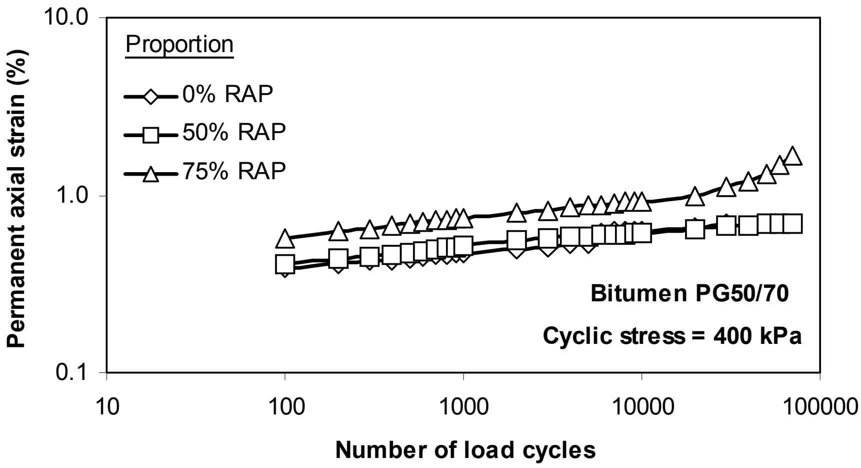


Figure 4.13 Influence of confining pressure on permanent strain of foamed bitumen stabilised crushed limestone

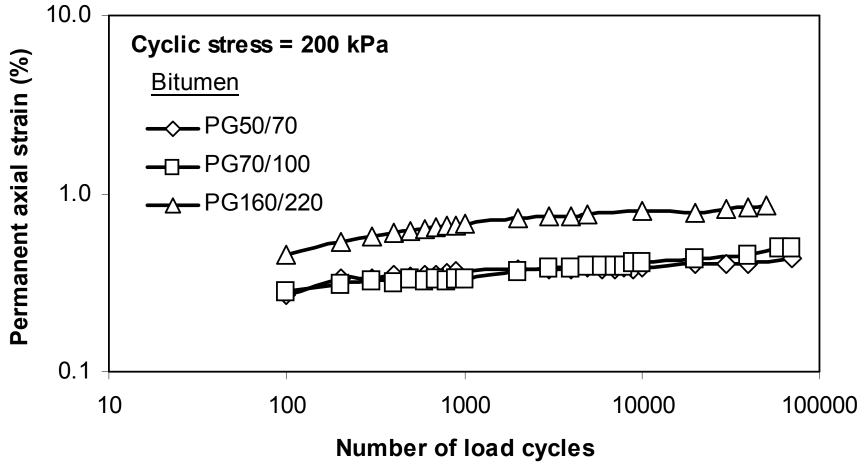


(a)

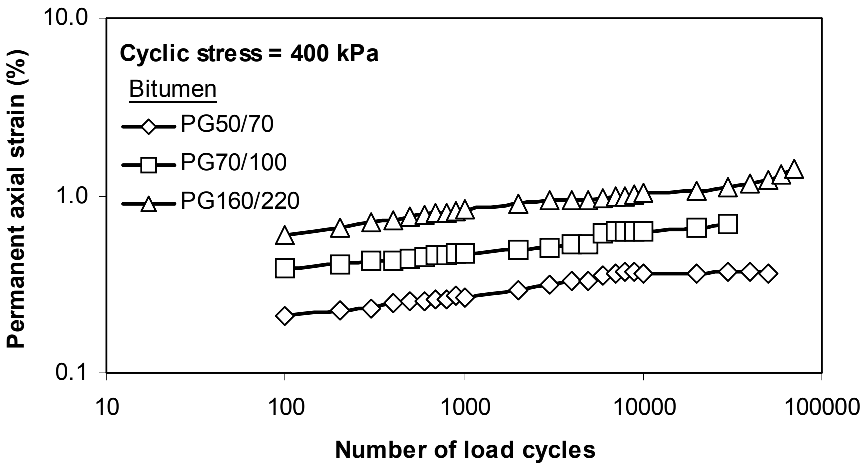


(b)

Figure 4.14 Influence of RAP proportion on permanent strain of foamed bitumen stabilised mixtures (a) cyclic stress = 200 kPa (b) cyclic stress = 400 kPa



(a)



(b)

Figure 4.15 Influence of bitumen type generating the foam on permanent strain of foamed bitumen stabilised crushed limestone

4.5.2 Resilient modulus

The resilient modulus in this study was estimated from the permanent deformation test. In the permanent deformation test, load and vertical deformation were acquired at appropriate intervals throughout the test. Permanent strain, resilient strain, and cyclic stress values were then determined. It is possible to estimate resilient modulus from the stress and strain data taken during a permanent deformation test as previously described. The variations of resilient modulus of foamed bitumen stabilised materials with number of load cycles and cyclic stress levels are presented in Figure 4.16 through to Figure 4.21.

Typically, during the first 10,000 cycles of most tests, there is an increase in the resilient modulus. This increase may be attributed to particle rearrangement and consolidation of the materials under repeated loading. After this stage, material reached a steady state. In the steady state, resilient modulus remains at an approximately constant level. Within the range of 70,000 load cycles, the resilient moduli of all materials were at an approximately constant level unless the materials collapsed. If the specimen failed, a steep drop in resilient modulus was observed.

Generally, the resilient modulus of the mixtures increased with an increase of cyclic stress level. The dependence on stress level indicates that foamed bitumen stabilised materials behave like granular type materials.

The effect of bitumen type generating the foam on the resilient modulus of foamed bitumen stabilised materials can be seen in Figure 4.20. It was found that the mix stabilised with foamed bitumen produced from the softest bitumen PG160/220 give the highest resilient modulus compared with the other two mixes. The mixes stabilised with PG50/70 and PG70/100 produced similar results. However, as indicated in Table 4.2, the average dry density of specimens stabilised with foamed bitumen PG160/220 was higher than the other mixes. The high density contributed to the high resilient modulus value. Therefore it cannot be concluded that bitumen type has any clear influence on the resilient modulus of the mixes.

Figure 4.22 shows the effect of proportion of RAP on the resilient modulus of the mixtures. The results did not show a consistent trend. At low cyclic stress the resilient modulus of the mix containing lower percentage of RAP is higher than the mix containing more RAP, while at the higher cyclic stress the inverse is true.

Figure 4.23 shows the influence of confining pressure on the resilient modulus of foamed bitumen stabilised materials. There is an obvious increase in resilient modulus when the confining pressure was applied. However, an increase in confining pressure from 50 kPa to 100 kPa did not seem affect the resilient modulus of the mixtures.

To illustrate the effect of stress condition on the resilient modulus of foamed bitumen stabilised mixes, specimens were subjected to various stress conditions in a single test and the resilient modulus was measured for each stress condition. The resilient modulus was plotted against the sum of the principal stresses $\theta = 3\sigma_3 + \sigma_c$, where σ_3 = the confining stress and σ_c = cyclic stress, as shown in Figure 4.24. It can be seen that the resilient modulus of the crushed limestone stabilised with foamed bitumen PG50/70 showed a slight stress dependency, with the modulus increasing as the sum of principal stresses increases. The result from the specimen stabilised with foamed bitumen PG160/220, on the other hand, did not seem to exhibit a stress dependency property. The stress dependant behaviour is that of granular materials and is expected for foamed bitumen stabilised mixtures as well. The lack of dependence of resilient modulus on the sum of the principal stresses indicates that the behaviour of these specimens is comparable to that of bound materials. In this case, the specimen was stabilised with 4 % foamed bitumen, which is a relatively high binder content. That the specimen stabilised with foamed bitumen produced from a soft penetration grade PG160/220 bitumen was independent of the sum of the principal stresses might imply that the binder was more thoroughly mixed with the aggregate and hence the mixture is closer to being a ‘bound’ material. It should be noted that the higher resilient modulus of the mix containing PG160/220 bitumen might be attributed to the higher density of the specimen.

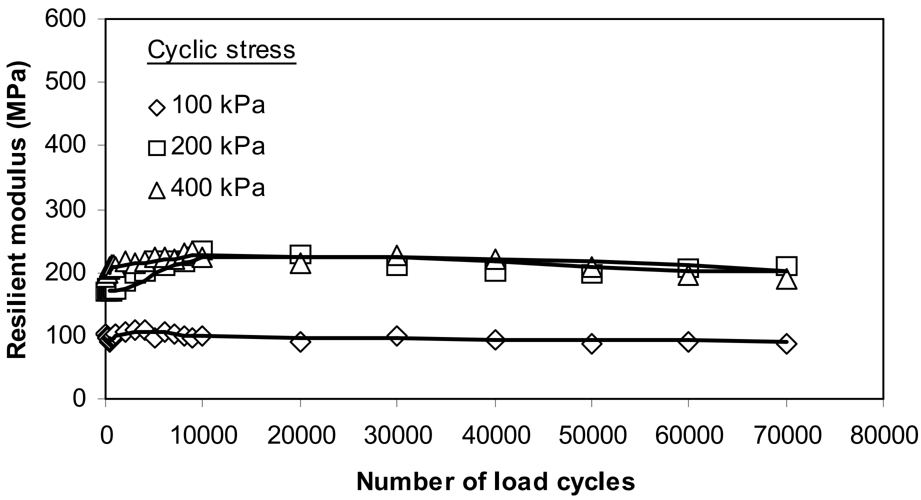


Figure 4.16 Resilient modulus of foamed bitumen (PG50/70) stabilised crushed limestone

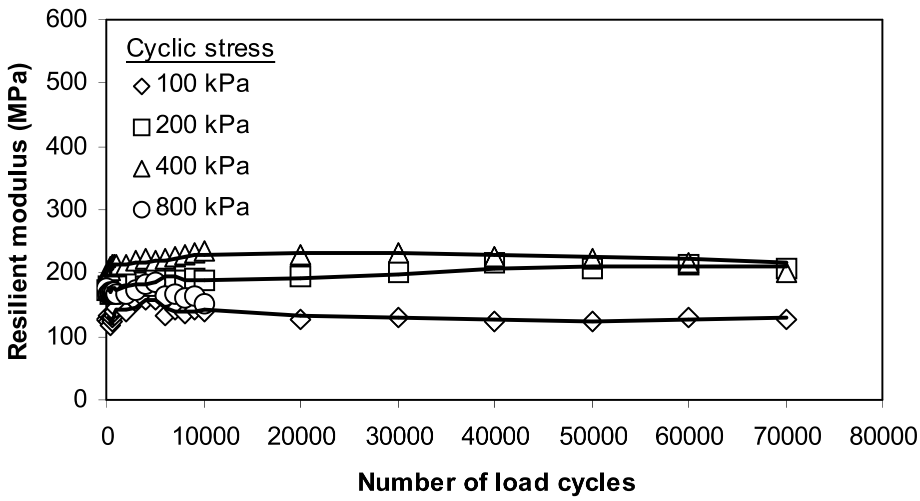


Figure 4.17 Resilient modulus of foamed bitumen (PG70/100) stabilised crushed limestone

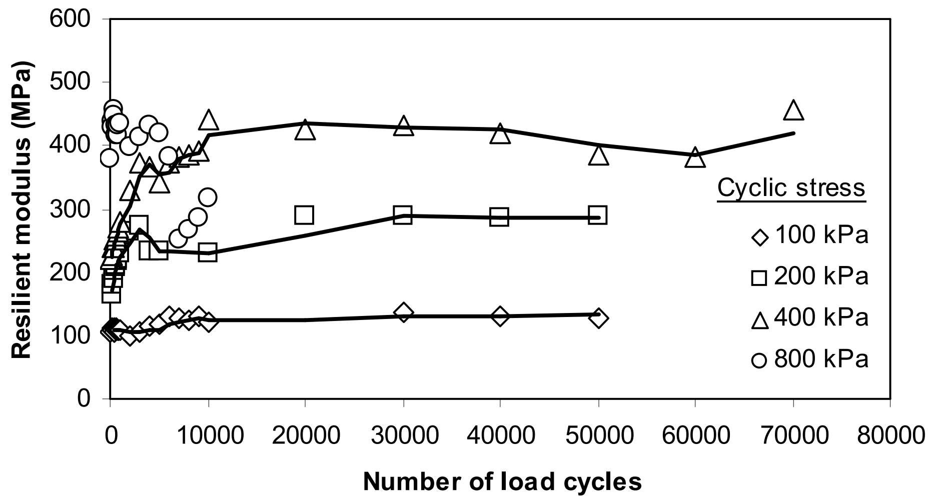


Figure 4.18 Resilient modulus of foamed bitumen (PG160/220) stabilised crushed limestone

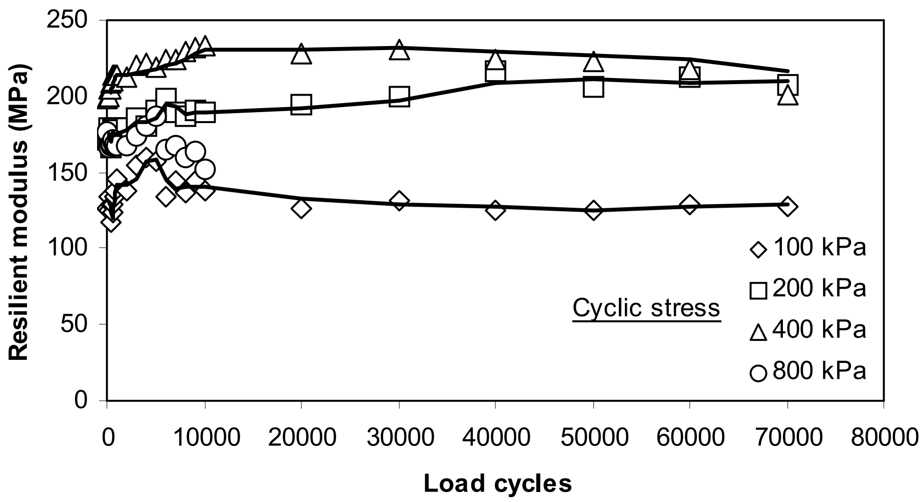


Figure 4.19 Resilient modulus of foamed bitumen (PG50/70) stabilised 50%RAP mixture

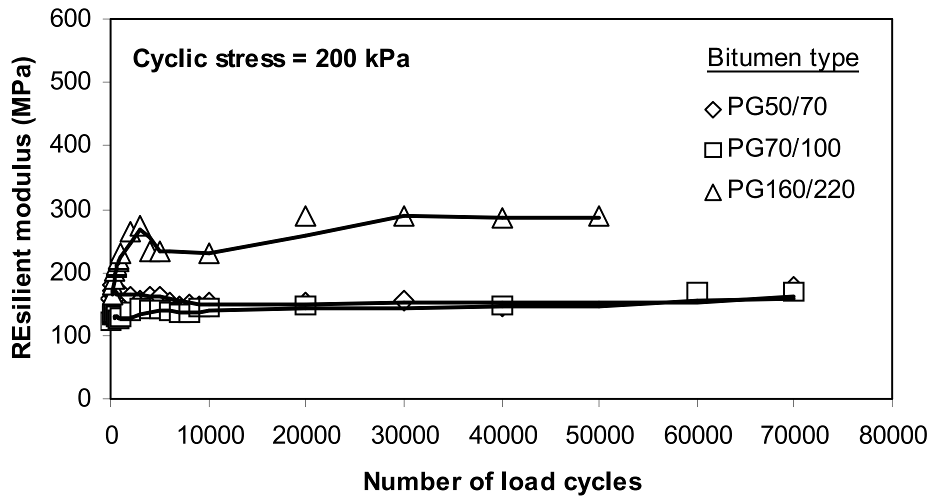


Figure 4.20 Influence of bitumen type generating the foam on resilient modulus of foamed bitumen stabilised crushed limestone

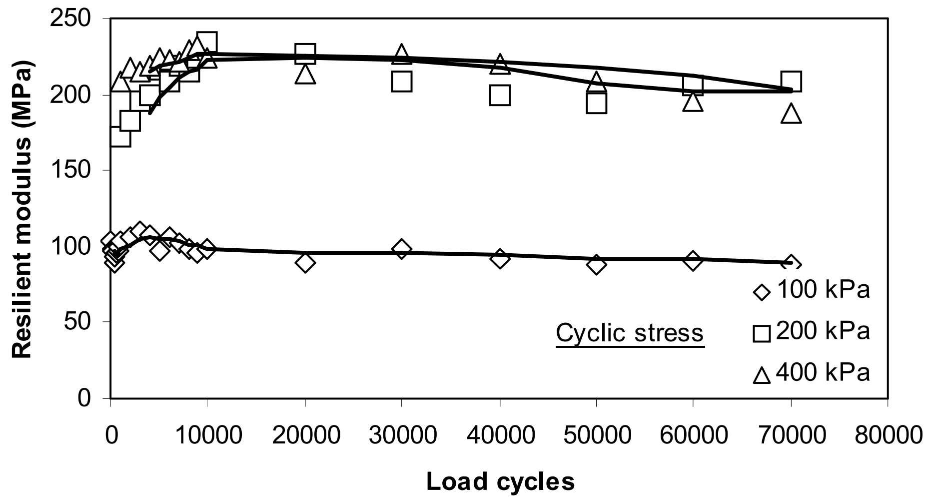
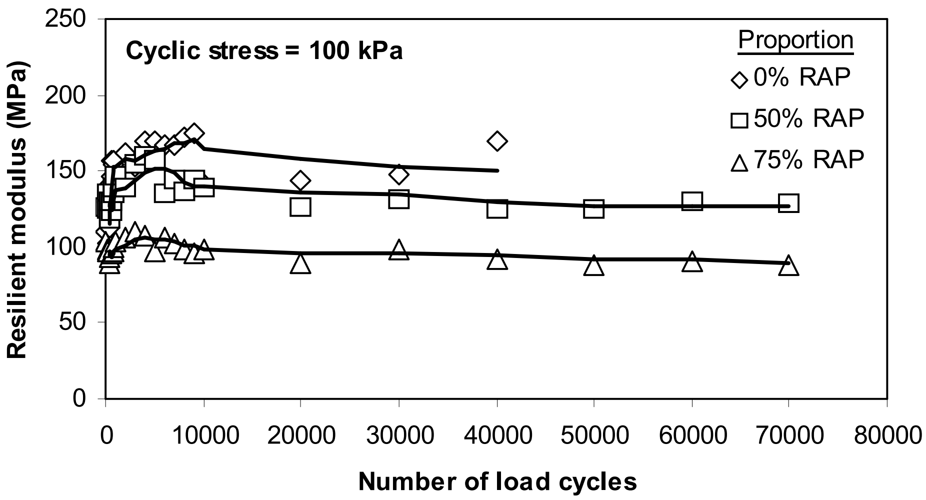


Figure 4.21 Resilient modulus of foamed bitumen stabilised 75% RAP mixture



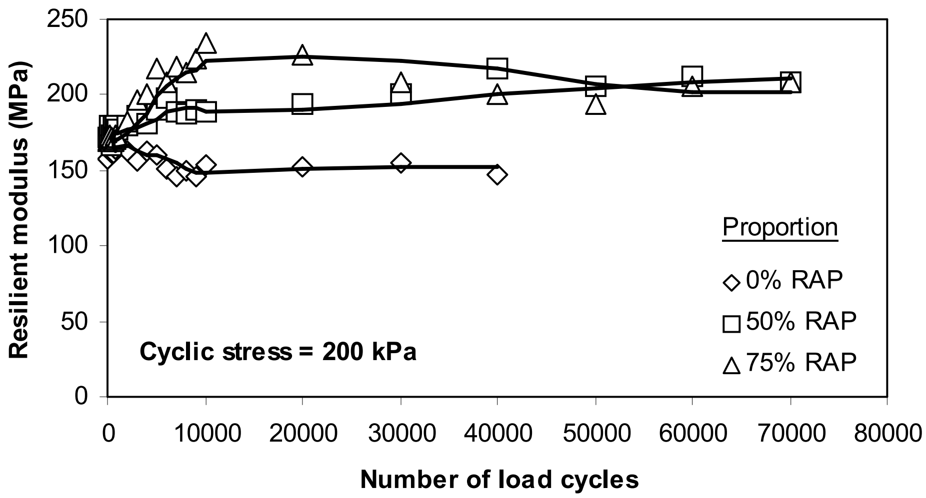


Figure 4.22 Influence of RAP proportion on resilient modulus of foamed bitumen stabilised mixtures

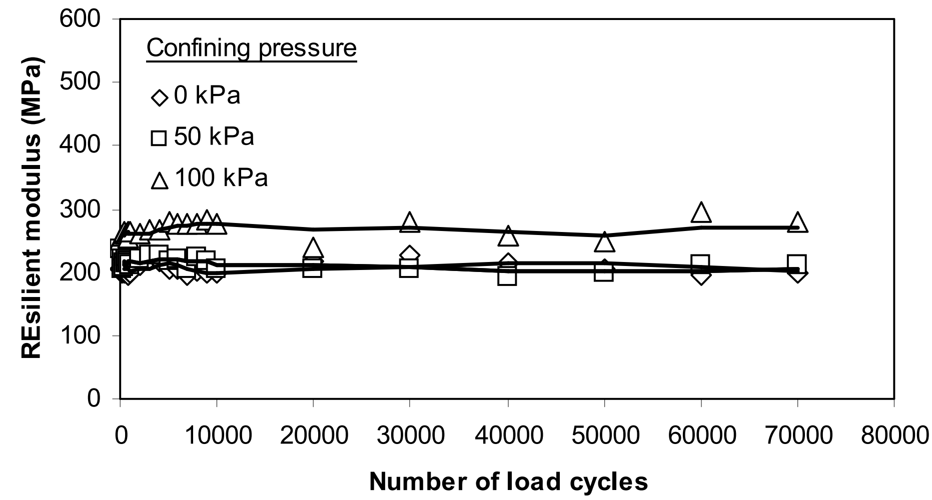


Figure 4.23 Influence of confining pressure on resilient modulus of foamed bitumen (PG50/70) stabilised crushed limestone

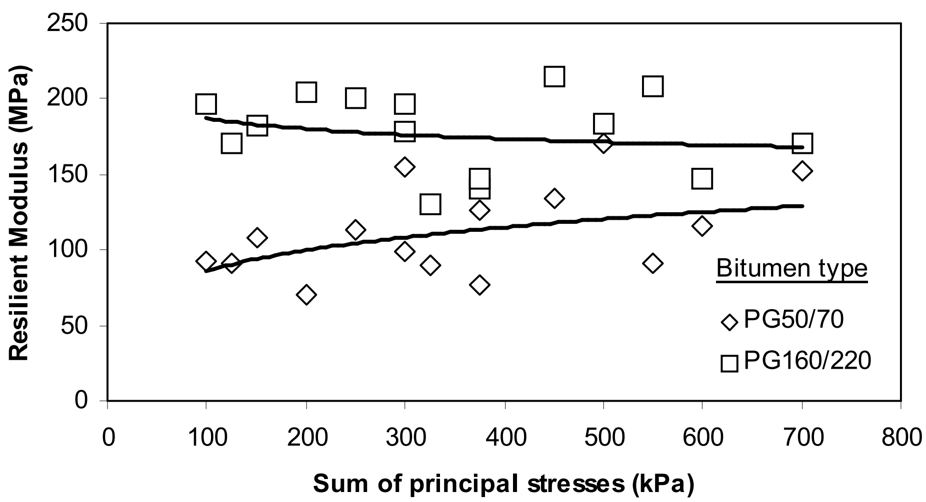


Figure 4.24 Resilient modulus as a function of total stress in triaxial condition.

A model for resilient modulus

An attempt was made to fit the resilient modulus test results into a model. A widespread and simple approach relating the resilient modulus to the sum of the principal stresses is the so-called K-θ model as expressed in Equation 4.4 [Seed et al, 1967].

$$M_r = k_1 \theta^{k_2} \tag{4.4}$$

where,

M_r = Resilient modulus
θ = Sum of the principal stresses
k_1, k_2 = Material parameters

In this study, there were two sets of modulus values, one from the long duration permanent deformation test and another from the short duration resilient modulus test. All resilient modulus values from the tests were plotted against their corresponding sum of the principal stresses as shown in Figure 4.25. The modulus values from long duration tests were taken after 40000 – 70000 load cycles and the modulus values from short duration test were taken after 100 load cycles. It can be seen from the figure that the resilient modulus from short duration tests seems to be independent of the sum of the principal stresses while the resilient modulus values from long duration tests show a slight stress dependency behaviour. Hence, only the latter set of resilient modulus results was used in the analysis. The parameters k_1 and k_2 were found to be 8.0587 and 0.535 respectively. However, the model gave a poor fit to the data as indicated by the low R^2 value of only 0.40. The values of resilient modulus were also lower than expected.

Jenkins (2000) stated that conditioning of foamed bitumen stabilised specimens before testing in the triaxial test has a profound effect on the magnitude of resilient modulus and behaviour at different stress levels. Exposure to at least 10000 load pulses changes the resilient deformation from stress-independent to stress-dependent behaviour. It highlights the need to extend a large number of conditioning load cycles to simulate field conditions in order to obtain representative results. The high content of foamed bitumen (4%) may also have contributed to the lack of stress dependent behaviour of the materials [Jenkins, 2000].

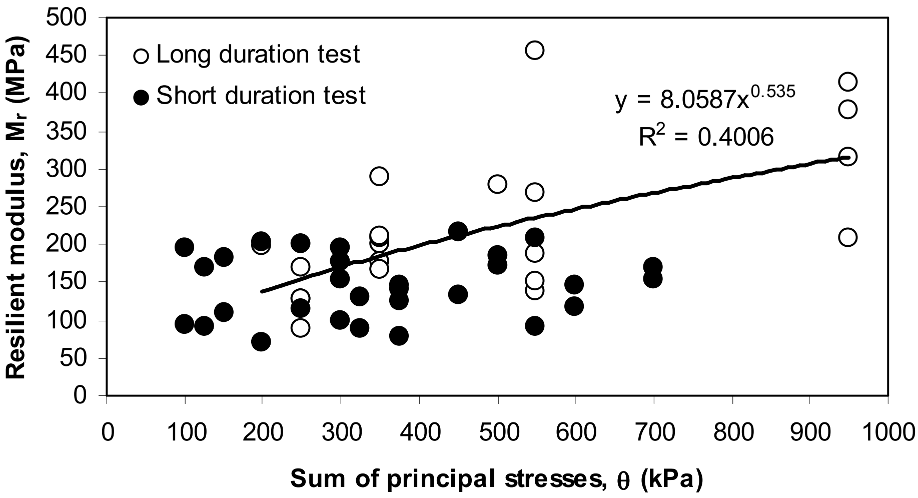


Figure 4.25 Resilient modulus from long duration and short duration tests as a function of sum of the principal stresses.

4.6 Discussion

The focus of this chapter is the laboratory evaluation of permanent deformation and the resilient modulus properties of foamed bitumen stabilised materials.

Foamed bitumen stabilised materials possess the typical characteristics of permanent deformation of granular type materials under repeated loading. They exhibited initial aggregate rearrangement and consolidation with the material undergoing a hardening stage, followed by a steady stage or constant rate of deformation under repeated loading. Failure can be indicated by an accelerated deformation.

The variation of resilient modulus with number of load cycles is somewhat similar to that of permanent deformation. The resilient modulus increased during the first stage of loading and then reached a steady state in which resilient modulus remained at an approximately constant level. As the specimens approached failure, the resilient modulus decreased with increasing permanent strain at a rapid rate.

Nevertheless, the resilient modulus and permanent deformation strain do not necessary follow the same trend. On the basis of resilient modulus test results, the mix stabilised with soft bitumen might exhibit high resilient modulus but on the basis of permanent deformation results, the softer bitumen yielded greater permanent deformation. The resilient modulus of foamed bitumen stabilised mixes also depended on stress level. At high cyclic stress, the results showed that resilient modulus of the mixes containing higher RAP proportion can be higher than the mixes containing less RAP. On the other hand, the permanent deformation test indicated that higher RAP produced more permanent deformation. In addition, in some cases, the resilient modulus of the specimen remained approximately constant while the permanent strain increased. This may imply that resilient modulus alone does not properly characterise foamed bitumen stabilised base materials. The permanent deformation characteristic should be incorporated in addition to the resilient modulus property.

The estimate of resilient modulus from the repeated load deformation test in this study did not seem to provide a reasonable value for the actual resilient modulus. The resilient modulus of foamed bitumen stabilised specimens was found to be in a range between 88 MPa and 508 MPa, which is relatively low. It has been reported that a foamed bitumen stabilised base has a stiffness modulus of 1100 to 1250 MPa, obtained from back-calculation of falling weight deflectometer data [Ramanujam and Jones, 2007]. The resilient modulus values reported herein, therefore, should be used with caution. However, the trend of change in resilient modulus was logical and the resilient deformation behaviour can be observed.

It is believed that the main reasons for the low resilient modulus results are the test arrangement and probably inadequate conditioning before conducting the resilient modulus test. In this experiment, the specimen deformation was only measured with LVDTs mounted on the loading ram outside the triaxial cell. In a more accurate resilient modulus test, the deformation should be measured with LVDTs placed on the specimen. The resilient modulus determined based on data from on-specimen LVDTs is expected to be greater and more reliable than those determined based on external LVDTs. The resilient modulus determined from external LVDTs in this study is therefore a conservative estimate.

Foamed bitumen stabilised materials showed slightly stress dependent behaviour in this study. The reasons for the stress dependence of foamed bitumen stabilised materials becoming less evident may be attributed to the fact that specimens have not been conditioned adequately with cyclic pulses, as well as the high foamed bitumen content of 4%.

4.7 Conclusions

This study has consisted of permanent deformation and resilient modulus tests on foamed bitumen stabilised base materials. These base materials included crushed limestone and blends of RAP and crushed limestone. Foamed bitumen was produced from three penetration grade bitumens. A series of repeated axial compression tests in the triaxial condition was performed on accelerated cured specimens. The findings of this study can be summarised as follows.

1. The resistance to deformation of foamed bitumen stabilised materials is affected by the amount of RAP in the mixtures and also the penetration of bitumen used to generate the foam.

2. Generally, foamed bitumen stabilised mixtures that contain a higher proportion of RAP and a softer binder exhibit greater deformation. However, limited experimental data showed that a mixture containing 50% RAP produced similar performance, in terms of permanent deformation, compared to a mix manufactured using only limestone.

3. Application of confining pressure caused a decrease of permanent deformation and an increase in resilient modulus of foamed bitumen stabilised mixtures.

4. The permanent strain can be fitted to a model that assumes the accumulated permanent strain (ε_p) is proportional to a power of the number of repeated load cycles (N).

5. Resilient modulus of foamed bitumen stabilised mixtures increased slightly with an increase in the confining pressure.

6. Bitumen type generating the foam did not significantly influence the resilient modulus of the mixtures. Nonetheless, the materials stabilised with softer bitumen tend to be compacted better and hence have higher resilient modulus.

7. Foamed bitumen stabilised mixtures show stress dependent behaviour. The material loses its stress dependency when the foamed bitumen content is high. It also seems that foamed bitumen stabilised material becomes less stress dependent when foamed bitumen is better distributed in the mixtures.

8. Resilient modulus alone does not properly characterise foamed bitumen stabilised materials. The permanent deformation characteristic was found to be another necessary property that should be incorporated in the assessment of the performance of foamed bitumen stabilised materials.

5 FLEXURAL CHARACTERISTICS OF CEMENT STABILISED RECYCLED ASPHALT PAVEMENT MATERIALS

5.1 Introduction

Cement is the most common stabilising agent in pavement and geotechnical engineering works. The main reason for this is probably its availability. It is also widely accepted as a construction material. Cement has been used to stabilise numerous road bases since ancient times. In fact, the use of cement stabilised base for roads dates back to Roman times [Margery, 1987]. Cement has been used as a stabilising agent in cold in-place recycling long before foamed bitumen became popular and is still widely used nowadays.

It is well known that cement treated pavement materials tend to fail in cracking by two reasons; first, the shrinkage that occurs during the cement hydration process, and second by the repeated loading from traffic. Shrinkage cracking can be controlled and is not necessary detrimental. The cracking caused by traffic, on the other hand, is clearly a failure mechanism, which leads to break-up of the material, and requires the rehabilitation of failed sections. Cracking could occur from a single large wheel load or after numerous smaller load applications due to fatigue effects.

This chapter focuses on traffic induced cracking of cement stabilised recycled pavement materials with two main objectives namely: determining the resilient modulus of cement bound recycled asphalt pavement materials, and; investigating the extent of fatigue damage in terms of degradation of elastic modulus with number of load applications.

The data from these experiments will be useful for the design of pavements incorporating cement stabilised recycled asphalt materials.

5.2 Reponses of cement stabilised materials to traffic load

When a base or subbase course layer of a pavement is constructed using cement stabilised material and the upper surface is asphalt, the construction is defined as a flexible composite (semi-rigid) pavement [Highway Agency, 1999]. From a mechanistic standpoint, a cement stabilised pavement layer is subjected to repeated tensile (flexural) stresses as a result of dynamic traffic loading on the surface. Figure 5.1 shows the locations of critical stresses and strains in a conventional flexible pavement (Figure 5.1(a)) and a similar pavement with a cement stabilised base course replacing the traditional granular base course (Figure 5.1(b)). The critical tensile stress is shifted from the bottom of the asphalt layer (Figure 5.1(a)) to the bottom of stabilised layer (Figure 5.1(b)). Cracking will initiate where the maximum tensile stress and strain occur.

Because concrete material is relatively weak in tension and the bound base is subjected to tensile stress, cement treated bases invariably fail in cracking mode. In order to simulate the traffic effect on the material, repeated flexure is an appropriate test method as it closely reflects the in-service behaviour of the material. Stiffness modulus and fatigue characteristics of cement stabilised pavement materials under repeated flexure can be determined.

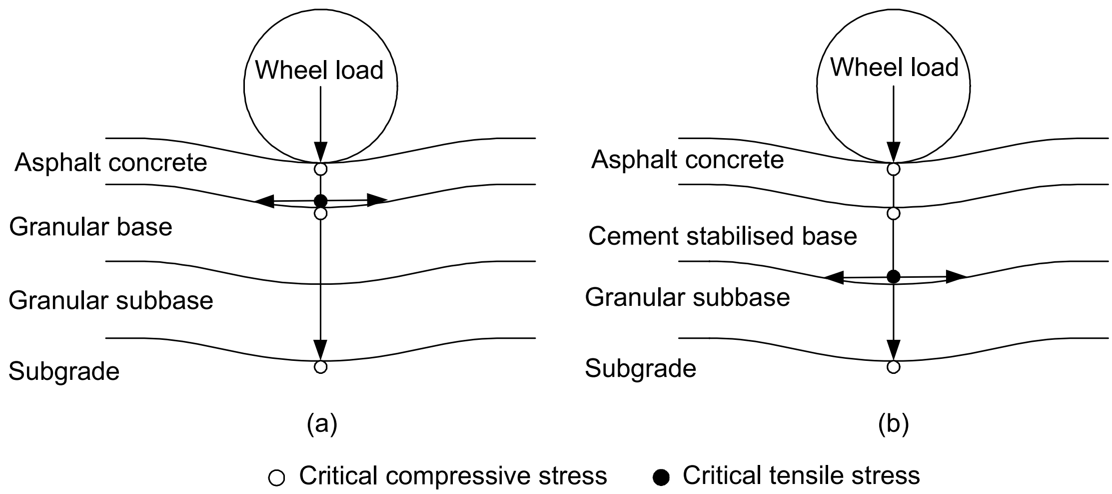


Figure 5.1 Locations of critical stresses and strains in a flexible pavement (a) containing a granular base course and (b) a cement stabilised base course

5.3 Repeated load flexural test

A number of methods have been developed for testing the fatigue characteristic of materials. One of the common methods employs the principle of the flexural beam because it simulates the stress and strain condition in a pavement layer in a realistic way. In beam tests, a simple beam with centre point loading (three-point bending) or third point loading (four-point bending), or a cantilever beam (two-point bending) have been used as shown in Figure 5.2. The simple beam test with third point loading was chosen as a fatigue test method for cement stabilised materials in this study. The advantage of third point loading over the centre point loading is the presence of a constant bending moment over the middle third portion of the specimens; so the failure will show up at a weak spot due to non-uniformity of the materials. This is not the case for centre point loading where failure is forced to occur at a specific point. Figure 5.2 (c) schematically illustrates the third point loading fatigue test where a simply supported beam specimen is subjected to a cyclic load under four-point loading.

The fatigue test can be performed in two types of controlled loading modes: constant stress and constant strain. The constant stress type of loading is generally considered to represent the response to traffic loading of a thick layer (more than 150 mm thick) as the main loading component of a pavement, while the constant strain type is suitable for thin pavement layers of less than 50 mm thickness [Huang, 1993].

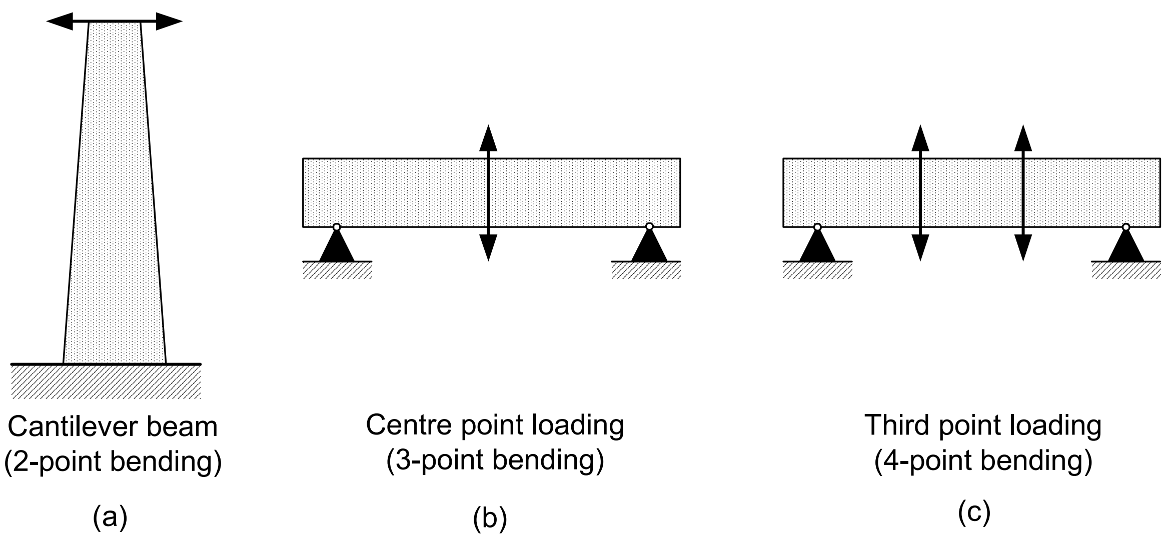


Figure 5.2 Beam fatigue tests

For Portland cement concrete, the fatigue test usually adopted applies a repeated flexural loading at the third point on beam specimens 375 mm long, 75 mm wide, and 75 mm deep [Huang, 1993]. The rate of loading does not have a significant effect on the fatigue life for frequencies between 1 and 15 Hz for concrete type materials [ACI Committee 544, 1990].

Considering the applicability of the test for thick pavement layers and allowing for the available equipment, the configuration chosen for flexural fatigue testing of the beam specimens is as follows:

(i) Four-point loading configuration
(ii) Span length 270 mm (i.e. load points 90 mm apart)
(iii) Beam cross section of 75 × 75 mm
(iv) Constant stress mode of loading
(v) Reversed sinusoidal cyclic loading
(vi) Load frequency of 5 Hz
(vii) Test at room temperature (20 ± 5 °C)

Stiffness modulus and flexural strength determinations

Referring to four-point bending configuration in Figure 5.3, the following formulae, which are based on elastic theory, have frequently been used to compute the stiffness modulus and flexural stress.

The general equation for calculating stiffness modulus from flexural tests is [Huang, 1993]:

$$E_s = \frac{Pa(3L^2 - 4a^2)}{4bh^3\Delta} \tag{5.1}$$

where,

a	=	distance between the load and the nearest support
P	=	total dynamic load, *P/2* applied at each third point
b	=	specimen width
h	=	specimen depth
E_s	=	stiffness modulus
L	=	span length between supports
Δ	=	dynamic deflection at beam centre

When $a = L/3$, Equation 5.1 becomes

$$E_s = \frac{23PL^3}{108bh^3\Delta} \tag{5.2}$$

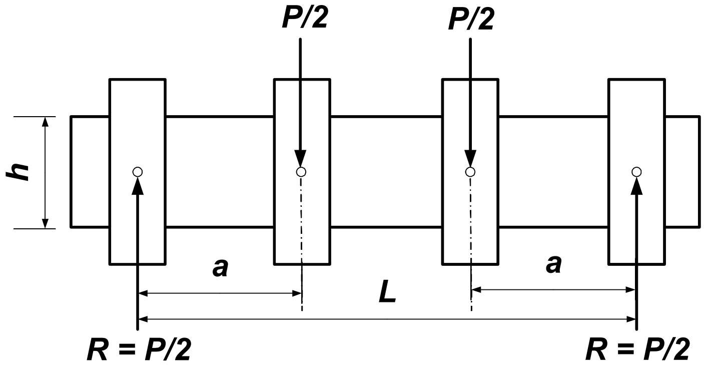


Figure 5.3 Loading and geometry of the specimen in four point bending test

If the square of the ratio of the height to beam span is much less than unity ($(h/L)^2 \ll 1$), the shear deformation can be neglected [Tayebali et al, 1994]. However, in this study the beam specimens were quite short compared to the height. Therefore, both bending and shear deformation of the beam were considered. Irwin and Gallaway (1974) suggested the use of following equation for E_s:

$$E_s = \frac{23PL^3}{108bh^3\Delta}\left[1 + \frac{216h^2(1+\nu)}{115L^2}\right] \tag{5.3}$$

where, ν = Poisson's ratio of the beam

The expression in the bracket in Equation 5.3 is the correction factor for shear deformation. The correction factor varies with h/L. For a beam with L = 270 mm and ν = 0.35, the correction factor is 1.16 for h = 75 mm.

The flexural strength of the specimen was determined by the following equation [BS EN 12390-5:2000];

$$f_{cf} = \frac{FL}{bh^2} \tag{5.4}$$

where,

f_{cf}	=	flexural strength
F	=	maximum load
L	=	distance between supports
b	=	specimen width
h	=	specimen depth

5.4 Experimental programme

5.4.1 Description of laboratory tests

Cement stabilised base course materials were manufactured in the laboratory by mixing RAP and limestone aggregate and treating with Portland cement. RAP and limestone aggregate were blended in three proportions, 0%RAP, 50%RAP and 75%RAP. All blends were stabilised with cement. Eight beams were prepared for each proportion. The static flexural strength test was conducted on three beams and the average was considered as a representative value for the material. The remaining five beams were subjected to repeated flexural load in stress control mode. The beam specimens were loaded at the third point by a continuous sinusoidal repeated load with constant amplitude at a frequency of 5 Hz. The repeated load is expressed in terms of stress ratio, which is the ratio of the applied flexural stress to the static flexural strength of the material.

5.4.2 Specimen preparation

All blends were stabilised with 6% cement by weight of dry aggregate at 6% optimum moisture content according to the mix design described in chapter 3. Predetermined amounts of RAP, aggregate and cement were mixed dry in a mechanical mixer for 30 seconds. Water was added gradually and was mixed for a further 60 seconds. Slabs were then prepared by compacting the mix in a detachable steel mould in a single layer using a slab roller compactor as shown in Figure 5.4. The slab dimensions were 305 × 305 × 75 mm. The compactor was designed to simulate the action of a roller compactor in the field. The mixtures were compacted to their corresponding maximum modified Proctor density. After compaction, the slabs were sealed at room temperature for 16 – 24 hours and then demoulded and stored in the 95 % humidity room for curing. After the slab had gained adequate strength (curing for at least 21 days), four beams were cut from each slab, having the dimensions 305 × 75 × 75 mm as shown in Figure 5.5. All beams were then kept at room temperature at 95 % humidity so that all specimens were cured for at least 28 days before testing.

Figure 5.4 Slab compactor

Figure 5.5 Sawn beams from a slab of cement stabilised material

5.4.3 Four-point bending equipment

The four-point bending apparatus used in this study was designed by Oliveira (2006) and manufactured at the University of Nottingham. Some modifications were made to the original apparatus so that it could accommodate beams with cross section up to 75 mm × 75 mm. Figure 5.6 shows the four-point bending assembly. The test frame was built such that the specimen's clamps and supports have free rotation in order to avoid internal stresses and strains imposed on the specimen. There was some horizontal restraint at the specimen clamps but it was determined that no major stresses would be induced into specimens during the test as a result of the lateral restraint [Oliveira, 2006]. The outer supports, however, could compress vertically at the pivot points during the test, which affects the results. Although the movement was a fraction of a millimetre, it was monitored and taken into account in the analysis.

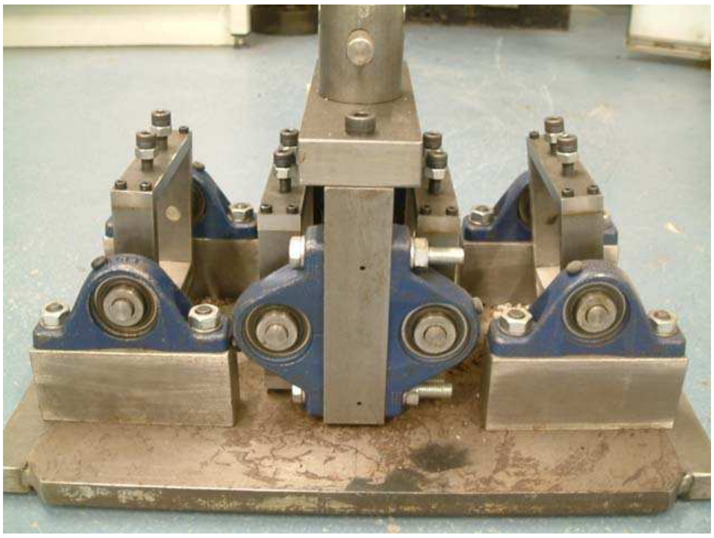


Figure 5.6 Four-point bending apparatus

A MAND servo-hydraulic testing machine was used to apply load to the 4-point bending frame. The load can be applied to a specimen under static or cyclic mode. The machine was operated by *Rubicon* digital servo control system. Figure 5.7 shows the testing machine.

Figure 5.7 MAND servo-hydraulic testing machine

5.4.4 Testing procedure

Test set-up

The four-point bending frame was attached to the testing machine and the alignment adjusted. A small metal bracket was glued at the neutral axis level to the beam centre on one side of the specimen as shown in Figure 5.8. The specimen's dimensions were measured. The beam was placed centrally into the test frame and clamped. The specimen was then instrumented. The dynamic deflection of the neutral axis of the beam at midspan was measured with a linear variable differential transformer (LVDT) whose foot rested vertically on the smooth surface of the bracket. Two additional LVDTs were used to measure the vertical movement of the outer supports which was then subtracted from the movement measured by the centre LVDT in order to obtain the correct vertical movement of the middle of the beam. The LVDTs and load cell of the testing machine were attached to a high-speed data acquisition system for recording of the cyclic load deformation response. Figure 5.9 schematically illustrates the test set up.

Figure 5.8 Beam specimen for four-point bending test

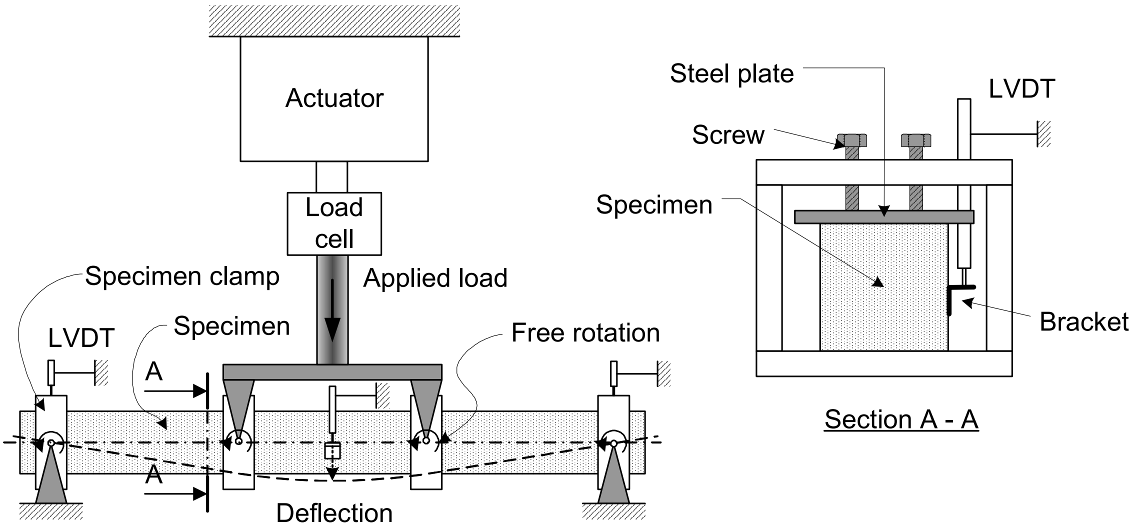


Figure 5.9 Schematic of four point bending test apparatus

Static flexural test

Three of eight beams of each mixture were subjected to static flexural tests. The average flexural strength would be considered to be representative of the material. After each specimen was placed in the four-point bending frame in position ready to be tested, a vertical load was applied so that the extreme fibre stress increased continuously at a constant rate of 0.05 MPa/s until the specimen failed by fracture. The maximum load was recorded and the flexural strength was determined.

Dynamic flexural test

Five beams of each mix were subjected to a repeated load flexural test. The tests were conducted under load-control using a sinusoidal load with a constant amplitude. Two types of test were carried out: a short duration test to determine the resilient modulus of the material, and a long duration test to determine the fatigue characteristic of the material. For each specimen, the test procedure included a short duration modulus test followed by a long duration fatigue test.

For the resilient modulus test, the stress amplitude was about one quarter of the flexural strength of the material to ensure that the specimen would still be in the elastic range and no damage would occur. The load was applied at two frequencies, 2 Hz and 5 Hz, for 100 cycles each. During the test, force and displacement amplitudes were recorded. The responses were used to determine the modulus of the beams at every load cycle.

In a long duration fatigue experiment, the repeated load is expressed in term of stress ratio, which is the ratio of the applied flexural stress to the static flexural strength of the material. The flexural strength was determined for each mix from the static flexural test described above. The selected stress ratios for this study were between 0.7 and 0.9. Each beam was tested at one stress ratio until the specimens failed or 150,000 load cycles had been applied, whichever was reached first. Failure was defined as the complete fracture of the specimen. The load and midpoint deformation data were recorded at appropriate intervals throughout the test.

5.5 Results

5.5.1 Flexural strength

Figure 5.10 presents the typical relationship between flexural stress and displacement of cement stabilised beams subjected to static flexural strength tests.

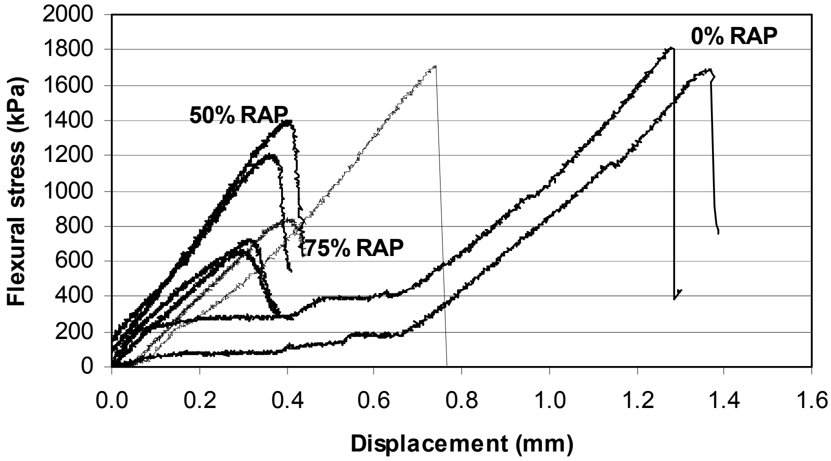


Figure 5.10 Flexural test results of cement stabilised recycled pavement materials

The average static flexural strengths, based on three specimens, of cement stabilised 0%RAP, 50%RAP and 75%RAP mixtures were obtained as 1.733, 1.305 and 0.747 MPa respectively. Increasing the RAP proportion caused a significant reduction effect on the static flexural strength of the mixtures. From this experiment, there was a decrease of 25 % and 57 % in the static flexural strength of cement stabilised recycled mixtures compared with cement stabilised crushed limestone (i.e. 0%RAP) with the addition of 50 % and 75 % of RAP to the aggregate respectively.

5.5.2 Stiffness modulus

Figure 5.11 presents the stiffness modulus results of cement stabilised mixtures. Two frequencies, 2 Hz and 5 Hz, were selected for the stiffness modulus test. Five specimens were used for each studied mixture. The results presented in the figure represent the average values obtained from five specimens. The stiffness modulus results from the 5 Hz tests are slightly higher than those from 2 Hz tests. Although the materials contain RAP, which incorporates bitumen, the stiffness modulus of cement bound recycled pavement materials is practically independent of the frequency of loading.

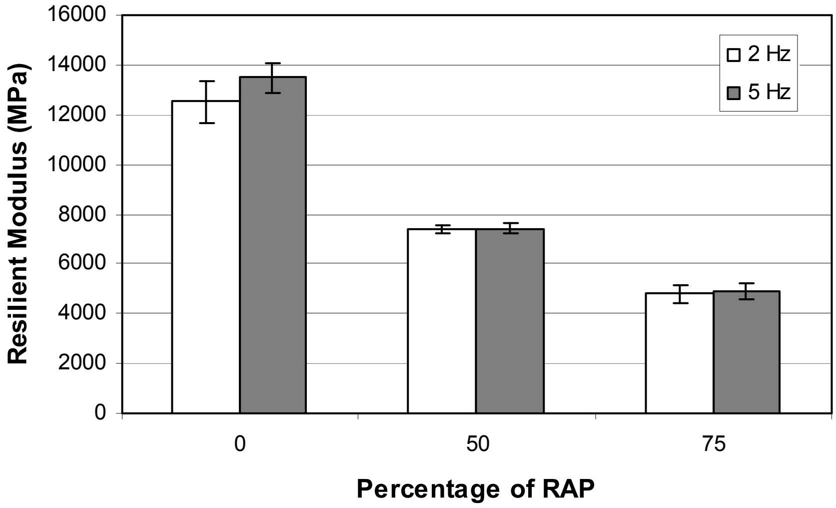


Figure 5.11 Influence of RAP proportion on the stiffness modulus of cement stabilised mixtures

As for the flexural strength results, it can be seen that the inclusion of RAP reduces the stiffness modulus of the mixtures significantly. In this study, stiffness modulus of cement stabilised recycled pavement materials reduced approximately 42 % and 62 % when 50%RAP and 75%RAP were included respectively.

5.5.3 Fatigue life

Figure 5.12 shows a typical set of stiffness modulus results plotted as a function of number of load applications obtained under different test conditions (mixture types and stress levels). The stiffness modulus of the material has been normalised by the stiffness modulus at the beginning of the test and the number of load applications was normalised by the number of load applications at failure.

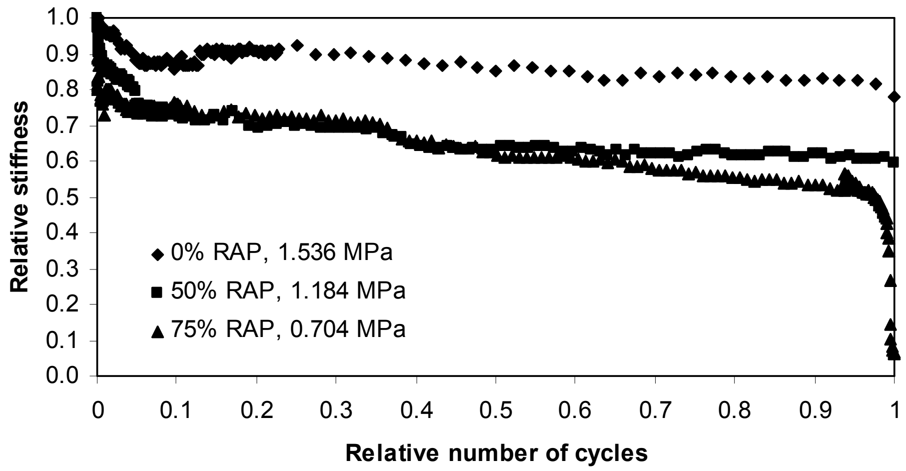


Figure 5.12 Normalised flexural fatigue test results

It can be seen from Figure 5.12 that the curves can be divided into three stages. The first stage is characterised by a rapid reduction of stiffness modulus. For most specimens in this study, the initial stage is up to approximately 5 % of the total fatigue life. The second stage is characterised by an approximately linear reduction in stiffness modulus with the number of load cycles. This stage accounts for the majority of the total fatigue life. The specimens show slight loss of stiffness modulus before the failure occurred. The second stage can end with an abrupt failure of the specimen. At lower stress ratios, in some cases, either a rapid reduction of stiffness modulus toward failure was observed or the duration of the second stage extended to more than 150,000 load applications. The stage of rapid reduction in stiffness modulus to failure, if it happened, was about 5% of the fatigue life.

Fatigue behaviour of concrete type materials is commonly expressed in terms of the S-N relationship, which is the relationship between the stress ratio (SR) and the corresponding number of cycles to failure (N_f). The SR is obtained by normalising the applied repeated extreme fibre stress (σ) with the respective flexural strength. The fatigue life data of cement stabilised recycled pavement materials with different RAP contents are listed in Table 5.1.

Table 5.1 Fatigue life data for cement stabilised recycled materials containing different proportions of RAP

0% RAP			50% RAP			75% RAP		
σ (MPa)	SR	N_f	σ (MPa)	SR	N_f	σ (MPa)	SR	N_f
1.733*	1.000	1	1.305*	1.000	1	0.747*	1.000	1
1.600	0.923	216	1.299	0.996	6039	0.704	0.946	107384
1.536	0.886	44091	1.220	0.935	472	0.634	0.852	8186
1.472	0.849	11761	1.078	0.826	105039	0.576	0.774	51276
1.344	0.776	12561	1.016	0.778	89050	0.512	0.688	83152
1.280	0.738	99062	0.914	0.700	39116	0.416	0.563	150000**

* average flexural strength of three specimens
** specimen did not fail

Figure 5.13 and Figure 5.14 present the S-N relationships for cement stabilised mixtures with different proportions of RAP. In Figure 5.13 the ordinate represents the maximum fatigue stress expressed as a proportion of the corresponding static flexural strength whereas in Figure 5.14 the ordinate represents the actual applied fatigue stress.

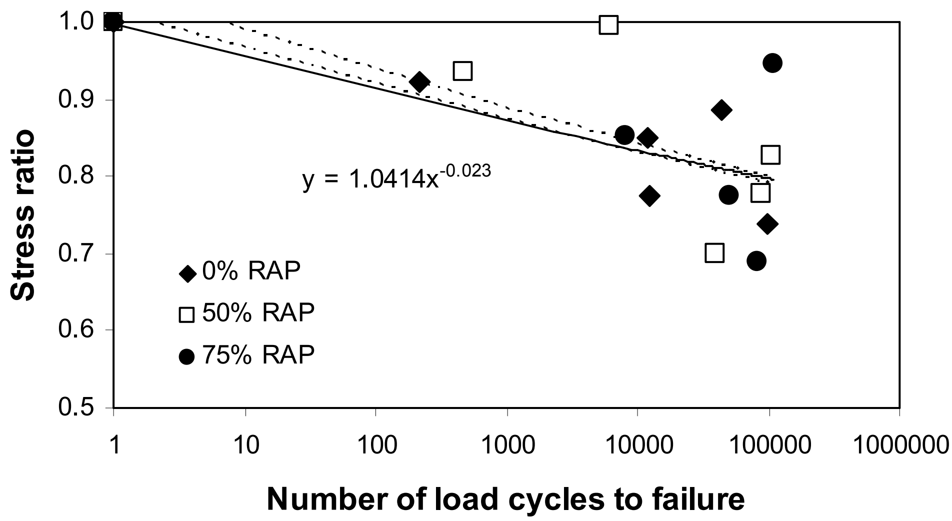


Figure 5.13 S-N relationships for cement stabilised recycled pavement materials based on stress ratio

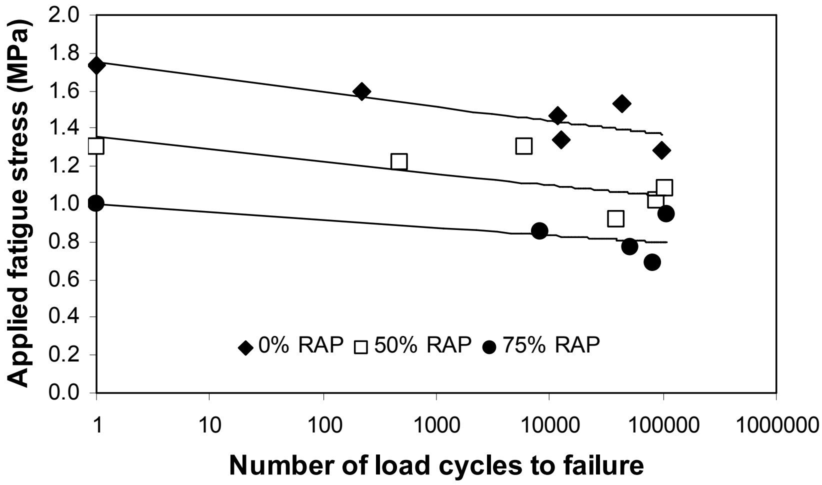


Figure 5.14 S-N relationships for cement stabilised recycled pavement materials based on actual applied fatigue stress

As expected, the fatigue test data for cement stabilised mixtures show considerable scatter similar to the results obtained from fatigue tests on other concrete type or cement stabilised materials. In the figures, the power trend lines were developed for the data points in order to analyse the general correlation. It can be seen from Figure 5.13 that when the fatigue performance is examined in terms of stress ratio, the trend lines obtained for the three mixtures are very close to each other and can be considered as a single trend line. When the fatigue performance is examined in terms of actual applied stress as shown in Figure 5.14, the ranking of the mixtures can be seen. Basically, the positions in which the fatigue curves are situated are related to the static flexural strength of the specimens.

5.6 Influence of temperature on the stiffness modulus of cement stabilised RAP

A separate experiment was conducted to examine the influence of temperature on the stiffness modulus of cement stabilised RAP.

In this experiment, RAP was stabilised with cement at three levels, 4%, 6% and 8% by weight at a water content of 6%. The mixtures were mixed as described earlier but the compaction was conducted in a gyratory compactor. For each cement content, three replicated cylinder specimens were produced. Each of them was 150 mm in diameter and 80 mm in height. The density for all specimens was 2.20 Mg/m^3 which is equal to 96 – 98 % of the maximum Proctor densities. After compaction, the specimens were cured in the moulds for 24 hours and thereafter in closed containers at room temperature.

The stiffness modulus of cement bound RAP was determined by the Indirect Tensile Stiffness Modulus (ITSM) test in accordance with British Standard DD213:1993 although the standard was developed mainly for bituminous bound mixtures. A brief description of the ITSM test is given in Chapter 7 or can be found elsewhere [e.g. Read and Whiteoak, 2003]. The ITSM test was carried out at 10, 20, 30 and 40 °C on three replicated specimens for each cement content when specimens had been cured for 28 days.

Figure 5.15 presents the stiffness modulus results of cement bound RAP. From the results, the influence of temperature on the stiffness of the mixes can be noticed. Generally, a 15 – 20 percent decrease in stiffness modulus was obtained from a 10 °C increase in the temperature. For comparison purpose, the variation in stiffness modulus with temperature of typical asphalt concrete cores [Ruenkrairergsa et al, 2002] is shown in the figure. It can be seen that, in the case of asphalt, a near 50% reduction in stiffness modulus occurs for every 10 °C increase in temperature. The stiffness modulus test was also conducted on cement bound granular materials. It was found that the results were constant over the range of temperatures as shown in Figure 5.16.

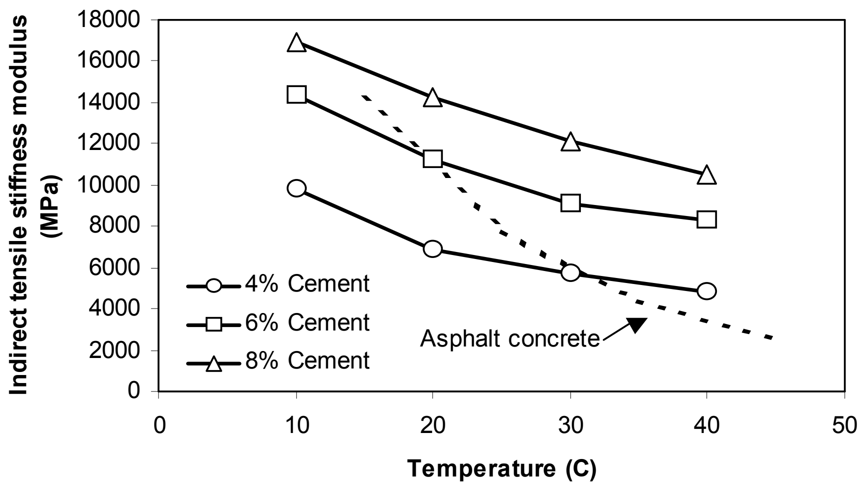


Figure 5.15 Influence of temperature on stiffness modulus of cement treated RAP

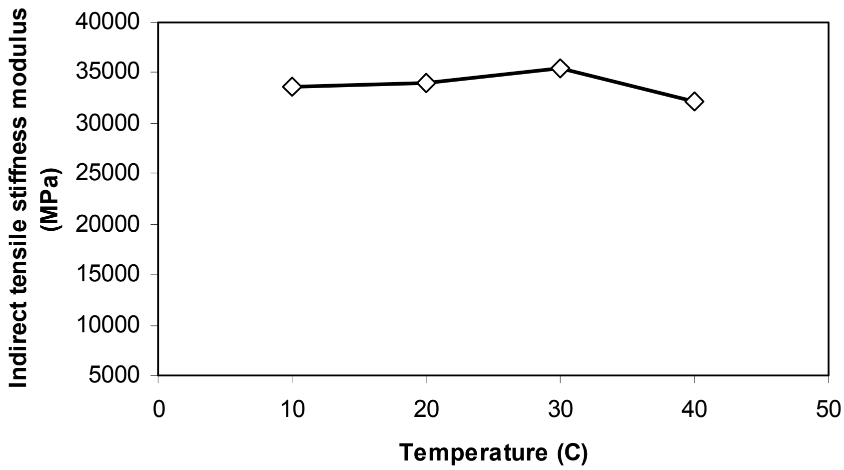


Figure 5.16 Stiffness modulus of cement stabilised crushed limestone at various temperatures

5.7 Discussion

The principal mode of damage of cement stabilised materials in pavement is cracking due to repeated flexural stresses induced by traffic load. The accumulation of fatigue damage and the behaviour of the material under repeated loading are therefore important parameters that affect the performance of the entire pavement structure.

The strength of RAP is obviously significant lower than the strength of limestone. The strength and stiffness modulus of cement bound recycled pavement materials in this investigation was also found to be significantly lower than a typical cement bound limestone due to the inclusion of RAP. Hence, it can be argued that the strength and stiffness modulus of cement stabilised materials are dependent to strength and stiffness of their aggregate. The rise in temperature obviously reduces the stiffness modulus of RAP, as RAP contains bitumen, and hence reduces the average stiffness modulus of aggregate of cement stabilised materials containing RAP. This reason can, therefore, explain the decrease in stiffness modulus of cement stabilised RAP with increasing temperature. Nonetheless, the stiffness modulus of cement bound recycled pavement is still comparable to or better than that of typical asphalt bound base course.

The fatigue characteristics of cement stabilised recycled asphalt exhibit typical cement bound materials behaviour. The fatigue test results of cement stabilised recycled asphalt materials suggests that if the fatigue line is expressed in terms of stress ratio, results from different mixtures fall on the same fatigue line.

Fatigue strength, expressed as a percentage of the flexural strength, is called the endurance limit, which ideally refers to a stress ratio at which the beam will never fail in fatigue. It was speculated that concrete will not fail by fatigue when the stress ratio is smaller than 0.5 – 0.57 [Ramakrishnan et al, 1989]. In this experiment, the results from limited data also indicate that at the stress ratio of about 0.5 – 0.6 the materials can withstand the load about 5×10^6 applications. The design thickness of the pavement incorporating this type of materials should, therefore, be selected so that the induced stresses are smaller than the endurance limit of the material.

The ITSM tests on cement stabilised RAP clearly illustrate that the material is temperature dependent. The temperature of pavement base course in actual roads can be as high as 40 – 50 °C. Hence, the reduction in stiffness modulus should be taken into account when design thickness of pavements incorporating this type of materials.

It is also expected that the fatigue life of cement bound materials incorporating RAP is likely to be temperature dependent. Time did not allow to investigate the effect of temperature on the fatigue characteristics of cement stabilised RAP in this study. It may be quite useful and should, therefore, be tested in the future.

Based on the stiffness modulus results, the reduction in stiffness modulus with increasing temperature indicates that the behaviour of cement bound RAP lies between those of cement bound granular material and asphalt concrete. Cement stabilised RAP may be an interesting material which combines some of the beneficial properties of an asphalt with the stiffness of a cement treated material. Nonetheless, the observed behaviour is based on very limited data and more research is recommended to fully evaluate the benefit of such materials.

5.8 Conclusions

In this chapter, recycled pavement materials composed of RAP and limestone aggregate had been stabilised with Portland cement. Beam specimens of the stabilised mixtures have been subjected to repeated flexural loading in four-point bending configuration to evaluate their flexural behaviours. The stiffness modulus and fatigue properties of cement stabilised recycled asphalt pavement materials has been reported. Significant findings from the study can be summarised as follows:

1. Flexural strength of cement stabilised materials depends on the strength of their aggregate. Therefore, an inclusion of RAP in limestone aggregate significantly influences the strength of cement stabilised mixtures. The increase of RAP proportion in cement stabilised base course considerably reduces the flexural strength and stiffness modulus of the materials.

2. The flexural characteristics of cement stabilised limestone which incorporated RAP are similar to typical cement bound materials behaviour. Stiffness modulus of cement stabilised mixtures with different proportions of RAP were found to be independent of loading frequency. The fatigue of cement stabilised recycled asphalt materials tend to be stress controlled. The fatigue test results suggest that if the fatigue characteristic of the materials is expressed in terms of stress ratio, result from different mixtures fall on the same fatigue line.

3. Stiffness modulus of cement stabilised materials incorporating RAP is susceptible to temperature. The increase in temperature decreases the stiffness modulus of materials. However, the degree of reduction in stiffness modulus with increasing temperature is not as high as that of hot mix asphalt.

6 PERFORMANCE OF PILOT-SCALE RECYCLED ASPHALT PAVEMENT UNDER TRAFFIC LOADING

6.1 Introduction

The previous chapters have presented the experimental results of recycled asphalt pavement materials on the laboratory element scale. This chapter presents results from the Nottingham Pavement Test Facility (PTF), used to traffic a pilot-scale cold recycled asphalt pavement. The objective of this investigation was to evaluate the performance of some recycled asphalt pavement mixtures, stabilised with foamed bitumen only, cement only and foamed bitumen plus a small amount of cement, realistically under traffic load. However, because stabilised materials normally require a curing duration ranging from several months to years in order to be fully cured and because of time constraints, it was decided to investigate the early life mechanical performance of cold mix stabilised asphalt layers.

The laboratory investigations of foamed bitumen stabilised mixtures in this research were mainly based on testing fully cured specimens. Also, a considerable amount of data available from the literature on cold mix in-place stabilisation is based on analysis of cores obtained from stabilised layers several months after completing the project. This is understandable as coring cold bituminous stabilised layers (i.e., emulsion or foamed bitumen) before this time is not readily achievable in many cases as a consequence of low early life strength of such mix types. Similarly, there are difficulties in testing fragile or weakly bound cold mixes in their early life.

The performance of a range of stabilised recycled mixes under realistic trafficking conditions with a moving loaded wheel during their early life was examined. It is believed that the data and results from this study should reflect the performance of actual cold recycled pavements. Nevertheless, because conditions in the PTF do not match those at full-scale exactly, the PTF was used for this work as an investigating tool on a comparative basis.

6.2 The Nottingham Pavement Test Facility

The Nottingham Pavement Test Facility, as shown in Figure 6.1, was developed at the University of Nottingham [Brown and Brodrick, 1981]. The PTF, which is an approximately half-scale facility, was used to produce repeated traffic loading onto the pavement via a moving wheel under controlled speed and load conditions. The PTF loading is applied through two hydraulic actuators mounted on the loading carriage. Reaction to this load is against the main longitudinal beams through bearings as the carriage is pulled over the test section. The test can be conducted at a maximum load of 12 kN and a velocity of up to 16 km/h. The tyre width is approximately 150 mm. The pressure applied at the surface of the pavement at 12 kN load was about 600 kPa, which is equivalent to that in typical heavy goods vehicle tyres. Loading can be applied in one direction or bi-directionally.

The pilot-scale trial pavement is constructed in a test pit 1.4 m deep, 4.8 m long and 2.4 m wide. Figure 6.2 shows a diagram of the PTF longitudinal-section. Normally, the PTF operates at room temperature although the pavement surface can be heated up to 35 °C by infra-red heaters.

Figure 6.1 The Nottingham Pavement Test Facility

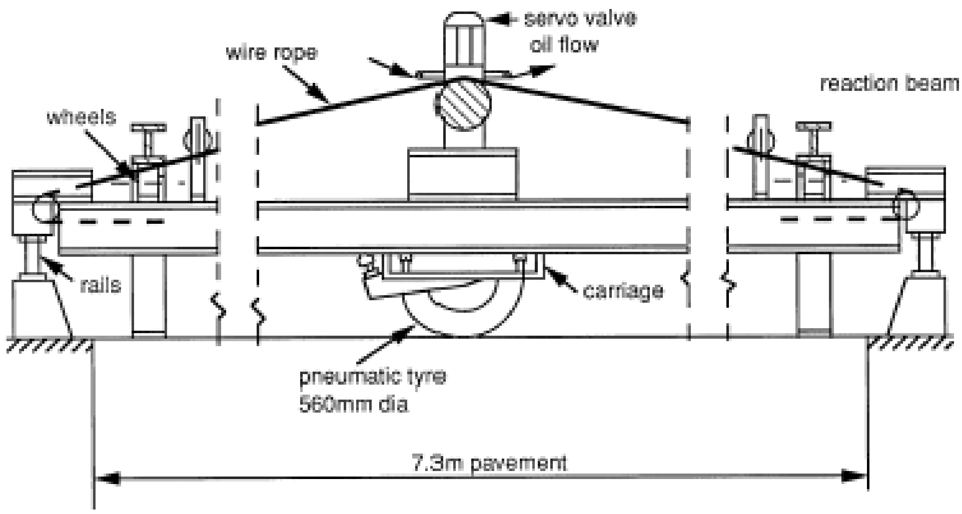


Figure 6.2 Cross section of the Nottingham Pavement Test Facility

6.3 Experimental programme

The cold recycled pavement materials were simulated by mixing reclaimed asphalt pavement (RAP), limestone aggregate, water and binders in the laboratory. A pilot-scale trial pavement of cold recycled pavement materials was constructed and cured in the PTF pit. The trial pavement was divided into several smaller sections composed of various mixture proportions and binder types. Once the pavement was fully constructed and allowed to cure for a short duration, the various sections were trafficked by the PTF. The performance of the pavement was assessed at frequent intervals by monitoring the magnitude of accumulated permanent surface deformations (rutting) and transient strains at the bottom of the stabilised layer in the wheel path during trafficking. Because of the time constraint, the experiment programme was designed to investigate the mechanical performance of a trial cold recycled asphalt layer during its early life period.

6.4 Materials

RAP, limestone aggregate and cement described in Chapter 3 were used in this study. Two penetration grade bitumens, PG50/70 and PG70/100, were selected for the production of foamed bitumen. A temperature of 160 °C and a water content of 2 % by mass of bitumen were selected to create the foamed bitumen for both bitumens. The corresponding maximum expansion ratio and half-life values were 15 times and 9 seconds for PG50/70 and 17.5 times and 16.5 seconds for PG70/100 respectively.

Two proportions of RAP to aggregate were considered in the experiment, namely:

Proportion 1 – (75%RAP) = 75 % RAP : 25 % Aggregate by mass.
Proportion 2 – (50%RAP) = 50 % RAP : 50 % Aggregate by mass.

RAP and virgin aggregate blends were treated with foamed bitumen and/or cement. Four mixtures were primarily treated with foamed bitumen and two with cement. In one of the foamed bitumen mixtures, 1.5% cement was added by mass of dry combined aggregate. In total, six mixtures were made up according to the following combinations:

Mix 1: Foamed bitumen PG50/70 treated 75% RAP
Mix 2: Foamed bitumen PG70/100 treated 75% RAP
Mix 3: Foamed bitumen PG70/100 treated 50% RAP
Mix 4: Foamed bitumen PG70/100 treated 50% RAP + cement 1.5 %
Mix 5: Cement treated 75% RAP
Mix 6: Cement treated 50% RAP

The compaction test was performed by the modified Proctor procedure [BS EN 13286-2:2004] on the two blends both with and without cement. The compaction curves of these mixes are shown in Figure 6.3.

The optimum binder content was determined by a procedure described in Chapter 3. Table 6.1 summarises the optimum foamed bitumen contents for the PTF experiment. The mix design adopted for both cement bound mixtures (Mixes 5 and 6) were 6% Portland cement and 6% moisture content by weight of dry aggregate.

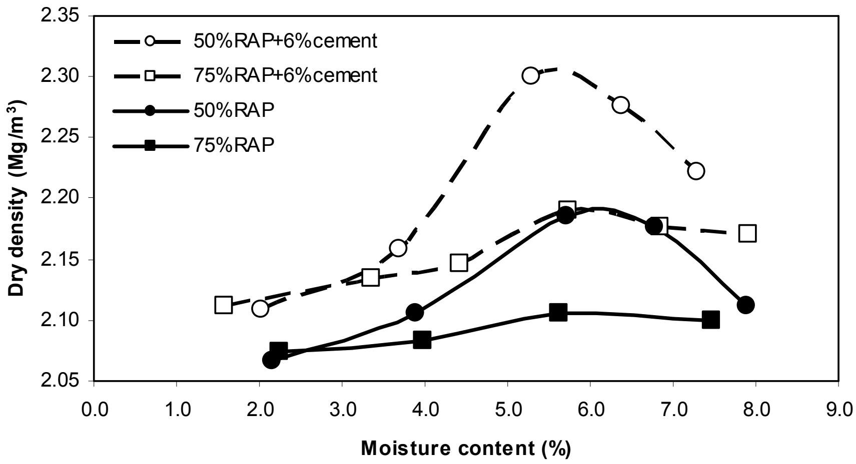


Figure 6.3 Compaction curves of RAP and limestone mixtures

Table 6.1 Mix design results of foamed bitumen bound materials

	50% RAP PG70/100	50% RAP PG50/70	75% RAP PG70/100	75% RAP PG70/100+1.5% C
Optimum binder content	2.7 %	2.7 %	2.5 %	2.5 %
Optimum moisture content	6.0 %	6.0 %	6.0 %	6.0 %
Max. dry density (Mg/m^3)	2.20	2.20	2.11	2.11

6.5 Pilot-scale pavement trial

6.5.1 Pavement foundation

The PTF foundation consisted of a 450 mm crushed limestone subbase sitting on top of a Keuper Marl clay subgrade. The bearing capacity of the foundation materials was estimated from the dynamic cone penetrometer (DCP) (Figure 6.4) and light falling weight deflectometer (LFWD) (Figure 6.5).

The detail of DCP is briefly described as follow [TRL, 1993]. The DCP consists of an 8 kg hammer that falls 575 mm onto an anvil, which drives a 60-degree cone of 20 mm diameter into the foundation layers. During the test the number of blows versus depth is recorded and used to calculate the penetration rate in millimeters per blow. Then, the average penetration rate of each layer can be used to estimate the California bearing ratio (CBR). The interfaces between foundation layers can also be observed from the change in penetration rate. DCP tests were conducted on foundations of each pavement section and the results are presented in Figure 6.6. It was found from the DCP results that an average CBR value of the limestone subbase was about 40 – 50 % and the clay subgrade was very weak with a CBR value of less than 1%.

Figure 6.4 Dynamic Cone Penetration (DCP) test

Figure 6.5 Light Falling Weight Deflectometer (PRIMA100)

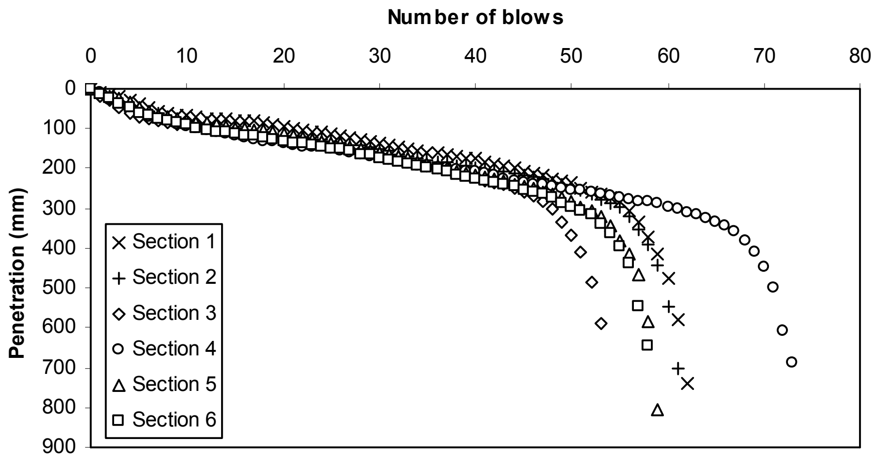


Figure 6.6 Dynamic cone penetrometer (DCP) test results

The LFWD used in this study was developed by Carl Bro Ltd. under the proprietary name PRIMA100 [Carl Bro, 2007]. The device consists of a 10 kg drop weight that falls freely onto a rubber buffer seated on a 300 mm diameter circular loading plate. The centre deflection of the loading plate was measured and used to estimate the surface modulus of the foundation. The LFWD test was conducted at 54 locations. The results indicated that the modulus at foundation level varied from 37 MPa to 81 MPa with an average of approximately 60 MPa. Figure 6.7 shows the subbase stiffness reading obtained from the PRIMA100.

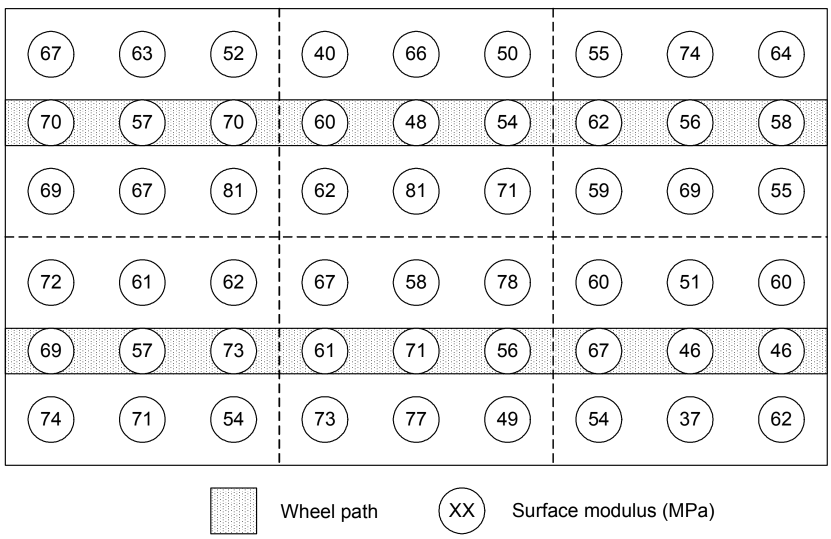


Figure 6.7 Surface modulus at the subbase level of PTF pit

6.5.2 Instrumentation

Embedment strain gauges, two in the transverse direction and one in the longitudinal direction, were installed at the bottom of the recycled pavement layer in each section and all gauges were installed directly underneath the wheel path. Two pressure cells were also installed in one of these sections. The fine particles of the mixtures were placed surround the strain gauges and pressure cells before they were installed in order to separate them from any large aggregate particle that could damage the gauges or change the results obtained by concentrate stresses at a single point. Figure 6.8 and Figure 6.9 show the installation of a strain gauge and a pressure cell respectively. The pavement layout and positions of the instrumentation are illustrated in Figure 6.10.

6.5.3 Pavement construction

Because the PTF is an approximately half-scale facility, the load applied to the trial pavement is significantly lower than that applied by an actual vehicle. In order to ensure that the pavement would suffer some degradation within a reasonable number of load applications, it was decided to construct the pavement as thinly as practically possible. Therefore, based on the compaction criteria, the trial stabilised pavement layer thickness was selected at 80 mm, which was around four times the maximum aggregate size.

For foamed bitumen bound mixtures, the materials were weighed and then mixed in a Hobart mixer at the adjusted optimum moisture and binder contents based on mix design results. Foamed bitumen was produced at 160 °C with 2% water content. However, due to the limitation of the mixer's capacity, the material could be only mixed in 6 kg batches. Thus the mixtures had to be stored in sealed containers at room temperature (20 ± 5 °C) and it took about 2 weeks to manufacture sufficient amount for construction of all stabilised pavement in the PTF pit (Figure 6.11).

The properties of compacted specimens manufactured from 2 weeks old loose foamed mixtures have not been tested in this study. However, it has been reported that foamed bitumen mixtures can be stored up to 3 months before compaction [Wirtgen, 2004]. Khweir (2006) also reported that stiffness modulus of foamed mix did not significantly reduce even if store up to 28 days.

For cement bound materials, the RAP, limestone aggregate, cement and water were weighed and thoroughly mixed in a concrete mixer (Figure 6.12). The mixture was then placed and compacted within 1 hour after mixing.

For the mixture with foamed bitumen plus cement, the RAP and limestone aggregate were treated with foamed bitumen first and the product was stored in a sealed container at room temperature for up to 10 days. On the day of compaction, cement and an additional quantity of water were then mixed with the foamed bitumen treated material using a concrete mixer.

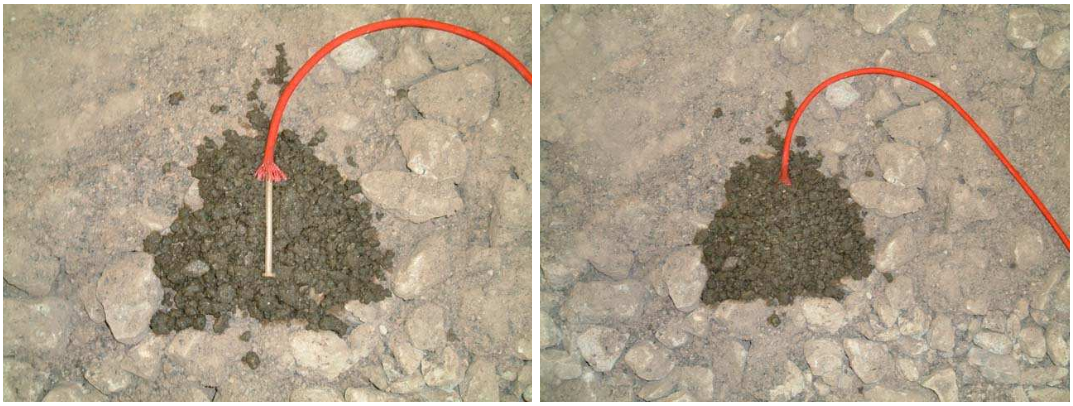


Figure 6.8 Installation of an embedment strain gauge

Figure 6.9 Installation of a pressure cell

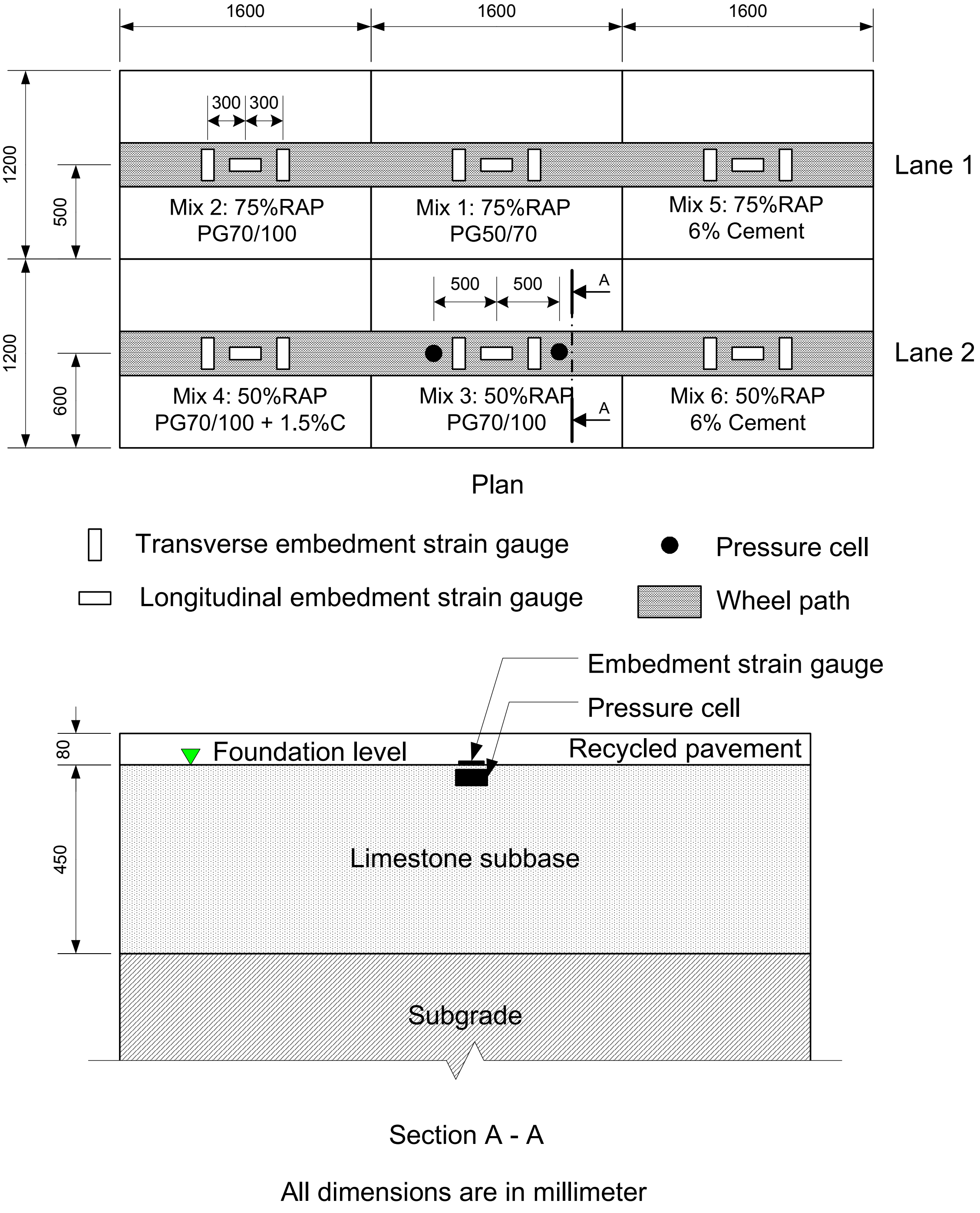


Figure 6.10 Trial pavement layout

Figure 6.11 Hobart mixer and a container ready for sealing

The materials were then placed into the PTF pit, spread, and compacted. A Wacker VP1340A vibrating plate compactor was used to compact the materials in a single layer (Figure 6.13). Before compaction, the moisture content of the materials was measured. The materials were weighed so that following compaction to the required thickness, their dry densities would not be less than 95% of their corresponding maximum dry density values as achieved in the laboratory compactability tests. The time required to lay and compact all the sections in the trial pavement was such that most sections would have been left to cure in the compacted state for at least 14 days before trafficking commenced. The exception was the section that included foamed bitumen plus cement, which was cured for 7 days before trafficking.

6.5.4 Trafficking

The performance of the pavement under traffic was evaluated by repeatedly applying wheel loads bi-directionally onto the pavement in the PTF. The trafficking was canalised, i.e. successive wheel passes were superimposed without lateral distribution of the wheel tracks across the pavement.

Figure 6.12 A concrete mixer

Figure 6.13 Spreading and compacting the material in PTF pit

The trial pavement was arranged into two lanes but was trafficked one lane at a time. Both lanes were trafficked with an equal number of load applications on each day of testing. This ensured that the performance of the different mixtures could be compared directly without having to consider the effects of differential curing between the various test sections. Trafficking was carried out at an approximate velocity of 3 km/hr. The number and magnitude of the loads applied to each lane are schematically illustrated in Figure 6.14. Since the early life strengths of the foamed sections were relatively low, and to avoid premature damage, the magnitude of the first applied wheel load was selected at 3 kN, which is the lowest practical level that can be comfortably applied using the PTF. The sections were trafficked and the accumulation of permanent deformation was monitored at this load level. When the rate of increase of surface deformation reduced to an insignificant level, the load applied was subsequently doubled in magnitude. In this experiment, the first 5000 passes were at a wheel load of 3 kN, the next 10,000 passes were at a wheel load of 6 kN, and the remaining passes were at a wheel load of 12 kN. Trafficking was terminated when the cumulative number of passes was equal to 45,000 passes per lane. Total duration of trafficking from the beginning to the end was about 45 days.

6.5.5 Monitoring

At intervals of approximately 1000 load applications, the strains at the bottom of the recycled pavement layer were recorded. At every 3000 to 4000 wheel passes, the pavement profile was measured using a straight edge, as shown in Figure 6.15, and the pavement surface was visually inspected. After completion of all trafficking, a number of cores were extracted from each foamed bitumen bound section using the dry coring technique and several beams were cut from cement bound sections to provide samples for laboratory testing

6.6 Results and discussions

6.6.1 Visual inspection

The pavement surface was inspected every 3000 to 4000 loaded wheel passes. No evidence of distress was observed in the cement bound sections. The foamed bitumen bound sections with no cement additive started to rut as soon as the first load was applied (i.e. at 3 kN). Rutting occurred only in the wheel path. Elsewhere, other than in the wheel path, the transverse profile of the pavement remained unchanged. It was also observed that, on every occasion that the magnitude of wheel load was increased, there was a significant immediate rise in rut magnitude. At each load level, the rutting rate was found to gradually decrease with increasing number of wheel passes. When the wheel load was increased to 12 kN, i.e. the maximum load selected in this investigation, and when no more significant increases in rut depth were noticed, it was decided to terminate the test. Trafficking was thus terminated after 45,000 wheel passes per lane.

Between 15,000 and 20,000 passes, i.e. soon after the wheel load was increased to 12 kN, longitudinal cracks were observed at both sides of the wheel paths on the foamed bitumen stabilised sections as shown in Figure 6.16. Cracks were not observed in the foamed section with added cement or on the cement stabilised sections. It is thought that these cracks were the result of excessive rutting and are not believed to have been caused by fatigue.

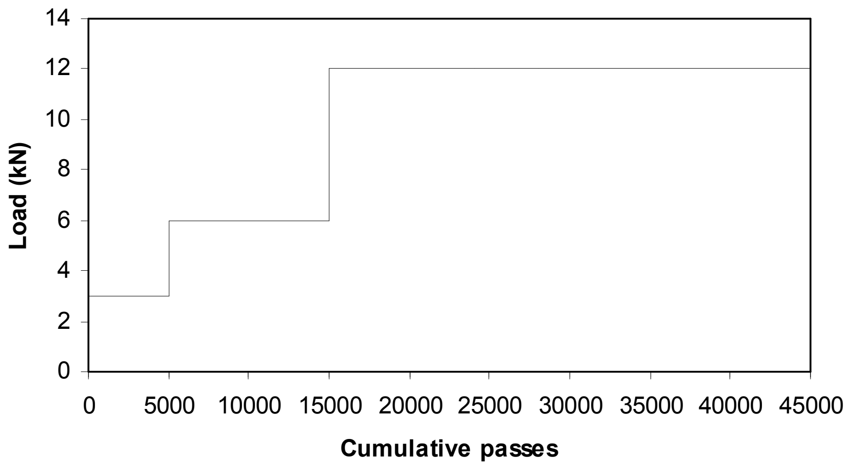


Figure 6.14 Loading schedule for PTF testing

Figure 6.15 Rutting measurement

Figure 6.16 Observed longitudinal cracks at both sides of the wheel path on the foamed bitumen stabilised pavement

6.6.2 Permanent deformation

The surface deformations of each section were measured at 8 points at equal intervals along the wheel path. The average permanent surface deformation under the wheel path of the trial sections is shown as a function of load applications in Figure 6.17. Rutting was developed to varying degrees in all foamed bitumen bound sections but no noticeable surface deformation was observed in either of the cement only sections. The condition of the wheel paths after finishing trafficking is shown in Figure 6.18. In terms of the amount of rutting, the mixtures can be ranked as follows (from best to worst):

- Cement bound mixtures
- 50%RAP PG70/100 + 1.5% cement,
- 50%RAP PG70/100,
- 75%RAP PG50/70,
- 75%RAP PG70/100.

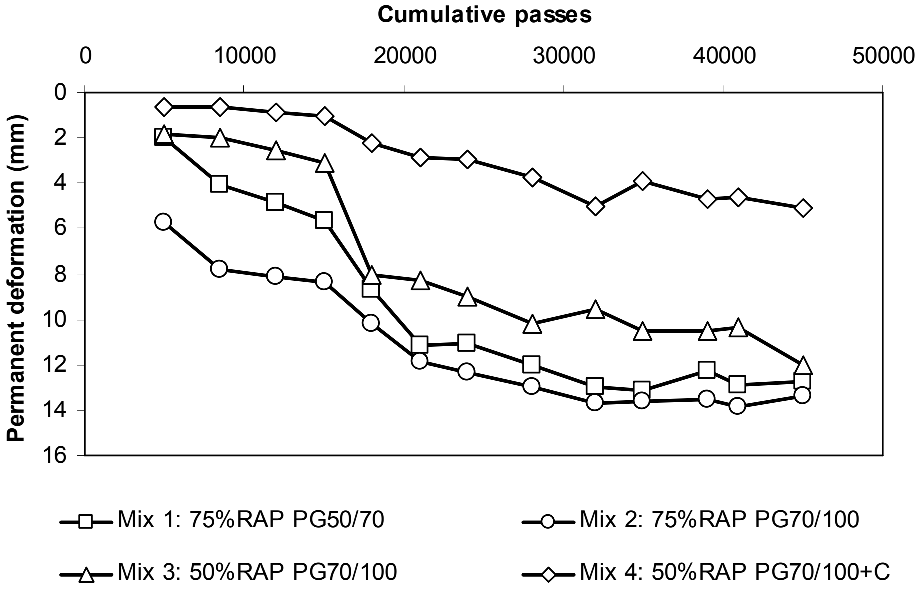


Figure 6.17 Deformation of trial pavement surface

For foamed bitumen bound materials, the differences in the rut depth values and profiles between different mixtures indicate that the rutting potential of stabilised mixtures depends on the binder type, mixture proportion and the presence of cement. In general, foamed bitumen bound mixtures that contain a higher proportion of RAP and a softer binder exhibited greater deformations.

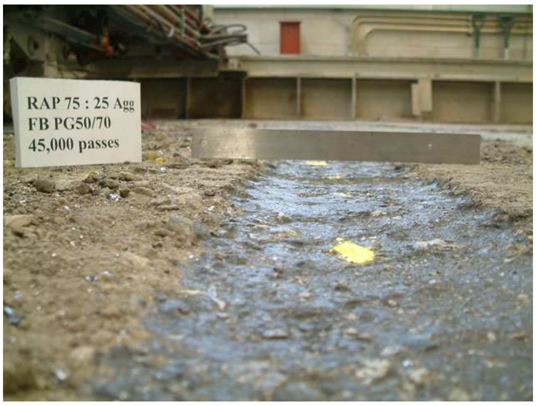
The effect of mixture proportions can be seen by comparing Mix 2 and Mix 3. Up to the first 5000 load applications, just before the load level was increased to 6 kN, the average rut depth in Mix 2 was approximately 6 mm and that of Mix 3, which contained less RAP, was 2 mm. At 15,000 load applications, the average deformation of Mix 3 was still about one third that of Mix 2. However, when the load was increased to 12 kN, the difference in rut depth between these two mixtures decreased. The final deformation at 45,000 load applications of Mix 2 was about 13.5 mm while the deformation of Mix 3 was about 12 mm, i.e. a difference of only about 1.5 mm. The effect of mix proportions on permanent deformation was more pronounced during early life and at low loads, but when the mixture was subjected to higher loads, this effect was less significant.

(a)

(b)

(c)

(d)



(e)

(f)

Figure 6.18 Pavement condition after trafficking

(a) cement treated 50%RAP

(b) cement treated 75%RAP

(c) foamed bitumen PG70/100 treated 50%RAP + cement 1.5%

(d) foamed bitumen PG70/100 treated 50%RAP

(e) foamed bitumen PG50/70 treated 75%RAP and

(f) foamed bitumen PG70/100 treated 75%RAP

The penetration grade of the binder also affected the permanent deformation during the early life and at low load. This effect can be shown by comparing the performance of Mix 1 and Mix 2, which contained the same amount of RAP but different binder grades. After the pavement had been subjected to 5000 passes at 3 kN load, the rut depth of Mix 1, containing the harder grade binder, was about 33% that of Mix 2. When the pavement was loaded with another 10,000 passes at 6 kN, the rut depth of Mix 1 increased to about 67% that of Mix 2. During the application of the 12 kN wheel loads, although Mix 2 deformed on average by a slightly greater amount, the permanent deformations of both materials were approximately equal with a difference of 1 mm or less. The harder penetration grade bitumen produces a foam that improves the mixture resistance to permanent deformation. However, when the mixture is subjected to higher loads, a harder penetration grade bitumen makes little difference compared to a softer grade bitumen.

Unlike conventional hot mix asphalts, foamed bituminous bound composites are visually very different from fully coated hot mixtures. In foamed mixes the fine aggregate and filler components are preferentially coated by the bitumen. It was clearly visible in this experiment that the RAP and coarse aggregate particles in the foamed bitumen bound mixtures, though bound together, were not fully coated by the bitumen. Also noticeable was the visibly voided nature of the mixtures. Mix deformation was therefore likely to depend on both particle interlock as well as binder stiffness. At low stress levels, the binder contribution to the mix response was relatively high resulting in improved response with the use of a harder grade binder. On the other hand, at high loads, the binder contribution to the mix behaviour was less evident and the deformation behaviour of the mixtures was primarily governed by the aggregate interlock regardless of the binder grade.

The effect of adding a small amount of cement is clearly observed from the performance of Mix 4. The magnitude and rate of deformation of Mix 4 was clearly smaller than that of all the other foamed bitumen mixtures. When the test was terminated at 45,000 load applications, all three foamed bitumen bound sections with no added cement had developed significant surface deformation with an average rut depth greater than 12 mm. On the other hand, the foamed bitumen bound section containing 1.5% cement deformed on average by only 5 mm. This 60% decrease in rutting indicates a significant increase in permanent deformation resistance caused by the inclusion of cement as an additive.

6.6.3 Strains at bottom of the stabilised layer

Strain gauges were located on the underside of the recycled materials. Data acquisition was performed at every 1000 passes. Typical horizontal transversal and longitudinal strain measurements are presented in Figure 6.19. The readings shown were taken at 28,000 passes. It can be seen that, longitudinally, as the wheel approached the strain gauge, compressive strain was developed, followed by tensile strain. Transversely, only tensile strain was developed.

There were 18 gauges in total and 16 of those gauges survived the compaction process undamaged. Each section had two transverse and one longitudinal strain gauge, and this allowed the elimination of data from gauges generating unreliable data. The measured horizontal strains of various sections are presented in Figure 6.20. Generally, tensile strains of all sections increased with the wheel loads as expected. Most strain values obtained from the longitudinal gauges were higher than those from transverse gauges. The measured strains were almost constant under the wheel loads of 3 kN and 6 kN, indicating no sign of local distress, followed by a slight decrease during the application of the 12 kN load. The reduction in strain may be due to the fact that the materials gain strength with time from the curing effect.

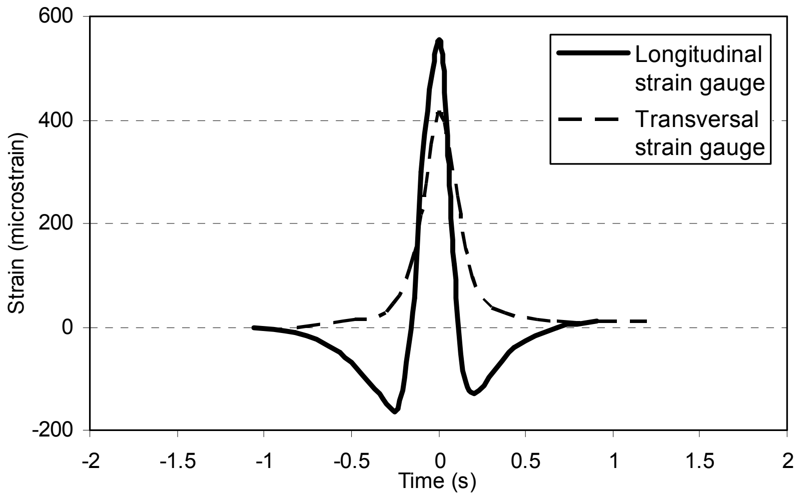
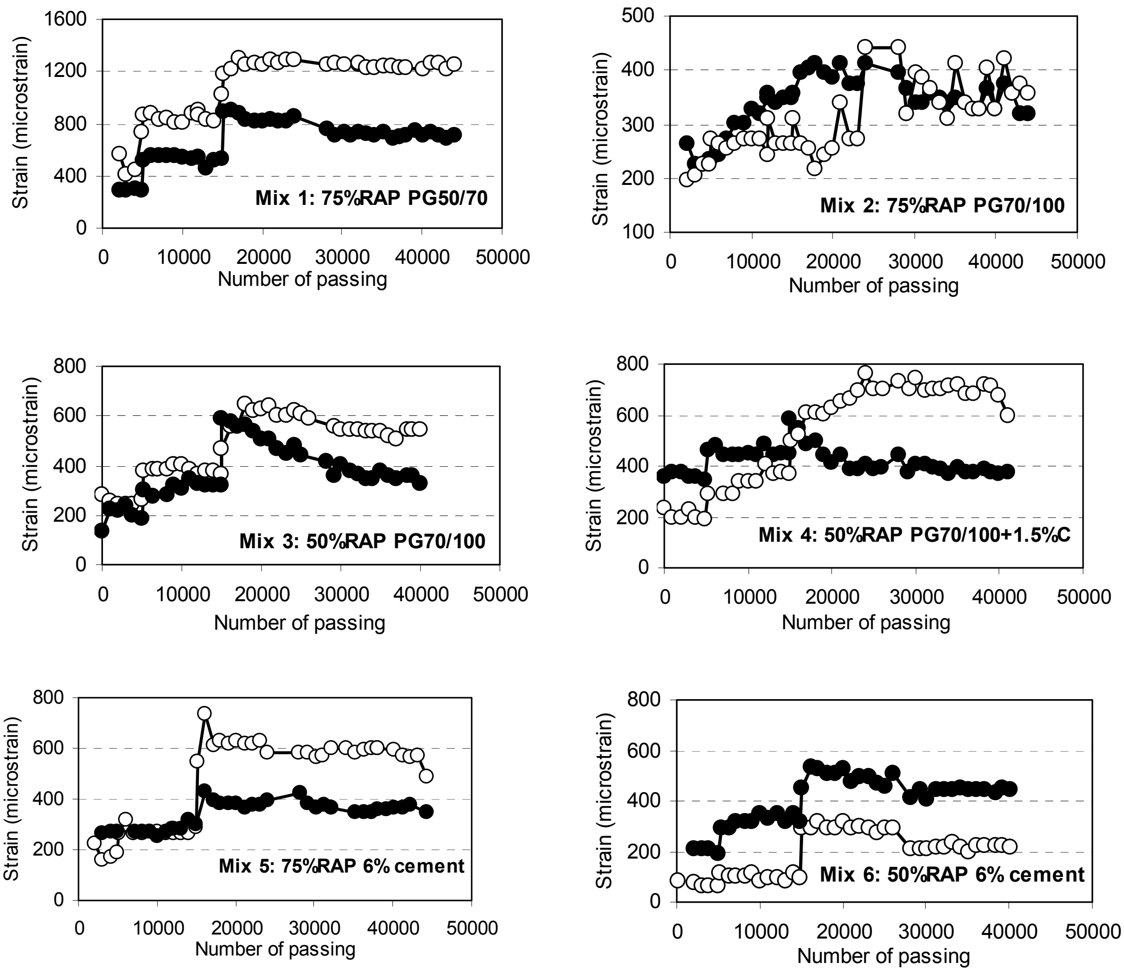


Figure 6.19 Typical tensile strain signal under stabilised layer



○ Longitudinal gauge, ● Transverse gauge

Figure 6.20 Tensile strain signals from embedment strain gauges under stabilised layer

However, there are exceptions in the strain responses from the Mix 2 and Mix 6 sections, which did not follow the above mentioned trend. The unexpected readings from the Mix 2 section may imply a lack of cohesion between the strain gauges and the surrounding material. In Mix 6, the strain values obtained from the longitudinal gauge were lower than those from transverse gauges, the opposite of other sections. This effect occurred in only one section and this is inadequate evidence to formulate an argument. Nevertheless, the overall trend was reasonable. The responses from one strain gauge in Mix 4 present an interesting feature. Unlike other gauges, the transient strain response increased gradually during trafficking especially at wheel loads of 6 and 12 kN, which implied a gradual reduction in modulus. It is speculated that, with respect to this gauge's response, one of the main reasons for the reduction in modulus of the stabilised layer is that cracks may have developed in the layer during trafficking.

6.6.4 Back calculated modulus

An attempt was made to estimate the modulus of the stabilised layer from the strain responses by using a computer program “BISAR” [Shell, 1998]. BISAR is a linear elastic pavement analysis program normally used to calculate pavement responses. The stresses, strains and deflections at any point in a linear elastic layered system, assuming infinite horizontal dimension, can be computed if layer thicknesses, elastic parameters, and loading characteristics are known. However, in this study, BISAR was used to back calculate the modulus of the stabilised layer by using its strain responses as the input. The following procedure was developed.

Firstly, an actual three-layered system was simplified to a two-layered system as the individual moduli of granular subbase and subgrade were unknown. From the LFWD test, the average modulus of the foundation, i.e. subbase + subgrade, was found to be 60 MPa. Therefore, the pavement was analysed as a two-layered system, consisting of the stabilised layer with a thickness of 80 mm as the upper layer, and the lower layer with infinite thickness and a modulus of 60 MPa. Secondly, a number of modulus values of the upper layer were assumed and the corresponding tensile strains at the bottom of the upper layer were calculated by BISAR at each applied wheel load. The results gave relationships between tensile strain and modulus of the upper layer as shown in Figure 6.21. Finally, by means of these relationships, the experimentally measured strains were used as the input and the modulus of the stabilised layer was determined. Table 6.2 summarises the backcalculated modulus results.

The modulus results appear to span a relatively wide range. This was primarily caused by the scatter of output from the various strain gauges within each test section. It is also easy to appreciate that the responses were greatly affected by fluctuations in load, plus variability in layer thickness and foundation stiffness across the test pit. Taking an overview of all the results, the modulus of an 80 mm foamed asphalt layer on a 60 MPa foundation at early life is a function of mixture composition and extent of curing. Foamed bitumen bound recycled asphalt mixes had stiffness values ranging from around 300 MPa at the lower end to about 2500 MPa at the upper end, whilst for the cement bound recycled layer, the stiffness values ranged from 1000 to 3000 MPa.

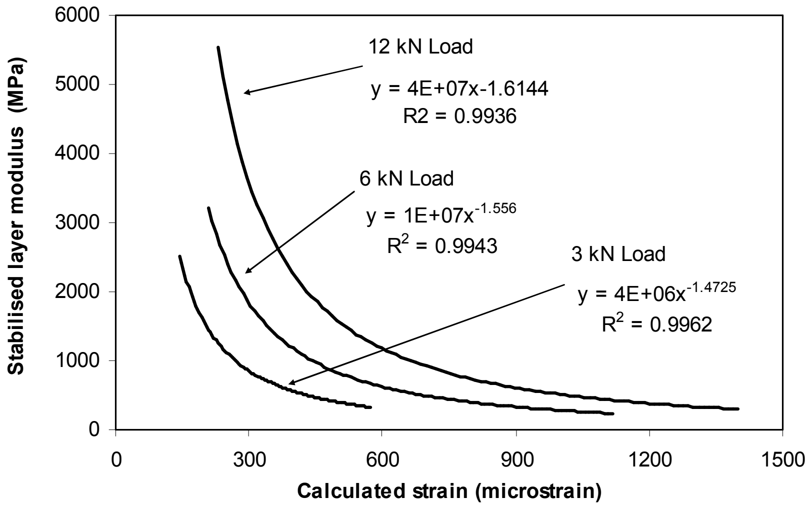


Figure 6.21 Relationships between calculated strain and modulus of stabilised layer based on two layered BISAR model

Table 6.2 An overview of calculated modulus values

Mixture	Calculated modulus during trafficking (MPa)	
	At the beginning	At the end
1. 75%RAP PG50/70	300 – 800	350 – 1000
2. 75%RAP PG70/100	Data was not reliable	
3. 50%RAP PG50/70	1000 – 1300	1500 – 2500
4. 50%RAP PG50/70+1.5%C	500 – 1500	600 – 2500
5. 75%RAP 6%C	1000 – 1500	600 – 2500
6. 50%RAP 6%C	1500 – 2000	2000 – 3000

6.6.5 Comparison with typical granular and bituminous materials

For comparison purpose, the moduli of granular and bituminous base courses with 80 mm thickness resting on a granular foundation having a modulus of 60 MPa were estimated.

According to the Shell design procedure [Claessen et al, 1977] the modulus of an unbound granular base layer can be calculated from the following relationship:

$$E_2 = kE_3 \qquad (7.1)$$

when, $k = 0.2h_2^{0.45}$ and $2 < k < 4$

where,

E_2	=	modulus of unbound base layer (MPa)
h_2	=	thickness of unbound base layer (mm)
E_3	=	modulus of underlying subgrade (MPa)

The modulus of a granular base course with thickness of 80 mm resting on a granular foundation having a modulus of 60 MPa was estimated to be about 120 MPa. The range of modulus values of crushed gravel base and crushed rock base in typical flexible pavements based on falling weight deflectometer tests has been reported to be 150 – 400 MPa and 250 – 600 MPa respectively [Lekso et al, 2002].

The elastic stiffness of an asphalt mixture can be estimated from that of the binder and the voids in mixed aggregate by the following equation [Heukelom and Klomp, 1964]:

$$S_{me} = S_b \left[1 + \frac{257.5 - 2.5VMA}{n(VMA - 3)} \right]^n \tag{7.2}$$

when, $n = 0.83 \log \left[\frac{4 \times 10^4}{S_b} \right]$ (7.2a)

where

S_{me}	=	stiffness modulus of an asphalt mixture (MPa)
S_b	=	stiffness modulus of binder (MPa)
VMA	=	voids in mix aggregate (%)

Stiffness of the binder (S_b) can be estimated from:

$S_b = 1.157 \times 10^{-7} t^{-0.368} 2.718^{-PI_r} (SP_r - T)^5$ [Ullidtz, 1979] (7.3)

and, $\log t = 5 \times 10^{-4} h - 0.2 - 0.94 \log V$ [Brown, 1973] (7.4)

where

t	=	loading time (seconds)
PI_r	=	recovered penetration index of bitumen
SP_r	=	recovered softening point of bitumen from the ring and ball test (°C)
T	=	temperature (°C)
h	=	thickness of asphalt layer (mm)
V	=	vehicle speed (km/hr)

The PI_r, SP_r and *VMA* of a typical dense bitumen macadam (DBM) base mixture using 50 pen bitumen as a binder can be assumed to be -0.2, 58.6 °C and 17.9% respectively [Brown and Brunton, 1988]. Hence, the modulus of a DBM road base of 80 mm thickness subjected to a vehicle speed of 3 km/hr and having a binder stiffness of 20.2 MPa at 20 °C was estimated to be approximately 3000 MPa. The modulus value of actual asphalt layer determined from falling weight deflectometer has been reported to be in the range of 2000 – 4000 MPa [Ruenkrairergsa et al, 2002].

It can thus be seen that in terms of modulus, foamed bitumen bound recycled mixtures exhibit excellent early life characteristics when compared with conventional unbound base materials and, as the curing time increases and the mixtures densify with additional trafficking, may become as good as those of typical asphalt road base materials. The modulus of cement bound recycled materials at the relatively short curing times tested in this investigation was also found to be similar to or better than that of typical dense bitumen macadam base mixtures. It is, however, significantly lower than typical cement bound base due to the inclusion of bitumen in the RAP. The comparison of back-calculated modulus of various stabilised materials with is presented graphically in Figure 6.22.

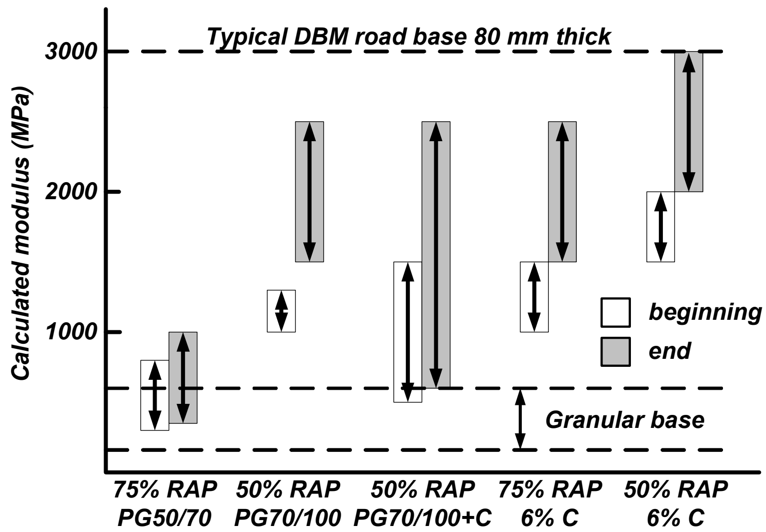


Figure 6.22 Back-calculated stiffness modulus of various stabilised sections

6.6.6 Stiffness of cored specimens

At the conclusion of the test, an attempt was made to core specimens from the foamed bitumen stabilised sections. Coring was carried out in the wheel path (trafficked) and away from the wheel path (un-trafficked). It was very difficult to core any suitable samples because the aggregate was weakly bound by foamed bitumen. However, some success was achieved by using a dry coring process with a 150 mm barrel. It was possible to core Mix 3 and Mix 4 at both positions but all the un-trafficked cores obtained from Mix 1 and Mix 2 were damaged during the coring process.

The stiffness modulus of the cored specimens was measured using the ITSM test and the results are shown in Figure 6.23. The calculated modulus values at the end of trafficking for each section are also superimposed on this figure.

The ITSM stiffness of the un-trafficked specimens in Mix 4 is about three times higher than those the trafficked specimens. The lower end of the calculated modulus band for this section coincides with the average trafficked ITSM value (about 500 MPa). However, in the case of Mix 3 the ITSM stiffness of trafficked and un-trafficked specimens were about 250 MPa and 350 MPa respectively, and far lower than the calculated modulus values as shown. Sample disturbance and damage during coring of low stiffness mixtures such as Mix 3 are likely to have caused a reduction in the measured ITSM values. This applies to both the trafficked and un-trafficked sections. The trafficked specimen ITSM stiffness values in Mix 1 also coincide with the lower limits of the calculated modulus for this section (about 400 MPa), whereas the ITSM values of the trafficked specimens of Mix 2 were the lowest of all (about 200 - 400 MPa)

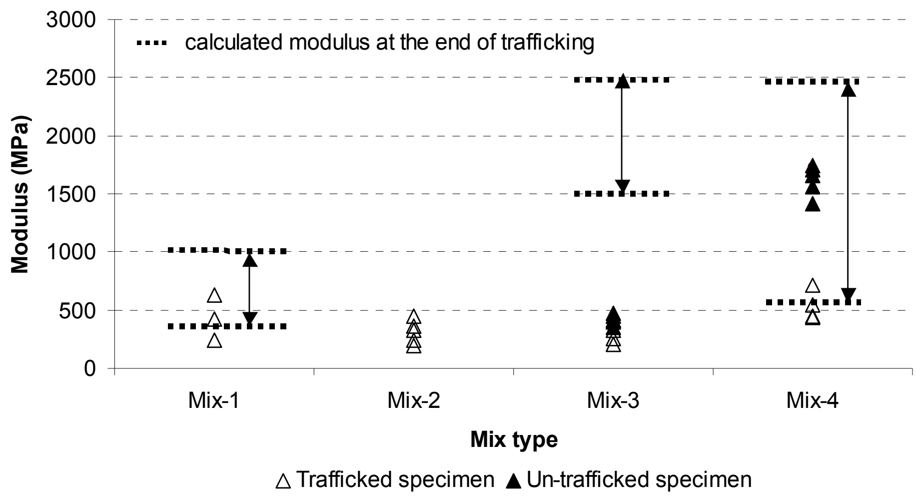


Figure 6.23 ITSM values of foamed bitumen cored specimens

In the case of cement bound materials, slabs were also cut from both cement bound sections. The slabs were then cut into several beams, having dimensions of 50 × 50 × 300 mm, to be tested for fatigue strength in the four point bending configuration. The test was conducted in displacement controlled mode. The results are shown in Figure 6.24.

It can be seen that the results obtained from four point bending test were very scattered. Apart from the brittle nature of cement bound materials, the reasons for this may be attributed to the specimen conditions. Firstly, the size of the beams (50 mm) were just above two times the largest aggregate size (20 mm) in the mixture. Thus, possibilities of disturbance that influence the fatigue life of the specimens, such as concentrations of binder, void, etc., were high. Secondly, the surfaces of some specimens were not sufficiently smooth. Cracking initiation and aggregate spalling occured during saw cutting. These defects tended to be starting points for the cracks during the bending test and made the specimens fail prematurely. Nevertheless, it can still be observed from the results that a greater fatigue life is found at lower applied strain levels. The materials can damage at relatively short life when subject to fatigue strain of at least approximately 300 microstrains.

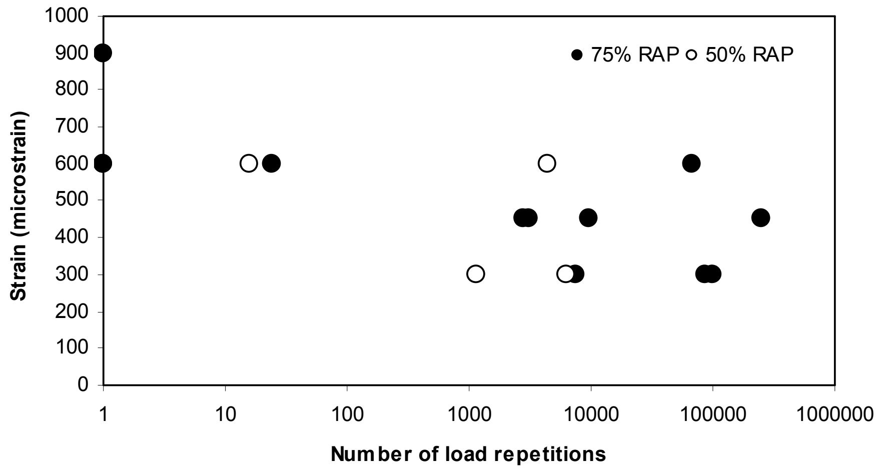


Figure 6.24 Results of four point bending tests on cement stabilised materials from PTF pit

6.7 Conclusions

The Nottingham Pavement Test Facility was used to traffic a pilot-scale simulated recycled pavement. The recycled pavement materials included RAP and limestone aggregate mixed in two RAP/limestone proportions namely, 75/25 and 50/50. The mixtures were treated with three types of binder including foamed bitumen, cement and foamed bitumen plus 1.5% cement. Two penetration grade bitumens, PG50/70 and PG70/100 were used to generate the foamed bitumen. Six different mixtures were tested. On the basis of the experimental work described, the following conclusions can be drawn:

1. The half-scale experiment indicated that foamed bitumen stabilised materials tend to fail in rutting mode rather than fatigue cracking.

2. The results were consistent to the laboratory element tests that the resistance to permanent deformation of foamed bitumen bound materials is affected by the amount of RAP in the mixture and also the penetration grade of the bitumen used to generate the foam. Foamed bitumen bound mixtures that contain a higher proportion of RAP and a softer binder exhibit greater deformation. However, this effect is less significant as the magnitude of load and curing period increase.

3. The addition of a small percentage of cement to the foamed bitumen bound mixtures significantly enhances the resistance against rutting failure. However, fatigue damage (seen as a loss of stiffness modulus) can occur when cementitious additives exist in the mix.

4. The deterioration of cement stabilised recycled pavement could not be observed as no apparent damage occurred on the test sections. The strain responses under the cement stabilised layer also implied that the materials were in an elastic state.

5. Overall analysis of the strain gauge results shows that the modulus of foamed bitumen bound recycled materials during early life is comparable to or can be higher than that of conventional unbound base materials and can be as high as that of traditional asphalt base material. The predicted modulus of cement bound materials was also comparable to that of traditional asphalt base material.

7 EFFECTS OF CEMENTITIOUS ADDITIVES ON MECHANICAL PROPERTIES OF FOAMED BITUMEN STABILISED MIXTURES

7.1 Introduction

Foamed bitumen can be used in combination with additives such as cement to stabilise recycled pavement materials. The laboratory work carried out on the pilot-scale recycled pavement, as described in Chapter 6, has shown that only a small addition of Portland cement can have a significant beneficial effect on the foamed bitumen stabilised mixture. Also the use of additives may improve the durability of mixes. However, the result from the trial pavement was merely a comparison and not comprehensive. The aim of the work described in this chapter is to quantify further the effect of Portland cement on the mechanical properties of foamed bitumen stabilised materials. In addition to Portland cement, the effect of hydrated lime ($Ca(OH_2)$) as an additive was also assessed. The methods used to measure the mechanical properties of materials in this study were mainly based on the Nottingham Asphalt Tester. The apparatus is based on pavement engineering principles and, therefore, the results it produces are more related to the fundamental properties of the mixtures.

7.2 The Nottingham Asphalt Tester

General description

Nottingham Asphalt Tester (NAT) was the main equipment in this study. The NAT was developed at the University of Nottingham as a simple apparatus for measurement of mechanical properties of asphalt mixes [Cooper and Brown, 1989]. Basically, it consists of a main test frame into which modules are placed in order to carry out a variety of tests. The main test frame has an actuator situated on the top to apply a dynamic load. Load is measured with a strain gauged load cell and deformation of specimens was measured with LVDTs. The acquisition of data and the control of the system are carried out using a computer connected to a digital interface and running using user-friendly software. The NAT test is normally carried out inside a temperature controlled cabinet. Figure 7.1 shows a schematic diagram of the NAT.

The NAT can be configured for testing specimens in the indirect tensile mode, for stiffness modulus determination, or for fatigue testing, or in the repeated load axial mode for assessing the permanent deformation of mixtures as shown in Figure 7.2.

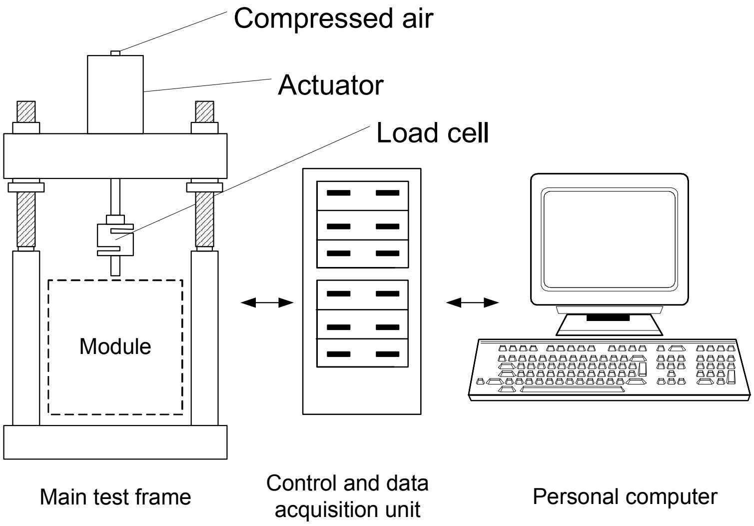


Figure 7.1 Schematic of the Nottingham Asphalt Tester

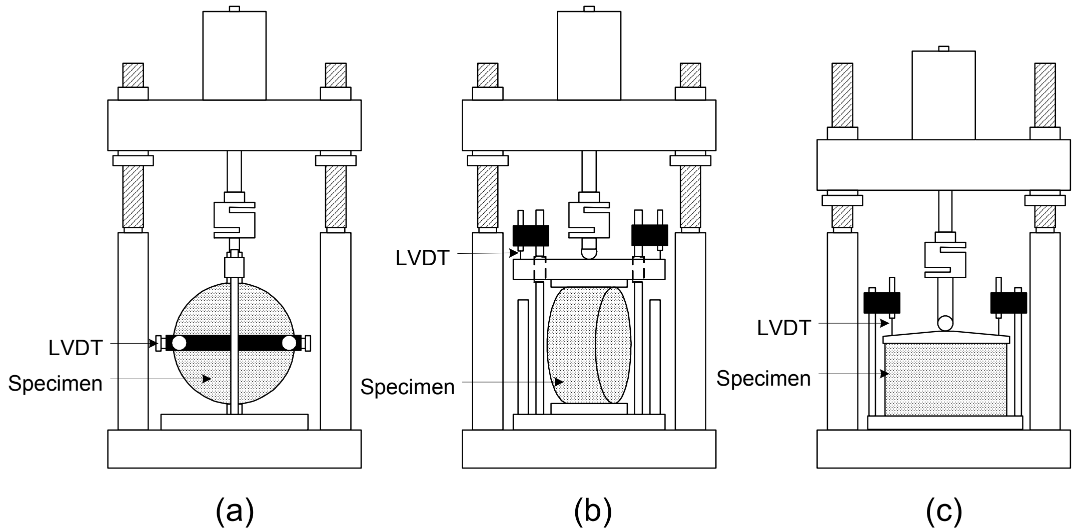


Figure 7.2 Diagrams of test modules for the NAT; (a) ITSM, (b) ITFT, and (c) RLAT

Indirect tensile stiffness modulus test

The stiffness modulus of a specimen can be determined by the indirect tensile stiffness modulus (ITSM) test. The specimen is loaded diametrically with a vertical compressive pulse, which generates a tensile stress indirectly across the horizontal diameter, as shown in Figure 7.3. The load pulse is applied so that the specimen deforms horizontally at a specified value and the peak applied vertical load is measured. According to elastic theory, the stiffness modulus (S_m) is a function of load, deformation, specimen dimensions, and Poisson's ratio as expressed in the following formula [DD213:1993].

$$S_m = \frac{L}{(\Delta h \times t)} \times (\nu + 0.27) \tag{7.1}$$

where,

L = peak value of applied vertical load
Δh = diametrical horizontal deformation
t = thickness of specimen
ν = Poisson's ratio

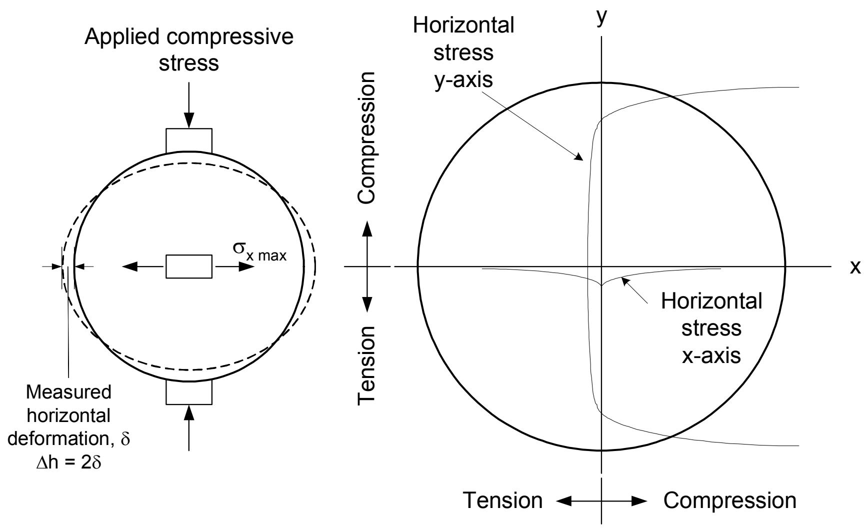


Figure 7.3 Horizontal stress distribution in indirect tensile mode of test [Adapted from Read and Whiteoak, 2003]

Indirect tensile fatigue test

The indirect tensile fatigue test (ITFT) uses the module shown in Figure 7.2(b). The design of this module is based on the ITSM test configuration. A specimen is subjected to a repeated controlled vertical stress pulse diametrically until it fails. Two LVDTs are used to measure the vertical deformation of the specimen. The failure is defined when the specimen is split or the vertical deformation reaches 9 mm. The maximum horizontal tensile stress, $\sigma_{x\ max}$, occurs in the middle of the specimen (Figure 7.3). This stress and the corresponding horizontal tensile strain can be calculated by Equations 7.2 and 7.3 [DD ABF:2003].

$$\sigma_{x\max} = \frac{2P}{\pi dt} \tag{7.2}$$

$$\varepsilon_{x\max} = \frac{\sigma_{x\max}}{S_m}(1+3\nu) \tag{7.3}$$

where,

P	=	vertical compressive force
$\sigma_{x\max}$	=	maximum horizontal tensile stress
$\varepsilon_{x\max}$	=	maximum horizontal tensile strain
ν	=	Poisson's ratio
d	=	diameter of the specimen
t	=	thickness of the specimen
S_m	=	stiffness modulus of the specimen

The ITFT is normally carried out on several specimens at several stress (strain) levels. The results of ITFT tests are usually presented as a relationship between tensile stress (or strain) and the corresponding number of load cycles to failure. Generally, the fatigue relationship can be expressed in terms of initial tensile strain as follows.

$$N_f = c \times \left(\frac{1}{\varepsilon_{x\max}}\right)^m \tag{7.4}$$

where,

N_f	=	number of load applications to failure
$\varepsilon_{x\max}$	=	maximum value of applied tensile strain
c, m	=	factors depending on the composition and properties of the mixture; m is the slope of the fatigue line

<u>Repeated load axial test</u>

The repeated load axial test (RLAT) is a simple method of assessing the resistance to permanent deformation of a specimen subjected to simulative of traffic loading. The configuration of the test is shown in Figure 7.2(c). The repeated stress pulses are applied for a duration of one second, with an interval of one second between load pulses. Permanent axial deformation is monitored with LVDTs attached directly to the top of the upper loading platen. The relationship between axial strain and number of load pulses can be obtained from this test.

7.3 Experimental programme

7.3.1 Description of laboratory tests

Because the objective of this study is to investigate the effect of binder (foamed bitumen plus additives) on the mixtures properties, only limestone aggregate, without the addition of RAP, was used. The limestone described in Chapter 3 was utilised. The cementitious additives used in this study were cement and lime. The cement was limestone cement (CEM II) [BS 197-1:2000] and the lime was hydrated lime ($Ca(OH_2)$). Cement was included in the mixture at 1 % and 2 % by weight of dry aggregate while hydrated lime was blended at 1 %. Foamed bitumen was produced from PG50/70 bitumen at 180 °C with foaming water content 2% by mass of liquid bitumen. The corresponding expansion ratio and half-life were 15 times and 9 seconds respectively. In addition to the foamed bitumen mixtures, hot mix specimens were prepared for comparison. The aggregate grading and density were the same. Hot bitumen content equivalent to the binder content of foamed bitumen treated mixtures was used.

The testing programme consisted of the determination of water loss, stiffness modulus, resistance to fatigue cracking, resistance to permanent deformation, and resistance to water damage.

7.3.2 Specimen preparation

The aggregate of all mixtures had the same gradation as shown in Figure 7.4. However, the filler component of the aggregate was replaced by active fillers, i.e. cement and hydrated lime, as given in Table 7.1. All specimens were prepared at optimum foamed bitumen content of 4% based on the mix design described earlier (section 3.3.3). The water content of the specimens was found to be 4.8 %. Cylindrical specimens, 100 mm in diameter, were manufactured using a Gyratory compactor. Each mixture was prepared in sufficient quantity to allow a 1100 g specimen to be produced. Each specimen was compacted to 200 gyrations at a pressure of 600 kPa and an angle of 1.25°. Compaction was performed at ambient temperature. The variation of the mean dry density and range of densities are shown in Table 7.2. Cold mix cores were extracted from their moulds after 12-16 hours and then stored at room temperature (20 ±

5 °C) until the required testing time. In the case of hot mix specimens, the aggregate gradation, bitumen type and bitumen content were identical to those of cold mix specimens. The specimens were compacted at 135 °C and no pre-wet water was added.

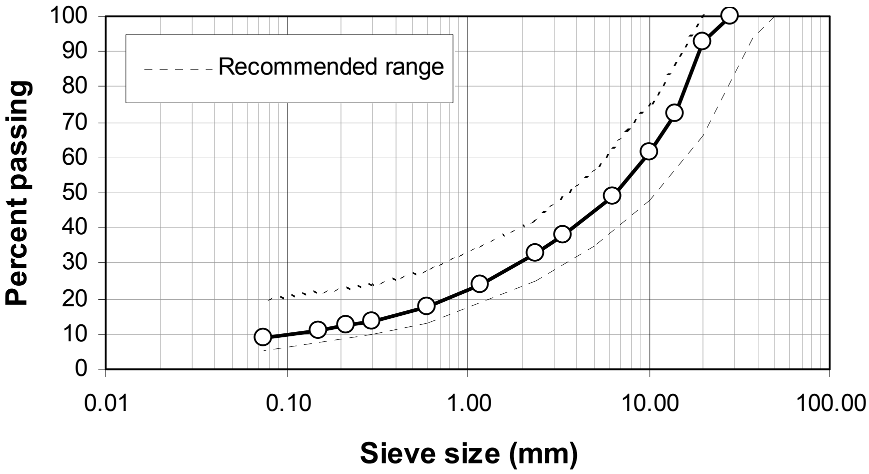


Figure 7.4 Gradation of studied aggregate

Table 7.1 Mixture compositions used to study effect of cementitious additives

Mix component		**Percentage of aggregate mix**			
Limestone aggregate	20 mm	26			
	14 mm	15			
	10 mm	9			
	6 mm	11			
	Dust	36			
	Filler	3	2	1	2
Active fillers		0 (Cement)	1 (Cement)	2 (Cement)	1 (Lime)

Table 7.2 Variations in dry density of specimens

Mixture	**No. of specimens**	**Range of dry density (Mg/m^3)**	**Average dry density (Mg/m^3)**	**Standard deviation (Mg/m^3)**
Cement 1 %	36	2.175 – 2.284	2.230	0.029
Cement 2 %	36	2.141 – 2.257	2.201	0.035
Lime 1 %	36	2.141 – 2.243	2.191	0.023
No additive	36	2.121 - 2.240	2.192	0.027
Hot mix	12	2.130	2.130	-

7.3.3 Test procedures

Test set-up

Prior to every test, the thickness and diameter of specimens were measured. For the RLAT, the end surfaces of specimens were coated with thin layers of silicone grease and graphite powder to eliminate unevenness of specimen surfaces and reduce friction between loading platens and the specimen surfaces. The specimens were placed in the temperature control cabinet at the required test temperature at least 2 hours before conducting the test.

It is noted that the term 'fully cured' specimen in this experiment means that the specimen was left at room temperature (20 ± 5 °C) for at least one month and then further cured in an oven at 40 °C for 3 days.

Stiffness modulus

The stiffness modulus was determined by the ITSM test in accordance with DD213:1993. Six sets of three (total of eighteen) specimens from each mixture were used. Specimens were tested at 20 °C, with a load target rise time of 124 ms and a target horizontal displacement of 5 μm. Poisson's ratio of the materials was assumed to be 0.35. Each specimen was tested in two orientations, 90° ± 10° apart. The result for each orientation was determined from average values of the 5 load pulses. If the values from these two orientations are within 10 % difference, the mean value of the two test results was then reported as the stiffness modulus of the specimen. If the difference between two test results was greater than 10 %, the test was repeated and if the difference persisted, the value for each test was reported individually. ITSM tests were conducted on the first set of specimens after 7 days of curing and another five times with different sets of specimens over the curing period of approximately 4 months.

Resistance to fatigue cracking

The resistance to fatigue cracking of the mixtures was assessed by the ITFT procedure as described in DD ABF:2003. Twelve specimens of each mixture were used and all specimens were tested at their fully cured state. The specimens were cured at ambient temperature for at least one month and then in an oven at 40 °C for 3 days. The specimens were firstly tested for stiffness, which is required for calculation of strain generated at the centre of the specimen during the fatigue test and were then placed in the ITFT module in the NAT frame. The test was performed at 20 °C in a stress-controlled mode at the rate of 40 pulses/minute. The horizontal stress was set to vary among any one set of specimens in order to achieve a range of lives and to generate a fatigue line. Once all tests were completed, the initial tensile strains generated at the centre of the specimens were calculated and were plotted against life to failure on a log-log scale.

Resistance to permanent deformation

The resistance to permanent deformation of the mixtures were evaluated by the RLAT procedure in accordance with DD 226:1996. Three fully cured specimens of each mixture were tested. In this test, the prepared specimen is positioned vertically between the upper and lower loading platens. The static conditioning load of 10 kPa was applied for 600 seconds prior to testing to ensure that the loading platens were properly seated onto the specimen. Repeated stress pulses of 100 kPa were then applied axially for a duration of one second, with an interval of one second between load pulses, and run for 3600 cycles, giving an accumulated loading time of two hours. The test was carried out at 40 °C in order to amplify the deformation result. The axial deformation and applied stress data were recorded automatically together with number of load cycles.

Water loss

The water loss measurement was simply taken as the loss of weight of the specimens. Three specimens from each mixture were used in this test. In order to prevent loss of the crumbled mass but to allow water loss, each specimen was kept separately in an open container of known weight. Both the specimen and its container were left in an ambient temperature and weighed together every day. When the ambient curing measurement had been completed the air-dried samples were fully dried in an oven at 40 °C for 3 days. The weight of mixtures without water can then be measured. The water loss over the curing period was back-calculated from the oven-dried weight. The oven-dried specimens were also used for ITSM testing to obtain the stiffness modulus values in a "fully cured" state. These specimens were further used for the resistance to water damage test.

<u>Resistance to water damage</u>

Three specimens from each mixture were cured for one month at room temperature and oven cured at 40 °C for 3 days. ITSM was first measured on the fully cured specimens in the dry condition. The same sets of specimens were then soaked in water at 25 °C for 24 hours. They were removed and surface dried and the ITSM was performed once again on the soaked specimens. The effect of cementitious additives on resistance to water damage of foamed bitumen bound mixtures in this study was evaluated by comparing the stiffness of mixtures before and after soaking.

7.4 Results

7.4.1 Stiffness modulus

Figure 7.5 demonstrates the effect of amount of additives and curing period on the stiffness modulus of specimens from the ITSM test. The results indicate the rate of gain in stiffness modulus for each type of mix. For hot mix specimens, there is only one data point representing the average of the resilient modulus for ten specimens determined approximately 1 month after compaction. It was assumed that the stiffness of hot mix specimens was constant over time. The fully cured data for foamed bitumen mixes were plotted at a curing time of 365 days (1 year).

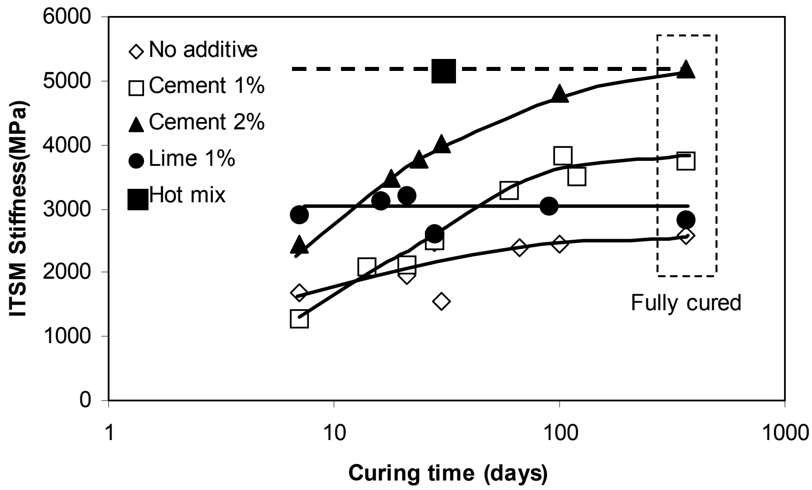


Figure 7.5 ITSM stiffness modulus of foamed bitumen stabilised mixtures with cementitious additives and hot mix specimens.

It can be seen that the stiffness modulus of specimens with cement additive increased continuously over several weeks. The stiffness also increases with increasing cement content. For foamed bitumen mixes, the stiffness modulus increases until about 100 days and then tends to be constant. The rate of gain in stiffness with time of specimens with cement is higher than that of specimens without additive. It can also be noticed that the rate of increase in stiffness modulus with time for both cement contents is approximately the same.

In the case of hydrated lime additive, the stiffness modulus is significant higher than that of foamed bitumen mixture without additive during the first few months. However, the stiffness modulus of specimens with hydrated lime is approximately constant over the curing period. On the other hand, stiffness modulus of foamed bitumen mixtures keeps increasing with time and in the fully cured state, the stiffness modulus of specimens with no additive comes close to that of foamed bitumen with hydrated lime mixtures.

In the fully cured state, the additions of 1 % hydrated lime, 1 % cement and 2 % cement increase the stiffness approximately 7.7 %, 45.8 % and 94.1 % of the stiffness relative to foamed bitumen mixture respectively.

It is noted from the figure that the curing of foamed bitumen stabilised specimens at 40 °C for 3 days is compatible with the assumption that it represents a year of curing period at ambient temperature as the data fall reasonably well on the stiffness–curing period trend lines.

7.4.2 Water loss

The percentage water loss is shown graphically in Figure 7.6.

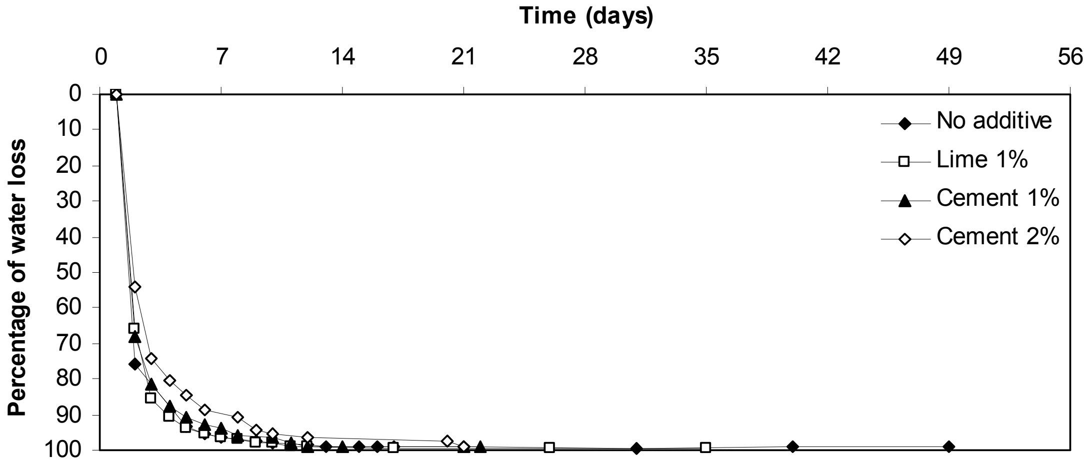


Figure 7.6 Water loss of foamed bitumen stabilised specimens

Generally, the majority of water loss occurred in the first week after specimens were produced. The remainder of the water seems to continue to evaporate over several weeks but after 14 – 15 days no further loss of weight of specimens could be detected.

The addition of cement additives tends to decrease the amount of water loss of the specimens at a specific time. More cement added results in slower rate of water loss. The rate of water loss of foamed bitumen mixes without additive is very close to the rate of water loss of specimens with hydrated lime.

7.4.3 Resistance to fatigue cracking

The NAT was used to conduct the ITFT. The specimen was loaded diametrically with a repeated vertical compressive force, which generated a tensile stress repeatedly across the vertical diameter.

Due to the duration of the test procedure, it was difficult in the laboratory to perform the ITFT for all specimens at the same specific stage of curing. The test was, therefore, conducted on fully cured specimens. All foamed bitumen stabilised specimens with and without additive failed in fully split apart manner. In order to generate a fatigue line, twelve specimens were used for each mixture.

The results from ITFT tests carried out on foamed bitumen mixtures with a range of cementitious additives are shown in the graph in Figure 7.7. The *c* and *m* values (from Equation 7.4) as well as R^2 for the data shown in Figure 7.7 are given in Table 7.3.

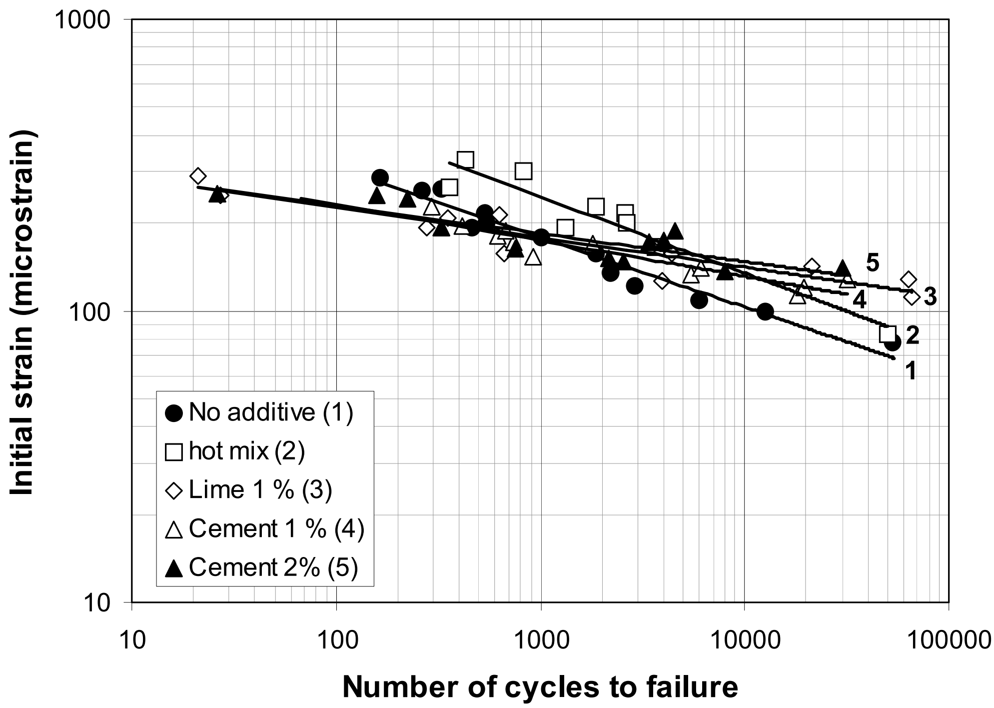


Figure 7.7 Relationship between initial strains and number of cycles to failure for foamed bitumen mixtures

Table 7.3 Linear regression values for foamed bitumen mixtures with cementitious additives

Additive	c	m	R^2
No additive	1×10^{12}	4.0285	0.9616
Hot mix	2×10^{11}	3.4412	0.9025
Lime 1%	2×10^{22}	8.5109	0.8672
Cement 1%	1×10^{19}	7.1714	0.8806
Cement 2%	2×10^{20}	7.6360	0.7371

From the data plotted in Figure 7.7, the slope of fatigue lines for hot mix and foamed bitumen stabilised mixtures was found to be similar but it is obvious that hot mix has a higher fatigue life compared to foamed mixes when they are subjected to the same initial tensile strain level. When cementitious additives were added to the foamed bitumen mixtures, their fatigue performances clearly change as can be seen from the slope of the fatigue lines. At an initial strain above 180 microstrain, the specimens with cementitious additive failed at a smaller number of load cycles when compared with specimens with no additive, but below 180 microstrain the reverse was true. That is when considering only the portion of the fatigue lines below 180 microstrain the result indicates that the addition of additives increases the resistance to fatigue cracking of foamed bitumen stabilised mixtures. The specimens stabilised with only foamed bitumen exhibit the poorest fatigue performance and the specimens with additives performed better in the following order, from best to worst; (1) cement 2%, (2) hydrated lime 1% and (3) cement 1%. The increase in cement content also increases the resistance to fatigue damage. It can be seen that the fatigue lines of specimens with 1% hydrated lime and 2% cement are close to each other. Nonetheless, above 180 microstrain the fatigue lines of all foamed bitumen mixes with additives seem to converge into one single line.

The R^2 values of the fatigue lines are shown in Table 7.3. It can be seen that the spread of data for the 0% cement and hot mix was quite narrow, as indicated by relatively high R^2 values, but when additives were added to the mixes distribution of data is wider, i.e. R^2 values become lower, especially for the 2% cement mixture.

Fatigue life can be divided into two stages namely, crack initiation and crack propagation. The repetitive application of applied tensile loads causes the coalescence of micro cracks to form one large crack (at the point of crack initiation N_1 load applications) which then propagates to failure (life for crack propagation N_f load applications). The definition of N_1 is achieved by plotting the number of cycles (N) against the number of cycles divided by vertical deformation (V_d) and this clearly shows the point of crack initiation at the peak of the graph when testing in the controlled stress mode. Once N_1 has been identified the additional life caused by crack propagation is easily determined. This method, based on dissipated energy concepts, assumes that stiffness is inversely proportional to the vertical deformation. Full detail of the method defining N_1 using stiffness change can be found in Rowe (1993).

Figure 7.8 shows typical plots of N/V_d versus number of load cycles of selected specimens at various initial horizontal tensile strains. The number of load cycles has been normalised by the number of load cycles at failure (N_f).

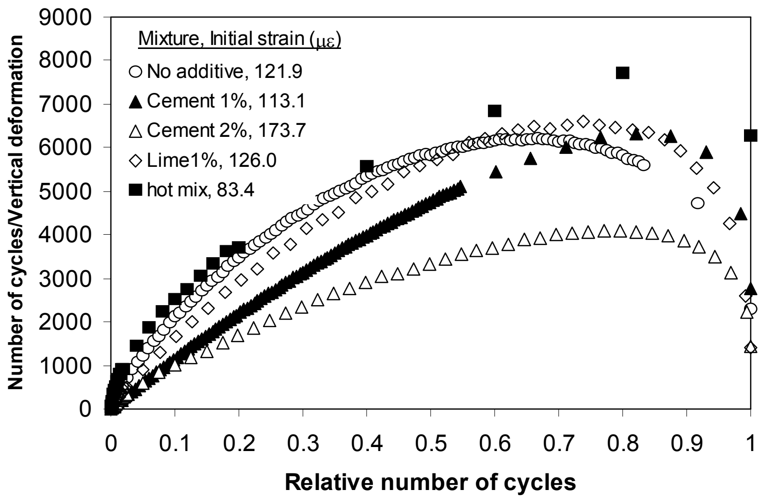


Figure 7.8 Crack propagation characteristics of foamed bitumen mixtures

From Figure 7.8, it can be seen that the crack propagation period of foamed bitumen mixtures is about 30 – 40 % of total fatigue life. On the other hand, the crack propagation period noted for hot mix specimens in this study is around 10 - 20 % of the total fatigue life. In other words, the hot mix displays more rapid crack growth after the crack has been initiated while the foamed bitumen mixtures exhibit a relatively slow propagation of the crack between initiation and failure. However, when additives were added to foamed bitumen mixes, the mechanism of crack propagation through the materials appears to be different. The mixtures with additives clearly show quicker crack growth after crack initiation to failure when compared to foamed bitumen mixes without additive. The crack propagation period of foamed bitumen mixtures with 1% and 2% cement was found to be about 15 % of total fatigue life. For foamed bitumen specimens with 1% hydrated lime, crack propagation life was around 25 % of total fatigue life.

7.4.4 Resistance to permanent deformation

The graph shown in Figure 7.9 presents the results from RLAT tests on triplicate specimens with different levels of cement additives.

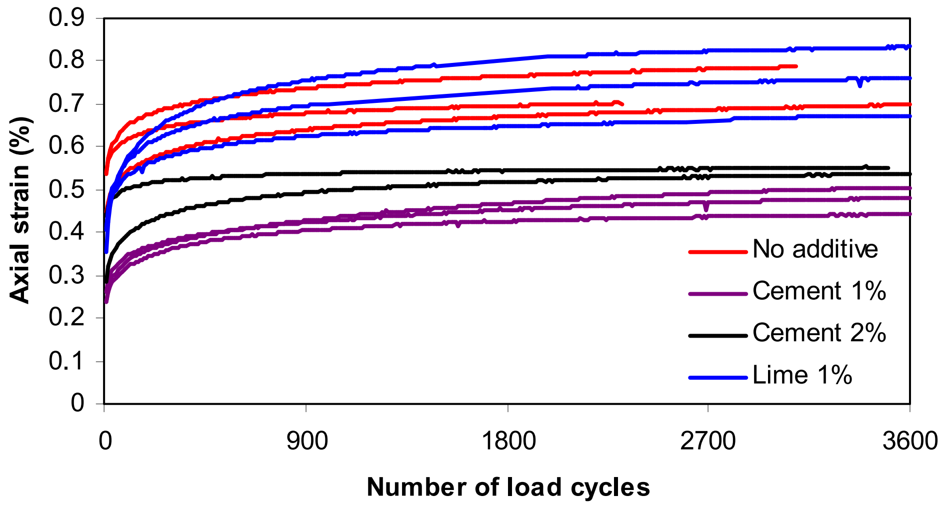


Figure 7.9 Resistance to permanent deformation of foamed bitumen mixtures

The results shown in Figure 7.9 indicate that the resistance to permanent deformation of foamed bitumen stabilised mixtures was improved by the addition of cement additives. However, the increase in cement content from 1% to 2% does not seem to improve the resistance to permanent deformation, as their RLAT results are fairly similar. Also the permanent deformation of specimens with 1% hydrated lime appears to be similar to that of specimens with no additive indicating that hydrated lime does not have significant effect in improving resistance to permanent deformation of foamed bitumen stabilised mix.

7.4.5 Resistance to water damage

The results of the dry and soaked ITSM tests of foamed bitumen mixtures are shown in Figure 7.10.

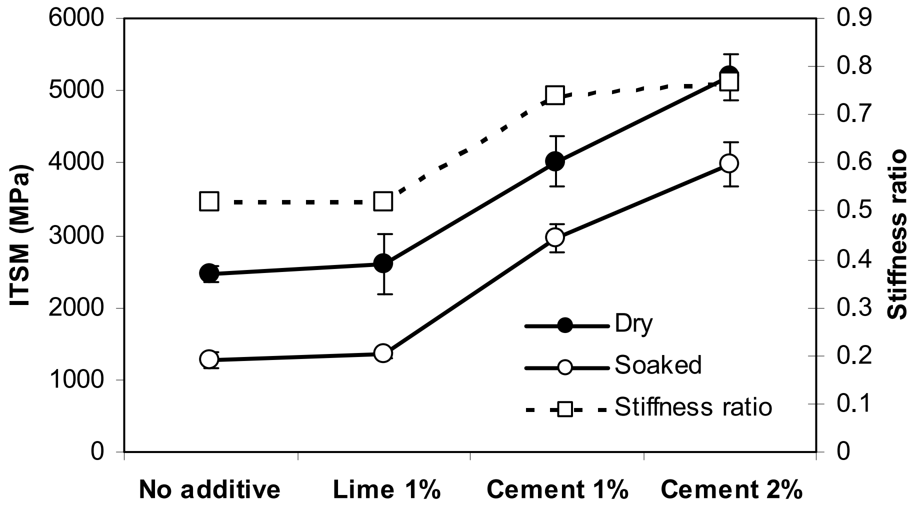


Figure 7.10 Stiffness modulus of foamed bitumen mixtures with additives in dry and soaked conditions

The comparison of dry ITSM and soaked ITSM is expressed in terms of stiffness ratio which is defined as the ratio between soaked and dry ITSM. The results shown in Figure 7.10 indicate that after soaking in water at 25 °C for 24 hours all mixtures exhibit lower ITSM values due to water damage. As shown in the figure, the mixtures with cement additive have relatively high resistance to water damage with a stiffness ratio of nearly 80 %. An increase in cement content from 1% to 2% slightly improves the resistance to water damage of the materials. On the other hand, hydrated lime seems to have no effect on improving water resistance of foamed bitumen mixtures. The stiffness ratio of the mixes with hydrated lime is almost the same as that of the mixtures without additive. Foamed bitumen mixtures and foamed bitumen with hydrated lime stabilised mixtures both retain about 50 % of ITSM values after being immersed in water.

7.5 Discussion

Cement, even only a small amount, influences the mechanical properties of foamed bitumen stabilised mixtures. It has beneficial effect in producing an excellent rutting resistance and a high stiffness modulus in laboratory specimens even after they were immersed in water.

Cement, in engineering terms, is a substance consisting of a range of mineral compounds which harden and adhere after being mixed with water. The constituent compounds of cement chemically react with water producing certain products which develop strength with time and become hard mass. This phenomenon is known as the hydration process. More details of the hydration of cement can be found elsewhere [e.g. Williams, 1986].

Needham (1996) found that cement is inert in hot mix asphalt. That is the addition of ordinary Portland cement into hot mix asphalt does not affect its strength in terms of stiffness modulus. In contrast, Needham (1996) and Giuliani (2001) showed that by adding cement to bitumen emulsion mixes, the performance of the stabilised mixtures was improved when compared with that of conventional emulsion mixes. Brown and Needham (2000) found that cement hydration took place in the emulsion mixtures because of the watery phase of the bituminous emulsion. Therefore, it can be argued that if there is water the presence of bitumen does not prevent cement from forming the cement hydrated compound. Thus, cement is not an inert filler in the process of cold recycling. A study at a molecular level [Montepara and Giuliani, 2002] was able to prove that when cement is added to bituminous emulsions, a new binder is not generated. The rigidity effects are due to two simultaneous mechanisms, namely, the emulsion breaking and the cement hydrating during the watery phase of the bituminous emulsion. The hydrate formed in-situ fills part of the porosity of the composite and contributes towards its cohesion and rigidity. In this study, the test results clearly suggest that cement also acts as a secondary binder in foamed bitumen mix. A very similar set of explanations are expected when foamed bitumen is used instead of a bituminous emulsion.

The foamed bitumen and the cement additives, in the presence of water, tend to form a mastic with the fine particles in the mix. Hydration is necessary to activate the cement and this can only occur in the cold mix material in which water exists. The mastic then forms a cohesive mixture which binds the larger aggregate particles generating a mixture with tensile as well as compressive properties. The cementitious additives such as cement and lime in the filler portion of the mixture react with the moisture present, converting it to a hydraulic product. The loss of moisture together with the creation of a secondary binder results in an increase in the strength of stabilised recycled asphalt pavement materials. Obviously, an increase in the cement content will increase the strength gain with time and yields a cold mix material with increasing stiffness modulus.

After compaction, foamed bitumen mixtures appeared to develop their stiffness while the moisture in the mixes was being lost. One of the differences between foamed bitumen mixture and hot mix asphalt is the way they achieve long-term characteristics after placement. Hot mix asphalt shows its long-term characteristic almost immediately after placing whereas foamed bitumen mixture needs curing time to gain its final characteristics due to the presence of water. There is a period of time in its early life when the material is weak and vulnerable to distress if time and environmental effects are not adequate to permit this process to develop to a sufficient level. The use of cold mix material therefore needs to be carefully considered both in respect of traffic volume and climatic considerations at the time of placement to ensure that the stiffness gain occurs in an acceptable period.

Hydrated lime has been used extensively in hot mix to improve adhesion and has also been found to offer similar benefits in bitumen emulsion mixtures [Needham, 1996]. In this study, it was found that the addition of 1% of hydrated lime to the foamed bitumen mixtures increases the stiffness of materials slightly. However, hydrated lime specimens achieve their 'peak' stiffness in less than 7 days, probably because hydrated lime reacts vigorously with water and generates a substantial amount of heat which assists curing soon after compaction. Therefore, hydrated lime may be a useful additive for foamed bitumen mixtures when considering the rate of stiffness gain with time. The shorter the curing time the shorter the time needed to wait before the surface course of hot-mix asphalt can be applied and hence the shorter the total construction time.

The specimens with cement additive, in the fully cured state, showed significantly higher ITSM values than the specimens without additive and with hydrated lime. However, it should be noted that materials with excessively high stiffness may be susceptible to fatigue cracking and should be used with caution. In addition, it is suspected that too high amount of cement used may adversely affect the workability of the mixtures.

Hot mix specimens also clearly exhibited higher stiffness modulus and longer fatigue life than foamed bitumen mixes when compared at the same aggregate gradation and binder content. The high stiffness and fatigue life of the hot mix can be attributed to the good homogeneous coating of bitumen on all aggregate particles that provide inter-particle movement resistance and flexibility to the mix. In contrast, foamed bitumen binder is not as continuous as in hot mix asphalt. In foamed bitumen mixes, the bitumen binder coats only fine aggregates and filler components. The combination of coated and uncoated particles leads to some aggregate particles contacting others without binder. The absence of binder at contacting points between aggregate particles will result an inter-particle movement within the aggregate and hence a low stiffness modulus [Thom and Airey, 2006]. The mechanism of crack propagation under fatigue loading through the two materials also appears to be different. The hot mix material displays a relatively quick propagation of the crack between initiation and failure while the foamed mix material shows slower crack growth after the crack has been initiated. The non-continuous nature of binder in foamed bitumen mixes is expected to retard the progress of propagation of cracks beyond the initiation point.

When cementitious additive is introduced to the foamed bitumen stabilised mix, the foamed bitumen plus additive tend to form a mastic with the fine fractions which continuously binds larger aggregates and this continuity of binder leads to a quicker propagation after the crack initiates.

It was observed that the more additive content in the mix, the wider was the distribution of data from fatigue tests. This phenomenon was believed to be due to the brittleness of the mixture with high cementitious additive content, which had the effect of causing the specimens to split very quickly once crack initiation occurred. Slight differences between specimens may have led to very different numbers of cycles to crack initiation.

The slope of the fatigue line shows that specimens' performance is more sensitive to stress level. It can also be seen that, above about 180 microstrain, the specimens with less cementitious additives survived a greater number of load cycles whereas below 180 microstrain, the reverse was true. It is likely that this is due to embrittlement caused by the inclusion of the cementitious binder. This imposes a threshold value on the level of strain which can be tolerated by the mixture. Below the threshold, a mixture would survive a very great number of load applications but above it would crack and fail very quickly. This is a typical behaviour of any cement treated mixture.

Theoretical analysis has been conducted and indicates that strains in the order of 30 to 200 microstrain are experienced under a standard axle load at the bottom of the main structural element of a pavement structure [Needham, 1996]. This is dependent on variables such as mixture and subgrade stiffness, load and layer thickness. Adding cementitious additives into foamed bitumen stabilised materials increases the stiffness of the mixtures and therefore helps reduce the tensile strain. In this case, it can be argued that the addition of cementitious additives extends the fatigue life of the foamed bitumen stabilised materials.

Many researchers [e.g., Mallick et al, 2002, Needham, 1996] have reported the effect of Portland cement on the resistance to water damage of cold mixes. Their results seem to conclude that Portland cement and hydrated lime are effective adhesion agents for emulsion mixtures. However, the results from this study indicate that only mixtures with Portland cement have an appropriate water resistance and hydrated lime does not seem to improve the water resistance of the mixtures. It is expected once again that because foamed bitumen does not coat aggregate fully hence hydrated lime has no contribution in adhesion of the binder to the large aggregate particles.

7.6 Conclusions

From the mechanical testing performed on foamed bitumen mixtures containing cement and hydrated lime, the following conclusions can be drawn.

1. Adding cement to the foamed bitumen mixes significantly enhances the mechanical properties, namely, stiffness modulus, resistance to permanent deformation, and resistance to water damage. The higher the addition level the greater the improvement obtained.

2. Adding hydrated lime to foamed bitumen slightly increases the stiffness modulus, resistance to permanent deformation and resistance to water damage of the mixtures.

3. Cement increases the rate of increase in stiffness modulus of foamed bitumen mixes. Hydrated lime rapidly brings the stiffness of mixtures to their peak in less than a week.

4. In terms of susceptibility to fatigue induced cracking under repeated indirect tensile load, both cement and hydrated lime increase resistance to fatigue cracking at initial tensile strains below 180 microstrain. The higher cement content cause better fatigue resistance. However, at initial tensile strains above 180 microstrain, cement and hydrated lime additives reduce resistance to fatigue cracking significantly.

5. The inclusion of cement and hydrated lime in foamed bitumen mixtures also alters the crack propagation mechanism of foamed bitumen mixes when subjected to repeated loading. The addition of cement and hydrated lime had the effect of causing the materials to split more quickly once crack initiation had occurred. That is to say that foamed bitumen stabilised mixtures with cementitious additive exhibit brittle behaviour. The greater the addition level, the greater was the brittleness observed.

8 IMPLICATIONS OF THE INVESTIGATION RESULTS

8.1 Introduction

Chapters 4 – 7 have presented an investigation into the behaviour of recycled pavement materials subjected to dynamic loading. The investigation, however, will only be beneficial if the findings obtained on the behaviour of the materials can be used. The stiffness modulus values may be employed in elastic analysis of pavement systems using existing techniques and computer programs. The permanent deformation behaviour of foamed bitumen stabilised materials can be of benefit to the prediction of rutting in recycled pavements. Similarly, the fatigue performance of cement bound recycled asphalt materials can be useful to design against damage of the material due to traffic induced cracking. This chapter presents a consideration of the usefulness of the recycled asphalt pavement materials investigated in this research for use as road base course. It also attempts to show that the investigation into the mechanical properties of the mixtures discussed in the previous chapters may be successfully applied to the problems of pavement performance prediction and pavement design.

8.2 Application of recycled pavement materials as road base course

8.2.1 Role of base course in a road pavement

A road pavement is a multi-layered system, which is formed of a number of layers of compacted unbound aggregate and/or bound materials. In general, a pavement primarily consists of three main components, from top to bottom, namely – surface course, base course and foundation. Base course is the main structural layer of a pavement. It is the layer which provides the major part of the strength and load-distributing properties of the pavement so that underlying weaker foundation materials are not over-stressed. Base course is normally required to spread the traffic load, withstand rutting and cracking due to repeated loading, and maintain adequate properties. Thus, the important mechanical properties related to the materials used for the base layer are stiffness modulus, resistance to permanent deformation, resistance to fatigue damage, and durability.

8.2.2 Mechanical properties of recycled asphalt pavement as road base course

Stiffness modulus

Foamed bitumen stabilised recycled pavement material tends to be weak during its early life. Stiffness modulus of the mixture was found to increase slowly but the ultimate stiffness is still low compared to conventional hot mix asphalt. However, the stiffness moduli of the mixtures during early life are comparable or higher than those of conventional unbound granular base materials.

The addition of as little as 1 % of cement has a marked effect on curing rate and final strength which brings the stiffness of foamed bitumen stabilised recycled mixtures close to that of hot mix. Higher levels of cement enhance the stiffness modulus even more. In pavement applications, the load spreading ability of a layer is dependent upon its stiffness modulus. A low value of stiffness modulus results in poor load spreading ability which leads to a high vertical strain in the material itself and in the foundation as well as a high tensile strain at the bottom of the base layer. From the results obtained in this investigation, the addition of a small amount of cement is recommended in cold in-place recycling work with foamed bitumen as cement serves to significantly raise the stiffness modulus and curing rate of the mixtures.

The stiffness moduli of cement bound recycled materials at the relatively short curing times tested in this investigation were found to be similar to or better than those of typical hot mixed asphalt base mixtures. They are, however, significantly lower than a typical cement bound base due to the inclusion of bitumen in the RAP. Moreover, the stiffness modulus of cement stabilised base course incorporating RAP was found to be susceptible to temperature. The stiffness modulus of the mixtures decreases with increasing temperature. Nonetheless, the degree of reduction in stiffness modulus with increasing temperature is not as high as that of conventional hot mixed asphalt. In pavement applications where the temperature underneath the surface course can be as high as 40 – 50 °C, it is expected that the stiffness modulus of these mixtures can reduce to 30 – 40 % of the modulus tested at 20 °C. It is, therefore, recommended that performance testing of these mixtures should be conducted at the expected temperature in the field.

Resistance to permanent deformation

Repeated load triaxial test results indicated that foamed bitumen bound recycled pavement materials exhibit typical characteristics of permanent deformation of unbound granular materials under cyclic loading. In the design of foamed bitumen stabilised mixtures, the optimum foamed bitumen and water content were determined so that high stiffness modulus of the mixes will be achieved with no regard to other properties. Excessive permanent deformation is the normal failure mode of granular type materials and therefore it is recommended that a permanent deformation check should be incorporated to assess the performance of foamed bitumen stabilised base course.

The investigation revealed that foamed bitumen stabilised base materials which contain a higher proportion of RAP and a softer binder exhibit greater permanent deformation. Rutting can be minimised by stabilising with foamed bitumen generated from harder grade bitumen. Data from the experiments also showed that a mixture comprising of up to 50 % RAP produced similar performance in terms of permanent deformation compared to foamed bitumen stabilised limestone.

The pilot-scale tests on foamed bitumen stabilised recycled pavement with and without cement revealed that the additive significantly enhanced the performance of the mixture in terms of rutting resistance. Not only does the addition of cement increase the resistance to permanent deformation but at the same time it also reduces the vertical strain in the underlying foundation due to the increasing stiffness modulus of the stabilised layer. Thus the addition of cementitious additives has a great benefit to this type of mixture.

On the other hand, the investigation has indicated that cement stabilised recycled asphalt pavement exhibits very good permanent deformation resistance even during its early life. It is believed that in an actual pavement application a well designed and constructed cement bound recycled layer would not fail in rutting.

Resistance to fatigue cracking

Indirect tensile fatigue testing has clearly shown that foamed bitumen stabilised mixtures have a lower fatigue life compared with hot mix asphalt when they are subjected to the same strain level. However, in the case of lightly bound foamed mixes, the materials behave like unbound materials and the dominant mode of failure is excessive permanent deformation rather than fatigue cracking.

The use of cementitious additives in foamed bitumen mixes transforms the mixtures closer to bound materials. Therefore, fatigue cracking will be significant form of distress in these materials. Test results also showed that the additives had an effect on the fatigue cracking resistance of the mixtures. It was found that with an initial tensile strain below approximately 180 microstrain the addition of cementitious additives increases resistance to fatigue cracking, but above 180 microstrain additives have the opposite effect. In former case, the addition of cementitious additives obviously extends the fatigue life of the stabilised base course mixtures. The results also showed that the resistance to fatigue damage increases with an increase in cement content of up to 2%.

The fatigue characteristics of cement bound recycled asphalt base showed typical cementitious type material behaviour. That is the fatigue of cement stabilised recycled asphalt base tends to be stress controlled. In this investigation, the results suggested that if the applied stress is low the material can withstand a significant number of applications.

Durability

The two primary factors that affect the durability of bituminous bound paving mixtures are embrittlement of bitumen due to age hardening and damage due to water [Scholz, 1995]. This investigation has considered only water damage. Water can degrade the strength and stiffness of the mixtures and, hence, its effectiveness to accommodate traffic-induced stress and strain. Tests showed that foamed bitumen stabilised mixtures with added cement have an acceptable resistance to water damage (retained stiffness modulus more than 70 %). On the other hand, foamed bitumen mixes with no additive demonstrated more stiffness modulus reduction after water soaking. Once again, the inclusion of Portland cement seems to be necessary in this mixture.

8.3 Application of the investigation results to pavement analysis and design

8.3.1 Mechanistic approach for pavement analysis and design

Basic principles

Generally, there exist two main approaches to analyse and design road pavement structures, namely, the empirical approach and the mechanistic (or analytical) approach. The mechanistic method is based on the mechanics of materials and relates an input, such as wheel load, to an output or pavement response, such as stress or strain. The response values are used to predict distresses based on laboratory test data [Huang, 1993]. The advantages of a mechanistic approach are the improvement in the reliability of a design, the ability to predict the distresses, and the feasibility to extrapolate from limited field or laboratory data [Huang, 1993].

Pavement model

Flexible pavements can be analysed using layered theory. In principle, a road pavement is modeled as a layered system of elastic materials. Each layer is homogeneous, isotropic, and linearly elastic with an elastic modulus E and Poisson's ratio ν. The layer interfaces are fully bonded. The material is weightless and infinite in horizontal extent and the lowest layer is infinite in thickness. External wheel load is considered as a uniform pressure applied on the surface of the top layer over a circular area. The responses of the pavement, such as stresses and strains, in the critical components, due to external load can be determined using elastic layered theory. The responses are then used with failure criteria to predict whether failures will occur.

Failure criteria

In the mechanistic approach, it is commonly agreed that fatigue cracking and rutting are the two main types of distress to be considered for flexible pavements. The failure criteria relate the allowable number of load repetitions to the critical stress or strain based on laboratory tests. The failure criteria are established for specific types of distress. It has been suggested that the vertical compressive stress or strain of pavement materials may be used as a failure criterion to design against permanent deformation, while the use of horizontal tensile stress or strain at the bottom of bound layers has been recommended to design against fatigue cracking.

8.3.2 Pavement design based on investigation results

Permanent deformation of foamed bitumen stabilised base course

Permanent deformation commonly occurs in flexible pavements. Two design methods have been used to control permanent deformation of pavements: one to limit the vertical compressive strain on top of the pavement foundation and the other to limit the total permanent deformation to an allowable amount. The latter computes permanent deformation of each component layer and should be more appropriate for deformation that is due primarily to the decrease in thickness of the component layers above the foundation.

Barksdale (1972) proposed a simple method for estimating the permanent deformation in a flexible pavement after a desired number of load repetitions. The method is based on direct interpolation from laboratory behaviour. Barksdale (1972) suggested that each layer should be divided into a number of sublayers. The major principal stress and the average confining pressure could be calculated at the centre of each sublayer beneath the wheel load. The permanent strain at the centre of each sublayer could be calculated by direct interpolation from the permanent stress-strain curves from laboratory test results. The total permanent deformation could then be obtained by summing the deformation of each sublayer assuming that the central strain is representative of the whole sublayer. This step can be mathematically expressed as;

$$\delta_{total}^{P} = \sum_{i=1}^{n} (\varepsilon_i^P \times h_i) \text{ [Barksdale, 1972]} \qquad (8.1)$$

where,

$\delta^P{}_{total}$	=	total permanent deformation beneath the wheel load
$\varepsilon^P{}_i$	=	permanent strain in the i^{th} sublayer
h_i	=	thickness of the i^{th} sublayer
n	=	total number of sublayers

This method can be applied to both non-stabilised and stabilised layers.

Fatigue cracking of foamed bitumen stabilised base course

It is well known that the allowable number of repetitions for actual pavements is much greater than that obtained from laboratory tests due to the fact that wheels on actual pavements do not always apply load at the same location and have longer times between loading cycles (rest periods), both of which increase the fatigue life. Also, it takes more repetitions for cracks to propagate through the entire thickness of an actual thick base layer before the pavement is considered to have failed. Therefore, the failure criterion for actual pavements must incorporate a correction factor (known as a shift factor) to account for the difference. Brunton (1983) proposed 440 for hot mix. This figure was derived from a combination of factors of 1.1 for lateral wheel load distribution, 20 for rest periods, and 20 for crack propagation (1.1 × 20 × 20 = 440). In the absence of any more appropriate figures this shift factor will be employed in the calculation although it is not yet known whether the figure is applicable to foamed bitumen mixes. The failure criterion for fatigue cracking can be expressed as

$$N_f = f \times c \times \left(\frac{1}{\varepsilon_t} \right)^m \tag{8.2}$$

where,

N_f	=	allowable number of load repetitions to prevent fatigue cracking
ε_t	=	tensile strain at the bottom of the layer
f	=	shift factor
c,m	=	constants determined from laboratory fatigue tests

<u>Fatigue cracking of cement stabilised recycled pavement base course</u>

Fatigue cracking has long been considered the only major criterion for concrete type pavement material. The fatigue of cement stabilised base course is similar to the fatigue of concrete in that the allowable number of load repetitions is related to the stress ratio, which is the ratio between the flexural stress and the flexural strength of the material. For concrete, it is often assumed that an induced flexural stress can be repeated indefinitely without causing rupture provided that the intensity of extreme fibre stress does not exceed approximately 50% of the flexural strength. This assumption has been used as the basis for concrete pavement design. Similarly, the fatigue experiment with cement bound recycled asphalt pavement in this investigation showed that at a stress ratio of approximately 0.5 – 0.6 the materials could endure a very large number of repeated flexural stress applications. Thus, for cement stabilised recycled pavement base course, the stress ratio of 0.5 may be used as a threshold value below which the material should not fail by fatigue damage although the value was not actually proved due to the limited number of tested specimens.

8.3.3 Calculation examples

Example pavement model

Figure 8.1 shows a modeled cross section of a road pavement. The base layer of the pavement is 225 mm thick foamed bitumen (PG50/70) stabilised limestone similar to the one in this investigation which for this example is taken to be at 20 °C. The pavement foundation has an elastic modulus of 100 MPa. A 50 mm thick asphalt surface is applied on the stabilised base. The pavement is expected to carry 5×10^6 equivalent standard axles throughout its design life. The standard axle load (80 kN) is considered as dual tyres, each tyre carrying a 20 kN load with a tyre pressure of 650 kPa, and a distance between the two tyres of 350 mm, as shown in Figure 8.1.

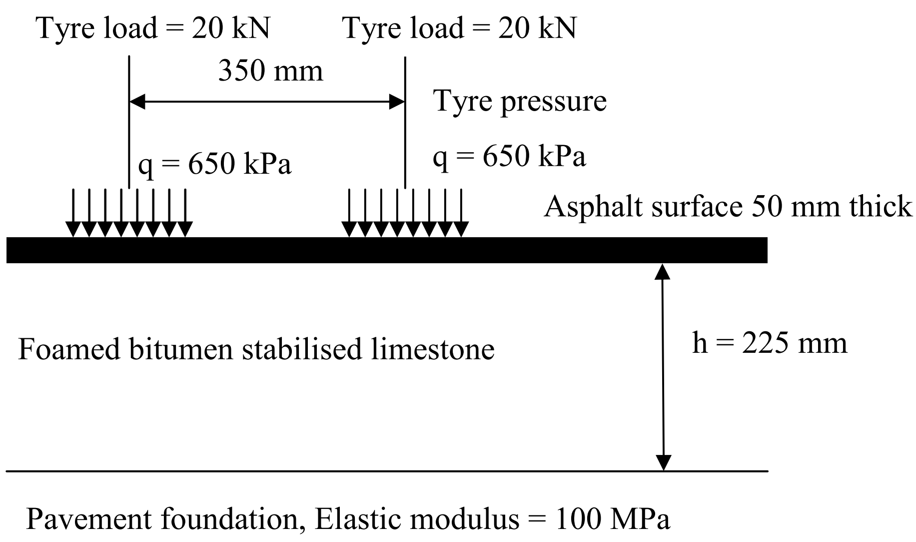


Figure 8.1 Example model for pavement analysis

The elastic analysis of the pavement system necessary to give the vertical and horizontal stresses and strains was calculated using the BISAR computer program [Shell, 1998].

Permanent deformation of foamed bitumen stabilised base course

The permanent deformation check at the end of a pavement's service life can be performed as follows;

1. The base layer is divided into three sublayers of equal thickness as shown in Figure 8.2.

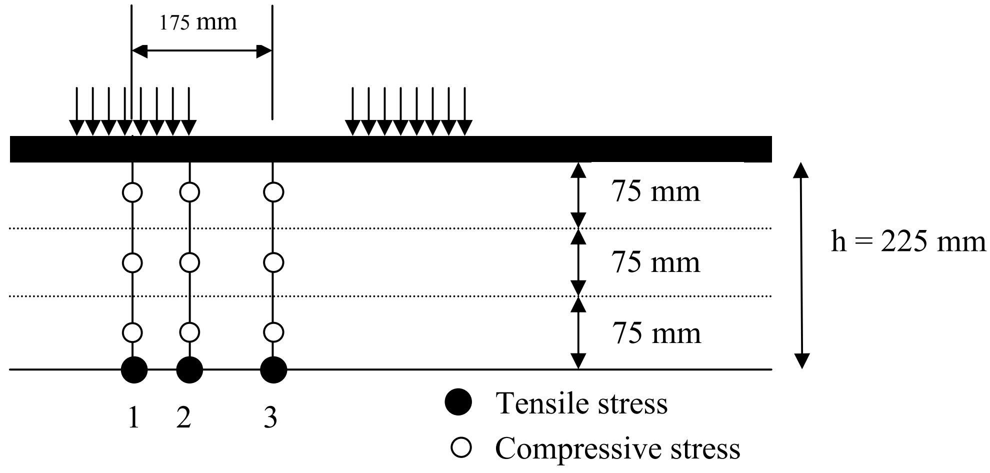


Figure 8.2 Locations of stress and strain points for pavement analysis

2. The non-linearity of the stabilised base is not accounted for in the calculation. The stiffness modulus value from the ITSM test of fully cured specimens (Figure 7.5) is taken as representative of the modulus of the entire base layer. That is the designed stiffness modulus of the base layer for this example is 1500 MPa. The asphalt surface is assumed to have an elastic modulus of 2500 MPa. All materials are assumed to be isotropic with a Poisson's ratio of 0.35.
3. The vertical compressive stresses (σ_z) at the centre of each sublayer, in lines 1, 2, and 3, as shown in Figure 8.2, are computed using the BISAR computer program and the largest stress in each layer is selected as the most critical. The results are summarised as follows;

Sublayer	Maximum σ_z (kPa)
1	415
2	182
3	62

4. The accumulated permanent strains (ε_p) are calculated for the number of repeated load cycles (N) equal to 5×10^6 at vertical stresses of 100, 200 and 400 kPa using Equation (4.3) and the data from Table 4.4. Although the relationships between permanent strain and number of load applications for foamed bitumen stabilised limestone are from the test results at a confining pressure of 50 kPa, it was found from the triaxial test that there was little difference in the results of

specimens tested at two different confining pressures (50 kPa and 100 kPa). Thus, the effect of differences in confining pressure is not taken into account in this calculation.

5. The permanent strain for vertical stresses 415, 182, and 62 kPa can be linearly interpolated. and the total permanent deformation beneath the wheel load is computed using Equation (8.1) as follows

Sublayer, i	h_i (mm)	σ_z (kPa)	$\varepsilon^p{}_i$ (%)	$h_i \times \varepsilon^p{}_i$ (mm)
1	75	415	0.543	0.407
2	75	182	0.448	0.336
3	75	62	0.244	0.183
			$\delta^p{}_{total} = \Sigma h_i \varepsilon^p{}_i =$	0.926

6. Therefore, the permanent vertical deformation in the foamed bitumen stabilised base after 5×10^6 equivalent standard axles is 0.926 mm.

<u>Fatigue resistance of foamed bitumen stabilised base course</u>

The fatigue life of the foamed bitumen stabilised base in Figure 8.1 can be estimated as follows;

1. Assuming the elastic modulus of stabilised base is equal to 1500 MPa, the tensile strains at the bottom of the base layer in the positions shown in Figure 8.2 are computed. The calculation shows that the maximum tensile strain at the bottom of the stabilised layer equals 158 microstrain.
2. The allowable number of load repetitions for fatigue failure (N_f) is determined by Equation (8.2) and the data from the ITFT test in Table 7.3. For foamed bitumen stabilised limestone with no additive, Table 7.3 gives parameters c and m as 1×10^{12} and 4.0285, respectively. The shift factor is taken as 440.

$$N_f = 440 \times 1 \times 10^{12} \times \left(\frac{1}{158}\right)^{4.0285} = 611{,}171 < 5 \times 10^6$$

Therefore, it is expected that the base course will fail in cracking before the end of its service life.

3. If there was 1% cement additive in the foamed bitumen stabilised base course, the elastic modulus of the material would be approximately 3500 MPa (see Figure 7.5) and the critical tensile strain at the bottom of the stabilised layer would be 95 microstrain.
4. From Table 7.3, for foamed bitumen stabilised limestone with 1% cement, the ITFT test gives the fatigue parameters c and m of 1×10^{19} and 7.1714, respectively. The N_f for base course with 1% cement would be:

$$N_f = 440 \times 1 \times 10^{19} \times \left(\frac{1}{95}\right)^{7.1714} = 28,867,263 >> 5 \times 10^6$$

It should be noted that the shift factor of 440 may be less applicable to mixture containing cement as the test results clearly show that the mixtures with cementitious additives exhibited quicker crack propagation after crack initiation to failure.

<u>Fatigue resistance of cement stabilised base course</u>

If the base layer in Figure 8.1 is cement stabilised recycled asphalt pavement material with 50 % RAP and 6 % cement, the fatigue damage of the stabilised base course can be checked on the basis of stress ratio value as follows;

1. From Figure 5.11, the elastic modulus of the base course material is approximately 7000 MPa. The tensile stress at the bottom of the base layer can be calculated. The critical tensile stress is found to be 571 kPa.
2. From Figure 5.10, the flexural strength of cement stabilised recycled pavement material with 50% RAP is 1.305 MPa. Thus, the stress ratio in this case is

$$SR = \frac{571}{1305} = 0.438 < 0.5$$

Therefore, it is likely that the cement stabilised recycled base course will not fail by fatigue cracking.

8.4 Discussion

This investigation has shown evidence that cold recycled asphalt pavement materials are suitable materials to be used in road pavement construction. The investigation provides results that can be used in mechanistic analysis of pavements. With this information, cold recycled pavement material is, therefore, a potentially viable material for use in all types of pavement. It was also found that the use of Portland cement as an additive offers a great improvement to the performance of foamed bitumen stabilised recycled pavement materials.

In order to demonstrate the applicability of the investigation results to the structural analysis of flexible pavements, some design examples of pavements incorporating stabilised recycled base have been given in this chapter. The methods shown in this chapter for the prediction of pavement distress may be considered as tentative due to limited results. The behaviour of cold recycled pavement materials depends on many factors. Thus, the acquisition of experimental data should be continued so that the necessary information is found for all materials and conditions likely to be encountered in the field. For example, experimental data with variation of temperature, the responses of materials with variation of load levels, and the properties of mixtures at various RAP proportions, moisture contents, gradations, etc should all be sought.

In addition, the calculations shown in this chapter are based on the properties of mixtures at a fully cured stage. As shown previously in this research, cold mixtures stabilised with foamed bitumen and cement develop their strength over a period of time which can be several weeks. During the curing period, the mixtures have lower stiffness and, consequently, the tensile and compressive stress or strain would be higher than those predicted using stiffness modulus values of fully cured mixtures. The mixtures will gain stiffness over time and eventually reach or may exceed the designed values. Therefore, cold recycled mixtures may not be suitable to be opened for high traffic volumes soon after laying. However, in the case of low to medium trafficked roads, the low traffic levels should not impose excessive stresses onto the pavement or alternatively it may be relatively easy to keep traffic off for a few weeks during the curing period after construction.

8.5 Conclusions

This chapter has described the beneficial characteristics of recycled asphalt pavement materials utilised as a road base course and also attempted to show that the techniques used in the investigation of the mechanical properties of the materials can be practically applied to problems of pavement analysis and design using a mechanistic approach.

From the results found in this investigation, it can be concluded that cold in-place recycled pavement base course, either stabilised with foamed bitumen or cement, is a potential alternative material for use in road pavements. However, for foamed bitumen stabilised mixtures, there may be some problems concerning the low strength and rut potential during the curing period and, therefore, the addition of cement in these types of mixtures is recommended.

9 SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

9.1 Summary

This last chapter summarises the main issues in this thesis and draws conclusions based on the findings from the present investigation. Some recommendations for future research are also proposed at the end of this chapter.

Cold in-place recycling of asphalt pavements

Cold In-place Recycling is a pavement rehabilitation method in which all of the old asphalt pavement depth and a predetermined amount of the underlying materials are treated to produce a stabilised base course. After that only a thin overlay or chip seal surfacing is required. Commonly, the thickness of recycled layer can be up to 300 mm with various choices of stabilising agent including foamed bitumen, bitumen emulsion or cement alone or a combination of cement and bituminous primary binder. Pavements with moderately deformed profile and with moderate to severe cracked surface are generally suitable for applying this technique. This technique can improve strength and riding quality of a deteriorated road at relatively low cost. Pavement recycling is popular because it is environmentally friendly and is normally cost effective compared to other conventional rehabilitation techniques.

Investigation into cold in-place recycled pavement materials

Many studies on engineering properties of cold recycled materials in the literature were performed using methods which may not relate to the performance of the materials under traffic load such as indirect tensile strength on foamed bitumen mixes and unconfined compressive strength of cement stabilised mixes. The present study reports a series of investigations into more fundamental mechanical properties of the cold recycled pavement mixtures which are suitable for applying in pavement engineering applications.

Material preparation

Cold recycled asphalt pavement materials were fabricated in the laboratory by mixing reclaimed asphalt pavement with new crushed limestone aggregate (representing unbound base material) and stabilised with stabilising agents. Two types of stabilising agents were studied in this research, foamed bitumen and cement. Foamed bitumen was created from penetration grade bitumen using a laboratory bitumen foaming plant. Mix design tests were carried out to optimise the properties and/or content of binder for each mixture type. The mixtures were subsequently mixed and compacted at optimum binder contents. A gyratory compactor was used to compact cylinder specimens for testing in the Nottingham Asphalt Tester and for repeated load triaxial testing. A slab compactor was used to prepare slabs of cement bound mixtures which were later cut into beam specimens for the beam flexural tests.

Foamed bitumen stabilised recycled asphalt pavement materials

The use of foamed bitumen as a stabilising agent in the cold in-place recycling process has gained popularity over the last decade due to the advance in the technology of cold recycling machines, as well as some perceived advantages of foamed bitumen over other alternative stabilising agents. In this research, some technical aspects of foamed bitumen stabilised recycled pavement materials were investigated.

An experiment was conducted to examine the manner in which foamed bitumen stabilised materials deformed when subjected to forms of loading similar to those imposed by traffic. Foamed bitumen bound recycled asphalt pavement materials were manufactured in the laboratory by mixing RAP with new crushed limestone and treated with foamed bitumen. The deformation behaviour of the materials was evaluated by using a repeated load triaxial apparatus which provided an approximate simulation of the traffic loadings experienced in actual pavements.

Cement stabilised recycled asphalt pavement materials

Portland cement was used as a stabilising agent for road bases long before foamed bitumen became popular. Cement is adaptable to most conditions and is readily available at reasonable cost. It is generally suitable for mixing with a wide range of pulverised aggregates comprising existing pavement materials. If there are any doubts that foamed bitumen may not perform satisfactorily, cement is likely to offer a safer option where unfavourable conditions exist. Portland cement as a binder for in-place pavement recycling produces a flexible composite pavement structure.

In this study, the repeated flexure test was chosen to simulate the traffic effect on the cement stabilised recycled asphalt pavement materials. Beam specimens were prepared by cutting compacted slabs. Stiffness modulus and fatigue characteristics of the materials under repeated flexure were evaluated.

<u>Cementitious additives in foamed bitumen stabilised mixtures</u>

Cementitious stabilising agents such as cement and lime can be used in combination with bituminous stabilising agents such as foamed bitumen and bitumen emulsion. Many researchers have reported the beneficial effects of cement and lime on bitumen emulsion mixes. In the present study, the effect of cement and hydrated lime additives on foamed bitumen mixtures was investigated.

Cylinder specimens of foamed bitumen stabilised limestone with a range of cement and hydrated lime additives were produced. The performance of the cured specimens, in three modes namely – stiffness modulus, resistance to fatigue cracking, and resistance to permanent deformation, was evaluated by the Nottingham Asphalt Tester.

<u>Pilot-scale recycled pavement test</u>

The Nottingham Pavement Test Facility was used to traffic pilot-scale simulated recycled pavements. Two types of bitumen were used to produce foamed bitumen which was used to stabilise two mixtures of different proportions of reclaimed asphalt and limestone aggregate. Cement was also used as a primary binder as well as an additive in this trial. The combination of binders and aggregate proportions allowed various simulated pavement recycled mixtures to be produced and constructed. The performance of the mixtures under realistic traffic load was assessed.

<u>Implementation of the investigation results</u>

The benefits of cold in-place recycled asphalt pavement materials as road pavement materials were described. The calculation examples bringing the investigation results to solve the problems of pavement analysis and design were also presented.

9.2 Conclusions

The conclusions that can be drawn from this investigation are as follows.

Foamed bitumen stabilised recycled asphalt pavement materials

1. Foamed bitumen bound recycled pavement materials, when subjected to traffic load, tend to fail in rutting rather than fatigue cracking. The material possesses the typical permanent deformation characteristics of granular type material under repeated loading.

2. The resistance to rutting of foamed bitumen bound materials is affected by the amount of RAP in the mixtures and also the type of bitumen used to generate the foam. Generally, foamed bitumen mixtures that contain a higher proportion of RAP and/or a softer bitumen exhibit greater deformations.

3. Foamed bitumen bound composites are visually very different from fully coated hot mixtures. In foamed bitumen mixes, the fine aggregate and filler components are coated by bitumen. RAP and coarse aggregate particles in the foamed bitumen mixes, though bound together, are not fully coated by bitumen. Mix deformation is therefore, likely to depend on particle interlock as well as stiffness of binder.

4. Foamed bitumen stabilised mixtures are stress dependent. Resilient modulus of foamed bitumen mixes increases with an increase in confining pressure and cyclic stress. However, it was found that the material loses its stress dependency when the foamed bitumen content is high and/or is well distributed in the mixtures. An increase in confining pressure also causes an increase of resistance to permanent deformation of the foamed bitumen mixes.

5. The type of bitumen generating the foam does not significantly affect the resilient modulus of the mixtures.

Cement stabilised recycled asphalt pavement materials

6. The strength and stiffness of cement stabilised materials depend on the strength and stiffness of their aggregates. Therefore, the inclusion of RAP in cement stabilised materials greatly reduces strength and stiffness modulus of the materials.

7. The flexural behaviour of cement stabilised recycled pavement materials is typical of cement stabilised materials. The fatigue characteristics of the mixtures tend to be stress controlled. The flexural stiffness modulus of the materials appears to be independent to the frequency of loading.

8. Stiffness modulus of cement stabilised materials incorporating RAP is dependent on temperature. Because stiffness of RAP decreases when temperature increases and stiffness modulus of cement bound materials seems to depend on the stiffness of the aggregate, the stiffness modulus of cement bound materials incorporating RAP decreases with increasing temperature.

Cementitious additives in foamed bitumen stabilised mixtures

9. Foamed bitumen stabilised mixtures develop their strength and stiffness as the moisture in the mixes is being lost. The mixtures need curing time which takes several weeks to reach their ultimate potential stiffness. When cementitious additive is presented, the additive reacts with moisture in the mixes converting it into hydraulic product which contributes to an increase in strength and stiffness of the mixtures.

10. The addition of a small percentage of cement in foamed bitumen bound mixtures significantly enhances the performance of the mixes, namely, rate of stiffness gain, ultimate stiffness modulus, resistance against rutting failure and resistance to water damage.

11. The addition of a small amount of hydrated lime to foamed bitumen stabilised mixes slightly increases final stiffness modulus, resistance to permanent deformation and resistance to water damage of the foamed bitumen mixes.

However, hydrated lime significantly increases the rate of stiffness gain with time.

12. The inclusion of cement and hydrated lime in foamed bitumen stabilised mixtures increase resistance to fatigue cracking of the mixes at low tensile strain. In contrast, if the materials are subjected to tensile strain higher than about 180 microstrain, the addition of cement and hydrated lime causes a detrimental effect to the mixtures.

13. Foamed bitumen mixes show slower crack propagation after the crack initiates when compared to equivalent hot mix asphalt. On the other hand, foamed bitumen mixtures with cement or hydrated lime exhibit a quicker crack propagation rate once the crack has been initiated. The mixtures with cementitious additives also exhibit brittle behaviour which becomes more evident with increasing quantity of additive.

<u>Performance of pilot-scale recycled asphalt pavement</u>

14. Results from pilot-scale pavement tests are generally consistent with the findings from element scale laboratory tests. Foamed bitumen stabilised recycled pavement materials fail in rutting when subjected to accelerated traffic load and the resistance to deformation of the materials is dependent on the mixture proportions and penetration grade of bitumen generating the foam.

15. It was found from the pilot-scale test that the effects of proportion of RAP and type of bitumen binder on the deformation of the mixtures were less significant as the magnitude of applied traffic load increased.

16. The analysis of the results measured from strain gauges reveals that the modulus of foamed bitumen bound recycled materials during early life is comparable or can be better than that of traditional unbound base materials and, with cement added, can be as high as that of conventional asphalt base course. The stiffness modulus of cement bound recycled asphalt pavement materials was also found to be comparable to that of traditional asphalt base material.

Implementation of the investigation results

17. Cold recycled asphalt pavement materials are potential alternative materials for use in road pavement. The investigation results in mechanical properties of these types of mixtures can be successfully applied to the problems of pavement analysis and design.

9.3 Recommendations

An investigation into some aspects of cold in-place recycled asphalt pavement materials has been conducted. However, cold in-place recycled mixtures are very complicated because of the diversity of materials that need to be accounted for. To completely understand the behaviour of cold recycled pavement mixtures and be able to predict their performance accurately is challenging and there are still plenty of areas to be further investigated. Some of them are:

- A study of the factors affecting the properties of cold recycled pavement materials. There are many parameters that influence the performance of cold recycled mixes including, aggregate origins, aggregate properties, type of binders, binder contents, moisture content, temperature, etc.
- A study to characterise the behaviour of cold recycled pavement materials. More investigation into the mixture behaviour under repeated loading is required in order to develop a performance model for the materials. This task may be achieved by comprehensive triaxial tests.
- A study into durability of cold recycled pavement mixtures. Recycled materials may not be as durable as new and controllable materials. The mix of deteriorated reclaimed asphalt and mineral aggregate, and the weak bond between of the aggregate-binder structure of foamed bitumen bound mixtures make the durability of the materials questionable.
- A large-scale study of cold recycled pavement. On the basis of satisfactory laboratory test results, before using recycled pavement, a field trial is recommended to verify the performance of the pavement in a realistic condition. Field trials also benefit the understanding of construction procedure and development of specification for cold in-place recycled pavements.

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