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TITOLO TESI

PERFORMANCES AND DURABILITY OF ASPHALT MIXTURES MADE WITH RECLAIMED ASPHALT PAVEMENT

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Esame finale anno 2016

To daddy, with love.

Non est ad astra mollis e terris via

(There is no easy way from the earth to the stars)

Seneca

It's a long way to the top (if you wanna Rock 'N' Roll)

AC/DC

The experimental programme of the present research has been conducted at the Elletipi Srl Testing Material laboratory, Ferrara, Italy. Aggregates and RAP have been provided by S.A.P.A.B.A. SpA asphalt production plant in Pontecchio Marconi, Bologna, Italy. Neat bitumen has been furnished by Valli Zabban SpA, Bologna, Italy.

Table of Contents

List of figures

List of tables

Keywords

Hot mix asphalt

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Recycling

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Durability of asphalt mixtures

Bitumen rheology

Blending between RAP and virgin bitumen

Abstract

According to most recent surveys, 265 mil tonnes of asphalt for road applications have been produced in the European area in 2014. In the same year, the amount of available RAP was more than 50 mil tonnes. The use of RAP in new blended mixes reduces the need of neat bitumen, making RAP recycling economically attractive. In spite of its economic and environmental benefits, the use of RAP is often subjected to restrictive policies pursued by agencies due to uncertainty related to RAP mixes field performances.

This thesis focuses on the effects of RAP on performances and durability of new asphalt mixes produced incorporating RAP with a standard hot-in-plant process. The influence of RAP being incorporated in new mixes has been investigated through an extensive laboratory testing campaign on asphalt mixtures and on bitumen recovered from the same mixes.

A specific typology of asphalt mixture has been produced with different RAP percentages of 10%, 20% and 30% by mass of aggregates. In the first phase of the research, asphalt materials have been characterized and compared in terms of resistance to fatigue, stiffness and volumetric properties. The second phase of the research aims to investigate the effects of RAP on asphalt binder as a composite blend of neat and RAP bitumen. Bituminous samples have been recovered from asphalt samples tested in phase 1 and then have been studied and compared with standard and dynamic laboratory tests.

Findings show that hot asphalt mixtures can be successfully produced incorporating up to 20% of RAP (by mass of the mix) in the fresh mix. For these percentages of RAP being recycled, no negative effects on performances and durability of recycled mixtures have been noted. However, the final bitumen grade of the mix may be adjusted if more than 20% of RAP is added. Furthermore, practical implications regarding production methods and paving of RAP mixes are proposed.

1 Introduction

1.1 Reclaimed Asphalt Pavement, RAP

During their service life, roads experience the action of a huge variety of external factors as traffic, weather condition, thermal shocks, chemical agents, which can all be considered “external loads”. Accumulation of small and routinely repeated external loads together with exceptional events that may occur lead distresses to raise and spread into the pavement over the years, with increased severity.

A well-planned road maintenance avoids excessive spread of distresses on pavements and contributes keeping the road system in good conditions. Depending on severity of distresses and hence on the required maintenance, one or more asphalt layers may be removed and then replaced with new layers. Old asphalt layers can be dismantled with an excavator or, better, removed with a milling machine. The first produces asphalt blocks, the second produces a granular material made of aggregates and old bitumen. Both are called “Reclaimed Asphalt Pavement” (RAP).

RAP is a source of raw materials, aggregates and old bitumen, 100% reusable or recyclable [1]. RAP recycling has both **economic and environmental advantages**: it reduces the need for virgin and non-renewable raw materials i.e. virgin aggregate and bitumen; RAP can be used as raw material to produce high quality asphalt mixtures; recycling reduces the amount of RAP being dumped to landfill and gives high value added to RAP; it also reduces transportation costs and related emissions; in cold and warm recycling, a lower energy input is required. On the other hand, the use of RAP may increase production costs on a short-term periods, for example: retrofitting of old asphalt plants; increased costs for RAP handling i.e. storage and processing; costs for lab tests to characterize stockpiled RAP; investments to buy special machines (cold-in-place recycling). However, all short-term costs turn into environmental and economic benefits in the mid-period. This makes RAP a resource for the road construction sector, making reusing or recycling RAP a valuable and profitable practice.

The most common RAP applications in Europe are: hot recycling in asphalt plant, hot in-place recycling, cold in-place recycling and full depth reclamation are the most commonly applied techniques [2]. Hot, warm and cold recycling are the most valuable procedures for recycling RAP because the less expensive RAP bitumen can replace a portion of the more expensive virgin bitumen [3]. Using the old asphalt bitumen in the newly blended mixtures and, therefore, reducing the required new bitumen content, makes the use of RAP in HMA mixtures economically attractive [4]. It is considered that the most economical use of RAP is in the intermediate and surface layers of flexible pavements because the less expensive RAP binder can replace a portion of the more expensive virgin binder [3]

The percentage of RAP (usually expressed by mass of the aggregate in the mix) that can be incorporated into asphalt mixtures depends on production process (plant type, production temperature, mixing time, and discharge temperature), paving technology and permitted emissions as well as RAP source and properties. These factors affect the interaction between RAP and virgin bitumens and consequently impact the performance of asphalt mixtures [5]. Considering hot in plant recycling, most conventional drum plants can accommodate 50% RAP, whereas the percentage of reusable RAP in batch plant ranges from 10 to 30% [6]. These limitations have been overcome thanks to multiple technologies readily available for production of up to 100% recycled hot mix asphalt [7].

On average, Europe recycles more than 70% of RAP. 61% of available RAP is used in hot mix recycling to produce asphalt mixtures with a certain amount of RAP. Cold and warm recycling involve 5% and 4% of available RAP. 13% of RAP is used as filling material into unbound granular layers, whereas 7% is dumped. The remaining RAP 10% is used in other civil engineering applications [8]. Italy uses only the half of its available RAP for valuable applications as hot, warm and cold recycling. 30% of RAP is dumped in landfill areas, 20% is used for filling unbound layers. Of the remaining 50%, only 20% is used in hot recycling and 30% is used in cold recycling.

Many agencies and public administrations insert **restrictions on RAP percentages** ranging from 10 to 30% in their regulations due to concerns for pavement performances and production technologies. Uncertainties concern the interaction between RAP binder and virgin binder and the effects on properties of the mixture. Inaccurate assumptions on the effects of interaction might create problems both in mix design and pavement performance [2, 5], leading to mixtures that might be subjected to premature failure to cracking, ravelling, moisture damage and rutting. At present, there is no industrial approved standard method to predict the degree of blending in laboratory [3], and tests shall be carried out for evaluating the effects of interaction between RAP binder and virgin binder on mechanical behaviour of RAP mixtures.

The success and spread of RAP recycling practices depend mainly on two factors:

1. Asphalt mixtures produced incorporating RAP, even in high quantities, shall perform at least as the same mixes made with only virgin materials
2. RAP mixes shall be profitable at least as virgin mixes

Experience suggests that if one of these two conditions fails, RAP will not be recycled or, in the worst case, it will be improperly recycled resulting in low-quality RAP mixes. In spite of all environmental benefits of RAP recycling, companies would not be able to afford higher costs without economic advantages.

1.2 Objectives and research approach

This thesis focuses on the ***effects of RAP on performances and durability of new asphalt mixes produced incorporating RAP with a standard hot-in-plant process***. The effects of RAP have been investigated through an extensive laboratory testing campaign on asphalt mixtures and on bitumen recovered from the same mixes. The laboratory testing activity of the present research aims to investigate performances and durability of asphalt mixtures produced incorporating different percentages of RAP.

The objectives of the present research are listed below:

1. Investigate the effects of RAP on **performances and durability** of asphalt mixtures. In detail, the research investigates the effect of increasing percentages of RAP (sampled from the same source) on performances of the same type of asphalt mixture
2. Study to what extent the **interaction between virgin and RAP bitumen** affects properties and response of asphalt mixes by measuring the effects both on asphalt mixes and bitumen
3. Define the **threshold value of RAP** that can be incorporated into a bituminous mix without compromising its long-term performances
4. Determine the correct **procedure** to incorporate RAP into mixtures produced in laboratory
5. Provide figures on the **use of RAP in Europe**: amount of asphalt being recycled, technologies, applications, by-products of RAP recycling
6. Give a comprehensive overview on **methods to incorporate RAP** into asphalt mixtures both in batch and drum plants

The main goal of the research is to answer to simple questions that often rise in the daily practice, as: "*how much RAP can be added into a specific mix?*" or "*how does the presence of RAP affects performances of the asphalt mixture?*"

Ensuring confidence in the design procedure and the success of using RAP would require addressing many durability concerns related to the interaction between virgin and recycled materials [9].

1.3 Materials and methods

The investigated material is a hot asphalt produced in plant and used in binder layers of flexible pavements. Four versions of the same mix have been produced in laboratory with different RAP contents.

The research is divided in two phases: in the first phase, the effects of RAP on asphalt mixtures are investigated. A specific typology of asphalt mixture is produced with different RAP percentages of 10%, 20% and 30% by mass of aggregates. Asphalt materials are characterized and compared in terms of resistance to fatigue, stiffness and volumetric properties. A great effort was put to produce mixtures with different RAP content but similar grading curve, bitumen content, which is sum of virgin and RAP bitumen, air void content and density. The process of manufacturing of asphalt mixes in laboratory is essential in order to obtain mixes with the desired properties and to produce samples that are representative of mix properties.

The second phase of the research aims to investigate the effects of RAP on asphalt binder as a composite blend of neat and RAP bitumen. Bituminous samples are recovered from asphalt samples tested in phase 1 and then are studied and compared with standard and dynamic tests.

In literature, laboratory testing are generally performed on blends made of neat and old bitumens, the latter recovered from RAP. Alternatively, RAP bitumen can be replaced with neat bitumen that has been aged in laboratory to simulate ageing that has occurred in RAP bitumen. A limitation of these two approaches is that neither of the two consider the actual bitumen and both perform tests on a bitumen that may differ from that present in the asphalt mix.

The experimental programme of the research was conducted with only standardized test procedures according to **European standards**. This for two reasons. First, test developed apart from the standards (thus with no standard procedure to follow) requires to be validated in terms of procedures, data processing and results. Repeatability and reproducibility are key issues when dealing with the high variability of RAP. Further uncertainty caused by test procedure is not required and desired neither. Second, the use of standardized tests allows to develop a sound methodology that can be used by everyone who needs to test mixtures containing reclaimed asphalt. A characterization procedure can be proposed standing on solid bases of European standard tests.

The underlying idea of the research is to investigate a common asphalt mixture produced with standard materials, i.e. natural limestone aggregates and unmodified 50/70 pen grade bitumen, and with properties that are consistent with the most common technical specifications. RAP has been incorporated into new mixes simulating the hot-in-plant recycling process. Recent surveys show that 61% of available RAP is used in hot-in-plant recycling. The percentages of 10, 20 and 30% of RAP being incorporated into the mixes reflect the most common usage of RAP into traditional batch asphalt plants in Italy i.e. with no specific retrofitting to handle high quantity of reclaimed asphalt. The choice of common materials and well-established procedures was made to give the wider validity to research findings and to make the testing procedure accessible and reproducible to other laboratories or asphalt plants.

1.4 The importance of testing both asphalt mix and bitumen

The testing activity of this research was conducted both on asphalt mixes and recovered bitumen as formerly described.

Firstly, test results grab different information of depending if they’re performed on asphalt mixes or on bitumens, highlighting different aspects and properties of the investigated mixes.

Tests on asphalt mixtures can describe performances, durability and expected behaviour of RAP mixes in field, but provide only indirect evidence of the interaction between virgin and RAP bitumen and almost no information about workability.

Tests on bitumen allow to investigate in detail the interaction between new and old bitumen and the workability of mixtures, but give only partial information about performances of mixes and no information about durability (with regard to the testing activity of this research). The expected behaviour of the investigated mixtures in field can be deducted only indirectly from results of bitumen testing.

Furthermore, results of asphalt and bitumen testing can be compared to evaluate their consistency. In other words, similar conclusions should be drawn from results of asphalt and bitumen testing. If this should not be the case, errors or inaccuracies in the testing procedures might be present.

1.5 Scope of study

The products of the research are presented below:

1. Present test results that clearly determine the effects of RAP on performances and durability of investigated mixes
2. Define a procedure for incorporating RAP into mix design of asphalt mixtures in laboratory that includes: sampling, handling, characterization, testing, interpretation of results
3. Find the maximum amount of RAP as percentage by mass of aggregate that can be incorporated into the investigated asphalt mix
4. Provide guidelines to extend the research to other hot asphalt mixtures produced incorporating RAP

1.6 Chapters

The present thesis is organized into 7 chapters each focused on a specific topic:

- The second chapter **"Use of RAP in pavement industry"** describes technical, environmental and economic reasons to consider reclaimed asphalt a valuable and profitable resource for the road sector. Firstly, the chapter provides a snapshot of the road construction market by analysing the most recent data about asphalt production, bitumen consumption, and availability of RAP in Europe. Figures show a relationship between the increased cost of natural resources i.e. neat bitumen and the use of reclaimed asphalt. Environmental and economic advantages are analysed and discussed in detail. Secondly, the chapter reviews all most common methods to incorporate RAP in hot-in-plant productions both in batch and drum plants. Factors that limit the use of RAP in batch and drum plants are presented together with the technological improvements that can increase the amount of RAP being incorporated into new mixes, e.g. parallel drum and double dryer. Examples of best practices of RAP recycling are also given in the final part of the chapter
- Chapter 3 **"Review of laboratory testing on asphalt mixtures"** and chapter 4 **"Review of laboratory testing on bitumen"** give a very detailed description of laboratory test methods thanks to the more than 6 years' experience of the Author in laboratory testing on road materials. Both chapters describe the objectives and procedures of each test accordingly to European standard references and they provide a practical guide to critically understand test results and to identify possible drawbacks in testing procedures. Laboratory testing are an essential tool to collect information about physical and mechanical properties of materials. Reliability and

soundness of test results are of primary importance in order to attain reasonable conclusions. Also, chapters 3 and 4 present examples of test parameter values for different types of road materials. For example, for different classes of pen grade bitumens, values of penetration at 25°C, softening point, viscosity at 160°C, penetration index and Fraass point are reported accordingly to European standards and most important technical specifications

- Chapter 5 **"Experimental programme"** describes the experimental part of the research of extensive laboratory testing on asphalt mixes and on recovered bitumen. The first part presents testing activity on asphalt mixtures that include: sampling and characterization of raw materials, mix design of RAP mixtures with different percentages of incorporated RAP, production of prismatic asphalt samples, fatigue testing with four-point bending device. The second part presents testing activity on bitumen samples that have been recovered from asphalt samples previously tested to fatigue. Bituminous samples were characterized with both standard tests, as penetration at 25°C and softening point, and rheological tests, which were performed using a rotational viscometer and a dynamic shear rheometer
- Results of the laboratory testing activity presented in the "Experimental programme" are reported in chapter 6 **"Results and discussion"**. The chapter reports the results of fatigue testing on asphalt mixes, with data being processed both with traditional Wöhler and energetic approaches, and results of tests on bituminous samples. Test results are used to deduct the influence of RAP on the investigated mixture
- All the most relevant considerations from previous chapter are collected in the final chapter 7 **"Conclusions"**, which is a clear and concise summary of research objectives, methods and findings

1.7 Literature review

In this section, a quick and easy-to-use literature review is reported with regards to the following RAP issues:

- Interaction between RAP and virgin bitumen
- Mix design of RAP mixes
- RAP content thresholds
- Performances and durability

The literature review collects the most important and recent findings that are commonly accepted and recognized by the scientific community about reclaimed asphalt. Also, it provides the necessary background for the research project presented in this thesis.

1.7.1 Interaction between RAP and virgin bitumen

a. A limited number of studies have investigated the blending process between aged and virgin asphalt binders [9] [10] [11]. In NCHRP 9-12, McDaniel et al. define three possible levels of interaction between aged and virgin binders were compared experimentally: black rock i.e. no blending, total blending i.e. 100% blending and actual practice i.e. blending as it usually occurs in practice

b. One major factor that is still unclear is the level of interaction between aged and virgin asphalt binders [9]. If RAP acts like a black rock, the aged and virgin binders will not interact. Hence, it would be assumed that RAP does not significantly change the virgin binder properties. However, it is usually assumed that RAP does not act as a black rock and that the aged asphalt blends with the virgin binder during mixing [12]. This means that the amount of virgin asphalt binder can be reduced by the full amount of asphalt binder in the RAP for the percentage specified [9]

c. The level of blending between RAP and neat bitumen occurring in asphalt mixes after mixing operations was investigated by Huang et al. [4]. The laboratory experiment consisted in staged extractions aimed to obtain bitumen from various layers coating the RAP aggregates. Results indicated that after blending, outside layers of asphalt binder around RAP aggregates were much softer than the inside layers of binder. In terms of percentages, about 60% of the aged binder did not blend with the virgin binder while 40% of the outside binder was a blend between aged and virgin binders. Although the authors cautioned that the mixtures used in that study do not reflect common HMA normally used in practice, it was evident that the level of contribution of the residual asphalt binder should be something substantially lower than the usually assumed 100%. Hence, it is clear that the appropriate amount of the RAP aged binder that effectively contributes to RAP mix needs to be further investigated [9]

d. An old RAP bitumen has increased viscosity and stiffness relative to the same neat bitumen. During its service life, bitumen reacts and loses some of its components due to oxidation, volatilization, polymerization, thixotropy, syneresis and separation. Because of the aging process, the rheological behaviour of bitumen will naturally differ from virgin materials. This suggests the importance of controlling the blending process between recycled and virgin bitumens. If the old binder is too stiff or too viscous, the blend of old and virgin bitumens may not perform as expected resulting in a poor quality RAP mixture

1.7.2 Mix design of mixtures incorporating RAP

Detailed guidelines for mix design of RAP mixtures are given in NCHRP reports 452 and 752 [13] [14]. A summary of most relevant conclusions is reported below.

a. The RAP content is usually expressed as percentage of mass of aggregate in the mixture

b. The weight of the binder in the RAP must be accounted for when batching aggregates. For example, when measuring the mass of a RAP fraction being incorporated into a new mix, the mass of RAP bitumen shall be considered apart from the mass of RAP aggregate: the former contributes to the bitumen content, the latter contributes to mix grading

c. The RAP aggregate is treated like another stockpile for blending and weighing, but must be heated gently to avoid changing the RAP binder properties. Guidelines in NCHRP-452 suggest to dry RAP at 110°C for not more than 2 hours

d. The total asphalt content is reduced to compensate for the binder provided by the RAP. The higher the RAP content, the larger the amount of virgin bitumen that can be saved

e. If total blending between neat and RAP bitumen is assumed, the RAP grading curve to be used in the mix design shall be determined from the extracted material

f. A change in virgin binder grade may be needed depending on the amount of RAP, desired final binder grade, and RAP binder stiffness

g. For RAP contents below 15 percent, the virgin binder grade should be the same as for a virgin mix. For intermediate RAP contents between 15 and 25 percent, the virgin binder should be one full grade lower than for a virgin mix. For RAP contents above 25 percent, blending charts or equations should be used to determine the appropriate virgin binder grade

1.7.3 RAP content thresholds

a. At small RAP percentages, generally 10-20% depending on the mix, an aged binder does not significantly affect the properties of the blend of virgin and RAP binder. When used at intermediate to higher percentages, a RAP bitumen can significantly influence the properties of the blend and may affect the resultant bitumen grade [10] [15].

b. RAP contains asphalt binder that has been aged. Because of this fact there has been a concern that incorporating higher RAP contents into HMA may lead to mixtures that are high in stiffness and accordingly may be prone to failures in the field [5]. In an attempt to mitigate this stiffness increase, state transportation agency specifications have suggested/recommended the use of a softer binder when RAP is used in high percentages (typically above 15-20%). If good blending occurs between the softer and RAP binder, the resultant binder in the mixture should have compatible properties to the typical specified asphalt binder used at low or zero percent RAP contents.

c. A study conducted by McDaniel and reported in NCHRP 9-12 concludes that there is no statistical difference between the values for 0% and 10% RAP. This would indicate that up to 10% of RAP may be added with no statistical effect on the performance of the mix

d. RAP mixes are generally divided into low, intermediate and high RAP content on the basis of RAP incorporated into the mix. Percentages are reported below:

RAP content	Percentage of RAP in the mix by mass of aggregate
Low	0-15%
Intermediate	15-25%
High	>25%

1.7.4 Performances and durability

Laboratory and field studies conducted to investigate performances of RAP mixtures agree to recognize the stiffening effect of incorporating RAP into a new asphalt mix. Nevertheless, many researches show that the effects of RAP on durability of mixes may be both positive and negative.

a. Numerous studies on RAP have indicated that addition of RAP to an asphalt mixture changes the physical behaviour of the mix. The increased stiffness of the RAP binder is believed to be the cause of increased modulus of asphalt mixture mixes [16]

b. The NCHRP 9-12 study on RAP mixes concludes that, at low RAP content, the properties of RAP mixture are not significantly different from those with no RAP [10]. However, at higher RAP contents, the indirect tensile test results and beam fatigue testing indicated increase in stiffness which would lead to cracking if no adjustments in mix design are made

c. NCHRP 9-46 study by West et al. [14] evaluated the use of 55% RAP mixes and showed that stiffness as measured by dynamic modulus at different temperatures and frequencies increased by 25-60% compared with virgin mixtures. The research also concluded that fracture energy, which is an indicator of fatigue cracking, was better for virgin mixes compared with high RAP mixtures. The study of long-term pavement performance (LTPP) for overlays of 20 years and 30% RAP content showed that fatigue, longitudinal and transverse cracking are the distresses that occur more often in RAP mixtures [14]. The study concluded that generally mixtures containing RAP performed better than or equal to virgin pavements for majority of the cases [2]

d. Testing conducted for the NCHRP 9-12 study confirmed that recycled mixtures with a RAP content greater than 20% have a lower fatigue life than virgin mixtures [10]. Decreasing the virgin binder grade may be an option to improve the mixture fatigue performance, especially at high RAP content [9]

e. To evaluate and compare fatigue performance of HMA with RAP, Shu et al. [17] prepared four asphalt mixtures consisting of 0, 10, 20, and 30% RAP with one source of aggregate and one type of binder. The fatigue properties tested included indirect tensile strength (ITS), failure strain, toughness index, resilient modulus, dissipated creep strain energy ($DCSE_f$), energy ratio, plateau value, and load cycles to failure. They observed that:

i. Inclusions of RAP into asphalt mixes generally increased tensile strength and reduced post-failure tenacity in indirect tensile strength tests

ii. The inclusion of RAP also generally decreased the $DCSE_f$ threshold and energy ratio calculated from IDT tests, which may result in the short fatigue life of HMA. Lower DCSEf values mean that the energy required to fracture the asphalt mix mixtures decreased as RAP percentage increased

iii. Based on the failure criterion of 50% reduction in stiffness (obtained from the beam fatigue test), incorporation of RAP increased the fatigue life of asphalt mixes, whereas based on plateau values from the beam fatigue test, inclusion of RAP would turn more input energy into damage, which may result in the shorter fatigue life

Two points are also present but not discussed in the study: results show an increase in the resilient modulus that indicates an increased stiffness of the mixtures with RAP content being increased. Furthermore, inconsistency between energetic and traditional fatigue approach suggest that

f. Huang et al. [4] evaluated fatigue resistance of a typical surface mixture commonly used in Tennessee at 0, 10, 20, and 30% RAP content. Fatigue characteristics of mixtures were evaluated with the indirect tensile strength test, semi-circular bending (SCB) test, semi-circular fatigue test, and semi-circular notched specimen fracture test. They found that

   i. Long-term aging influenced the ranking of fatigue characteristics for mixtures containing different percentages of RAP

   ii. Inclusion of RAP into the limestone surface mixture generally increased tensile strength, reduced post-failure tenacity, increased the mixture’s modulus (stiffness), and reduced viscosity characteristics.

   iii. In the study, total dissipated energy to failure at 20% of SCB tensile strength also indicated that inclusion of RAP generally increased fatigue life for unaged mixtures, whereas for long-term aged mixtures, dissipated energy increased with inclusion of 20% RAP and dropped to the same level as the mix without RAP

   iv. The inclusion of RAP in the mixtures improved the mixtures’ resistance to fracture failure. The inclusion of less than 20% of RAP material had very limited influence on mixture stiffness and indirect tensile strength characteristics. The inclusion of a high percentage (30%) of RAP tended to significantly change the mixtures’ fatigue cracking characteristics

g. The distresses in high RAP mixtures are mostly associated with the aged binder [18]. The stiff, less elastic binder in RAP typically increases mixture stiffness and can cause fatigue damage and low temperature brittleness. Other potential problems are associated with the unknown amount of actual blending that occurs between virgin and RAP asphalt binders and the effective contribution of the RAP binder towards the total binder content of the mix (often referred to as “black rock”) [2]. These are some of the main reasons for reluctance for government agencies to allow high RAP content

h. The addition of RAP to an asphalt mix may decrease the mixture workability as compared to the respective control mixture without RAP due to the high viscosity of RAP bitumen. The workability reductions were generally larger at higher RAP contents. It is common belief that the use of a softer binder could improve the workability of RAP mixtures [5].

2 Use of RAP in pavement industry

2.1 Infrastructures and development

Infrastructure is a heterogeneous term that includes physical structures of various types used by many industries as inputs to the production of goods and services [19].

Infrastructures can be distinguished into **economic** or **social**. Economic infrastructures (EOC, Economic Overhead Capital) are primarily oriented toward the support of productive activities or toward the movement of economic goods. Social infrastructures (SOC, Social Overhead Capital) may also increase productivity but in a less direct way than in the case of EOC. Roads, highways, railways, airports, maritime transport, waste pipelines, aqueducts, gas pipelines, power networks are all economic infrastructures, and they directly support productive activities. Schools, police stations, hospitals, sport facilities, green areas and rest homes are social infrastructures, and they're aimed to increase social wealth with indirect positive impact on productivity.

Road networks have a decisive impact on the economic development of an area. The impact is measured in terms of income, productivity and employment with reference to the specific area [20]. Aschauer (1989) says that "a region with adequate presence of infrastructures has a comparative advantage over another region fairly served by infrastructures. The larger the infrastructural assets, the higher the economic advantages of the area". Thus, economic development of a region is highly related to the quantity and quality of its assets.

2.2 Factors impacting demand for road infrastructures

Roads are fundamental assets for economic and social development of an area, and hence of a country. So why road industry is struggling in some European countries, especially in Italy?

Argus [21] lists some of the most impacting factors that contributes to road development:

- Economy
- Government budgets for roads
- Credit

Natural disasters and unplanned events e.g. acts of terrorism and accidents may also play a role. Some other less explicit and evident factors as corruption and constant decrease in valued added of the construction sector negatively affect the development of roads.

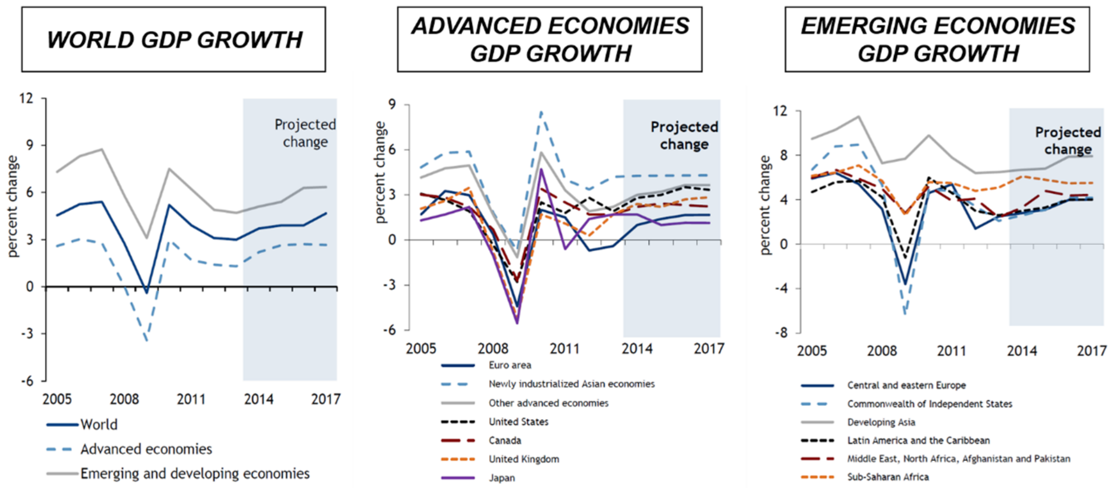
2.2.1 Economy

Economic growth contributes to the need for roads and for the demand for roads. Under conditions of economic growth, the mobility demand for people and goods increases together with the need of efficient roads and the financial resources of public administrations. Therefore, economy has a strong influence on public investments for strategic assets, as road and highways, both in negative and positive ways.

GDP growth provides a peek into the future, forecasting which markets will potentially have asphalt requirement, potential changes in balances for individual countries and changes in the trade patterns [21]. Economy growth leads to a larger number of roads being constructed.

Global economic prospects have improved following the drop in world GDP by -0.4% in 2009. In 2013, world GDP growth was 3%, down from previous 3 years. World growth is projected to increase 3% to 5% in next years. Advanced economies fell to -3.4% in 2009, bounced back to 3% in 2010 and then slowed in 2011 and again in 2012. Euro area had the slowest in 2013, whereas newly industrialized Asian economies led the pack in recovery in the same year. GDP growth of emerging and developing economies was 3.1% in 2009, increased to 4.7% in 2013. Middle East, North Africa and Sub-Saharan Africa weathered global recession well, however North Africa continues struggling with difficult internal transition.

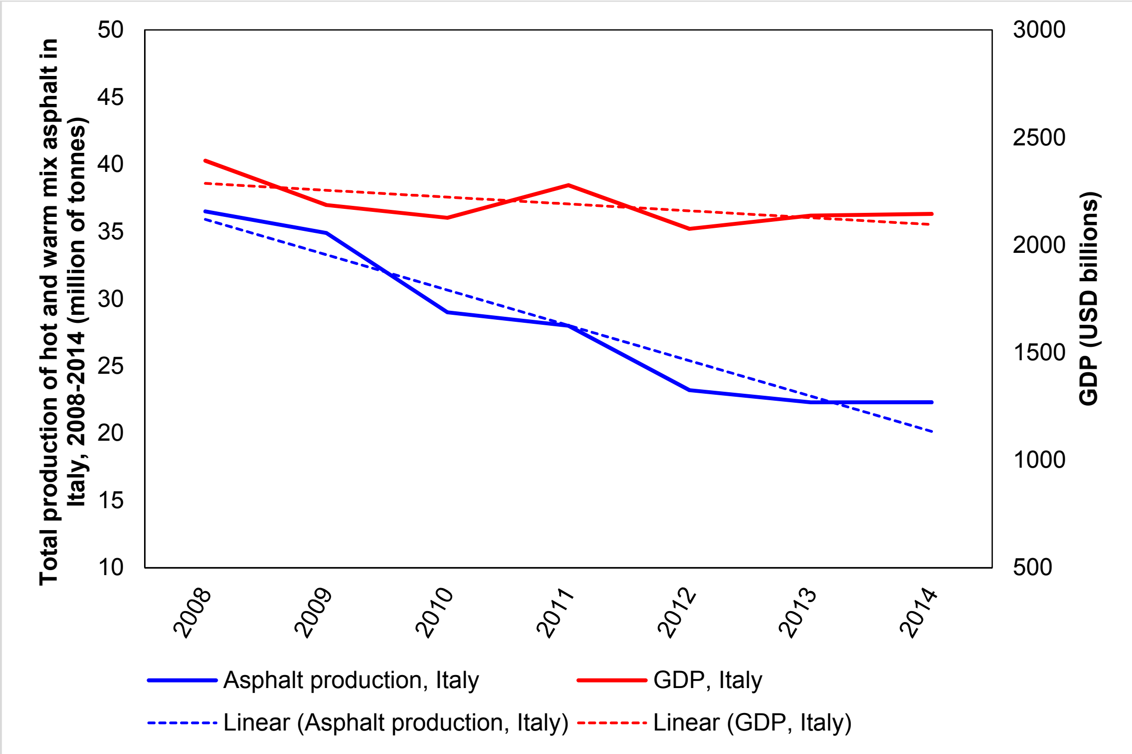
Fig. 2.1: Gross Domestic Product (GDP) growth of different economies



An example of the relationship between GDP and production of asphalt mixture (hot and warm) is discussed for the Italian case. The trend of GDP and asphalt production in the period 2008-2014 is illustrated in Fig. 2.2, showing a strong correlation between asphalt production and economic growth. The Italian GDP has significantly dropped down in 2008, reducing from 2,392 billion USD to 2186 billion USD, and in 2011, from 2,278 to 2,075 billion of USD. The production of asphalt mixtures follows a similar but more constant trend over the same period, with a global decrease of 39% in 7 years.

Data show a strong correlation between economic growth and production of asphalt. Negative variations of GDP lead to decreasing asphalt production as a consequence of decreased demand for new roads or maintenance works.

Fig. 2.2: plot of asphalt production and GDP in Italy in 2008-2014



2.2.2 Public capital

Public capital has a decisive role in the economic growth of core infrastructures of a country as roads, highways and airports [22]. However, government intervention exposes infrastructure investment to an additional layer of risks and decision-making biases [23]:

a. Political risks, reflecting the inability of the political system to deliver cross-party consensus around strategic plans for infrastructure and stable policy frameworks to support their implementation
b. Analytical risks, reflecting the dual relationship between the prevailing political ideology and economic mainstream
c. Unbiased project appraisal, reflecting the deficit in project evaluation grounded in sound and independent expert analysis and comprehensive assessment of policy alternatives
d. Limitations of the planning system and compensation mechanisms, reflecting a current planning system that does not properly share the asymmetric benefits of development
e. Public accounting distortions, reflecting practices that fail to incorporate the value of public sector assets and concentrate solely on public sector debt

2.2.3 Credit

Infrastructure suffers from a series of market failures that impede the optimal level of investment from being reached [23]. What usually sets investment in infrastructure apart from other types of investment is its long-term, capital intensive that typically generates long-lived assets with high sunk costs. This creates a gulf between (short term) marginal and (long term) average costs, which in turn, creates a time-inconsistency problem. However, investing wisely in infrastructure is critically important as over-investment can lead to projects that are inefficiently large, and therefore have low marginal returns.

2.2.4 Corruption

Unfortunately, a significant part of the public capital addressed to road construction is drained by corruption, resulting in the worst use being made for public funds. The last point of the list above can be related to corruption phenomena that are sadly a plague especially in Italy. The majority of corruption can be found on infrastructure public tenders [24]. In 2011, public contracts for infrastructures and services contributed to Italian GDP for 15.9%. The total value of public tenders was 18.3% of public contracts in the same year. 14% of public tenders were assigned without any tender notice, increasing the risk of fraudulent behaviours and corruption.

Alteration to public tenders may be pursued in different ways: tailor-making of technical documentations with the precise aim to favour specific companies; abuse of negotiate procedures. In some cases, public tenders may be by-passed with direct negotiations between public authority and companies. The abuse consists in an unnecessary use of this solution instead of public tenders, again with the aim to favour some companies; conflict of interest causes alteration of public contracts as well. Typically, the employer i.e. a public official (ab)uses of his position to pursue a private economic interests; agreement in public tenders between the two parts; definition of ambiguous evaluation and selection criteria in documentation; participation of tenderers in writing technical specifications; abuse of urgency reasons to by-pass public tenders.

Corruption strikes mostly during the post-tender phases of road construction. In particular, the quality control process is highly vulnerable since it involves raw materials, which are the biggest part of direct cost of road construction (including transportation) [25]. In this phase, illegal practices are able to produce the greater margins for illicit profits. An example: suppose the bitumen content of an asphalt mixtures is 5%. The production plant may decide to produce the material with 4% of bitumen content and sell it as an asphalt mixture with 5% bitumen content, with a profit on the 1% of bitumen saved. Furtherly, the testing laboratory, which provides the quality control on materials, may give in to the pressure of the plant, or other involved subjects, for certifying 5% bitumen instead of 4%. Laboratory may also grab a part of the profit that results from bitumen saving. However, the result is a material poor in bitumen that will probably fail before expectances and hence will require extraordinary maintenance paid by taxpayers. Sometimes these practices, despite fraudulent and illegal, are used by companies as survival strategies aimed to overcome increased prices and hence reduced margins. Nevertheless, this attitude negatively affects road construction, but it has its roots on the low valued added of construction industry instead on corruption itself.

2.2.5 Value added of construction sector

The low value added of construction sector is a factor that negatively affects road industry at all levels: asphalt plants, paving companies, testing laboratories and so on. The lower the value added, the less profitable the sector of activity. Also, the low value added creates condition for corruption to spread.

The Italian National Institute of Statistics (ISTAT) published a report where the value added at basic prices by sector of activity is presented [26]. The value added of construction industry, which includes roads and highways construction, has dropped from 5.2% to 4.9% in the period 1995-2014 (see Fig. 2.3). Services have a valued added of 74.4% basing on 2014 surveys. This makes services the sector with the largest number of employers (Fig. 2.4). Compared to services, construction sector has a significantly lower profitability. This confirm that companies have to face tremendous challenge when margins become too small or even zero (which means paying for working).

Fig. 2.3: value added at basic prices by sector of activity in 1995 and 2014, percentage composition

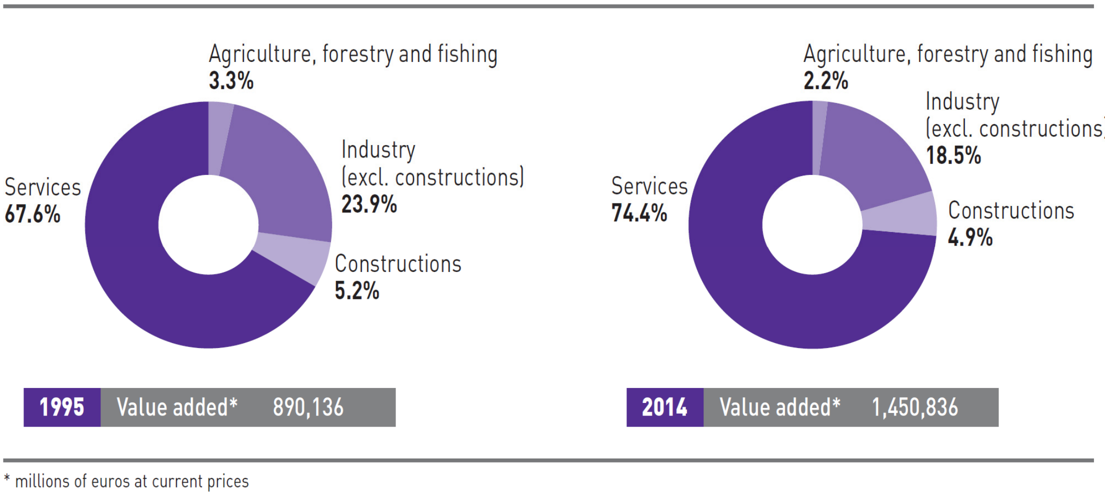
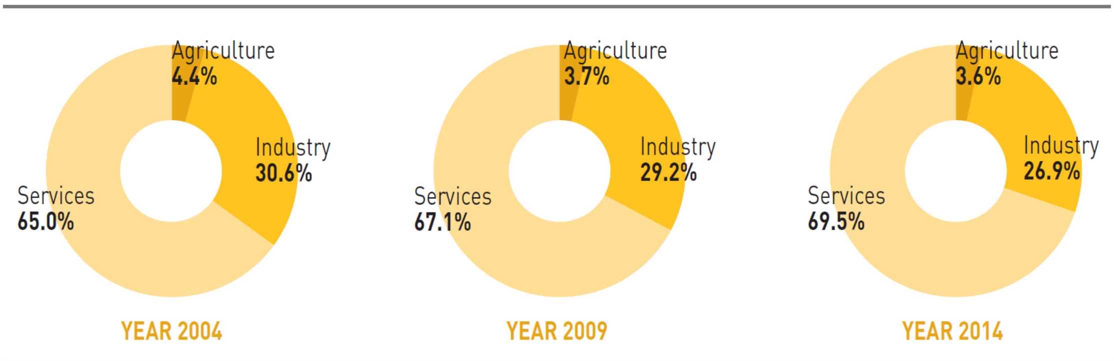


Fig. 2.4: employed by sector of activity, 2004-2014



2.2.6 Strategies to overcome road sector crisis in Europe

Underinvestment and under-maintenance define the current transport challenges. Growing demand on existing connections and dense concentrations at certain times of the day are putting the system under serious strain. The most heavily used and economically significant parts of the transportation network as urban areas; inter-urban corridors, and key international gateways, are showing signs of increasing congestion, unreliability, and overcrowding. At the same time, environmental costs demand a different policy context for a transportation sector that needs to contribute to reductions in the greenhouse gas emissions [23].

The London School of Economics Growth Commission points out the following suggestion to overcome investment issues for road infrastructures:

- Improving public accounting to generate management tools that report a comprehensive range of data on infrastructure assets and effectively use these new accounts as the basis for policy-making
- Introduce regulated road pricing systems to generate dedicated revenue streams that would provide a long-term solution to the problem of road investment, maintenance, and finance
- Institutionalise a flexible system of compensation for those who stand to lose from new developments
- Continue to develop and improve the broadband infrastructure in order to respond flexibly and promptly to a rapidly changing technological environment

2.3 Asphalt industry in figures

The European road network (EU-27) is more than 5.9 million kilometres long and comprises, in thousands of kilometres [27]:

- motorways: 73.2
- national roads: 345.8
- Secondary or regional roads: 1,551.9
- Other roads: 3,946.8

The European traffic population is estimated at 230 million passenger cars and over 31 million commercial vehicles. Over 90% of the total road network has an asphalt surface. Almost 260 million tonnes of hot and warm mix asphalt is produced per annum and approximately 5 million tonnes of cold mixtures. Asphalt is 100% recyclable and recycling has increased significantly in recent years. Quantities recycled directly back into road surfaces vary from country to country, but can be as high as 70%. There are over 1,900 asphalt production sites in Europe and over 10,000 companies are involved in the production and/or the laying of asphalt.

2.4 Asphalt mixtures production in Europe

Hot and warm asphalt mixtures production data are taken from European Asphalt Pavement Association (EAPA) [8]. The data have been established with the assistance of the members of the EAPA, and are at the moment the best available and reliable for the asphalt industry. A blank or ‘no data’ means no data are supplied or available. EU-28 means: Austria, Belgium, Bulgaria, Croatia, Cyprus, Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungary, Ireland, Italy, Latvia, Lithuania, Luxembourg, Malta, The Netherlands, Poland, Portugal, Romania, Slovakia, Slovenia, Spain, Sweden, United Kingdom. Asphalt production data are available for Iceland, Norway, Switzerland, Turkey and United States as well. EU-27 refers to EU-28 except for Croatia. In further considerations, Europe area will include all countries except for United States.

Production data referred to 2014 are reported in Fig. 2.5. Data include hot and warm mix production. Germany shows the highest asphalt production, with 39 mil tonnes, followed by France with 31.9 mil tonnes. Turkey is the third asphalt producer in the (extended) European area with a production of 30.9 mil tonnes. Italy follows with 22.3 mil tonnes of asphalt. United Kingdom’s production is slightly above 20 mil tonnes. Italian Asphalt and Road Association (SITEB) says that 40 mil tonnes of asphalt are necessary to properly maintain the national road network, which is almost twice the annual production.

The asphalt production in Europe has changed from 338 mil tonnes in 2008 to 265 mil tonnes in 2014, dropping of 21.6% in the seven years. United States production was 440 mil tonnes in 2008 and 319 mil tonnes in 2014, decreasing of 27.5% in the same period. However, data in Fig. 2.6 show different trends for US and Europe: the former experienced a significant drop of asphalt production in 2008 maintaining almost constant values in successive years; the latter has been experiencing a constant production decrease, with a significant drop from 2011 to 2012. In this period, Europe started to face the sovereign debt crisis that has reduced economical resources of countries for public investments and has impacted on strategic industrial sectors as construction and maintenance of roads.

Greece and Spain show the most significant decrease in asphalt production, -67% and -66% respectively in 2008-2014 as plotted in Fig. 2.7. Production loss in Italy is -39%. In Germany and France it’s -24%. However, some countries show positive trend: the increase in asphalt production in Belgium, Turkey, Luxemburg, Denmark, Poland, Switzerland and Norway ranges from 6% to 22%. Romania and Hungary made the best with +36% and +52%, respectively. This trend is confirmed by the large number of private companies that establish temporary headquarters in Eastern Europe. The average asphalt production variation in Europe in 2008-2014 is -21%.

Fig. 2.5: chart of hot and warm mix asphalt production in Europe area, 2008-2014 **[8]**

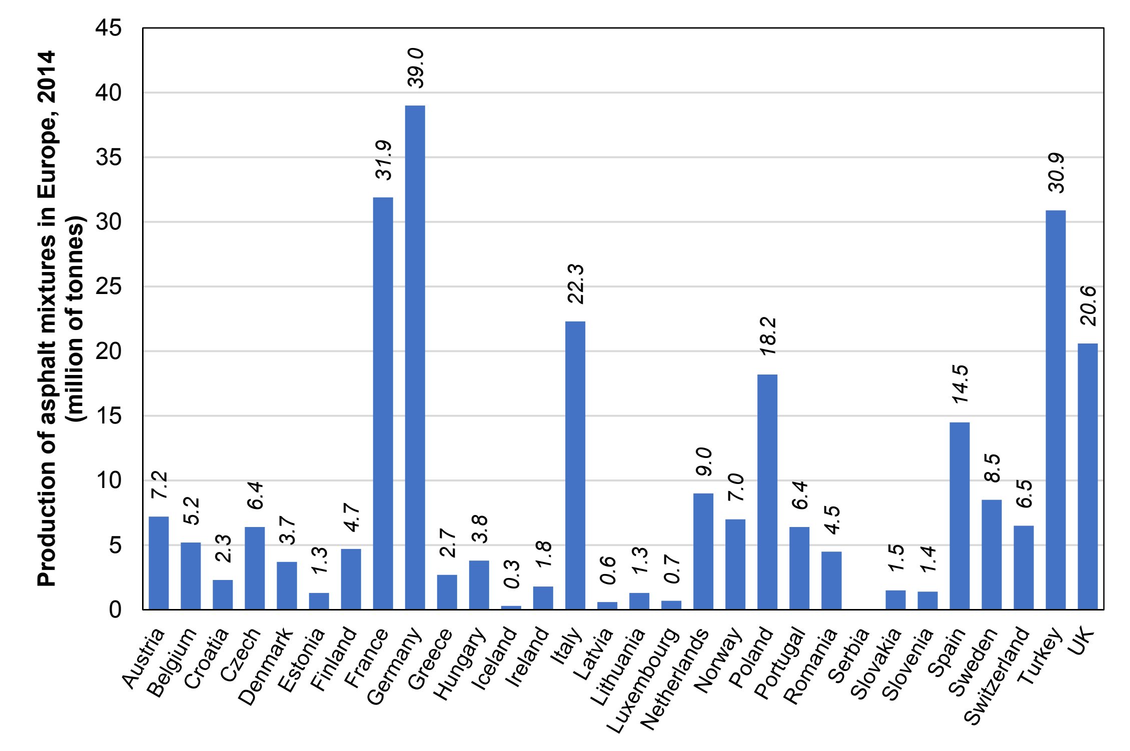


Fig. 2.6: comparison of Europe area and United States production of hot and warm mix asphalt from 2008 to 2014 **[8]**

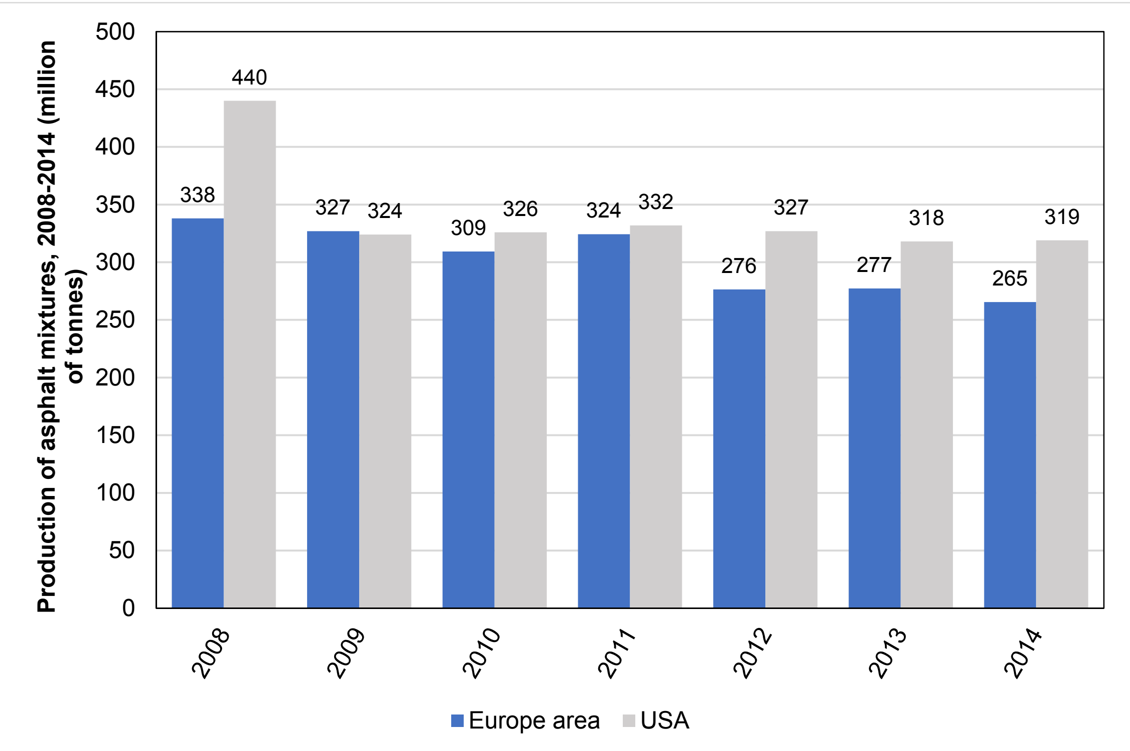
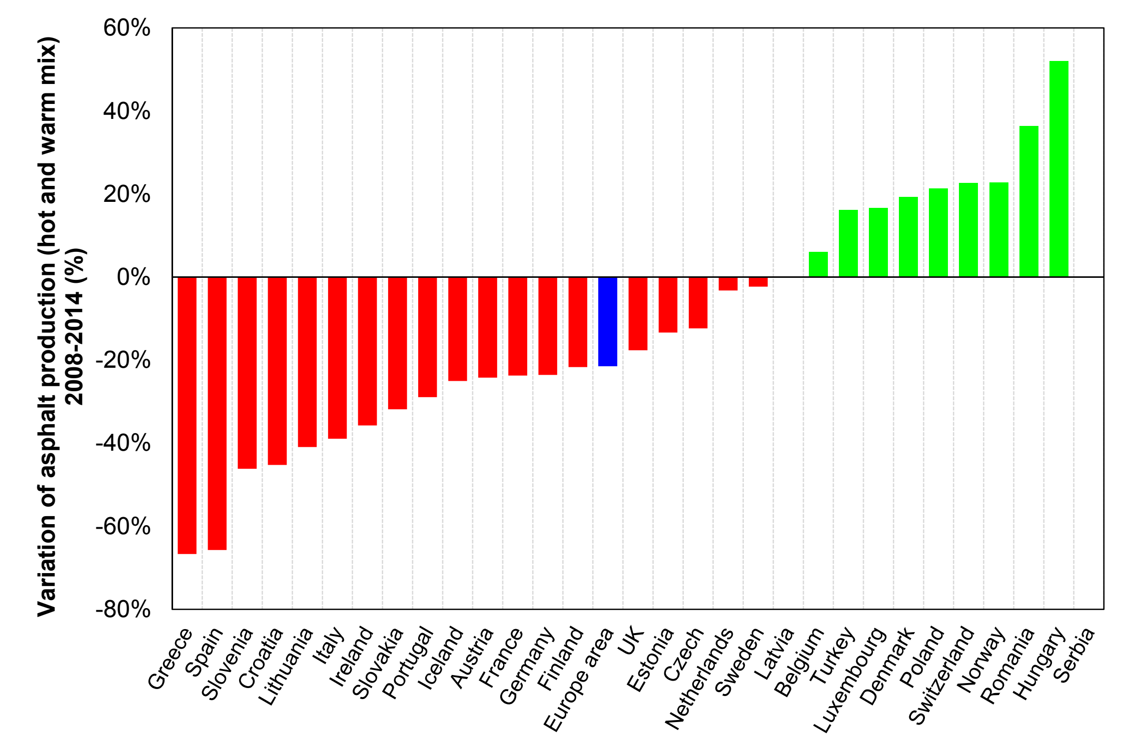


Fig. 2.7: variation of asphalt production in Europe, 2008-2014



Different countries have different asphalt production, thus different impact on the trend of European asphalt market. In order to evaluate the impact of each country on European asphalt production, the variation of asphalt mixture production of each country has been referred to the total production of the same country. Values recorded in 2008 are used both for countries and EU production. The annual asphalt production of a member State and Europe are noted as $AP(country)_{2008}$ and $AP(EU28)_{2008}$ respectively. The production variation in the period 2008-2014, which is plotted in Fig. 2.7, is noted as Δ(country). The impact of each country on variation of the European asphalt production is noted as I_{AP}(country) and it's calculated as follows:

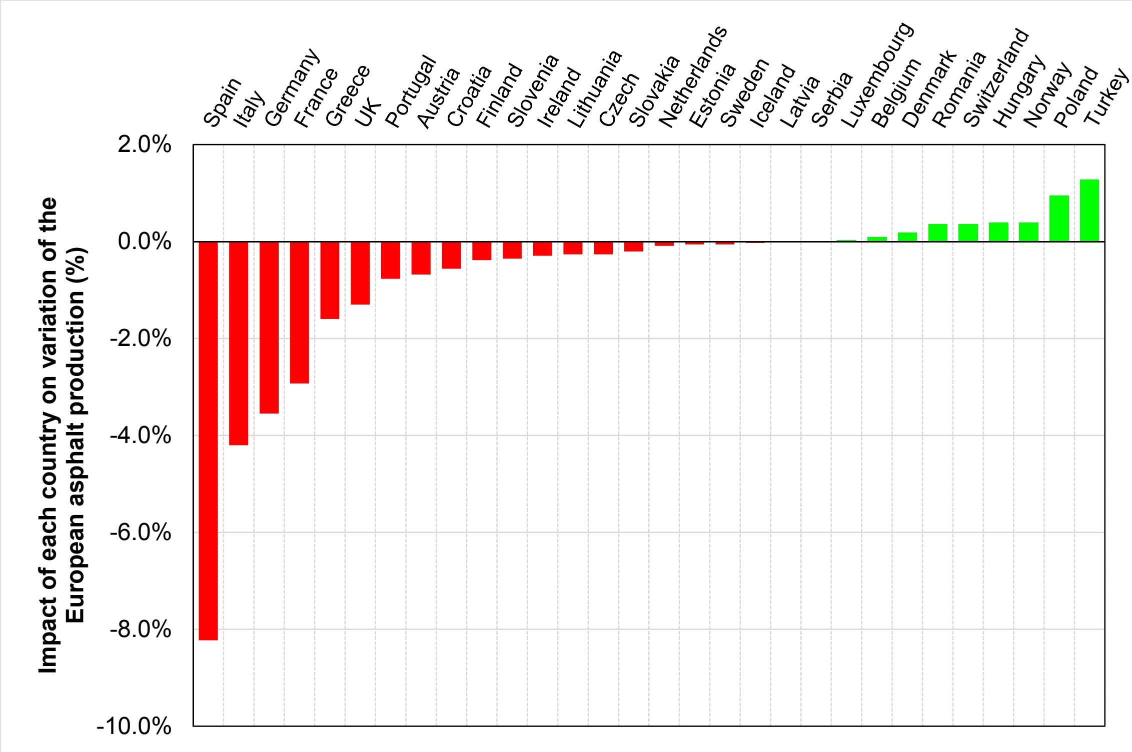
$$I_{AP}(country) = \frac{AP(country)_{2008} \cdot \Delta(country)}{AP(EU28)_{2008}} \quad [\%]$$

Results are reported in Fig. 2.8. Spain and Italy have the highest impact on asphalt production variation in Europe, with production loss of -8.2% and -4.2%. Germany and France follow with -3.6% and -2.9%. Greece impacts with -1.6% on total production. It is not a case that the firsts four biggest economies (in terms of GDP) of Eurozone have lowered down the asphalt market in Europe. Countries of Eastern Europe confirm the growth in asphalt production discussed above, with significant contributes of Romania (+0.4%), Hungary (+0.4%), Poland (+0.9%) and Turkey (+1.3%).

This results is not unexpected for two reasons: first, developed Western countries are facing an expected slowdown on asphalt production caused by the reduced need of new roads. Financial efforts have been moving from construction of new roads to maintenance of the existing system.

Material required for maintenance works is much less than that required for building new roads. Second, the road network growth of Eastern Europe partially explains the larger volume of asphalt production. Eastern Europe is still significantly less developed than Western Europe, and high growth rates are expected in such countries.

Fig. 2.8: impact of each country on European asphalt production variation, 2008-2014



In conclusion, European asphalt production has dropped of 21% from 2008 to 2014, with most significant decreases in Greece, Spain and Italy. Western Europe countries have all negative variation of asphalt production, whereas Eastern Europe countries have been experiencing an increase in asphalt production, especially Romania, Hungary, Poland and Turkey.

2.5 Bitumen consumption in Europe

Bitumen is a viscous liquid or semi-solid petroleum residue. It is predominantly used as a binder to consolidate aggregates in concrete and asphalt. Bitumen is obtained as a refined residue from fractional distillation or in other cases crude bitumen may be found as a naturally occurring petroleum deposit.

Bitumen consumption is strongly correlated with asphalt mixture production, since the majority of bitumen is used for road applications.

European bitumen consumption in 2014 is 12,385 tonnes [27]. This value comprises standard, hard (pen at 25°C below 50 decimillimetres), soft (pen at 25°C above 50 decimillimetres) and industrial grade bitumens. Surveyed data for each country are reported in Fig. 2.9. With reference to paving grade bitumen consumption, France, Germany and United

Kingdom have a market share of 20.5%, 17.7% and 13.4% respectively. Italy has a lower position, with 6% of European consumption. Details are reported in Fig. 2.10.

Fig. 2.9: total bitumen consumption in the European area, 2014 (Source: EAPA)

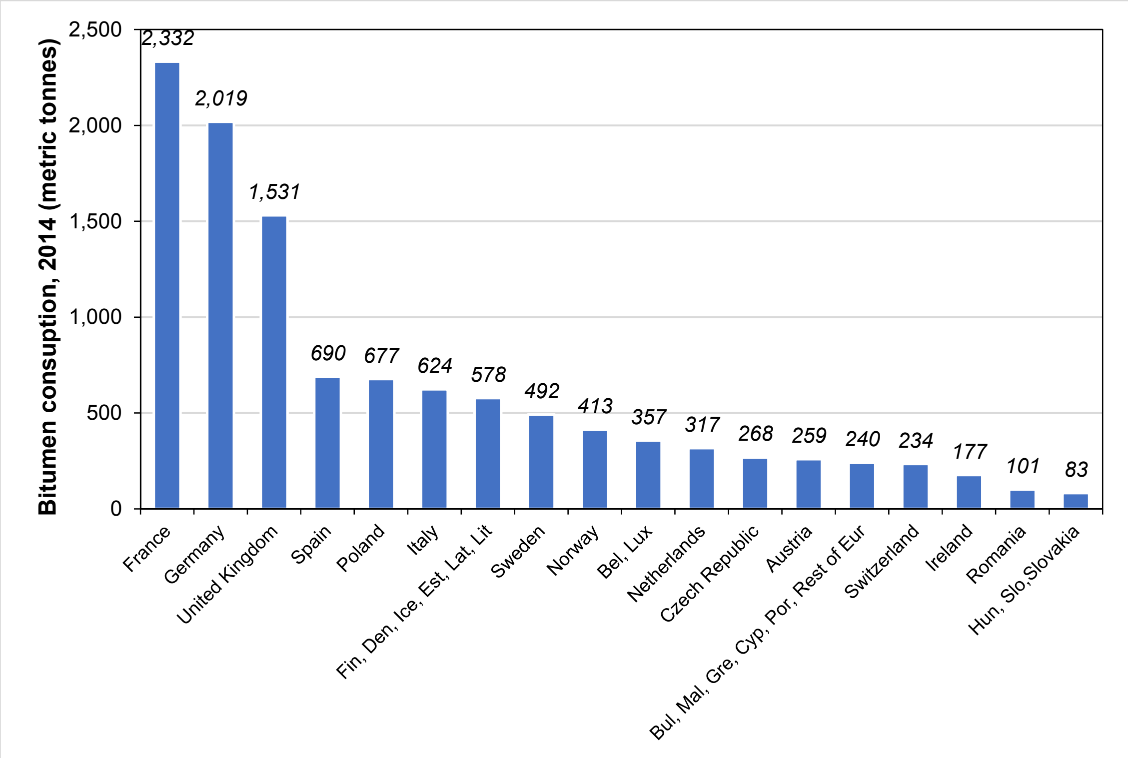
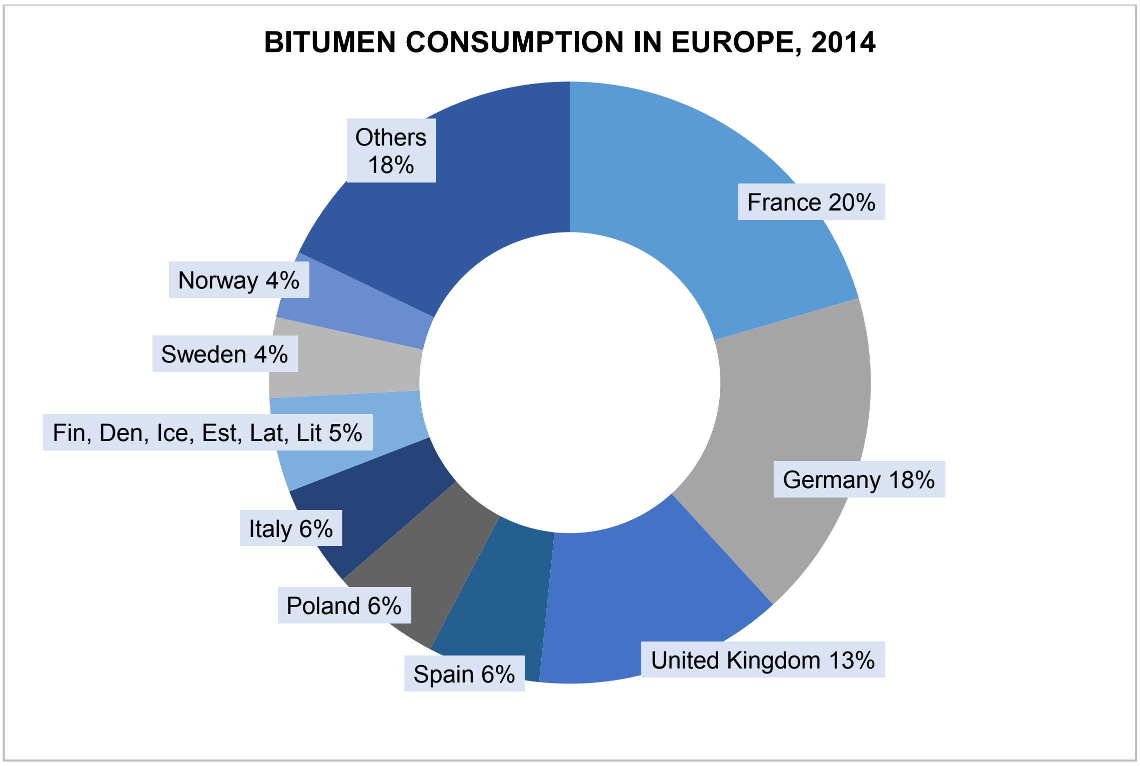


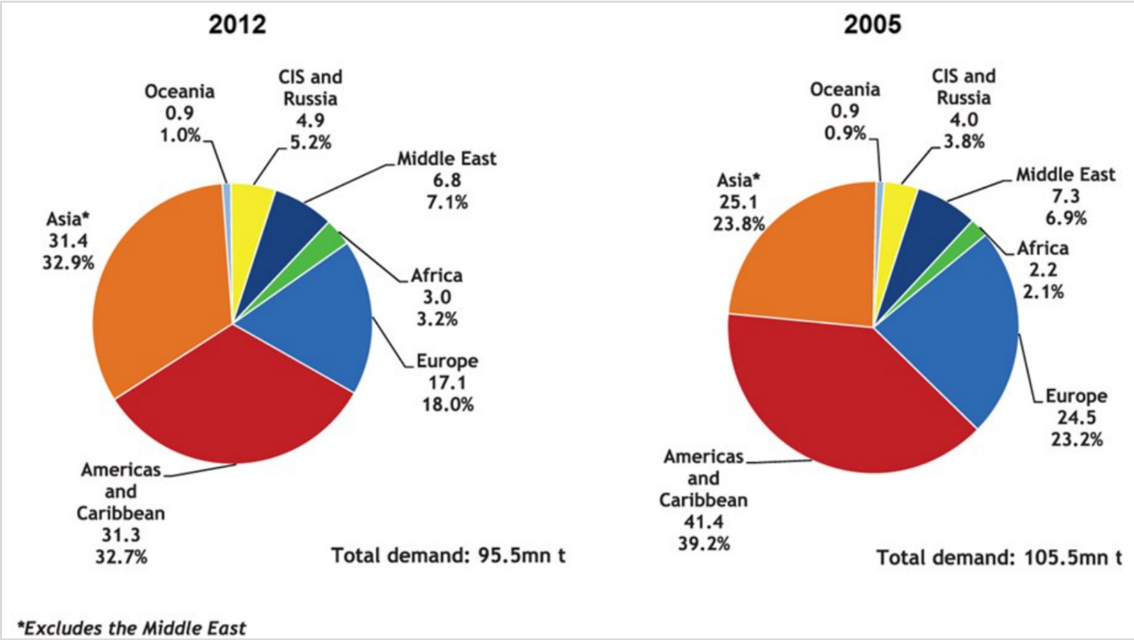
Fig. 2.10: market share of bitumen consumption in European area, 2014 (Source: EAPA)



2.5.1 Evolution of demand in the bitumen market

The last seven years have seen significant shifts in the demand for bitumen globally. In 2005, according to figures from Argus Asphalt Report, a weekly publication that records bitumen prices worldwide, the Americas led the demand, using 39% of the world's total 105.5 million tonnes; second was Europe and third Asia. In 2012, Asia had just overtaken the Americas to become the biggest consumer of bitumen, with 32.9% of a total 95.5 million tonnes [28]. Europe had slipped into third place with 18% (see Fig. 2.11).

Fig. 2.11: change in global bitumen demand: 2012 vs 2015 (Source: Argus Media)



World demand for primary bitumen is projected to rise 3.6% yearly through 2017 from a weak 2012 base to 121 million metric tons [29]. Many developing countries, particularly those in the Asia/Pacific region, will post strong growth in bitumen demand as they work to improve transportation infrastructure. Mature Bitumen markets, which saw declining demand between 2007 and 2012, will recover steadily, though in most cases 2017 asphalt demand will still be below 2007 levels. Gains will be fuelled by especially strong growth in asphalt consumption in China and recovering demand in most developed countries.

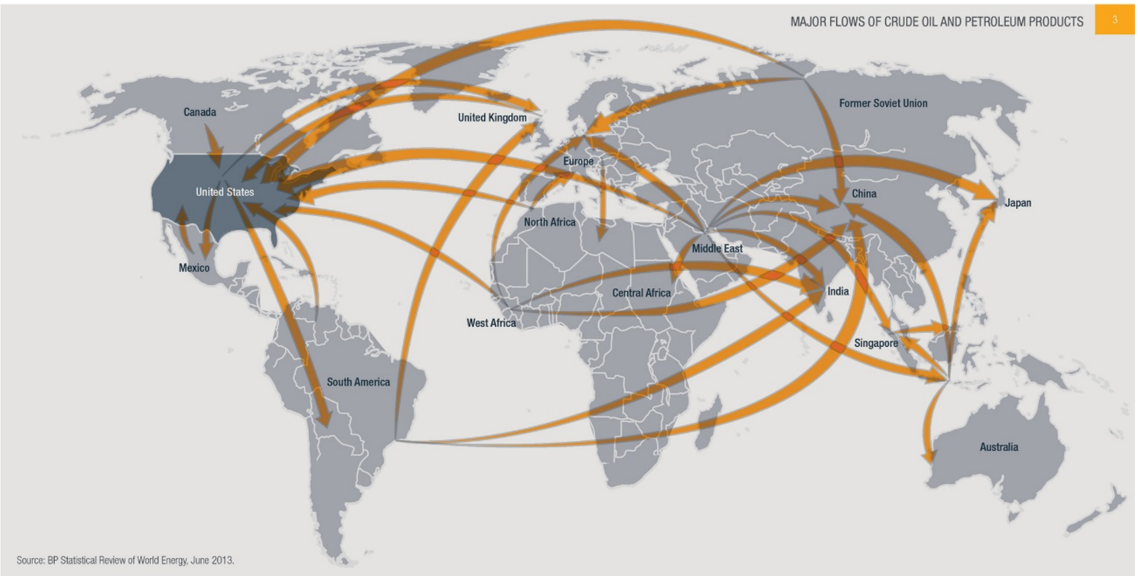
The **Asia/Pacific** region, which overtook North America in 2009 as the world's largest regional consumer of asphalt, is forecast to see the most rapid advances through 2017. China is expected to post robust growth through 2017, surpassing the United States as the largest national market for asphalt worldwide, despite deceleration from near double-digit annual growth in the 2007-2012 period. India, a much smaller bitumen market than China, will also post large annual gains through 2017 that will further boost regional demand. The massive transportation infrastructure building programs in those countries will fuel strong regional increases in asphalt demand as the region constructs new paved roads at a pace far exceeding that of any other region.

Developed countries are also projected to see good gains through 2017, yet in most cases, they will not make up for the sharp contractions in asphalt consumption following the global economic recession. North America is a major asphalt consumer because of the need for maintenance on the region's massive network of roads and highways. In addition, the US and Canada are major consumers of asphalt for roofing products; demand for those products will rise with increased building construction expenditures through 2017.

Recovery in bitumen demand in **Europe** and Japan will be driven by heightened expenditure on road maintenance and repair applications after years of very low spending on infrastructure improvements. Many countries maintain backlogs of necessary road repairs, which will bolster demand for bitumen for paving applications. While demand for asphalt will grow in both Western Europe and Japan through 2017, neither will register gains that make up for the declines exhibited in the 2007-2012 period. That picture is likely to change again in the coming years, as Asia's share continues to grow. European refineries are already feeling the impact of Asia's dominance in demand. The Financial Times reported in July that Russian oil shipments to Europe had reached a 10-year low, with Moscow setting up a deal to supply China through a new pipeline. All European refineries which process mid-heavy crude are expected to face problems in sourcing raw materials and in buying at a premium light crude oil. Prices will rise over the next three to five years immensely in Europe [28].

While demand in Europe falls and Asia's large slice of the pie continues to grow, **Africa** will be growing its small slice. There are initial difficulties in getting into the market as some African companies require investment from the suppliers, but the growth is so great that everyone is looking for ways to get in. Seven of the world's 10 predicted fastest growing economies between 2011 and 2015 are African countries. And there is only 204 km of road for every 1,000 km^2, only a quarter of which are paved, compared to 944 km for the rest of the world, with over half paved.

Fig. 2.12: major flows of crude oil and petroleum products (Source: British Petroleum)



To conclude, bitumen supply outlook provided by Argus [30] observes the following:

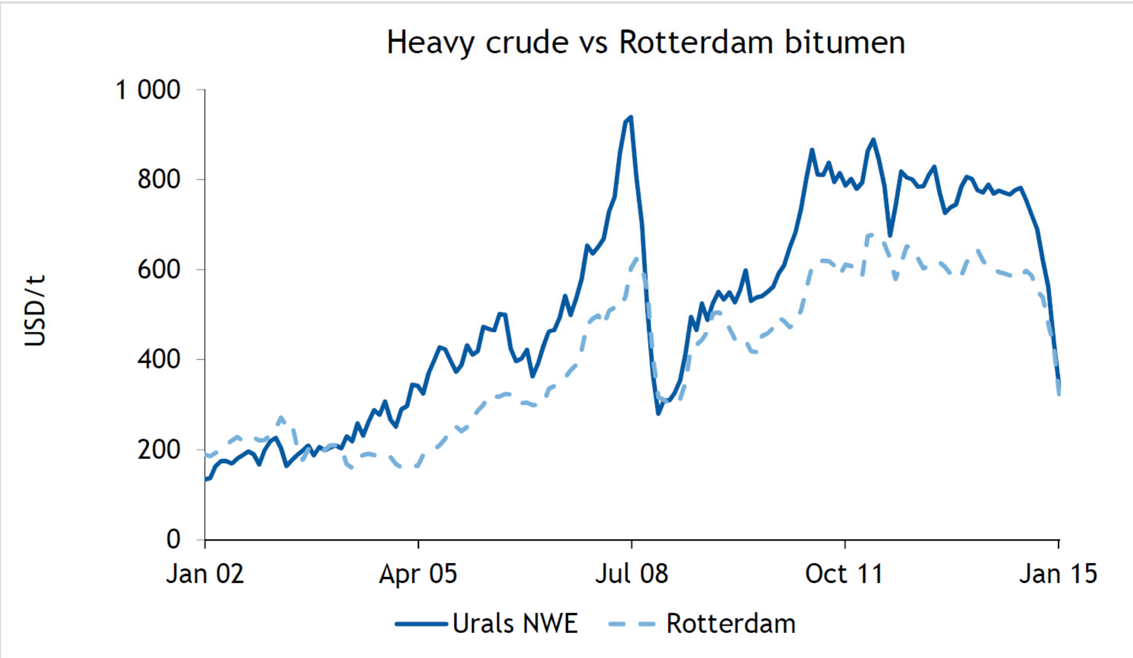
- Less bitumen production comes from fewer refineries, however is adequate. Bitumen supply reductions despite refinery closures has not kept up with demand reductions (there is still overcapacity)
- Bitumen will be produced only if economies justify it. This put pressure on the quality produced. It's a fact that quality of bitumen have been decreasing in the last years, with the need to use polymers to increase performances
- Refining capacity in Europe has been falling and refiners are not investing in increasing bitumen

2.5.2 Crude oil and bitumen price

The trend of crude and bitumen prices in the period 2002-2014 is reported in Fig. 2.13. First, a very high volatility of crude price can be observed. Oil price in 2008 was close to 1000 USD/t, five times the price of 200 USD/t in 2002. Oil price dropped down again in 2009 to slightly above 200 USD/t and rose with very irregular trend till the end of 2014. In 2015, oil price has been experiencing a significant decrease.

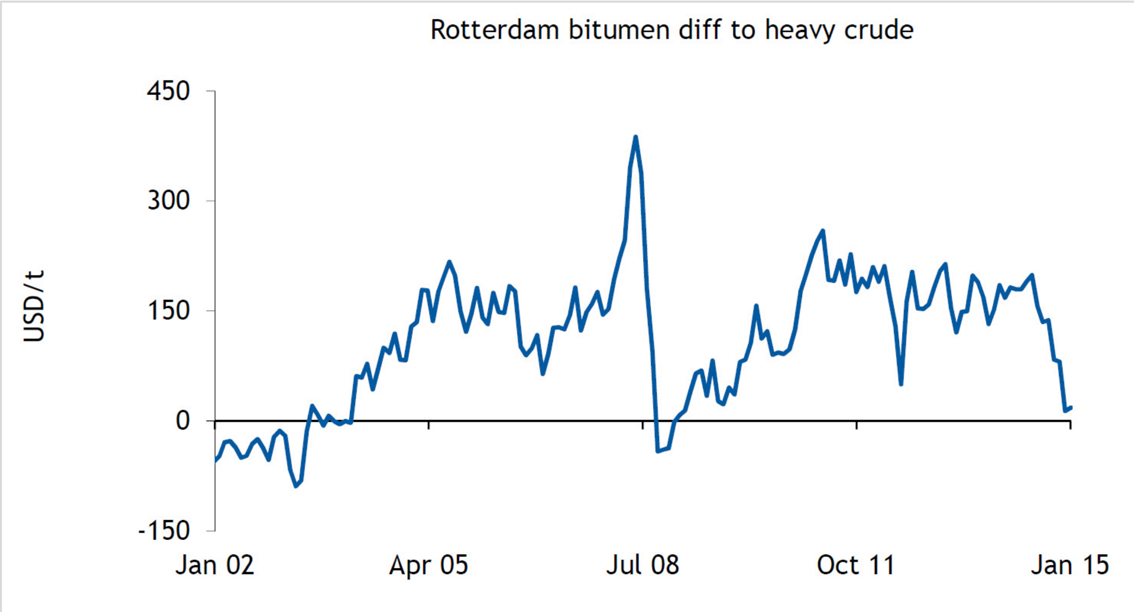
Many factors affect crude price. Traditional factors, such as oil supply and demand, technological advances, the marginal cost of production, seasonal and severe weather patterns and crude oil inventory levels all continue to be important in determining the overall direction of crude oil prices. On the other hand, new emerging factors such as the devaluation of US dollar have had a growing influence on oil prices. According to analysts, the growing presence of finance into oil market is likely to have contributed to increase volatility of oil price by making the price spikes higher, and the price falls lower. Deregulation of the crude oil market allowed financial interests to significantly increase their volume of trades, and this may have supported increased oil prices fluctuations.

Fig. 2.13: crude volatility and bitumen price reaction (Source: Argus Media)



However, bitumen price is slow to respond to crude oil variations. This is particularly evident analysing the spread between crude and bitumen price, which is reported in Fig. 2.14. From 2006 to the first half of 2014, the increase in bitumen price was 87.8% and the increase in crude price was 61.5% [1]. In the second half of 2014, oil prices declined 44% whereas bitumen prices increased 1.5%. Price lag indicates a weak correlation between crude and bitumen price variation. The current fall in crude price is expected to drive bitumen price down to a greater extent. But, price lag has become close to zero in 2015. This means that price of bitumen is affected by crude price only to limited extent.

Fig. 2.14: the bitumen lag to heavy crude (Source: Argus Media)



Summarizing, bitumen production decreases, but its price increases. This apparent contradiction can be explained by some factors:

a. Evolution in bitumen demand
b. Improvements in technology, which allow the production of higher value products from the heavy residues (bitumen production is reduced)
c. Long supply chain that causes difficulties in handling bitumen
d. Uncertainty in bitumen trading

All these factors cause refineries to change their business, reducing bitumen production in favour of more productive activities. Unfortunately, bitumen production is strongly related to volatility and hence to risk. Factors mentioned above are discussed.

Point “a”, **evolution in bitumen demand**: for some, bitumen just does not stack up economically. Small, specialist refineries cannot justify the production of bitumen and are having to shut down. For integrated refineries, bitumen has to make money for them, otherwise they will look at alternatives; or some may switch to lighter crudes and therefore make less bitumen. In

Europe, 13 refineries have closed in the last few years. A further four where production had stopped and perhaps would not restart; 15 more had changed ownership recently, with three more looking for buyers.

Point “b”, **improvements in technology**: coupled with the shifting global picture of supply and demand are improvements in technology which mean that many refineries are stopping the production of bitumen altogether. Processes such as hydrocracking and coking mean that refineries can create higher value products from the heavy residues used to produce bitumen. For example, cokers crack long chain bitumen molecules into more valuable short chain molecules as heavy gas oil, light gas oil and naphtha, leaving behind the excess carbon in the form of petroleum coke. The resulting crude oil is a blend of naphtha, light gas oil and heavy gas oil. Cokers reduce or eliminate bitumen or liquid asphalt production by pulling more gas or diesel fuel from the crude oil. This could reduce bitumen supply at a time when demand is increasing, causing prices to increase.

Point “c”, **supply chain**: the huge supply chain with many complications negatively affects bitumen price: refineries, terminal owners, ship owners, ship brokers, container carriers, packaging companies, importers, distributors, modifying companies and many others are all passages that increase the cost, and then the price, of the final product. Very sophisticated equipment is required to transport, store and distribute bitumen from source to end-user.

Point “d”, **uncertainty in trading**: for those that resell bitumen, and those that use bitumen, closures and changes in trading patterns mean uncertainty for both supply and prices. For contractors who are bidding for projects, sometimes years in advance, price fluctuations are bad news. Larger contractors who have the capital have been looking at the price volatility and creating more storage to give themselves flexibility, looking to do deals directly with refineries.

In conclusion, bitumen market has changed, and it’s experiencing a roller coaster ride with crude oil prices. This has a strong impact on refinery production costs and margins, changing the mix of asset ownership or refinery closures. Resulting ripple effects are:

- Bitumen can be disconnected from the international crude market since it’s treated as a by-product by integrated refiners [1]
- Advanced economies have decreased their bitumen need after the economic crisis in 2008, especially European (Western) countries. On the other hand, bitumen demand has been increasing in emerging economies. Thus, the bitumen market has shifted towards developing/emerging countries with an global increase of 3.6% in bitumen demand through 2017
- Refiners: integrated refiners have to justify bitumen as a business unit, while specialized refiners have to break even
- Resellers: unpredictability in availability and their wholesale costs, leading to fluctuating reselling margins
- Contractors: bitumen costs can fluctuate during the year and consequently contractors may experience difficulties in quoting long-term road projects

2.6 Reclaimed asphalt: the second life of road materials

Bitumen has the greatest impact on road construction cost. As discussed above, it significantly affects road construction and maintenance costs. As previously discussed, price fluctuations and a complicated supply chain make bitumen a source of uncertainty that creates issues to long-term planning of road projects.

Fortunately, there is an alternative source of bitumen we step on every day: roads. Asphalt roads and highways are sources of both aggregates and bitumen that, after expiring their service life, can be reused or recycled to build new asphalt pavements.

During their service life, roads experience the action of a huge variety of external factors as traffic, weather condition, thermal shocks, chemical agents, which can all be considered “external loads”. Accumulation of small, routinely repeated external loads together with exceptional events that may occur e.g. a massive snowfall lead to rising and spreading distresses into the pavement over the years, with increased severity. There are several types of distress: cracking, rutting, ravelling and weathering mostly affect asphalt layers (wearing and binder course). Distresses that affect base layers, foundations and subgrade are generally related to an insufficient bearing capacity or to a poor quality of materials cause layers to be vulnerable to water, traffic passages, thermal cycles.

A well-planned road maintenance avoids excessive spread of distresses on pavements and contributes keeping the road system in good conditions.
Depending on severity of distresses and hence on the required maintenance, one or more asphalt layers may be removed and then replaced with new layers. Old asphalt layers can be dismantled with an excavator or, better, removed with a milling machine. The first produces asphalt blocks, the second produces a granular material made of aggregates and old bitumen. Both are called “Reclaimed Asphalt Pavement” (RAP).

RAP is a source of raw materials (aggregates and bitumen) that change with time since the moment they’re paved. For example, an original paving grade 50/70 virgin bitumen may has changed its chemical composition during the service life due to exposition to oxygen, which causes old bitumen to become stiffer than the original. In general, natural aggregates may change in shape and grading curve, especially after being milled, but they’re stable in terms of chemical compositions. RAP is not the same material that was paved before being milled. This is a key issue that should always be taken into account. In this regard, a complete characterization of RAP shall always be performed in order to understand what type of material we are dealing with.

In conclusion, RAP is a source of raw materials, aggregates and old bitumen, 100% reusable or recyclable [1]. ***This makes RAP a resource for the construction sector, making reusing or recycling RAP a valuable and profitable practice with economic and environmental advantages***.

2.7 Asphalt recycling in figures

Recycling data are provided by EAPA and report on RAP availability and applications. Data on RAP availability are illustrated in Fig. 2.15 and reports only largest availabilities (above 1 mil tonnes), except for Spain that has been included as well.

Germany, France and Italy have the largest availability with 10.9, 9.2 and 9 mil tonnes of RAP produced in 2014. Other European countries have lower availabilities: Netherlands 4.5 mil tonnes, UK 3.4 mil tonnes, Turkey 2.3 mil tonnes in the same year.

There are many ways for handling and using RAP. Reclaimed asphalt can be recycled, reused or dumped at landfill. Recycling and reusing are aimed to produce road materials that incorporate a certain amount of RAP. Mixtures produced incorporating RAP shall have at least the same performances of the same mixes produced using only virgin materials in order to be economically attractive. However, RAP can be used for other civil engineering applications as well.

The most common RAP applications in Europe are presented below:

a. *Hot, warm or cold recycling in place or in plant*: RAP is processed i.e. sieved or heated and recycled to produce a new asphalt material for pavement layers. Depending on production technology, RAP can be recycled on site or in plant. When recycled on site, RAP is milled and recycled in the same time. Cold in place recycling is the most common example of in place recycling. When recycled in plant, RAP is first milled and then transported from a site to the plant, where is stored, processed and incorporated as raw material into new asphalt mixes. Hot in plant recycling is the most common example of in plant recycling. RAP mixes produced with hot in pant technology are used in binder, base and even surface pavement layers. Use of RAP for road resurfacing has been increasing in the last years thanks to retrofitting of asphalt plant, which allows to produce high quality RAP mixtures
b. *Unbound layers*: RAP is reused as a common aggregate class into structural unbound pavement layers as foundations or sub-bases, or reused as filling material in embankments
c. *Landfill*: RAP may be dumped into landfill areas when demand is temporarily low and availability is high or, in the worst case, when dumping is the most profitable or less costly solution

Hot, warm and cold recycling are the most valuable procedures for recycling RAP because the less expensive RAP bitumen can replace a portion of the more expensive virgin bitumen [3]. RAP is pre-processed and then it’s put into a new production cycle, which is the definition of “recycling”.

The use of RAP as filling material is somehow valuable, since it mainly reuses the aggregate. In this case bitumen, which is the most valuable RAP component, doesn’t bring any advantage to the new material, as its own name “unbound” suggests. The use of RAP as aggregate into unbound layers is referred as “reuse” of RAP because it does not involve old RAP bitumen.

Dumping into landfill is the worst way to handle reclaimed asphalt. RAP is a resource with economic value. Storing RAP in landfill means to freeze its economic value, involving other resources i.e. an area to store the “wasted” material with no profits. For some reasons, dumping might be the best economic choice for plant managers since it might represent the cheapest solution for handling RAP.

All previous considerations are summarized in Table 2.1.

Table 2.1: types of end products and value related to different RAP uses

RAP used in	Value	End product
Hot, warm, cold in place/in plant recycling	High (RAP bitumen is used as binder in the new asphalt mix)	Hot asphalt mixtures for waring, binder and base courses Cold asphalt mixtures (bitumen stabilized materials with foamed bitumen or emulsion) for base or sub-base layers
Unbound granular materials	Low-medium (RAP bitumen is not used as binder)	Unbound granular material incorporating RAP as aggregate
Landfill	Negative-very low	-

The fundamental requirement to produce RAP mixtures is that the resultant material has the same or improved properties of the virgin mix. Recycled asphalts can continue to, and increasingly, be re-used (back into the same product or application) at the top of the recycling hierarchy, before being recycled as ingredients in new asphalt. These are certainly preferable to them being “down-cycled” as aggregate or in to other construction materials (although after several re-uses and recycling phases this may eventually be appropriate) [1]. Hot, warm and cold recycling are considered to be “valuable applications” of RAP.

Different uses of RAP expressed in percentage of total availability are reported in Fig. 2.16. Data include Turkey and United States.

Italy uses only the half of its available RAP for valuable applications as hot, warm and cold recycling. 30% of RAP is dumped in landfill areas, 20% is used for filling unbound layers. Of the remaining 50%, only 20% is used in hot recycling and 30% is used in cold recycling.

On average, Europe recycles more than 70% RAP in valuable applications. This percentage raises to 75% excluding Turkey from the analysis.

Considering its availability, Germany is the European country that recycles asphalt in the most valuable way, with 90% of asphalt being recycled with in processes (mostly hot in plant). Netherland and Sweden follow with 85%, United Kingdom with 75% and France with 64% (see Fig. 2.17). Finland has 100% of RAP being valuably recycled, but its availability is significantly below levels of other European countries. The same consideration can be made for Spain. Italy uses only 50% of the available RAP for valuable applications, wasting the other half. The result is that RAP is considered to be “waste” instead of a profitable by-product.

A quick picture of different uses of RAP is given in Fig. 2.18. 61% of available RAP is used in hot mix recycling to produce asphalt mixtures incorporating a certain amount of RAP. Cold and warm recycling involve 5% and 4% of available RAP. 13% of RAP is used as filling material into unbound granular layers, whereas 7% is dumped. The remaining RAP 10% is used in other civil engineering applications.

With improvements in technology, asphalt plants are now capable to incorporate high percentages of reclaimed asphalt into new mixtures, making hot in plant recycling economically attractive. Due to different technological development of asphalt production plants and public awareness, European asphalt recycling rate varies from 10% to 90% [31], as discussed above. Investments in retrofitting of asphalt plants aimed to incorporate higher quantities of RAP into production process are significantly scattered over the European area.

Fig. 2.15: RAP availability of largest RAP producers in Europe, 2014 (Source: EAPA)

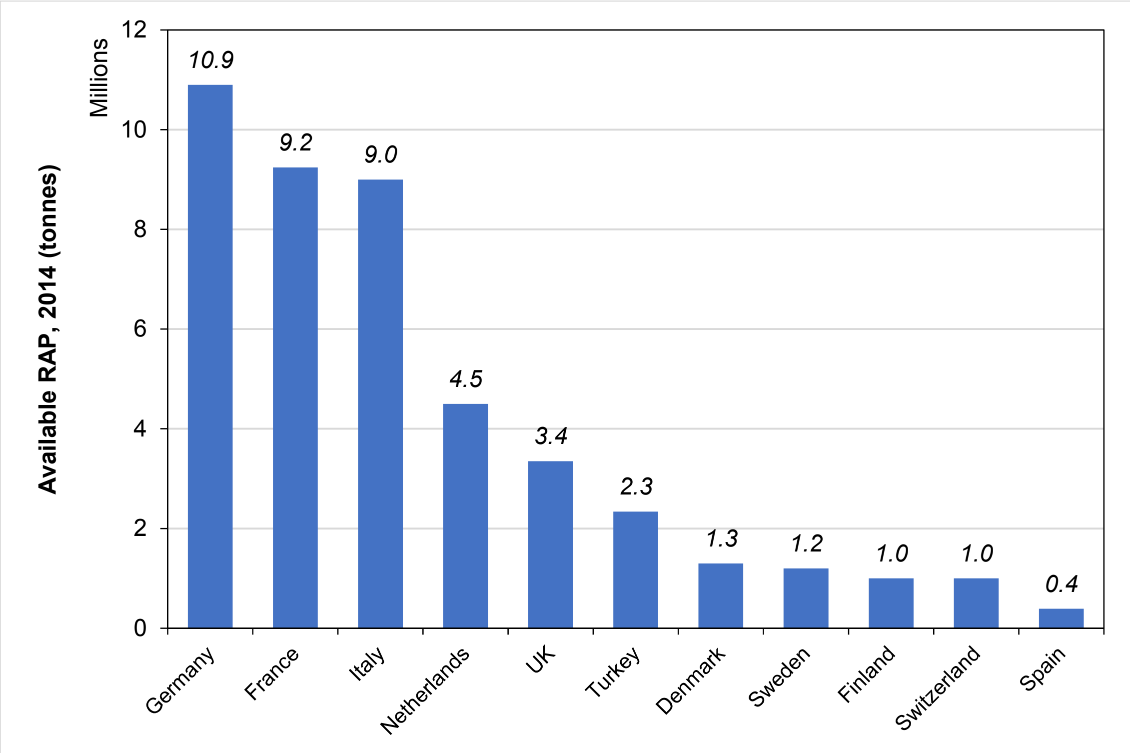


Fig. 2.16: different applications of available RAP in largest RAP producers countries in Europe and United States (Source: EAPA)

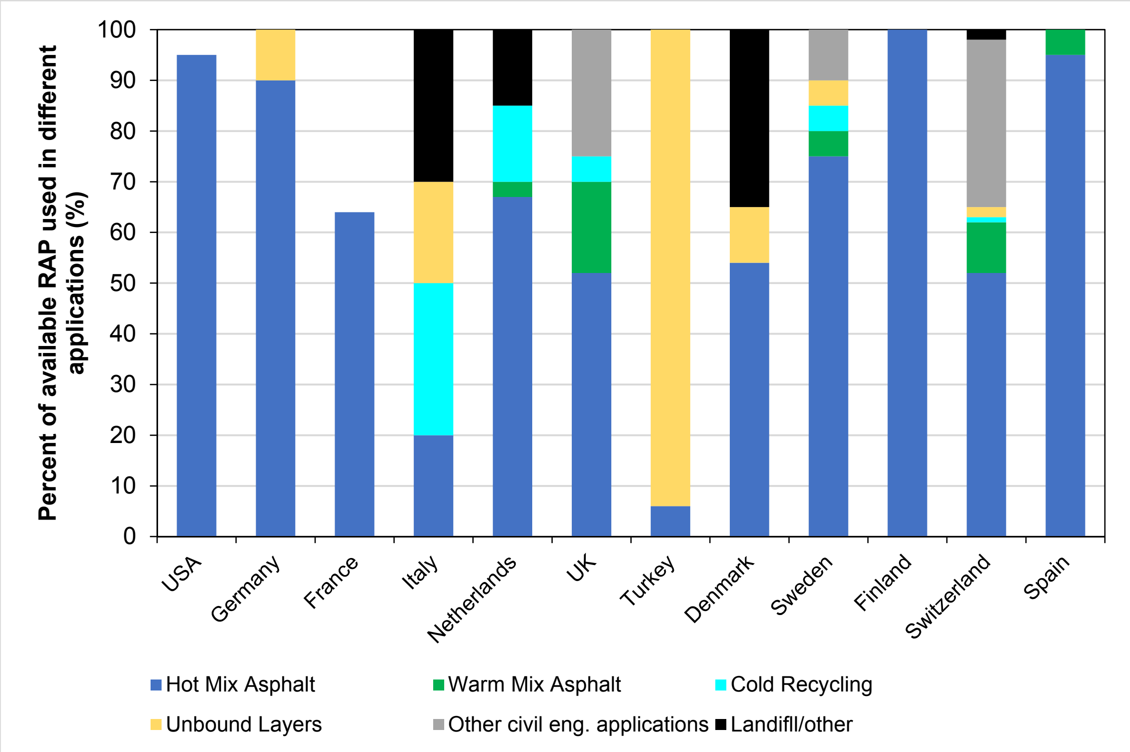


Fig. 2.17: percent of RAP used in valuable applications (hot, warm and cold recycling)

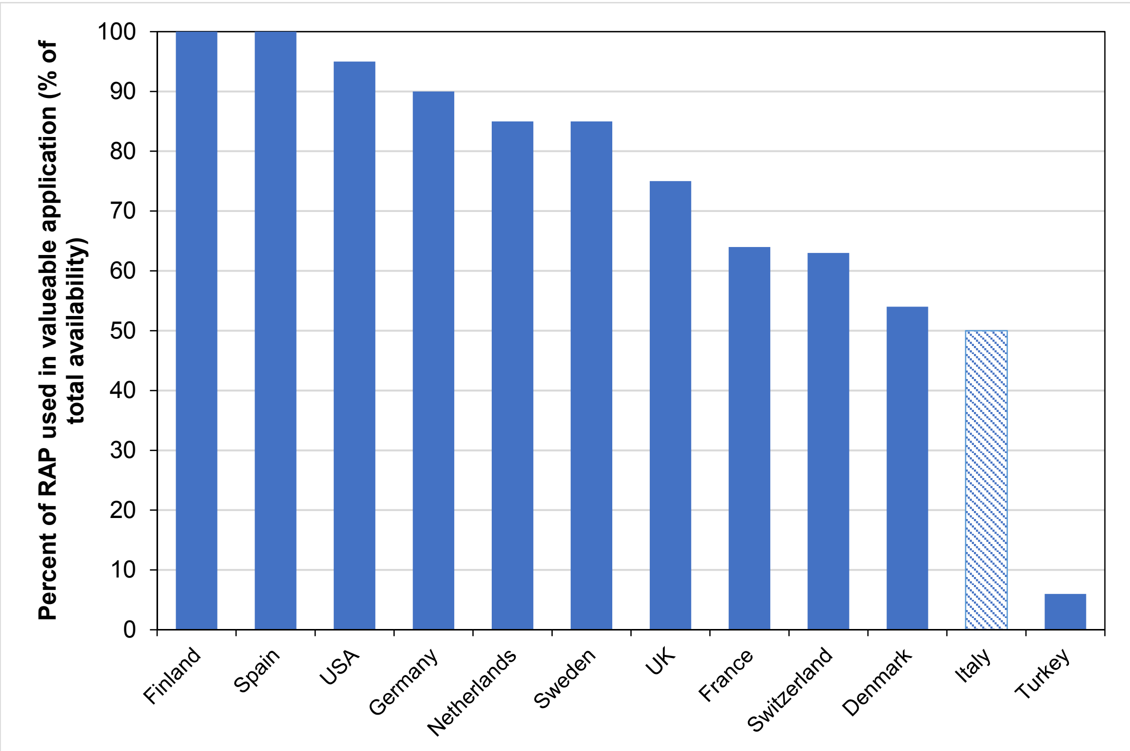
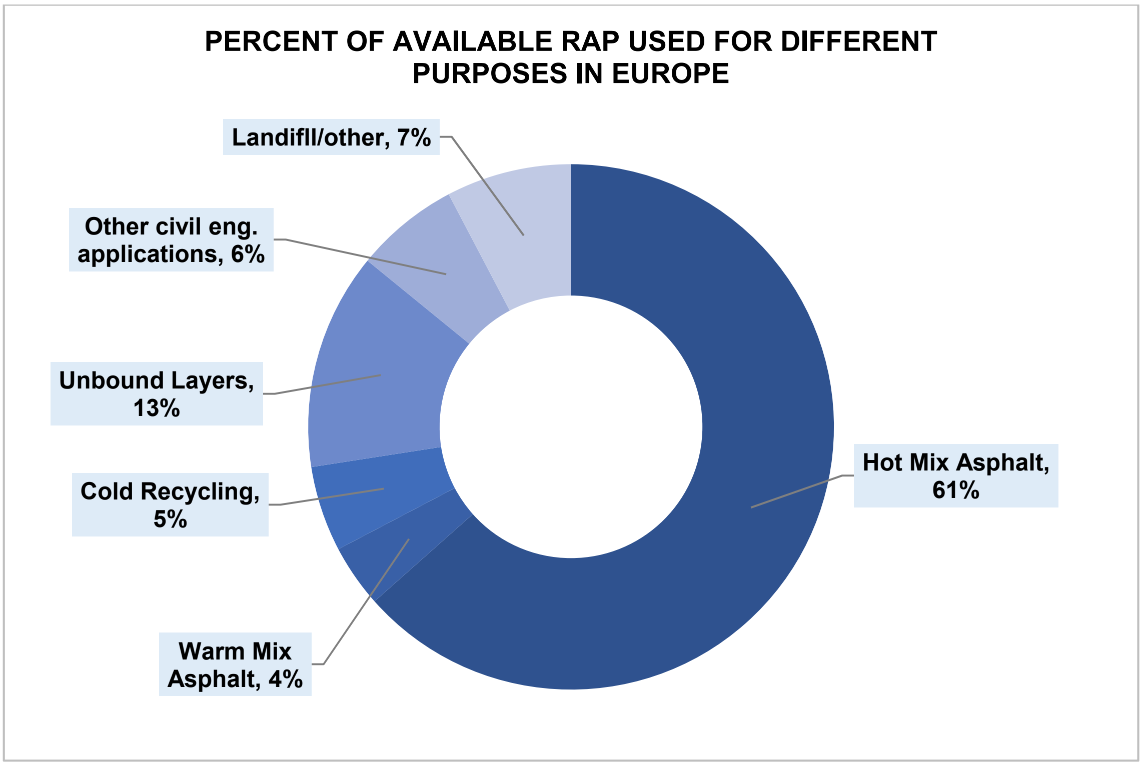


Fig. 2.18: different uses of RAP in Europe (Source: EAPA)



In conclusion:

- The most valuable use of available RAP is recycling to produce new asphalt mixtures. Different technologies are available as hot, warm and cold in plant/in site recycling
- The most profitable use of RAP can be pursued using RAP bitumen as binder in the new asphalt mixture. In this case, RAP bitumen can replace a portion of the more expensive virgin bitumen
- Asphalt mixtures produced incorporating RAP shall have at least same properties of the same mixtures produced with virgin materials only
- Europe recycles 70% of available RAP in valuable processes as hot, warm and cold recycling
- Hot in plant recycling is the most common practice for recycling RAP in Europe, with 61% of available RAP being used
- Italy addresses only 50% of available RAP for valuable applications. Hot in plant recycling is used only in 20% of cases

2.8 Effects of RAP recycling

2.8.1 Economic and environmental benefits

The success and spread of RAP recycling practices depend mainly on two factors:

3. Asphalt mixtures produced incorporating RAP, even in high quantities, shall perform at least as the same mixes made with only virgin materials
4. RAP mixes shall be profitable at least as virgin mixes

Experience suggests that if one of these two conditions fails, RAP will not be recycled or, in the worst case, it will be improperly recycled, resulting in low-quality RAP mixes. In spite of all environmental benefits of RAP recycling, companies would not be able to afford higher costs without economic advantages.

The first of the previous points is the topic of the present research, where performances and durability of a particular type of asphalt mixture will be investigated for different RAP contents. This is aimed to determine the effect of RAP content on mix properties and performances.

Regarding the second point, it's common to find in literature the idea that milling of asphalt pavements results in mountains of asphalt chunks that "have" to be recycled. The concept of recycling as something that has to be done "a priori" leads to a very weak reason for using RAP. One may or may not agree, but profitability is the most impacting factor able to determine the success of RAP recycling. Pursuing good practices in RAP recycling shall be a profitable business. Otherwise, all the positive environmental effects of recycling would be missed.

Environmental and social benefits are the objective of RAP recycling. In order to make them feasible and sustainable, there is a need of an economic reason to make RAP recycling a reliable and affordable practice. ***RAP recycling should be considered as a profitable business in order to develop all its potential environmental benefits***.

RAP usage may have both economic and environmental positive effects, which are strongly correlated. Economic advantages shall result in environmental advantages, and vice-versa. On the other hand, RAP recycling may pay the price of higher costs for production and handling, which might make RAP mixes less competitive than virgin asphalt mixes. Thus, the effects of RAP recycling can be divided into:

a. Economic and environmental advantages of RAP recycling
b. Increased production costs (disadvantages)

A list of main advantages and disadvantages of RAP recycling is proposed in Table 2.2.

Table 2.2: economic and environmental advantages and increased production costs of RAP recycling

Economic and environmental advantages of RAP recycling	Factors increasing production costs (disadvantages)
• Reduces need for virgin and non-renewable raw materials (virgin aggregates and bitumen • It's 100% recyclable • Is used as raw material to produce high quality asphalt mixtures • Reduces dumping to landfill of valuable material • Recycling gives high value added to RAP • Damps bitumen price fluctuation • Reduces transportation costs and emissions • Lower energy input is required for warm and cold recycling	• Modification to plants are required • Increases costs for handling (storage, processing) • Requires lab tests to determine physical properties • Special mix design may be required (especially for high %RAP incorporated in a new mix) • Special machines may be required (e.g. cold recycling in site) • Market deregulation

Economic/environmental advantages are:

- **Less natural resources**: recycling of RAP allows to reduce the amount of natural and non-renewable resources needed to produce a new asphalt mixture, since RAP can partially replace virgin aggregates and neat bitumen. RAP is 100% recyclable when appropriately handled and processed
- **Reduced emissions**: transport of materials and construction of roads produce pollutants being dumped in the atmosphere. A lower usage of natural resources results in lower emissions related to aggregates transport and supply from quarries, bitumen production, and movement of goods for long distances. In fact, RAP can be used directly on site or it can be transported to the nearest asphalt plant. Combustion of fossil fuels during manufacturing causes the emission of carbon dioxide (CO_2) and nitrogen dioxides (NO_2). This contributes to greenhouse gas (GHG) emissions and global warming, atmospheric pollution (ground level ozone creation and acidification) as well as nutrient enrichment. Also, the traditionally very high temperatures associated with heating of bitumen consume a large amount of energy. Reclaimed asphalt requires lower heating temperatures compared to those of virgin aggregates
- **Reduced landfilling**: the higher the quantity of RAP recycled, the lower the space necessary to store RAP somewhere i.e. landfills
- **Reduced use of virgin bitumen**: The most significant economic advantage of RAP recycling is the reduction in the use of bitumen, which is the most impacting factor on road costs [7]. While bitumen only constitutes approximately 5-6% by mass of the materials used in asphalt, it can contribute to over half of the total cost and this price tends to rise further according to increasing world crude oil prices [32]. As observed in previous sections, bitumen has been experiencing roller-coaster price variation that can be correlated to crude price only to a certain extent. Also, bitumen market has been changing rapidly moving demand towards expanding economies, with an impact on refineries production. In other words, bitumen price is one of the most important variable to take into account when planning maintenance and construction of roads, but still the less predictable. Roads are a source of bitumen we daily step on. For this reason, RAP is a material that can acquire value added depending on how it's used. Employing RAP as a feed (reuse) achieves important sustainability targets, but the key driver for companies using RAP in asphalt production (recycling) is that it provides a highly cost-effective source of good quality material that helps boost the company's profitability. Recycling offers the same functions as in the original application, while reuse is the utilization of a lesser function than in the original application i.e. foundation layers or fill for embankments

Increased production costs related to short/mid-term investments of companies are presented below:

- **Modifications to asphalt plants**: RAP mixes shall perform as good as similar mixes made with only virgin materials. In order to achieve this goal, traditional asphalt plants must be retrofitted with specific equipment i.e. parallel drum for heating RAP. Recycling in dated plant may result in increased emissions and fumes and faster equipment deterioration, which leads RAP being considered as a problematic thing to throw away somehow. Also, unfit equipment has high probabilities to produce low quality RAP mixes due to equipment itself or procedures followed to incorporated reclaimed asphalt

- **RAP handling/managing procedures**: RAP requires specific handling procedures from the moment it's stored at the plant to the moment is incorporated into a new asphalt mixture. Procedures include: stockpiling on the basis of RAP source, storage of stockpiles in covered area, processing i.e. crushing and sieving, periodic characterization of physical properties i.e. grading, bitumen content and bitumen. In addition to this, specialised consulting may be required. In case of high percentages of RAP incorporated into mixtures (>30%), performances and durability of RAP mixtures should be investigated in laboratory to avoid uncertainties related to RAP homogeneity. The higher the amount of RAP incorporated into asphalt mixtures, the more profitable recycling. Despite high quality asphalt mixtures can be produced incorporating high quantity of RAP, retrofitting, handling and specialised consulting are necessary to obtain competitive RAP mixes compared to virgin mixes. This inevitably increases production costs of RAP mixtures

2.8.2 RAP recycling turns short-term costs into benefits

Together with many environmental and economic benefits, RAP recycling requires investments that may lower competitiveness of RAP mixtures compared with virgin mixes on a short-term period. However, governments are aware of the fact that initial costs of RAP recycling turn into benefits on the mid/long period.

As discussed above, costs related to RAP recycling range from retrofitting of asphalt production plant and handling procedures. In other words, production costs of RAP mixes might be higher than costs of the same asphalt mix made with only virgin materials. Costs consist in short/mid-term investments that private companies should afford to produce high quality RAP mixes. In many cases, production costs of RAP mixes are not competitive compared to those of virgin mixes. In these cases, market will always chose the short-term cheaper solution for handling RAP, which may be dumping instead of recycling [1].

Retrofitting of asphalt plant to incorporate RAP has the largest impact on recycling cost. However, many benefits are associated to improvement in recycling technology: the use of specific instruments allows to increase the amount of RAP incorporated into asphalt mixtures. The volume of production increases as well.

Traditional batch plants can produce 10%-20% RAP mixtures. When using a parallel drum or a double dryer to heat RAP separately, mixtures containing up to 50% of reclaimed asphalt can be routinely produced [2]. The higher the amount of RAP being incorporated into mixes, the more profitable recycling since less virgin bitumen is needed. Separate RAP heating contributes to reducing emissions. Heating RAP into the same dryer used for aggregates exposes RAP to direct flame, burns the aged bitumen and generates emissions. This significantly limits the amount of RAP being processed by traditional asphalt plants. To overcome the issue, a common but incorrect practice is to introduce cold RAP directly into the mixing chamber with superheated virgin aggregates and neat bitumen. Superheating increases fuel consumption and emissions. Also, RAP's moisture content generates little steam explosions when cold RAP comes in contact with hot aggregates. Furthermore, improper RAP usage may cause clogging of screen (if used) and of both hot and cold elevators. RAP recycling in dated and hence unfit plants may result in additional machinery maintenance or, in worst cases, into work stoppages due to plant breakdown.

RAP recycling made into dated or unfit asphalt plants hides costs in the form of additional costs i.e. maintenance or loss of earnings i.e. reduced production rate, underuse of RAP and hence unnecessary overuse of virgin bitumen and aggregates. Furthermore, poor-quality asphalt mixtures are produced using unsuited instruments or following incorrect production processes, making RAP recycling unattractive.

One of the major issues that limits the use of reclaimed asphalt is the high variability of its physical properties as grading, bitumen content and bitumen type. Handling and managing of RAP may significantly help to reduce uncertainty, and hence to produce RAP mixes with consistent properties. Good practices are: storage of RAP in different stockpiles depending on RAP source, covering stockpiles to help maintaining a constant moisture content, periodic characterization RAP properties with laboratory tests. All the aforementioned practices are necessary to ensure homogeneity of RAP being incorporated into new asphalt mixes. Consequently, following good practices causes production costs to raise. However, the increased costs for RAP handling and managing allow to produce RAP mixtures on the basis of well-defined RAP properties. The better the handling, the higher the quality of produced RAP mixes.

Short-term costs can be reduced rejecting best practices of RAP handling. In these cases, RAP, which may come from different sources, is stored in the same uncovered stockpile. Then, it is taken from stockpiles and recycled. Clearly, this cheaper process generally lead to producing a final RAP mix with highly variable physical properties, as grading curve, bitumen type and bitumen content. Unfortunately, a short-term period saving may result in further remedial actions necessary to fix the low-quality asphalt paved on site. This has negative impact on public and private companies involved in asphalt production and on the quality of road network as well.

A list of short-term "savings" with associated negative impact on recycling process, and hence on RAP mixes production, is reported in Table 2.3.

Table 2.3: negative impact of short-term "savings" on production of asphalt mixtures with RAP

Short -term cost "savings"	Negative impacts of short/term "savings" on RAP mix production
No retrofitting	
RAP is not screened	• Poor control of mix properties i.e. grading curves • Low quality of RAP mixes
RAP is introduced directly into the mixing chamber	• Steam explosions in the mixing chamber • Need to superheat virgin aggregates, which causes increased emissions and fuel consumptions • Moisture and RAP temperature lower the temperature of the mixture. The higher the incorporated RAP, the greater the temperature drop
RAP passes through screens	• Clogging of screens; periodic maintenance is necessary to ensure quality in production and final mixtures
RAP is dried in the same drum used to heat aggregates	• Increased emission due to exposition of RAP bitumen to direct flame • Production rate shall be lowered
No RAP handling/managing procedures	
RAP from different sources is stored into the same uncovered stockpile	• Uncontrolled moisture content • High variability of RAP properties
Laboratory characterization to determine RAP physical properties is not performed on periodic base or is not performed at all	• Uncertainty about performances of RAP mixes • Concerns in RAP recycling • Low quality RAP mixes

In conclusion, short-term "savings" on retrofitting and handling may result in poor quality RAP mixes, increased emissions, increased fuel consumptions and reduced usage of a valuable resource as RAP.

2.8.3 Strategies to encourage and promote RAP recycling

In certain countries, the cost of producing a RAP mix might be higher than the cost of the equivalent mixture made with only virgin aggregates. This lead RAP being landfilled instead of being recycled or, at least, reused.

This is clearly unreasonable, since despite increased costs, many benefits can be obtained pursuing good RAP recycling practices. The point is that only a part of these benefits returns to companies that produce and pave RAP mixes in form of revenues. If this portion is lower than production costs, then RAP is going to be dumped or improperly used, instead of being recycled. The other part of benefits goes to the society in form of high-quality roads, reduced consumption of not-renewable natural resources and reduced emissions. These aspects have an economic value that, under conditions of **market deregulation**, represents a loss of money.

In many European countries, governments have been encouraging recycling of RAP with an intervention strategy aimed to make RAP mixes at least competitive as virgin mixtures. United Kingdom's government has introduced a levy on aggregates to stimulate reuse and recycling of reclaimed asphalt [33]. Some of the revenues are used to promote the use of waste as resource (WRAPP, Waste Resources Action Promotion Programme). At the same time, a landfill tax has been introduced in order to limit unnecessary landfilling.

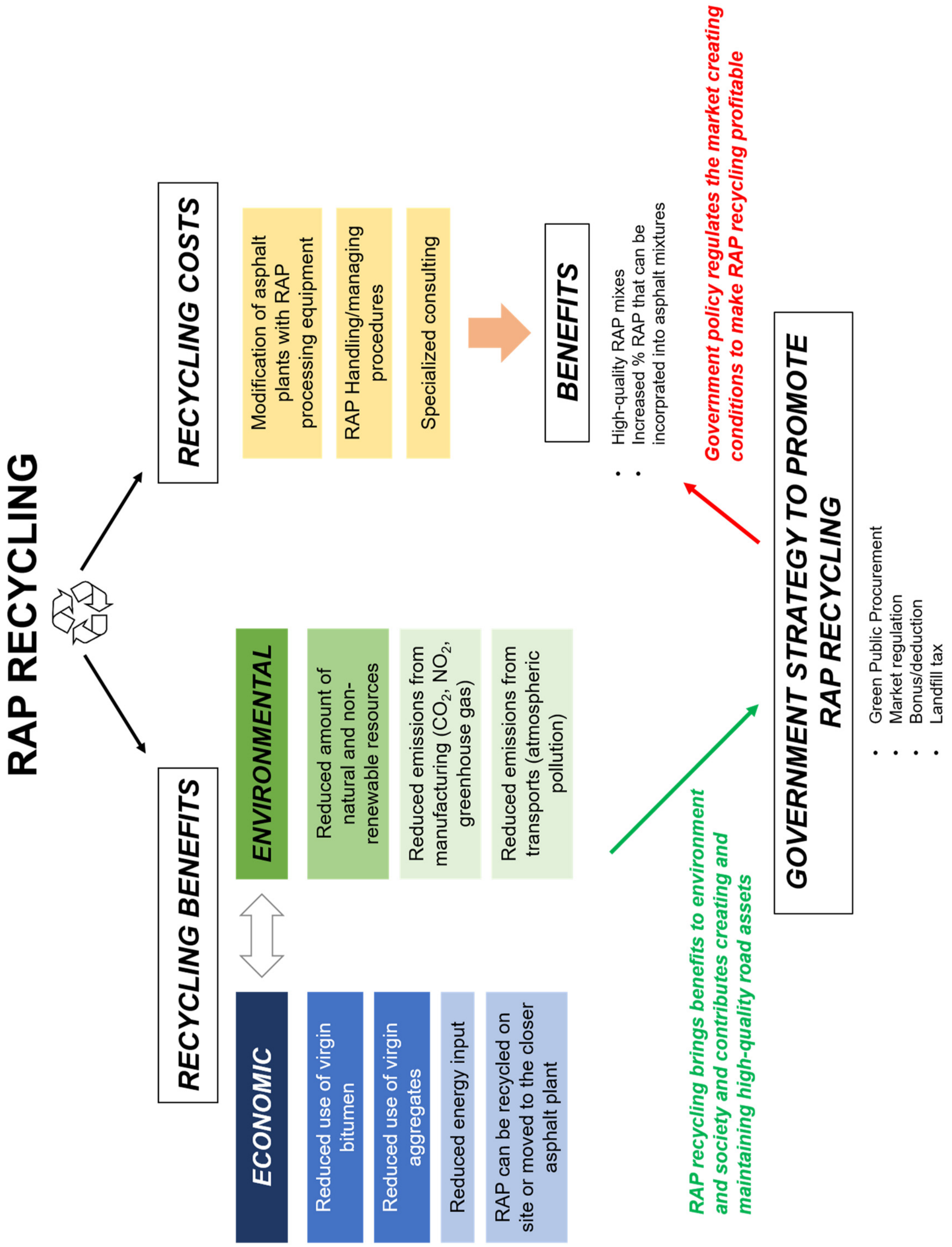
In Sweden, extra pays or deductions are provided on the basis of the amount of RAP incorporated into new asphalt mixes. The Gothenburg case is illuminating: for mixes that contain more than 10% RAP, a bonus is given. For mixes made with less than 10% RAP, a deduction is applied to the final price [1].

In Netherlands, a country with very limited natural resources, the cost of asphalt mixes that contain RAP is lower than that of the same quality of asphalt containing virgin materials only.

Germany makes large use of Green Public Procurement as tool to encourage RAP recycling [32]. In Hamburg, green public procurement principles are included in the city's overarching procurement policy and there are additional environmental contract requirements imposed by the City, which are specific to road construction. Under German National Law on Recycling and Waste, departments within local councils are required to increase the proportion of waste recycled. Furthermore, <u>laws require the use of any recyclable material in the highest possible position of any value chain</u>. Asphalt is a recyclable material and is therefore subjected to this legislation. Hamburg has taken a significant step forward to encourage an increase of recycling rates in construction materials by establishing an online exchange for soil, debris and construction materials. Hamburg prescribes the use of a minimum 35% white aggregates in order to brighten road surfaces and to have a colder, deformation resistant road surface in summer. The high cost of these aggregate materials increased the incentive to recycle more than just base courses.

In all these cases, governments provide for a regulation of asphalt market aimed to compensate short-term investments that companies have to afford to make recycling a profitable practice.

Fig. 2.19: benefits of RAP recycling and relationship with government's strategies to encourage and promote recycling



2.9 Methods to incorporate RAP into asphalt mixtures

2.9.1 Hot recycling technologies

Hundred per cent of the reclaimed asphalt can be recycled [4] with different methods: hot recycling in asphalt plant, hot in-place recycling, cold in-place recycling and full depth reclamation are the most commonly applied techniques [2].

The percentage of RAP (usually expressed by mass of the mix) that can be incorporated into asphalt mixtures depends on production process (plant type, production temperature, mixing time, and discharge temperature), paving technology and permitted emissions as well as RAP source and properties. These factors affect the interaction between RAP and virgin bitumens and consequently impact the performance of asphalt mixtures [5]. In Europe, 61% of available RAP is used in hot in plant recycling, as illustrated in 2.7.

The maximum amount of reclaimed asphalt that can be incorporated into a new asphalt mixture is mainly limited by the production technology [34]. Considering hot in plant recycling, most conventional drum plants can accommodate 50% RAP, whereas the percentage of reusable RAP in batch plant ranges from 10 to 30%. These limitations have been overcome thanks to multiple technologies readily available for production of up to 100% recycled hot mix asphalt [7].

RAP heating is the part of hot in plant process that mostly affects the quality of the final mixture and contributes determining the maximum amount of RAP being recycled. RAP can be heated in the dryer together with virgin aggregates (direct heating), or by contact with superheated virgin aggregates (indirect heating). Factors as plant type, production process and presence of equipment specifically designed for recycling affect the quantity of RAP being incorporated in new hot mixes.

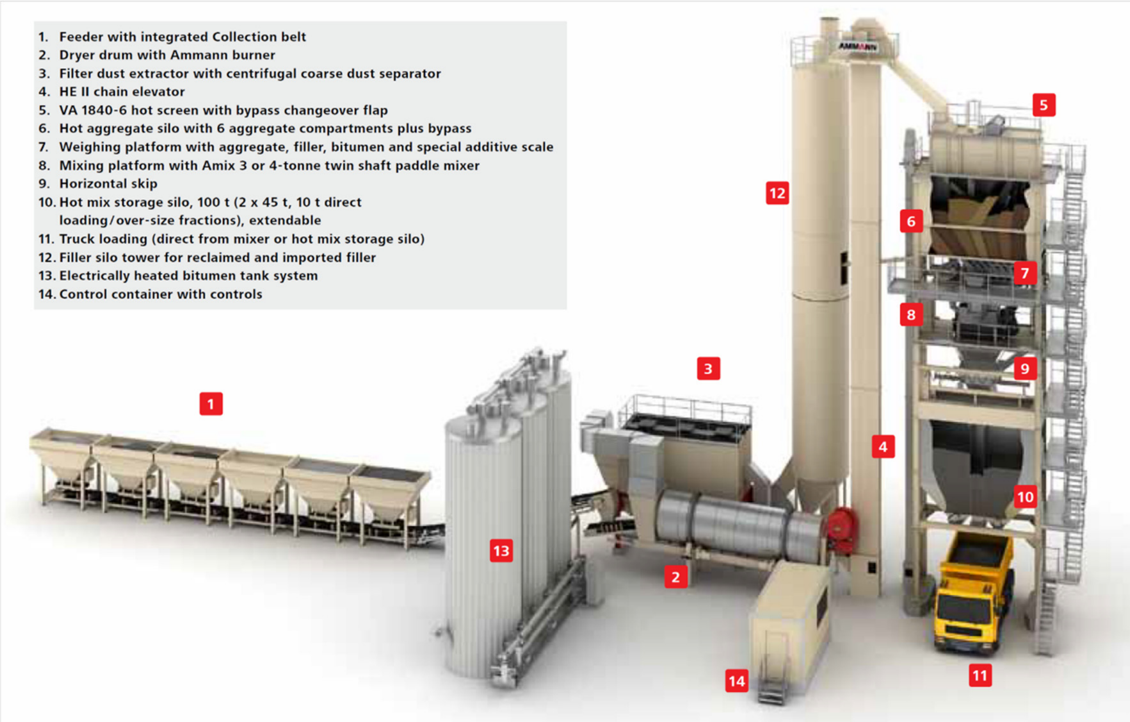
Asphalt plants for hot mix asphalt production are usually divided into two categories:

- Batch or discontinuous plants
- Drum or continuous plants

2.9.2 Addition of RAP in traditional batch plants

Today in Europe most asphalt plants are of the batch type [35]. Processing begins as the aggregate is hauled from the storage piles and is placed in the appropriate hoppers of the cold feed unit. Raw aggregate used for the production of asphalt is normally stockpiled near or at the actual plant site. It is advantageous for energy economy to store the raw material at a location where bulk moisture content can be kept at a minimum. The material is metered from the hoppers onto a conveyer belt and is transported into a rotary dryer (typically gas or oil fired). Dryers are equipped with flights designed to shower the aggregate inside the drum to promote drying efficiency. Aggregates are first dried via convection and then heated via radiation. As the hot aggregate leaves the dryer, it drops into a bucket elevator and is transferred to a set of vibrating screens, where it is classified into as many as four different grades (sizes) and is dropped into individual “hot” bins according to size. At newer facilities, RAP also may be transferred to a separate heated storage bin. To control aggregate size distribution in the final batch mix, the operator opens various hot bins over a weigh hopper until the desired mix and weight are obtained. Concurrent with the aggregate being weighed, liquid bitumen is pumped from a heated storage tank to an asphalt bucket, where it is weighed to achieve the desired quantity in the final mix.

Fig. 2.20: typical batch plant (Source: Amman)



Sometimes, aggregate screening may be by-passed. In this case, aggregates move from the hot elevator to the mixing chamber or to a temporary by-pass silo. RAP being incorporated the mixture is metered by volumetric batching at the hoppers. This practice is mostly used when producing mixes for base courses. For these mixes, an accurate control of grading and bitumen content is perceived to be of secondary importance relative to mixes used for wearing courses. However, by-passing screens reduces their wearing, resulting in savings on maintenance works.

There are several methods to process RAP in traditional batch plants. The most common methods are:

- Addition of RAP into the dryer together with virgin aggregates: RAP is transported from the feeder by a belt conveyor and put into the dryer (see Fig. 2.21)
- Addition of cold RAP into the mixing chamber: cold RAP addition in the mixer is based on weight batching. RAP is transported from the feeder to a weighing belt, fitted at mixer level, by an inclined belt conveyor or by a bucket elevator. A pre-set amount is batched on to the weighing belt and then added swiftly into the mixer (see Fig. 2.22)
- Addition RAP cold RAP before the hot elevator: cold RAP addition to the hot elevator is based on volumetric batching and the RAP fraction is counted as a cold feed fraction added after the drying drum. RAP is heated and dried in the hot elevator.

Furtherly, RAP can be screened or put into the by-pass silo in the tower (see Fig. 2.23)

Fig. 2.21: addition of RAP into drying drum (Source: KVM)

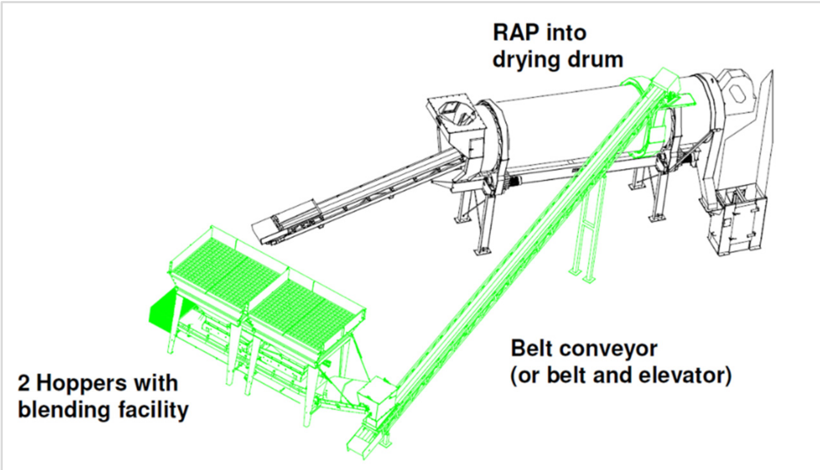


Fig. 2.22: addition of RAP into mixing chamber (Source: KVM)

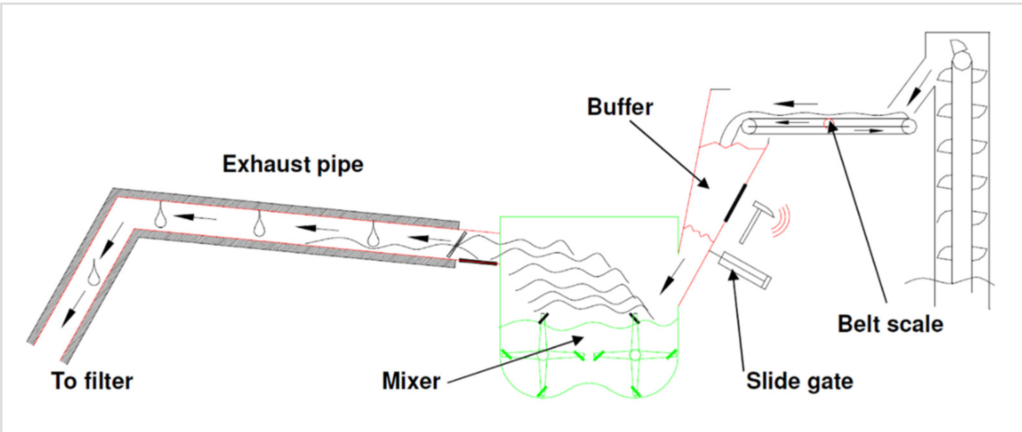
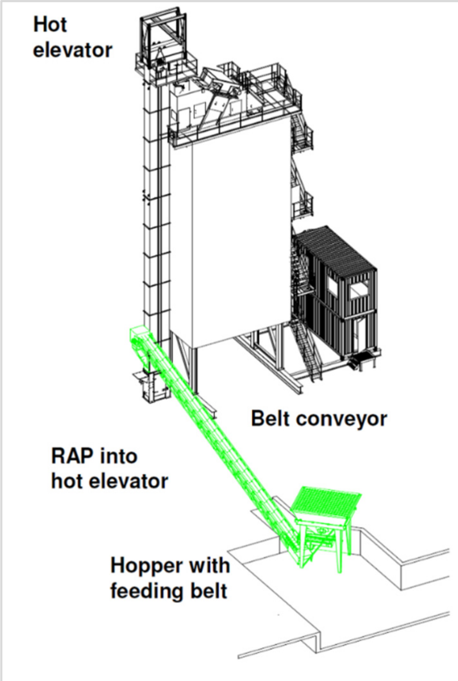


Fig. 2.23: addition of RAP in the hot elevator (Source: KVM)

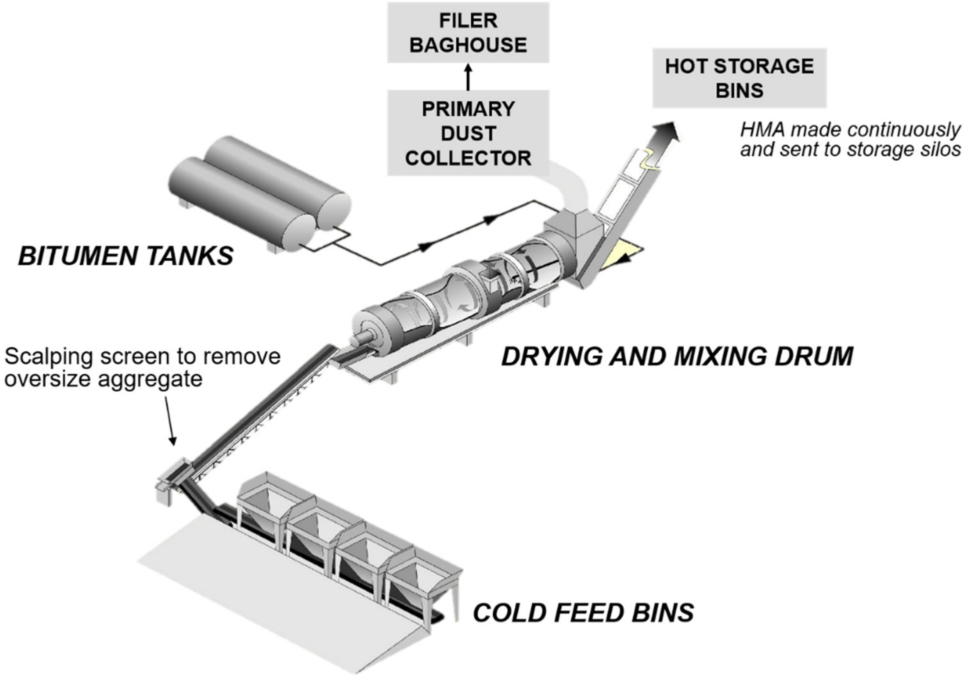


2.9.3 Addition of RAP in traditional drum plants

Drum mixing is a relatively simple process of producing asphalt mixture. The mixing drum from which this type of plant obtains the drum mixing name is very similar in appearance to a batch plant dryer drum. The difference between drum mix plants and batch plants is that in drum mix plants the aggregate is not only dried and heated within the drum, but also mixed with the binder. There are no gradation screens, hot bins, weigh hoppers, or pugmills in a drum mix plant. Aggregate gradation is controlled at the cold feed. As the aggregates (correctly proportioned at the cold feed) are introduced into the drum mix plant for drying, the binder is also introduced into the drum. The rotation of the drum provides the mixing action that thoroughly blends the binder and the aggregates. As the asphalt mixture is discharged from the drum, the mixture is carried to a surge bin and subsequently loaded into trucks.

Controlled gradations of aggregates are deposited in the cold feed bins from which the aggregates are fed in exact proportions onto a cold-feed conveyor. An automatic aggregate weighing system monitors the amount of aggregate flowing into the drum mixer. The weighing system is interlocked with the controls on the binder storage pump, which draws binder from a storage tank and introduces binder into the drum where binder and aggregate are thoroughly blended by the rotating action of the drum. A dust collection system captures excess dust escaping from the drum. From the drum, the asphalt mixture is transported by hot mix conveyor to a surge bin from which the mixture is loaded into trucks and hauled to the paving site. All plant operations are monitored and controlled from instruments in the control van [36].

Fig. 2.24: process and components of traditional drum (continuous) plants



The mixing process is essentially similar in all drum mixing plants; however, there are several plant designs available. These include the parallel-flow drum, as shown in, the counter-flow drum, which has the burner located near the outlet end of the drum, and the unitized counter-flow drum, which has an outer mixing drum that surrounds the dryer drum. The use of counter-flow drum prevents some hydrocarbon emissions and increases fuel efficiency over the parallel flow drum mixer [37].

In a conventional drum plant, a centre entry is used to introduce RAP to the superheated virgin aggregates. The hot virgin aggregates are required to dry, heat and mechanically mix with the RAP. Reclaimed is protected from the burner flame by a veil of aggregate in a parallel flow drum or is introduced behind the flame in counter-flow drum, but there are multiple modifications to these systems. To ensure maximum blending, it is recommended to use early entry RAP collars and long mixing chambers [2].

2.9.4 Factor limiting RAP usage in batch plants

RAP can be added to new mixes using one or more methods presented above. For example, the asphalt plant of Socob Cesena, Italy, which is a traditional batch plant with no specific equipment for recycling, adds half of the total RAP into the drier and half into the mixing chamber. Coupling two adding methods contributes limiting temperature of overheated aggregates and reduces issues related to the presence of RAP into the dryer.

However, traditional methods for adding RAP significantly limit the amount of RAP that can be incorporated into new mixes. Batch plants generally do not allow as high RAP use as drum plants. The typical range of RAP is 10–20% and very rarely RAP content exceeds 40%, although as high as 50% RAP has been used [2].

Limiting factors for increasing RAP content are reported below [38] [5] [2]

a. RAP moisture content
b. Production rate
c. Equipment unfit for RAP recycling

2.9.4.1 RAP moisture content

RAP moisture content plays a significant role in defining the amount of reclaimed asphalt being recycled. Increased moisture content requires RAP to be dried or heated for longer time. The higher the moisture content, the lower the RAP being incorporated in new mixes. The reasons are:

- In traditional (not-retrofitted) plants, cold RAP can be dried by direct contact with superheated aggregates. Temperature of aggregates increases in proportion to mix temperature and RAP moisture content, and cannot be increased above a certain threshold. Also, overheating of the aggregates involves some additional hardening of the new virgin bitumen and to some extent of the old one as well. Therefore, as RAP moisture increases, the amount of RAP being incorporated in new mixes shall be reduced (see Fig. 2.25)
- Mixing time increases when cold RAP is filled into the pugmill because of its moisture content. The higher the moisture content, the higher the mixing time, resulting in decreased production rate

Fig. 2.25: amount of RAP incorporated into asphalt mixture as function of aggregate's temperature (Source: Re-Road, 2012)

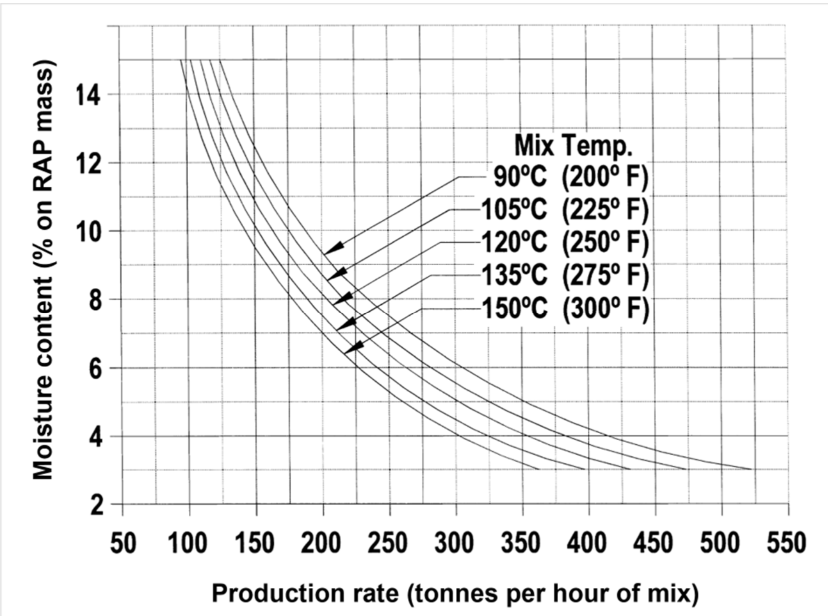


Table 2.4: elevation of drying/heating temperature of aggregates depending on RAP moisture content and RAP being added in the hot asphalt mix

RAP content (% by mass of the mix)	RAP moisture content (% by mass of RAP)					
	1	2	3	4	5	6
	Elevation of temperature (°C)					
10	4	8	12	16	20	24
15	6	12	18	24	30	36
20	8	16	24	32	40	48
25	10	20	30	40	50	60
30	12	24	N/A	N/A	N/A	N/A

2.9.4.2 Production rate

Volume of asphalt production is not a factor that directly impact the amount of RAP being added into new mixes. Nevertheless, production rate changes as consequence of RAP addition.

The production rate of the plant may be reduced if RAP is added, especially if the plant is not retrofitted with specific equipment. Depending on the size of the plant, production rate may lower from 300 tonnes per hour to 250-270 tonnes per hour (10% reduction). Mixing time is increased of 10 seconds when cold RAP is used. In general, the higher the RAP content, the larger the decrease in production rate. Decrease in production rate may vary from 10 to 25% respective to standard production.

Large drops in production rate due to addition of RAP may contribute to make RAP less competitive than virgin aggregate because of the increased production costs.

2.9.4.3 Equipment unfit for RAP recycling

The systems for adding cold RAP into the asphalt mixer may face problems and inaccuracies. Dosing of RAP is done via a belt scale. Since the batch is small the variation of each dosing is large, resulting in large variations of RAP in each batch. Today's high tech asphalt technologies do not leave gaps for inaccuracies of that scale. After the belt scale, each RAP batch will temporarily be stored in the intermediate hopper for RAP. When the RAP must be added into the batch, a bottom gate valve opens and the RAP should via gravity fall down into the mixer. The intermediate hopper often clogs up and manual workforce may be needed to get the clogged RAP to flow into the mixer.

When the cold RAP gets in contact with the hot aggregates and bitumen, a steam explosion occurs. The water trapped in the RAP boils away in seconds and a fast evacuation is needed to lead the thousands of litres of steam away from the mixer in just a few seconds. During this steam explosion bitumen and filler will also be sucked away into the vent piping. During the long travel to the filter the water in the air condenses and transforms into a sludge of water, bitumen and fine filler. Result is that pipe ducting clogs up with sludge and the amount of possible RAP % will decrease accordingly. Ducting must be cleaned manually and regularly.

If RAP is added before the hot elevator, part of its bitumen being in contact with overheated virgin aggregates liquefies, sticking on elevator's shovels. The elevator may get stuck because of bitumen clogging and must be cleaned on regular basis.

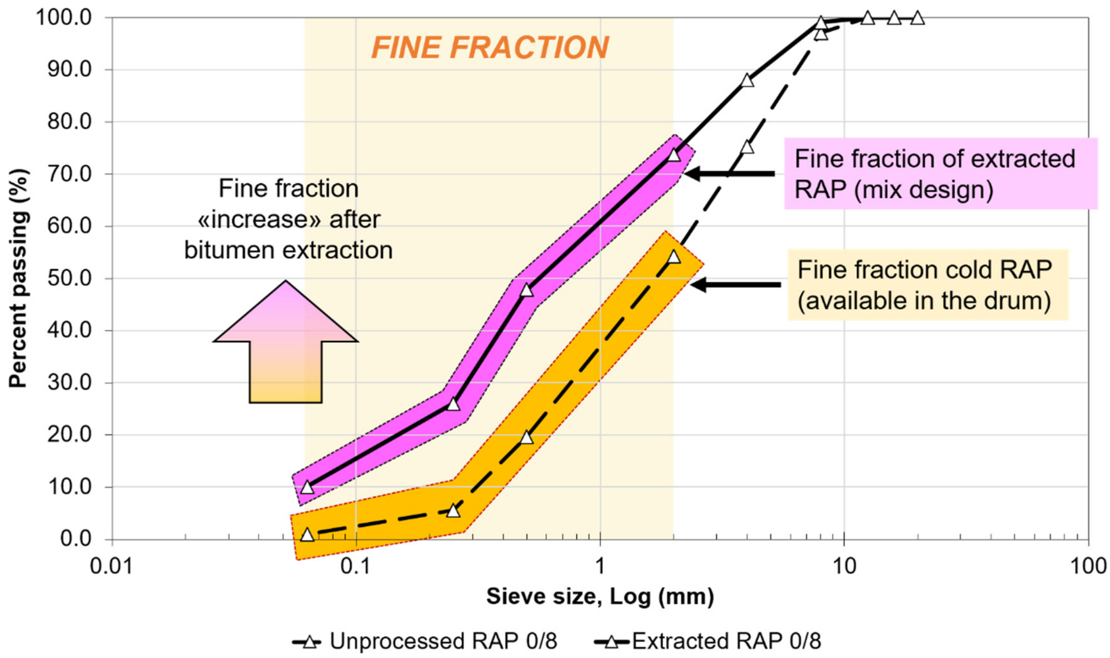
Similarly, part of the liquefied bitumen that is present on warmed reclaimed asphalt shingles may clog screens in the batch tower. A prolonged condition of dirty and plugged screens may lead to strong inaccuracies in screening of aggregates, because of reduces size of screen's mesh and of the presence of sticky material.

RAP may be dried into the drum together with virgin aggregates. Traditional drums are not designed to handle RAP because of the presence of bitumen, which may liquefy at high temperatures (approximately above 130°C). Liquid bitumen contributes getting the dryer dirt in proportion to RAP being dried into the drum. Consequently, dryer shall be periodically cleaned. Furthermore, this issue is even more critical in the case of RAP with modified bitumen. Polymers increase bitumen stickiness, leading to more time consuming cleaning operations.

A more complicated and subtle issue is related to the effects of the sand fraction present in reclaimed asphalt on the production process of RAP mixes. As consequence of milling, RAP contains a significant portion of sand/fines, which have a nominal dimension of aggregates of 0/2 mm. The sand fraction contained in RAP shall be taken into account during the mix design. Generally, RAP mixes require only limited addition of virgin sand to comply with grading requirements, since a large part of the necessary sand/fine fraction may be already present in the reclaimed asphalt. The higher the RAP content, the smaller the amount of virgin sand needed. If we determine the grading curve of cold (unprocessed) RAP, we'll see a lack of fine fractions (0-2 mm), but if we perform the same test on the same material after extraction, we'll see a higher presence of fines. The fine fraction of cold RAP added in the dryer is apparently different from the fine fraction of the extracted RAP. This difference is caused by the presence of bitumen, which causes fines to be lumpy.

Grading curve of extracted RAP must be considered in the mix design and generally leads to limiting the addition of virgin fines in the final mix. Fines are therefore lumpy, and the resulting grading curve has lower fine content relative to the curve of extracted material, as illustrated in Fig. 2.26. The consequence is that less fines are added in the dryer. Unfortunately, lowering the amount of sand and fines into the dryer causes an overheating of the drum. Fine particles shield dryer's inside from heat. As their content drops down, dryer may be subjected to overheating, which may lead to a shutdown of the drum.

Fig. 2.26: variation of fine fraction in the same RAP material



The consequence of this “sand-issue” is a limitation in the amount of RAP being added into the dryer, and hence less RAP is incorporated into new mixes.

As the RAP is added into the drum, there are possibilities for bitumen to be burned by the dryer’s flame. Burning of bitumen increases emissions and air-pollution. The emissions due to the production of high RAP amount are a concern. RAP is commonly introduced in the path of the hot gasses, creating ‘blue smoke’ from volatilisation of RAP binder. Studies show that 30–50% RAP is the limit that the conventional plants can accommodate to comply with the emissions requirements.

It has been suggested [39] that most of the smoke problem is caused by the light oils in soft grade of asphalt binder used to rejuvenate the aged asphalt in the RAP. This means that most of the rejuvenating agents being added to soften RAP bitumen may gone lost when RAP is added into the dryer.

2.9.5 Factors limiting RAP usage in drum plants

In drum plants, RAP is almost entirely processed into the dryer/mixer. Most of the problems related to dryer’s use in batch plants can be found in continuous plants as well. Other issues more closely related to batch plant’s specific equipment as clogging of elevator and steam explosions into the mixing chamber does not clearly involve drum plants.

For the case of continuous plants, factors that limit RAP quantity into new hot asphalt mixes are:

a. RAP moisture content
b. Blue smoke (emissions from drum)
c. Production rate

d. Drum typology

The effects of moisture content have already been discussed for batch plants, and the same considerations are valid for drum plants as well.

2.9.5.1 Blue smoke

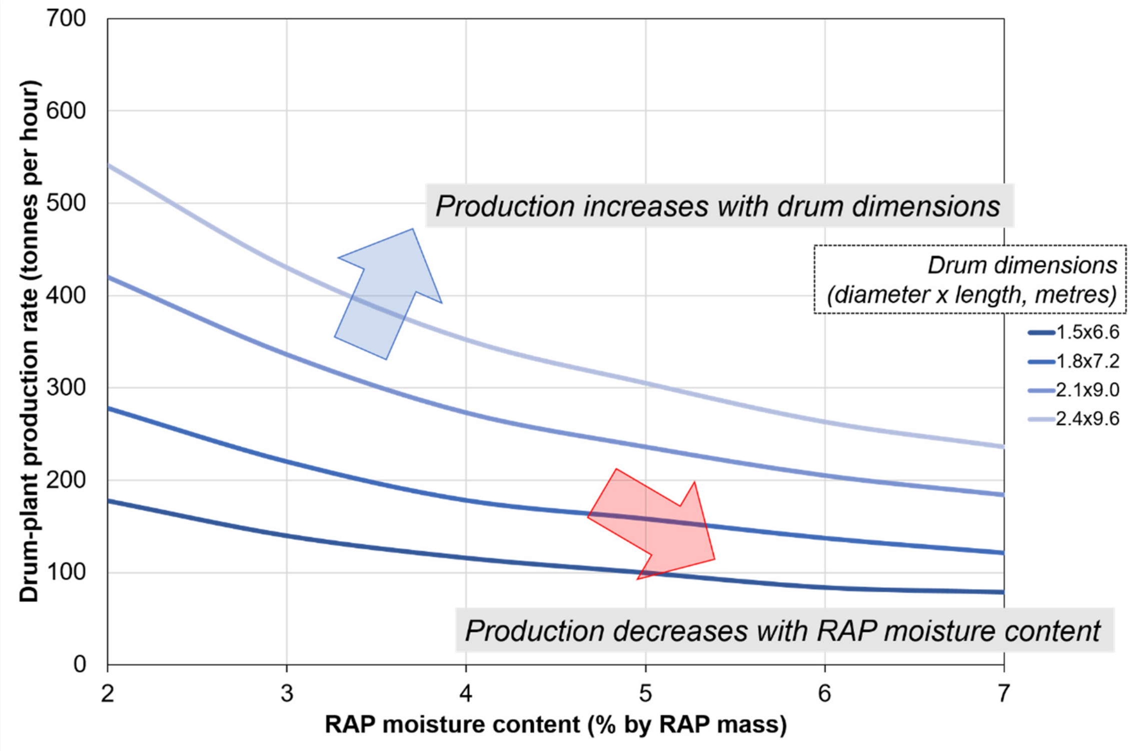
Blue smoke is a visible aerosol emission composed for 94% of Volatile Organic Compounds (VOCs), which are hydrocarbons that readily vaporize at room temperature and pressure, and 6% of Hazardous Air Pollutants (HAPs), fuel dependent wastes suspected or known to be dangerous for humans and environment. Vaporization of lighter fraction of bitumen can cause blue smoke formation during mix production and storage. Chemistry of bitumen, presence of additives as anti-strip products or rejuvenator, dryer/mixer type and addition of reclaimed asphalt contributes producing blue smoke emission. With similar conditions, the choice of thee drying technology i.e. parallel-flow or counter-flow dryer/mixer can limit blue smoke emissions.

2.9.5.2 Production rate

Production rate of RAP mixes in continuous plants is affected by RAP moisture content, discharge temperature (temperature of the produced mixture delivered to trucks) and drum-mixer dimensions. As for batch plants, moisture content plays a significant role in the production volume of the plant: the larger the amount of moisture being dried from RAP; the longer the time required to complete the production process.

Data reported in Fig. 2.27 indicate that at an average moisture content of 5%, a drum-mix plant having a diameter of 1.8 m has a theoretical production capacity of 143 tonnes (158 tons) of mix per hour. If a drum 2.44 m in diameter is used, the manufacturing rate increases to 276 tonnes (305 tons) of mix per hour. For a drum mixer 3.0 m in diameter, the capacity increases to 492 tonnes (541 tons) of mix per hour at 5% moisture removal.

Fig. 2.27: effect of RAP moisture content and drum's dimensions on production rate of continuous plants



As the moisture content in the aggregate decreases from 5% to 3%, the production rate for a drum mixer that is 2.4 m in diameter increases from 276 tonnes per hour to 391 tonnes (430 tons) of mix per hour. If the aggregates have a higher moisture content (for example, 8%), a drum mixer of the same 2.4-m diameter can produce only 191 tonnes (210 tons) of asphalt mix per hour. Thus the average moisture content of the aggregate directly affects the capacity of a drum-mix plant [40].

Most conventional drum plants can routinely accommodate 50% RAP. The maximum amount of RAP that was found reported in conventional drum plant is about 70%, although problems with blue smoke can occur from volatilisation of RAP binder [2].

2.9.5.3 Drum typology

Conventional drum mix asphalt plant technology has been geared to parallel-flow drying and mixing drums. In this configuration the aggregates enter the drum at the burner end and travel downhill to the discharge chute. This is called parallel-flow because the aggregates travel the same direction as the superheated air-stream. About two thirds of the way down the drum the asphalt is injected into the heated aggregate. The problem with this configuration is that the superheated air from the burner impinges on the asphalt oil and burns it, releasing blue smoke. By extending the drying zone and shortening the mixing zone it's possible to reduce the time that the liquid oil is exposed to the superheated air stream. Blue smoke problem is exacerbated when RAP is introduced since heat shall be ran up to compensate for the cold RAP [41].

Most manufacturers are now focusing on how to blend RAP without the associated blue smoke problems of the past. Counterflow dryer/mixers are the dominant theory. In this configuration the aggregates enter the drum at the opposite end from the burner. They travel down the drum in the opposite direction of the superheated air. Nearly every drum designed strictly for aggregate drying has been this configuration since it is the most cost efficient way of drying rock. However, the use of RAP in conventional dyers is associated with blue smoke issues in spite of the dryer/mixer configuration.

2.10 Methods to increase RAP usage in asphalt plants

The typology of asphalt plant and the available equipment play a significant role in RAP recycling. As discussed in previous sections, traditional batch plants can accommodate 10-20% RAP usage, with maximum of 40% in very limited cases. Traditional drum plants can handle up to 50% RAP. For these plants, the biggest issue concerns blue smoke that is generated by burning the aged bitumen.

However, in spite of the typology of asphalt plant, **the most impacting factor on RAP usage is related to heating/drying of reclaimed asphalt in relation to its moisture content**. Traditional and not retrofitted plants must use the same equipment used to heat virgin aggregates. Unfortunately, the equipment has not been designed to process bound materials like RAP, and this generates limitations to RAP usage and issues to the plant's equipment. A summary of equipment related drawbacks is reported in Table 2.5.

Table 2.5: summary of factors limiting the amount of RAP used in asphalt plants to produce new hot mix asphalt

Method to incorporate RAP into production process	Drying process	Plant issues	Consequences on RAP mix
RAP added in the mixing chamber (Batch plants)	Contact with superheated aggregates	• RAP is roughly batched via belt scale • Steam explosions in the mix chamber	• Inaccurate control on properties of final mix • Possible issues during paving due to low temperature of the final mix, especially if high RAP contents are used
RAP added before hot elevator (Batch plants)	Contact with superheated aggregates	• Clogging of hot elevator • Clogging of screens (if screened)	• Inaccurate grading and bitumen content of the final mix
RAP added in the dryer (Batch and drum plants)	Dryer's flame	• Liquid bitumen get the dryer dirt • Dryer's flame may burn bitumen, increasing emission (blue smoke) and	• Dryer's flame may contributes ageing RAP bitumen, resulting in an excessively stiff bituminous blend (neat and RAP binder).

		furtherly ageing bitumen • Rejuvenating agent may be burned in the dryer contributing to blue smoke • "Sand-issue" may cause overheating of dryer with possible shutdown	• Bitumen ageing may become critical if rejuvenating agent are burned

There are several technologies that enable re-use of recycled asphalt for the production of fresh mix including [42]:

a. Parallel drum dryers
   - Parallel counter-flow dryers that theoretically enable use of 100% hot recycled material
   - Parallel parallel-flow dryers that can use up to 60% hot recycled material

b. Double dryers (or double barrel dryers) for the use of up to 90% recycled material

2.10.1 Parallel drum

The parallel drum is an additional dryer specifically designed for RAP drying/heating. It's similar to standard dryers used in traditional batch plants, with some differences due to the optimization for recycling of RAP. It's used in parallel to virgin aggregate's dryer.

The parallel drum is aimed to dry/heat RAP at temperatures between 120-150°C in order to vaporize RAP's moisture and to avoid thermal shocks between RAP and hot aggregates being mixed. When added into the parallel drum, RAP is dried at relatively low temperatures, generally up to 150°C, to avoid a further short-term ageing of bitumen and excessive emissions (blue smoke). The parallel drum is generally placed on top of the batch tower (over the vibrating screens) in order to prevent clogging of the elevator caused by the hot and sticky reclaimed asphalt. RAP is transported from two or more different cold feeders. If the drum was at ground level, RAP should be lifted upwards to vibrating screens via the hot elevator, causing the same clogging that occurs in traditional batch plants. After being dried, RAP is weighed on a belt weighing scale, and generally transported into the mixing chamber, bypassing the screens for keeping them clean.

The parallel drum can be both of parallel-flow or counter-flow types. Using the parallel-flow technology, up to 60% of RAP can be incorporated into new mixes. When using counter-flow technology, the percentage lifts up to 80%.

Fig. 2.28: parallel drum technology (Source: Re-Road)

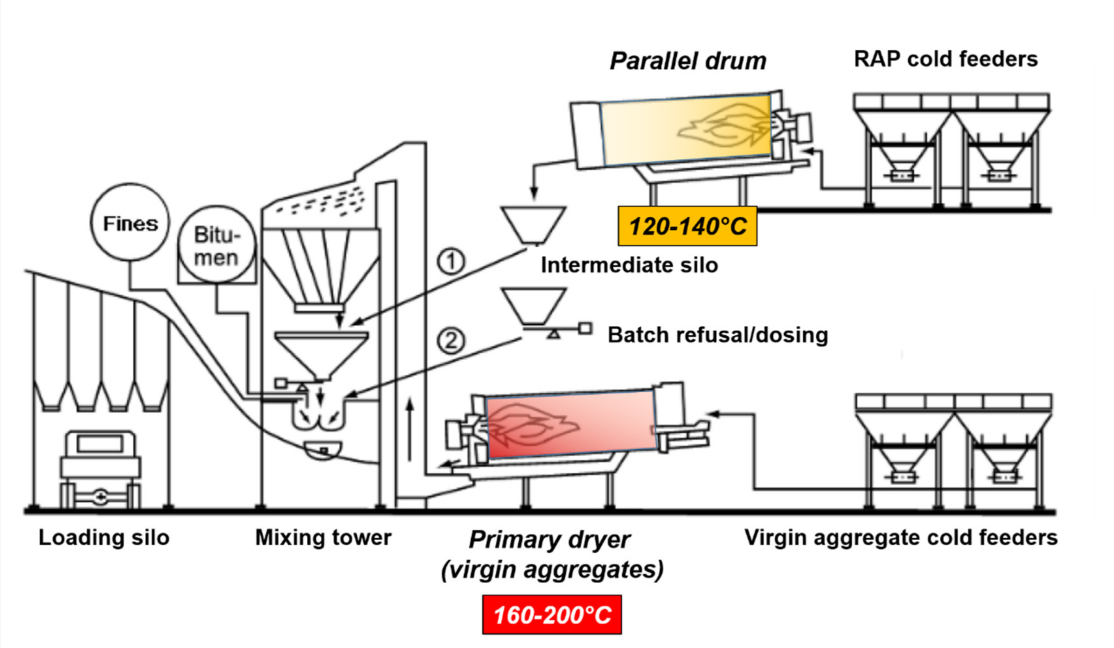
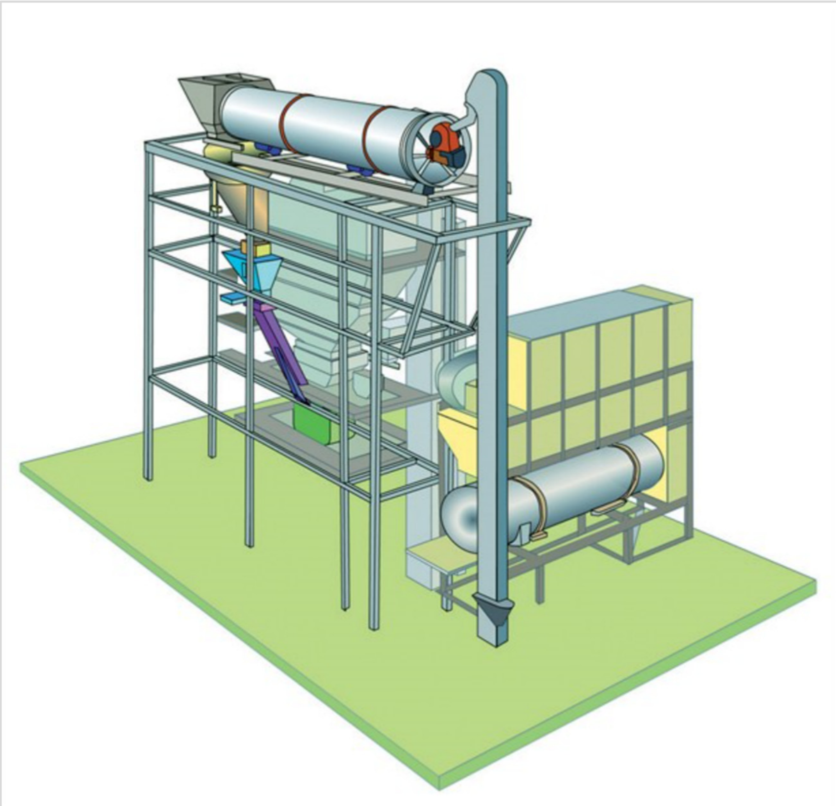


Fig. 2.29: illustration of a batch plant with parallel dryer on top, used to dry RAP at 140-160°C



As already discussed, the recycling capacity depends on moisture content of RAP. The higher the moisture content, the lower the recycling capacity. Considering the Ammann RT25140 parallel-flow drier, the recycling capacity decreases from 180 t/h at 3% moisture to 150 t/h at 5% moisture. The counter-flow drier RT29120 drops from 240 t/h at 3% to 200 t/h at 5%.

Parallel drums are generally used to produce high volumes of asphalt mixes. Despite the recycling capacity of counter-flow dryers is up to 80%, the recycling rate of batch plant with parallel dryer is generally 60%. This is because a minimum amount of virgin aggregates must be added to the primary dyer to prevent superheating of the drum. For example, producing mixes with 80% RAP requires a too low quantity of virgin aggregates being added into the primary dryer, which will cause the production to stop.

The best use of parallel drum is to produce RAP mixes for base courses, for two reasons: first, base courses are generally allowed to incorporate higher RAP contents than surface courses. This aspect depends on guidelines included in technical documents of a specific project. Second, base courses require more asphalt than surface or binder courses, therefore volumes of production are larger compared to other asphalt bound pavement layers.

2.10.2 Double RAP dryer

The Double-dryer, also noted as double-barrel, consists of an ordinary counter-flow drum surrounded by a fixed outer drum where RAP is added. The inner and the outer chambers are separated, thus RAP is never exposed to the burner's flame. Double dryers are used in continuous plants.

Virgin aggregates are dried into the inner chamber. The inner drum can be divided in different portions corresponding to different phases of the drying process. When the aggregate first enters the drum, conditioning flights break up any clumps or sticky material. Then, special flights installed in the second portion of the drum form a uniform veil of aggregate through the gas stream, in order to maximize the efficiency of the drying process. In the last portion closer to the flame, combustion flights prevent aggregate from impinging on the flame while spreading the material to maximize radiant heat transfer (see Fig. 2.30).

Reclaimed asphalt is dried into the external drum, which is separate from the inner chamber. This prevents excessive heating of RAP bitumen due to direct contact with the flame, avoiding related short-term ageing and blue smoke emissions. When added to the external drum, RAP moves in opposite direction. The outer chamber is heated by convection, using the heat generated by the flame in the inner chamber. The external drum is provided with a system to vent steam that generates from RAP drying. Dried RAP and heated aggregate are discharged into a pugmill mixer with liquid bitumen (see Fig. 2.31).

Drying RAP without a direct exposition the flame significantly limit the need to superheat virgin aggregates. The higher the RAP temperature (but below certain limits i.e. 140°C as previously discussed), the lower the superheating temperature.

Most conventional counter-flow drum plants can routinely accommodate 50% RAP. However, typical issues related to RAP usage as blue smoke and bitumen ageing can be avoided using a double drum dryer. Double dryers can increase the RAP amount up to 80-90% without the need to excessively superheat virgin aggregates or expose RAP to direct flame [2].

Fig. 2.30: double dryer inner chamber used to heat virgin aggregates (Source: Astec)

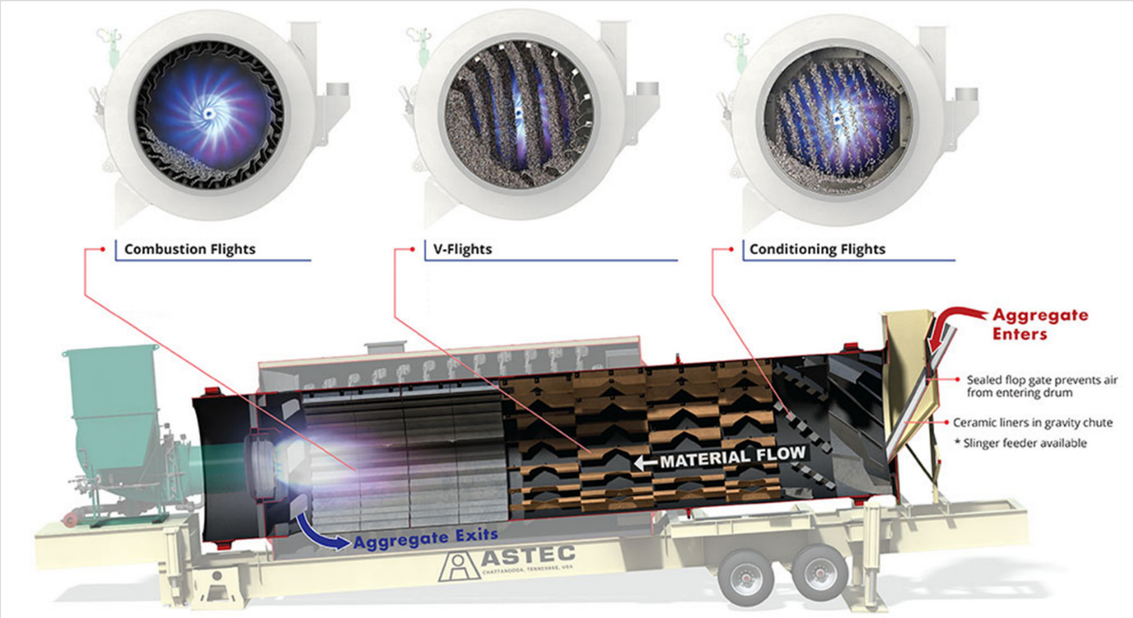
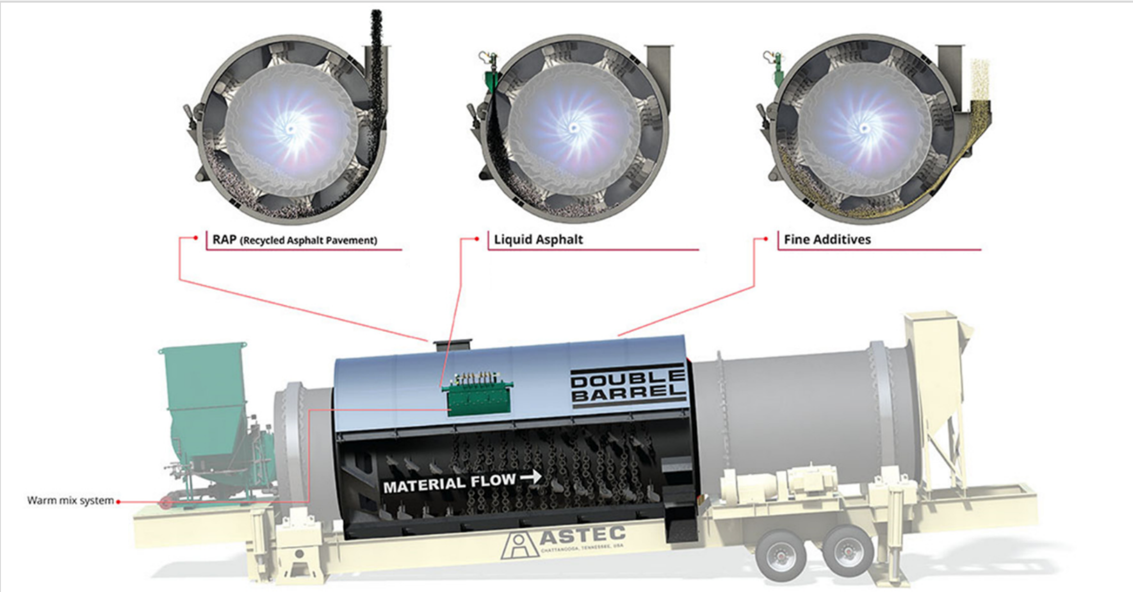


Fig. 2.31: double dryer external chamber used to dry RAP (Source: Astec)



2.10.3 Feeder system

A conventional cold feed system can be used to supply crushed RAP to the drum mix plant. However, to allow easy discharge and avoid stacking problems, the bin should have relatively low capacity, with steep sides and long and wide bottom. The RAP material should not be supplied to the bin as a unit drop as it can cause compaction of the RAP with resultant bridging, sticking and discharge problems. Instead, the material should be dribbled as much as possible. Also, the bin should not be vibrated as this may lead to RAP compacting. Both belt and slat type feeders have been used successfully. On warm days RAP should not be left in the bin for more than two hours in case of plant shutdown. It is better to keep the bin half full and feed frequently. Feeders should be fairly wide and should have sufficient horsepower to be used in a start-stop operation as necessary. Vibratory type feeders are not recommended to avoid consolidation and sticking problems.

Table 2.6: modifications to increase recycling capacity of asphalt plants

Modification to asphalt plant	Purpose	Recycling capacity	Best uses
Parallel drum (batch plants)	• Dry/heat RAP before mixing with virgin aggregates using a drum operating at lower temperatures (120-140°C) than the primary dryer • Limit aggregate superheating	• 60% with parallel-flow drum • 80% with counter-flow drum • Recycling capacity is limited to 60% to prevent superheating of the primary dryer caused by insufficient amount of virgin aggregates	• High production volumes • RAP mixes for base courses
Double dryer or double barrel (counter-flow)	• Dry RAP into a separate chamber avoiding direct contact with the burner’s flame • Limit aggregate superheating	• Up to 80-90%	• Same as traditional drum plants

2.11 RAP recycling case history

Due to different step of technological development and public awareness, asphalt recycling rate in Europe vary from 90% to less than 10%. In the present section, some cases of successful retrofitting of existing asphalt plants are presented.

2.11.1.1 Bunte batch plant with parallel dryer, Germany [43]

One example comes from Germany where local contractor Johann Bunte, a road specialist with three asphalt plants, aims to optimise production of hot asphalt and to reduce the consumption of energy: a "green approach for black asphalt." To meet this target, the rates of RAP in the mixture must be dramatically increased. The production process needs some optimisation and the electricity consumption of all major drives must be minimised using frequency converters.

The existing 2005 batch mix plant was retrofitted two years ago by an ATS/Marini (Fayat Group) recycling ring on the existing dryer to increase recycling rates. The RAP cold feeding into the mixer was insufficient with only 20%, but by retrofitting the recycling ring the feed rates could be increased up to 40% and more. However, for big production runs and higher RAP rates it was necessary to install a powerful parallel dryer with burner for lignite dust and gasoil. The concept developed with Bunte proved itself during the first major productions with daily production of above 2,500 tonnes and a recycling rate far above 60%.

Fig. 2.32: parallel dryer installed on top of the Bunt batch plant

The project required a parallel flow dryer drum for RAP with a nominal output of 180 tonnes/hr with variable rotation speed to achieve a temperature of the RAP material at the dryer discharge of between 140-160°C. The exhaust fumes temperature could not be higher than 10-15 °C above product temperature, and to meet this target the drum was sized with a 2.5m diameter and a length of 14m. The drum and drives were designed by Marini and ATS in close cooperation. A 32m high heavy steel structure included the elevated mixer with a buffer hopper of 26 tonnes, a separate weighing hopper of 3 tonnes, and a highly inclined chute into the mixer. The hoppers for the hot RAP are insulated and electrically heated, and the chute into the mixture is highly inclined with special steel surfaces and is heated and insulated to avoid any sticking.

The heat for drying the RAP is generated by a double fuel burner for coal dust/oil. The application of these burners in recycling parallel dryers is a new technology and needs special care when designing the combustion area. The problem is to transfer the heat into the sensitive RAP material by avoiding overheating and thus damaging the bitumen. RAP should never become too sticky, otherwise all the lifters will be blocked. Special flights were designed and installed. So the drum performed in reality better than expected.

For the parallel operation of the two dryer drums (virgin aggregates and RAP) the regulation of low-pressure at the burner front wall is quite complicated and needs two servo-controlled dampers for both fumes ducts and a constant under-pressure in the raw gas duct ahead of the bag filter. The existing exhauster is speed controlled with a new frequency converter to keep this constant low-pressure in the main duct. The reduced noise is an additional benefit.

The newly developed ATS/Marini burner control system receives additional input by belt weighing scales indicating material flow into the drum. It calculates any change in material flow for changing the burner capacity in time for a constant product temperature. This offers additional savings in energy and improvements of the asphalt quality.
Erection and commissioning took place within two months without major interruptions to production.

2.11.1.2 Recycling techniques in Europe [31]

In Netherlands, of 41 asphalt plants, 35 are batch type. The majority of available RAP is used in hot in plant applications, whereas in situ recycling is not commonly performed. Separate heating of aggregates and RAP is the preferred method to maximize the amount of reclaimed asphalt a plant can accommodate. Asphalt plants are retrofitted with parallel drum or double drier. RAP is generally stored under roofed areas, in order to control its moisture content and to limit all related issues. Furthermore, RAP is stored in different stockpiles on the basis of RAP source and nominal diameter size. This is particularly useful when reclaimed asphalt contains modified bitumen. The amount of RAP added to hot asphalt mixes varies with asphalt type: 45-50% RAP for base courses and 25-30% for wearing courses. The production rate generally decreases of 10% when using RAP, whereas fuel consumption may increase of 10-15% depending on RAP quantity.

In Belgium, the majority of asphalt plants are equipped to perform hot-in-plant recycling with separate heating of aggregates and reclaimed asphalt using or parallel drum or double drum technologies.

In Denmark and Germany, asphalt plants were often equipped with parallel drum for separate heating of RAP, but nowadays the parallel drum has been dismantled in favour of the use of double dryer. The greater amount of available RAP is used in hot/warm asphalt applications (see 2.7).

In Portugal, the A1 motorway has been restored using asphalt mixtures made with reclaimed asphalt. The material milled from existing surface and binder courses was first stored and accurately characterized. Grading curves of RAP before and after extraction were determined. Bitumen content was determined together with penetration and softening point of recovered binder. Characterization of RAP provides for clear and precise information on material properties, and it’s a fundamental tool to check homogeneity of reclaimed asphalt that shall always be considered as part of the production process. RAP mixes were produced in a continuous counter-flow plant with production rate of 320 t/h. RAP was introduced half way through the drum mixer, into an external ring, in percentage of 25% by mass of virgin aggregates. The external ring prevented a direct contact with burner’s flame, limiting short-term ageing and blue smoke emission. A soft neat bitumen 70/100 pen grade was used to compensate for the stiffer RAP bitumen, which has showed a 10/20 pen grade from characterization tests. The resulting blend is a 35/50 pen grade bitumen.

In Sweden, most of the available RAP is used in hot/warm applications (see 2.7). In 1998, the amount of RAP incorporated into new mixes was 15-50% depending on the asphalt plant type. Traditional batch plants can accommodate up to 20% RAP. The percentage increases to 60% using a parallel drum (black drum) to separately dry RAP. Drum plants can handle up to 50% RAP. As for batch plants, the amount of RAP that can be processed increases as consequence of plant retrofitting, which can rise RAP usage up to 80%.

2.12 Annexes

Table 2.7: production of hot and warm mix asphalt in Europe, period 2008-2014

Country	Total production of hot and warm mix asphalt in 2008-2014 (mill. tonnes)						
	2008	*2009*	*2010*	*2011*	*2012*	*2013*	*2014*
Austria	9.5	9	8.2	8	7.2	7	7.2
Belgium	4.9	4.7	4.8	5.9	5.6	5.3	5.2
Croatia	4.2	3.2	2.7	2.6	2.5	2.8	2.3
Czech Rep.	7.3	7	6.2	5.8	5.6	5.4	6.4
Denmark	3.1	2.7	3.2	4	3.6	3.7	3.7
Estonia	1.5	1.2	1.1	1.3	1.1	1.2	1.3
Finland	6	5.2	4.9	5	4.5	4.5	4.7
France	41.8	40.1	38.8	39.2	35.3	35.4	31.9
Germany	51	55	45	50	41	41	39
Greece	8.1	8.7	5.2	2.3	1.6	2.7	2.7
Hungary	2.5	1.6	3.4	2.3	2.5	2.7	3.8
Iceland	0.4	0.3	0.2	0.2	0.2	0.2	0.3
Ireland	2.8	3.3	2.3	1.8	1.9	1.8	1.8
Italy	36.5	34.9	29	28	23.2	22.3	22.3
Latvia	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Lithuania	2.2	1.5	1.6	1.6	1.3	1.3	1.3
Luxembourg	0.6	0.6	0.7	0.65	0.61	0.7	0.7
Netherlands	9.3	9.8	9.5	9.6	9.2	9.7	9
Norway	5.7	6.5	5.9	6.7	6.3	6.4	7
Poland	15	18	18	26.5	21.1	18.2	18.2
Portugal	9	9	6.7	6.4	6.4	6.4	6.4
Romania	3.3	3.6	3.2	3.6	3.2	4.1	4.5
Serbia	1.3	No data	No data	No data	No data	No data	No data
Slovakia	2.2	2.2	1.9	2.2	1.9	1.6	1.5
Slovenia	2.6	2.3	1.8	1.3	1.1	1.2	1.4
Spain	42.3	39	34.4	29.3	19.5	13.2	14.5
Sweden	8.7	8.1	7.9	8.1	7.7	7.6	8.5
Switzerland	5.3	5.4	5.3	5.4	4.8	4.8	6.5
Turkey	26.6	23.1	35.3	43.5	38.4	46.2	30.9
UK	25	20.5	21.5	22.4	18.5	19.2	20.6

Table 2.8: production of HMA and WMA in Europe and United States from 2008 to 2014

Area	Total production of hot and warm mix asphalt in 2008-2014 (mill. tonnes)						
	2008	*2009*	*2010*	*2011*	*2012*	*2013*	*2014*
Europe (28)	338	326.9	309.3	324.3	276.4	277.3	265.4
USA	440	324	326	332	326.9	318.1	319

Table 2.9: paving grade bitumen consumption in EU-27 area, 2014 **[27]**

Country	Bitumen consumption in Europe (tonnes)			
	Total bitumen consumption	*Harder paving grade*	*Softer paving grade*	*Normal paving grade*
Austria	259	2	216	217
Belgium, Luxembourg	357	56	239	295
Finland, Denmark, Iceland, Estonia, Latvia, Lithuania	578	64	473	537
Norway	413	0	381	381
Sweden	492	0	432	432
France	2,332	1,054	1,181	2,235
Germany	2,019	69	1,192	1,261
Hungary, Slovenia, Slovakia	83	0	60	60
Czech Republic	268	1	237	238
Ireland	177	20	157	177
Italy	624	40	445	485
Netherlands	317	43	235	278
Poland	677	134	399	533
Romania	101	0	95	95
Bulgaria, Malta, Greece, Cyprus, Portugal, Rest of Europe	240	55	104	159
Spain	690	166	494	661
Switzerland	234	22	177	199
United Kingdom	1,531	78	1,224	1,302

3 Review of laboratory testing on asphalt mixtures

An accurate laboratory characterization is necessary (but not sufficient) to attain consistent and representative results. The present chapter reviews standard and advanced laboratory testing procedures used to investigate properties of asphalt mixtures. The procedures, presented in accordance with the European standard references, are analysed in detail in every aspect: testing equipment, best practices, expected results and interpretation of test data are reviewed for each test type.

3.1 Particle size distribution

The test consists of dividing and separating a material into several particle size classifications of decreasing sizes by means of a series of sieves. The mass of the particles retained on the various sieves is related to the initial mass of the material. The cumulative percentages passing each sieve are reported in numerical and graphical form [44].

When asphalt mixtures are investigated, the granulometric analysis of the aggregates is performed after the binder extraction as indicated in the EN 12697-2. After being extracted, the aggregate is first weighed and then washed in order to remove the finest particles (maximum dimension smaller than 0.063 mm), then is dried to constant mass by heating at 110±5°C. The sample rests at room temperature and the weight is determined. The series of sieves is prepared: the sieve column comprises a number of sieves fitted together and arranged, from top to bottom, in order of decreasing aperture sizes with the pan and lid. The sieve size generally ranges from 40 mm to 63 µm. Details are reported on the European standard EN 13043 [45]. The aggregate is poured into the sieve stack which is closed and mechanically shaken for at least two minutes. No particles shall be lost during this phase. The retained material is weighed for each sieve starting from the upper sieve to the ban at bottom of the column. For each sieve, the percent of material passing the correspondent sieve is calculated with the following equation:

$$p_i(\%) = 100 - \left(100 \cdot \frac{\sum_i R_i}{M_1}\right)$$

Where p_i (%) is the percent of material passing the sieve “i”, R_i is the retained material (in grams) for the sieve “i” and M_1 is the dried mass of the sample. An example of calculation of percent “p” is illustrated in Fig. 3.1.

The size of aggregate is identified in terms of lower (d) and upper (D) sieve size as d/D, where “d” and “D” are respectively the lower and the upper sieve size of the aggregate in the mixture, in millimetres (mm). The parameter “D” refers to the sieve size and not to the particle size, thus it shall not be intended as the maximum dimension of the aggregate.

The percent of material passing each sieve size is plotted versus the logarithm of sieve size in millimetres. The resulting curve is noted as *grading curve* of the material. The grading curve is of primary importance in road engineering because it gives information on the internal structure of aggregate which is the skeleton of the asphalt mixture. Gradation of aggregates influences properties of the asphalt mixture as optimal bitumen content, air voids, maximum density, compaction and resistance to fatigue and rutting.

Composite aggregates can fall into one of the following types of gradation of particle size distribution [38]:

- *Well-graded* or *dense-graded*: structure has an extensive distribution of particle sizes ranging from fine to coarse. The smaller aggregate particles fill the in gaps and voids between larger particles. Permeability of the aggregate structure is low and interlock between aggregate particles is high. Asphalt mixtures with such aggregate structure may be easily compacted but tend to tightly pack
- *Gap-graded*: gradation characteristic in which there is a gap in the particle size distribution. There is a range of particle size fractions (typically the mid-size range) that is either devoid of particles or the amount of particles in that range is low. The amount of voids between aggregate particles is higher than that of the well-graded aggregates. Asphalt mixtures have the greater air voids content so that the mix can contain an adequate amount and coating thickness of bitumen
- *Uniformly-graded*: aggregate structure consists of particles that are primarily of one size or their range of sizes is very narrow

Fig. 3.1: sieving of aggregate sample and calculation of percent of material passing a specified sieve size

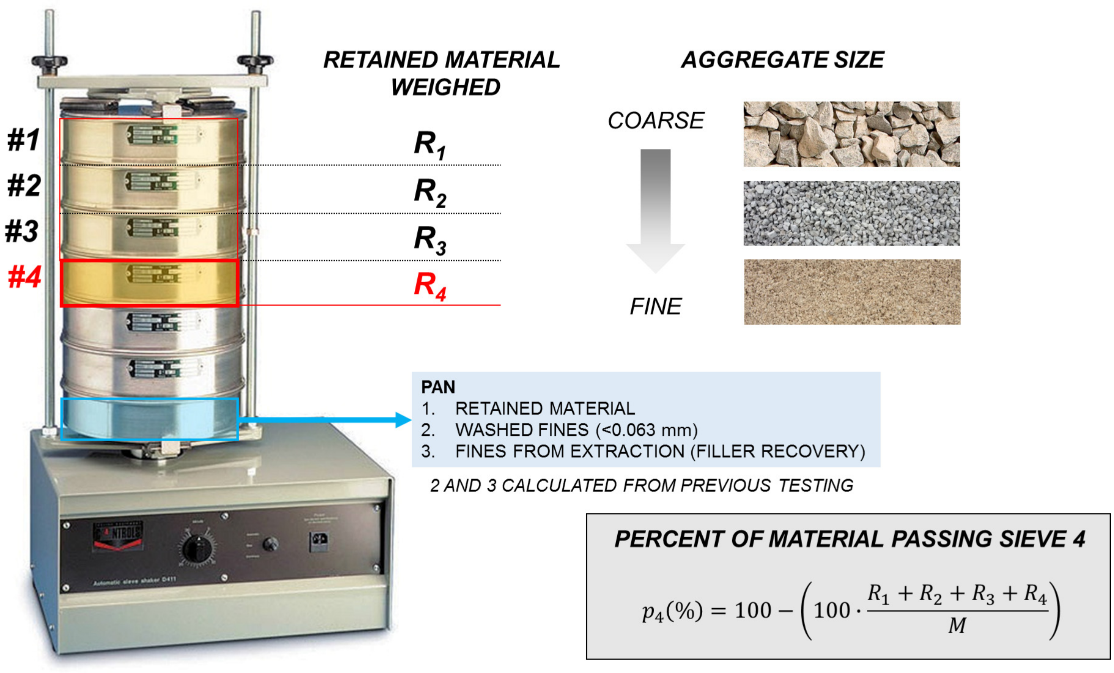
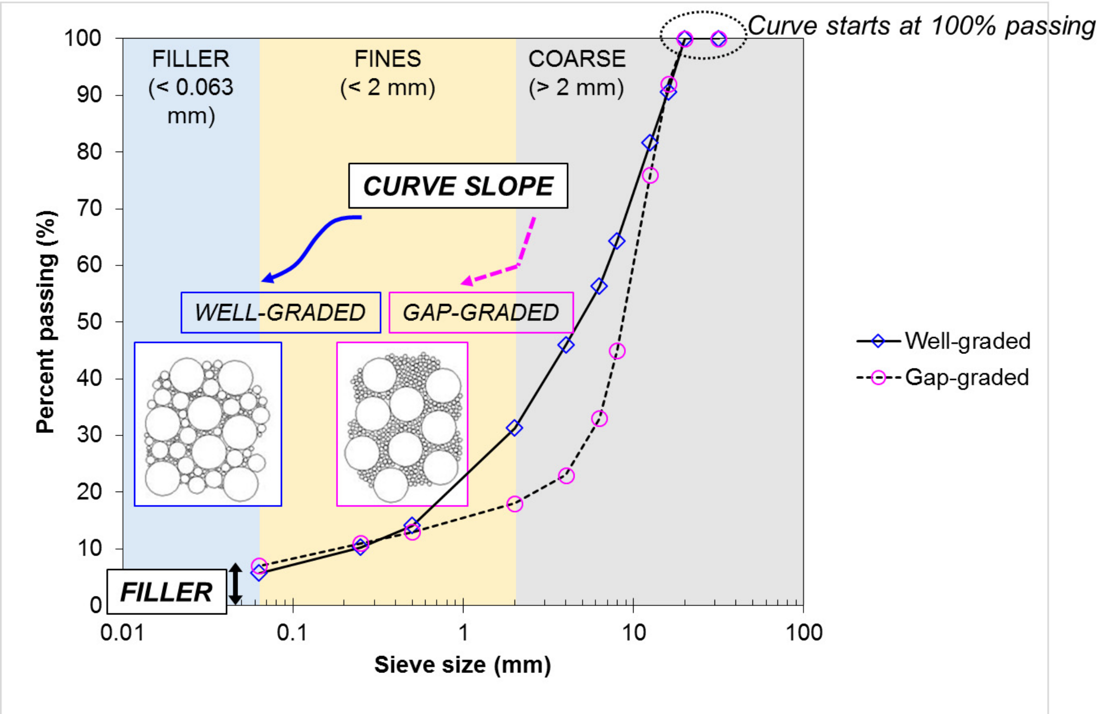


Fig. 3.2: example of grading curve for well-graded and gap-graded materials with interpretation of the curve's shape



When this test is carried out after a binder extraction procedure in which a sieving stage is incorporated containing the applicable sieves, and when the aggregate is thoroughly washed during the execution of test EN 12697-1, or when the aggregate remains from EN 12697-39, the particle size distribution may be determined by dry sieving only.

If the proportion of material passing the 0.063 mm sieve remaining with the aggregate is found to be greater than 1.0 % of the total aggregate, the particle size distribution shall be re-determined after washing the aggregate.

If one size portion contains visually different aggregate types, this variation should be reported and the type aggregate shall be described. It's important to note that the assumption of similar density of the aggregates in the asphalt mixture underlies the accuracy of the current procedure for calculating the percent of material passing each sieve. The particle size distribution gives information on the volumetric distribution of aggregates into a reference volume, defining the amount of coarse and fine fractions into the blend. If materials with similar density are blended, the mass can be used to determine the volumetric distribution of aggregate particles, because all aggregate classes into the blend have the same density. When aggregates with significantly different density are blended together, the similarity between mass and volume may not be valid, leading to erroneous calculation of the grading curve. In this case, the grading curve shall be determined with volumetric calculation only.

3.2 Density

The density of a compacted asphalt or aggregate specimen is defined as its unit mass or the mass of a specific volume of mix [38]. Density is closely related to volumetric characteristics and performances of asphalt mixtures, and it's considered to be one of the most important properties of mixes. Low density are often associated with poor quality aggregates that may lead to pavement performance problems.

Density is calculated as the ratio of the sample mass (m) and a reference volume (V) with the following equation:

$$\rho = \frac{m}{V}\ [Mg/m^3\ \ or\ \ Kg/m^3]$$

The European standard EN 12697-6 expresses density in Mg/m^3. However, density is commonly reported in Kg/m^3 accordingly to units used by operators of production plants.

There are different measures of density, depending on the reference volume that is measured in laboratory and used in the previous formula. Density can be classified in the following types:

- Bulk density of compacted asphalt mixture/aggregates
- Maximum density of asphalt mixture
- Apparent density of aggregates

3.2.1 Bulk density

The bulk density ρ_b or G_{mb} of an intact compacted bituminous specimen is determined from the mass of the specimen and its volume [46]. The mass of the specimen is obtained by weighing the dry specimen in air.

The bulk density of compacted bituminous samples is determined according to the European standard EN 12697-6. Four procedures are presented:

a. **Dry**: the dry sample is weighed (m_1), then the weight of the sample submerged in water is measured (m_2). The temperature of the water is recorded and its density ρ_w is calculated. The bulk density dry is calculated with the following formula:

$$\rho_{bdry} = \frac{m_1}{m_1 - m_2} \cdot \rho_w\ [Mg/m^3\ \ or\ \ Kg/m^3]$$

b. **Saturated surface dry (SSD)**: the mass of dry specimen (m_1) and water density ρ_w are measured. The specimen is submerged in water for at least 30 minutes, allowing the water to saturate the sample. The saturated sample submerged in water is weighed (m_2), then the mass of the saturated specimen is measured (m_3). The bulk density SSD is calculated with the following formula:

$$\rho_{bSSD} = \frac{m_1}{m_3 - m_2} \cdot \rho_w\ [Mg/m^3\ \ or\ \ Kg/m^3]$$

c. **Sealed specimen**: the mass of dry specimen (m_1) and water density ρ_w are measured. The specimen is sealed in order to make it impermeable to water and weighed (m_2) after being sealed. The density of the sealant material is ρ_{sm}. The sealed specimen is submerged in water and weighed under water (m_3). The bulk density sealed specimen is determined with the formula:

$$\rho_{bsea} = \frac{m_1}{\left(\frac{m_3 - m_3}{\rho_w}\right) - \left(\frac{m_2 - m_1}{\rho_{sm}}\right)} \; [Mg/m^3 \;\; or \;\; Kg/m^3]$$

d. **By dimension:** the mass the dry sample is measured (m) and the dimensions of the specimen are determined according to EN 12697-29. For cylindrical samples with height "h" and diameter "d", the geometrical density of the specimen is determined with the following formula:

$$\rho_{bdim} = \frac{m}{\pi \frac{d^2}{4} h} \; [Mg/m^3 \;\; or \;\; Kg/m^3]$$

For the first three procedures, the volume of the specimen is obtained from its mass in air and its mass water. In the dry procedure, the mass in water is determined without pre-treatment. In the SSD-procedure, the specimen is first saturated with water, after which its surface is blotted dry with a damp Chamois. In the sealed specimen procedure, the specimen is sealed before immersion in water to prevent access of water to the voids in the specimen. In the fourth procedure, by dimensions, the volume of the specimen is obtained by measurement of the dimensions.

The four methods listed before are used with the aim to incorporate different void contents depending on the type of material being tested. The main issue in selecting a procedure for the determination of the bulk density of bituminous materials is whether the voids in the specimen surface are taken into account as a part of the specimen volume sufficiently precisely [46]. The ideal procedure takes into account exactly those voids that are part of the volumetric material composition, but neglects those voids that occur as specimen irregularities due to the specimen preparation method.

In other words, superficial texture of asphalt samples (and hence accessibility of internal voids) is the key factor for determining the best method to measure bulk density. The higher the void content, the higher the accessibility of water or sealant, the less accurate the methods from A to C.

Procedure A is suitable for measuring the bulk density of very dense, practically non-absorptive bituminous specimens. It is a quick, easy method and is particularly convenient for many dense laboratory prepared specimens.

Procedure B is suitable for measuring the bulk density of dense-graded bituminous specimens having a low water absorption level or a slow drainage of absorbed water.

Procedure C is generally used for measuring the bulk density of bituminous cores sampled on site. The paraffin wax is commonly used as sealant. The paraffin is heated until it becomes liquid, then the core is submerged and sealed. The sealing shall be done with care in order to create a thin and uniform film of paraffin on the sample. The asphalt specimen shall be submerged only for few seconds otherwise it will crumble because of the high temperature of paraffin.

Procedure D is suitable for measuring the bulk density of bituminous specimens whatever the voids content may be. Specimens should have a regular surface and a geometric shape to facilitate the measurement of their dimensions. Procedure D described in this European Standard is suitable for void contents greater than 15 %. For these materials, the other procedures would not give accurate results.

A summary of the previous considerations is given in Table 3.1.

Table 3.1: choice of the bulk density method on the basis of the investigated material

Bulk density procedure (EN 12697-6)	**Suitable materials**
A – Dry	• Dense-graded asphalt produced in laboratory, air voids < 8% • Gap-graded asphalt, air voids < 5% (SMA)
B – Saturated Surface Dry (SSD)	• Dense-graded asphalt produced in laboratory, air voids 8-15%
C – Sealed specimen	• Asphalt cores, air voids < 15% • Same as Dry and SSD. In this case, procedure A or B are preferred because quicker and with same accuracy
D – by dimensions	• Open-graded asphalt produced in laboratory, air voids >15% • Asphalt cores, air voids > 15% • All materials that cannot be submerged in water

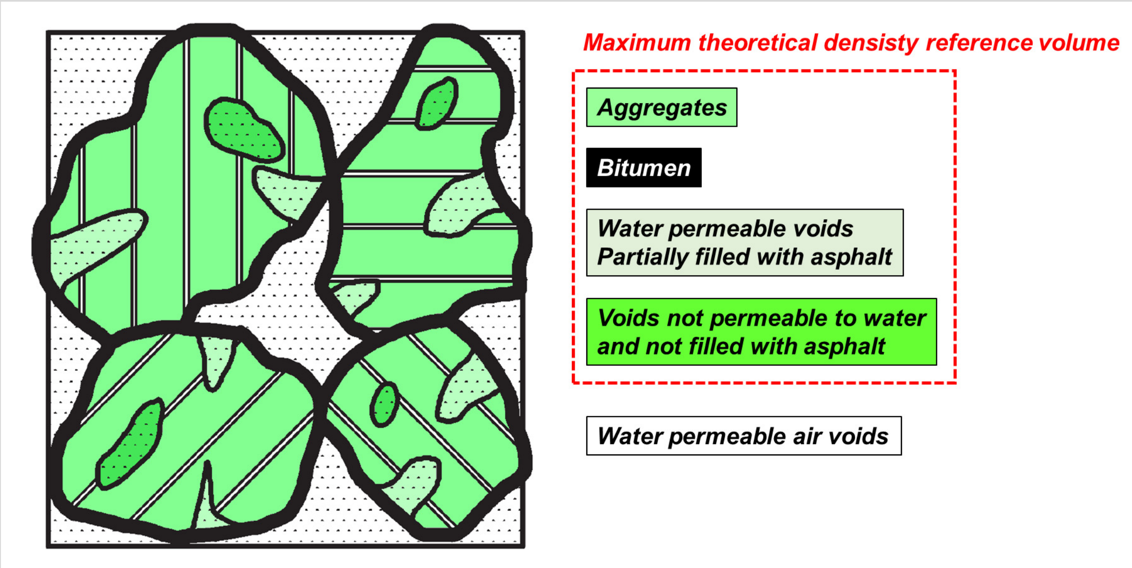
3.2.2 Maximum theoretical density

The maximum theoretical density ρ_m or G_{mm} of an asphalt mixture is the ratio of the mass in air of a unit of volume of the bitumen and aggregate in the mixture at a stated temperature to the mass in air of equal density of an equal volume of gas-free distilled water at a stated temperature. In other words, ρ_m is the mass of bitumen and aggregate mixture divided by the volume of the asphalt coated particles.

ρ_m measures the density of the asphalt mixture at zero voids. The density is noted as theoretical since it’s impossible to produce an asphalt mixture with zero voids.

The reference volume comprises aggregates, bitumen, air voids not accessible to water and air voids accessible to water filled (partially) with bitumen. Air voids accessible to water are not comprised. A representation of the reference volume is given in Fig. 3.3.

Fig. 3.3: reference volume of maximum theoretical density of aggregates



The volumetric procedure reported in the European Standard EN 12697-5, *Procedure A*, is the most commonly adopted in testing material laboratories and it will be presented. All the procedures aim to measure the reference volume of the mixture, as described before.

In the volumetric procedure, gas-free distilled water is poured into a pycnometer (comprising the cap) up to a reference mark. The pycnometer filled with water is weighed in order to determine its volume V_p expressed in m^3. This first procedure, which is often called calibration, can be done once for a specific pycnometer. The empty and dried pycnometer is weighed (m_1). A sample of loose asphalt is weighed (m_2) and poured into the pycnometer.

The asphalt shall be at low temperature and not clumped. In order to that, asphalt is taken out from the oven and spread onto a sheet of filter paper, trying to separate the single aggregates by hand. This shall be done every 20 minutes until the asphalt mixture becomes similar to aggregate.

Water is poured into the pycnometer with the test portion and it's de-aired with a vacuum pump. The vacuum system shall be able to evacuate entrapped air from the immerse test portion by applying a partial vacuum that results in a residual pressure of 4 kPa or less. The partial vacuum shall be achieved within not more than 5 min and shall be maintained for at least 30 min [47]. At the same time, the pycnometer is agitated using a source of external vibration with sufficient amplitude to displace the whole of the test portion within the pycnometer. Water vapour sucked by the vacuum pump and mixed with the oil, producing emulsion, may cause serious damage to the pump when in notable concentration. For this reason it is important to use a suitable de-airing system with air drying unit filled with the silica gel (desiccator salt). When using hydraulic pumps, the lubricating oil shall be changed periodically.

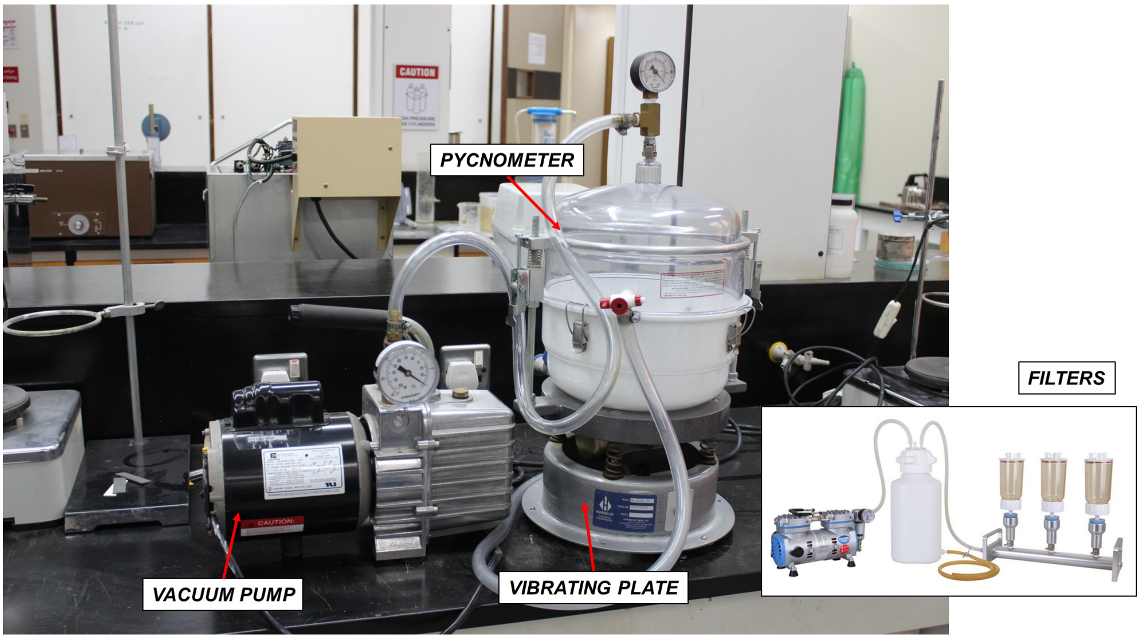
Then, the pycnometer is filled with air-free distilled water up to the reference mark (with the cap). Water and test portion occupy the reference volume V_p of the pycnometer. The mass of the pycnometer is measured (m_3).

The maximum density of the material is calculated with the following formula:

$$\rho_m = \frac{m_2 - m_1}{V_p - \frac{m_3 - m_2}{\rho_w}} \; [Mg/m^3 \; or \; Kg/m^3]$$

The testing apparatus is represented in Table 3.1.

Fig. 3.4: apparatus for measuring the maximum density of asphalt mixtures with volumetric procedure, EN 12697-5



The maximum density of asphalt mixtures can be calculated with a mathematical procedure (Procedure C indicated in the EN 12697-5) from the following parameters: percent of bitumen in the mixture (% by weight of the asphalt mixture) p_b, percent of aggregate in the mixture p_a, density of bitumen ρ_b, apparent density of aggregate ρ_a. The maximum density is calculated with the following formula:

$$\rho_m = \frac{100}{\frac{p_a}{\rho_a} + \frac{p_b}{\rho_b}} \; [Mg/m^3 \; or \; Kg/m^3]$$

The procedure to calculate the apparent density of aggregate is reported in the European standard EN 1097-6 and it's very similar to that for determining the maximum density of asphalt mixtures. The reference volume of the apparent density of aggregate comprises the aggregate and the voids accessible to water. Typical values of the apparent density of aggregate are given in Table 3.2 for different types of rock.

Table 3.2: apparent density of aggregates for different types of rock

Type of rock	Average apparent density of aggregate (g/cm^3)	Range of apparent density of aggregate (g/cm^3)
Basalt	2.80	2.6 – 3.0
Flint	2.54	2.4 – 2.6
Granite	2.69	2.6 – 3.0
Sandstone	2.69	2.6 – 2.9
Hornfels	2.82	2.7 – 3.0
Limestone	2.66	2.5 – 2.8
Porphyry	2.73	2.6 – 2.9
Quartzite	2.62	2.6 – 2.7

Artificial aggregates may be used into bituminous mixtures as steel slag. The apparent density of aggregate of these materials may be very different from that of natural aggregates. Apparent density of steel slag ranges from 3.1 g/cm^3 to 3.3 g/cm^3, which is a significantly higher value compared to densities of natural aggregates presented above.

When the apparent density of an aggregate class is significantly different from that of other classes, testing procedures shall be adjusted to take into account of this difference.
For example, the binder content shall be adjusted with the coefficient α calculated according to EN 13108-1 at point 5.3.1.3-Binder content:

$$\alpha = \frac{2.650}{\overline{\rho_a}}$$

The coefficient α is a ratio between a reference apparent density of aggregate mixture of 2.650 g/cm^3 and the actual apparent density of the aggregate mixture $\overline{\rho_a}$, calculated as the average apparent density of each aggregate class in the blend.

A similar adjustment shall be done for the grading curve of the mixture when handling blends with artificial aggregates. In these cases, the assumption of similarity between mass and volume leads to inaccurate results, as already discussed. This shall be considered in quality control processes and mix design as well.

For this reason, a detailed knowledge of aggregates properties is of primary importance to characterize asphalt mixtures in laboratory.

3.3 Voids

Recently, asphalt has been defined as a mixture of aggregate, bitumen and voids. Voids play a primary role in bituminous mixtures, affecting mechanical behaviour, compaction and workability.

Three different types of voids are usually identified in asphalt mixtures:

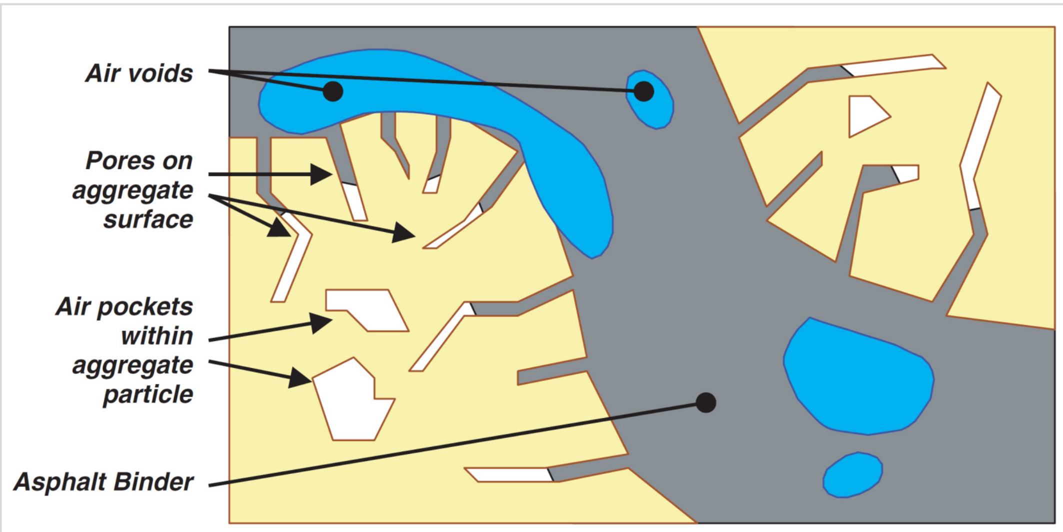
- Air voids, V_m

- Voids in the Mineral Aggregate, VMA
- Voids Filled with Bitumen, VFB (in the US are noted as Voids Filled with Asphalt, VFA)

3.3.1 Air voids, V_m

Air voids are small pockets of air between the coated aggregate particles in the compacted mixture [38]. A certain percentage of air voids is necessary in the finished asphalt mixture to allow for a slight amount of compaction under traffic and a slight amount of asphalt expansion because of temperature variations. The durability of an asphalt pavement is also a function of the air void content of the in-place asphalt pavement. The lower the in-place air voids, the less permeable the mixture becomes. An air void content that is too high provides passageways through the mix that allow damaging air and water to enter. An air void content that is too low can lead to flushing or bleeding. Density and in-place air void content are directly related. The higher the density, the lower the air void content and vice versa.

Fig. 3.5: representation of air voids in asphalt mixtures



The air voids content is defined in the EN 12697-8 as the volume of the air voids in a bituminous specimen, expressed as a percentage of the total volume of that specimen.
The air void content of an asphalt mixture depends on the type of specimen analysed [48] and the compaction method as well. Air voids can be measured on the following specimens:

- Specimens compacted in laboratory when developing a mix design. A further division can be made in:
    - Air voids of cylindrical specimen produced with Marshall compactor (Marshall voids)
    - Air voids of cylindrical specimen produced with gyratory compactor
    - Air voids of asphalt slabs produced with roller compactor

- Specimens compacted in laboratory from material produced in plant as part of quality assurance testing. The same division based on compaction method can be done for this specimens too
- Specimens taken from the roadway as cores immediately after construction as part of quality assurance testing
- Specimens taken from a wheel path of a roadway as cores after several years or more of traffic loading

The typical air void content of these different types of specimens will usually vary substantially, so it’s important when discussing or specifying the air void content of asphalt mixture to make sure it is clear what type of specimen has been tested or is required to be tested.

3.3.2 Voids in the mineral aggregate, VMA

Voids in the mineral aggregate are the intergranular void spaces between the aggregate particles in a compacted paving mixture. VMA includes air voids and spaces filled with asphalt. VMA is a volumetric measurement expressed as a percentage of the total volume of a compacted mix, and it can be calculated with the following formula:

$$VMA = V_m + B \cdot \frac{\rho_b}{\rho_B}\ [\%\ v/v]$$

Where V_m is the air void content of the specimen in 0.1% (v/v), B is the binder content of the specimen in 100% asphalt mixture, ρ_b and ρ_B are respectively the bulk density of the specimen and the density of the binder in kg/m^3.

VMA represents the space that is available to accommodate the effective volume of bitumen (i.e. the asphalt that is not absorbed into the aggregate) and the volume of air voids necessary in the mixture. Typical values of VMA ranges from 11% to 18% by volume of the material. Air voids and VMA are represented in Fig. 3.6 and Fig. 3.7.

Fig. 3.6: schematic representation of voids in a reference volume of asphalt mixture. White areas represent the air voids, black and white areas represent the VMA

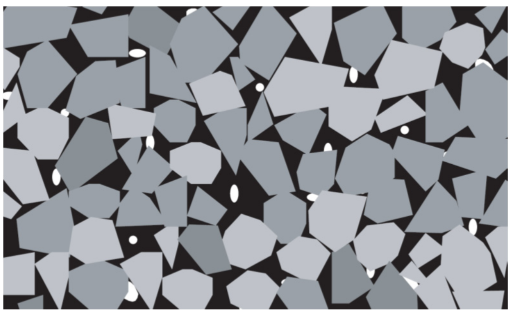
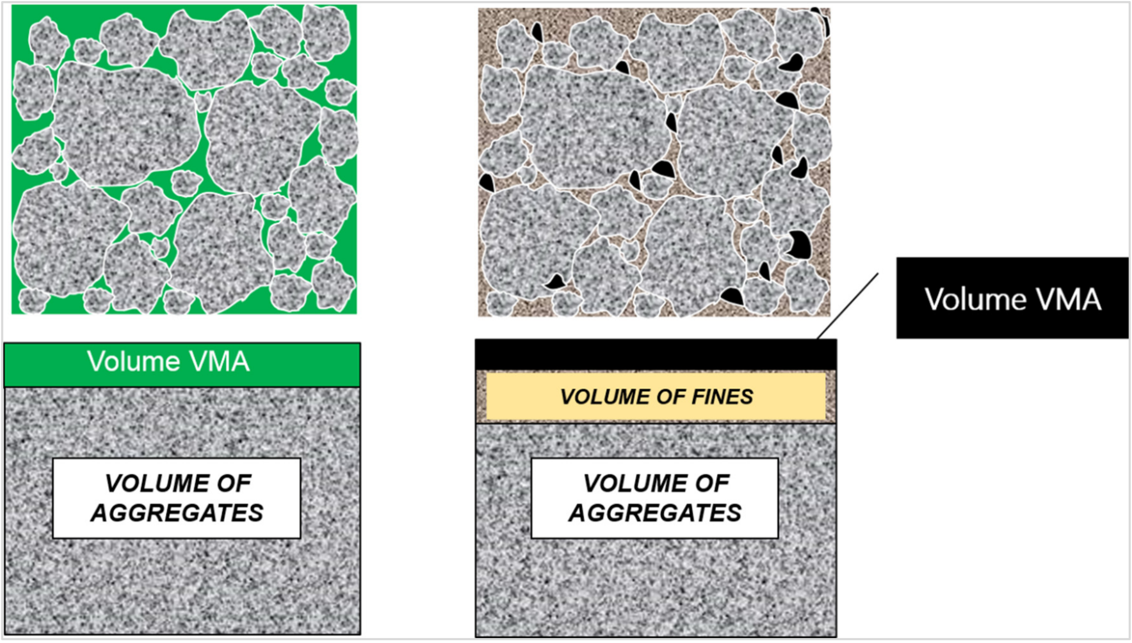
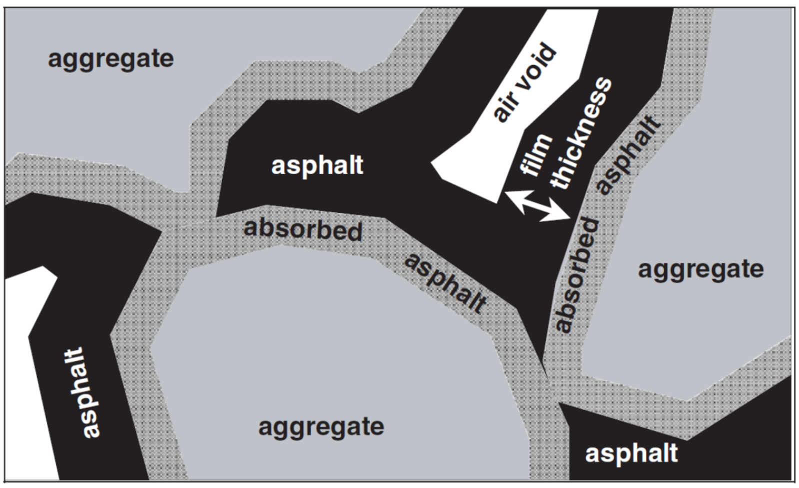


Fig. 3.7: representation of voids in the mineral aggregate for an aggregate blend



Voids in the mineral aggregate are often related to the concept of "*bitumen film thickness*" on the aggregate particles. The validity of a film thickness, calculated simply by dividing the total surface area of the aggregate (obtained from its gradation) by the effective asphalt content may be questionable. It is highly unlikely that all the particles in a mix have the same film thickness of asphalt coating. Fine aggregate particles may have a much thicker coating as compared to the coarse aggregate particles, and in fact, for all practical purposes, some very fine particles might simply be embedded in the asphalt cement/filler mortar system. Therefore, the term "film thickness" is elusive and difficult to define. Based on the past research experience, an average asphalt film thickness of 8 microns is recommended as optimal value. However, the minimum average film thickness also needs to verified and related to field performance [49].

Fig. 3.8: concept of apparent film thickness **[48]**

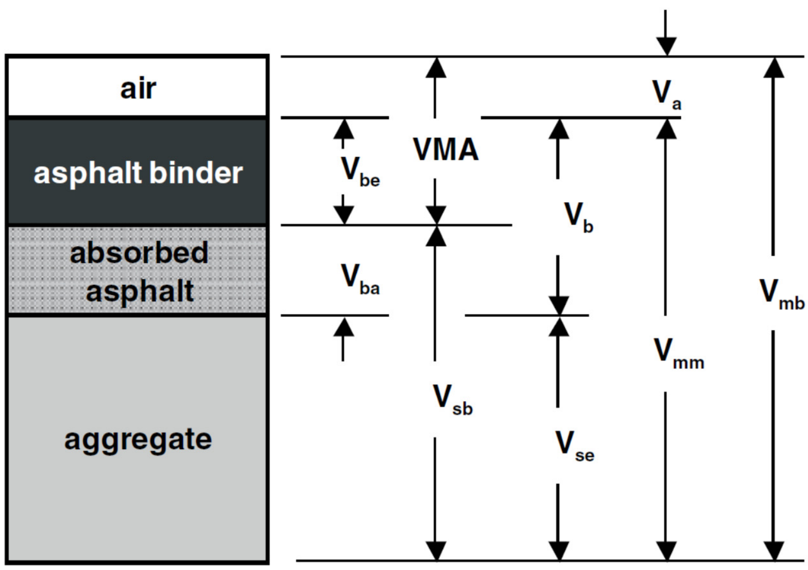


3.3.3 Voids filled with bitumen, VFB

Voids filled with bitumen VFB is the percentage of intergranular void space between the aggregate particles (VMA) that contain or are filled with asphalt [38]. VFB is used to ensure proper asphalt film thickness in the mix. If it is too low the mix will have poor durability or if it is too high the mix can show bitumen bleeding during its service life. The acceptable range of VFB varies depending upon the traffic level for the facility. The higher the traffic level, the lower the VFB.

In designing asphalt concrete mixtures, VFB is closely related to both VMA and V_m. This is because with the design air void content constant, as VMA increases, V_m increases and VFB also increases. Therefore, in most cases VFB should be thought of as simply an indicator of mix richness, like VMA or V_m [48]. If design voids are fixed or allowed to vary only over a narrow range, there is little point in simultaneously controlling VMA, V_m, and VFB. In fact, simultaneous control of strongly interrelated volumetric factors can lead to confusion and conflict during the mix design process and during construction.

Fig. 3.9: relationship between volumes different phases (aggregate, bitumen and voids) of an asphalt mixture



V_{be} = Volume of effective asphalt binder
VBE = Effective asphalt content, percent by volume
V_{ba} = Volume of absorbed asphalt binder
VBA = Absorbed asphalt binder, percent by total mix volume
V_{ma} = Volume of voids in mineral aggregate
VMA = Voids in mineral aggregate, percent by volume
V_{sb} = Volume of aggregate, bulk (including all permeable surface pores)
V_b = Volume of asphalt binder, total
VB = Total asphalt binder content, percent by volume
V_{se} = Volume of aggregate, effective (excluding surface pores filled with asphalt)
V_a = Volume of air voids
VA = Air void content, volume percent
V_{mm} = Volume of aggregate and asphalt
V_{mb} = Volume of specimen, total

The optimal void content of an asphalt mix shall be always determined through an accurate mix design, investigating the effect of voids together with other variables as bitumen content on physical (density) and mechanical (indirect tensile strength, Marshall stability, stiffness, fatigue resistance) properties of the mixture.

3.4 Compaction

3.4.1 Gyratory compaction

The gyratory compactor is a device used for the compaction of asphalt mixtures and other road materials as cohesive soils, hydraulically treated granular materials, cold mixes. The basis for the gyratory compactor (GC) was the Texas gyratory compactor developed in the 30s by the U.S. Corps of Engineers. The GC was modified by French researchers of “Laboratoire Central des Ponts et des Chaussées” (LCPC) in the late 50s. It is part of the Superpave mix design process and it’s essential to determine volumetric and mechanical characteristics of asphalt mixtures.

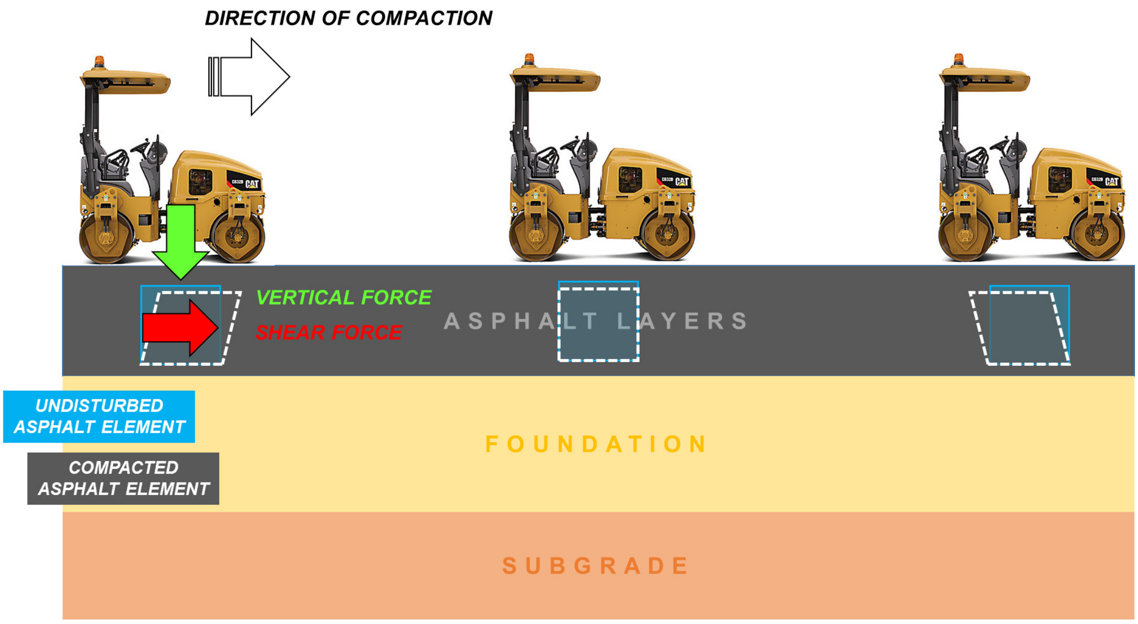
The gyratory compactor can be used for different purposes:

- Production of cylindrical bituminous samples for further laboratory testing (indirect tensile strength, stiffness modulus, indirect tensile fatigue)
- Quality control and acceptance of volumetric characteristics of asphalt mixtures
- Support to mix design of asphalt mixtures through the determination of optimal properties of the mixture (bitumen content, density and voids)
- Investigation of compaction characteristics of the material and optimization of the blend (aggregate grading, bitumen and fines content)

The gyratory compactor simulates the effect of the passage of the roller compactor on asphalt layers in the field. In field, an undisturbed element in the asphalt layers is subjected to a vertical and horizontal load produced by the passage of the roller compactor. The external load compresses the asphalt element in the vertical direction and stretches it horizontally changing its shape. The material lowers its volume under the action of the vertical pressure and the shear force of the roller compactor. The shear force increase the effectiveness of the compaction since it constantly reduce the mutual friction between aggregates rearranging the internal structure of the asphalt mixture.

Rearrangement of the asphalt structure is optimized by vibrating systems mounted on the cylinders of the roller compactor. Vibrations produce a dynamic compaction that further increase the compaction energy and make densification process faster and more effective compared with static compaction. This is essential for reaching the expected densification of the material. The compaction mechanism on asphalt layers is illustrated in Fig. 3.10.

Fig. 3.10: compaction mechanism on asphalt material during paving operations on site



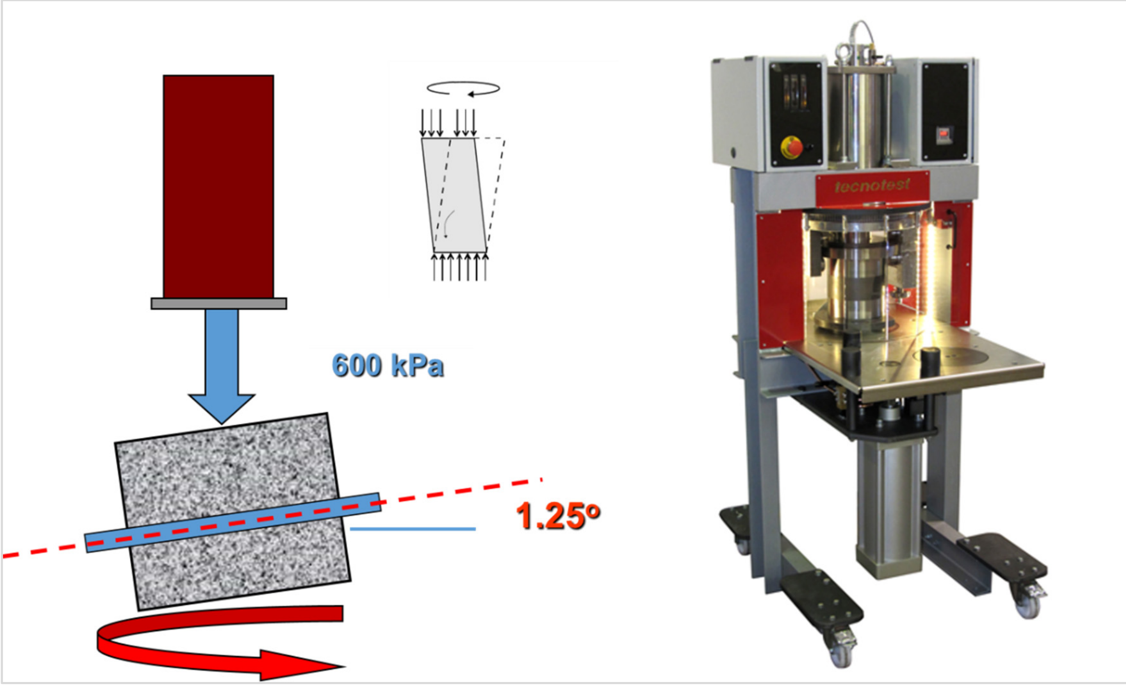
With gyratory compactor, the asphalt specimen is subjected both to vertical and shear forces in the same way as in field compaction.

The gyratory compactor is a device comprising the following components:

- Reaction frame, rotating base and motor
- Loading system, loading ram and pressure gauge
- Height measuring and recordation system
- Mould and base plate
- A specimen extruding device

The samples of loose hot asphalt are put into a metallic mould 100 or 150 mm diameter. The mould is placed into the gyratory compactor inclined at 1.25° with respect to the vertical axis of the machine. When the compaction starts, the mould rotates at 30 turns per minute and a loading ram applies a vertical and constant pressure of 600 kPa on the top of specimen throughout the compaction. A pressure gauge measures the ram loading to maintain constant pressure during compaction. The gyratory compactor mould has an inside diameter of 100 or 150 millimetres, and a base plate in the bottom of the mould provides confinement during compaction.

Fig. 3.11: gyratory compactor configuration



The height of the specimen is measured at each gyration throughout the compaction by recording the position of the ram throughout the test at each gyration. Using the mass of the material placed in the mould, the diameter of the mould and the specimen height measured at each gyration, the density of the specimen can be estimated dividing the mass by the volume of the specimen. The same methodology is referred in the EN 12697-6 as "bulk density by dimensions".

The densification of the specimen at a specific gyration is calculated as a function of measured height and it's expressed as percentage of the maximum density G_{mm}. The % G_{mm} is plotted versus the logarithm of gyrations, as reported in Fig. 3.12. The curve is noted as "densification curve" or "gyratory compaction curve". The slope of the gyratory compaction curve can provide an indication of the properties of the compacted mixture [50] [51] [52]. The slope differentiates between different aggregate gradations. Finer gradations exhibit flatter compaction slopes. However, Superpave mixture analysis and wheel tracking testing did not support the idea that mixes with flatter slopes had weaker aggregate structures.

There are three characteristic compaction levels or number of gyrations characterizing the densification of an asphalt mixture:

1. N_{ini}: usually corresponds to 10 gyrations and represents the compaction of the material just after being paved
2. N_{des}: compaction of the material after passages of roller compactor and before the road is opened to traffic
3. N_{max}: compaction of the material after some years of service

The values of N_{des} and N_{max} may vary depending on the purpose of the compaction. In mix design procedure, the design number of gyrations is chosen to reach the desired density (air void content). In quality control, asphalt specimen are usually compacted at N_{des} and then tested in laboratory. The value of N_{des} depends on the material: for dense-graded mixtures N_{des} ranges from 100 to 130. A greater number of gyrations is required when processing mixtures with modified bitumen. The more the bitumen is modified, the higher the number of gyrations required in technical documents for quality control. For open-graded materials, N_{des} ranges from 60 to 80 gyrations.

Extensive laboratory testing shows that density measured by dimension is smaller compared with density measured with dry procedure, on the same asphalt specimen. A cylinder approximates by excess the volume of an asphalt sample because of the rough surface texture of the specimen. For the same mass, increasing volume reduces the calculated density. The geometrical density calculated at each gyration is adjusted to fit the actual bulk density measured with dry procedure at the end of the compaction with the following formula:

$$c = \frac{G_{mb,G}}{G_{mb,D}}$$

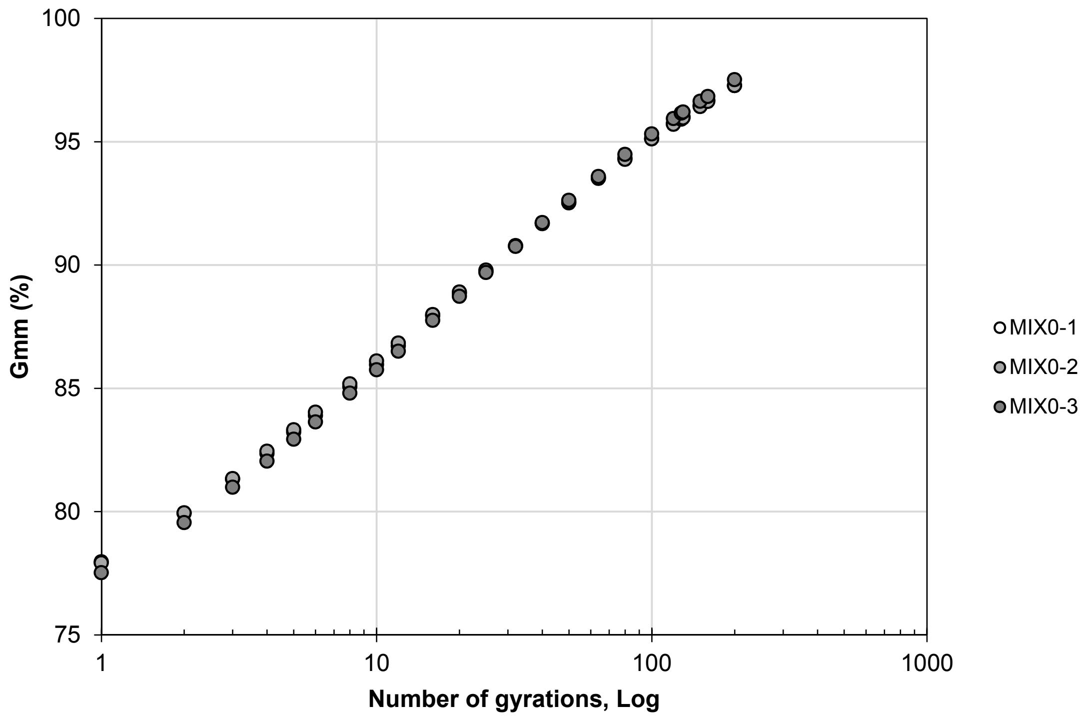
Where $G_{mb,G}$ is the geometrical density calculated at the end of the compaction and $G_{mb,D}$ is the bulk density measured on the compacted asphalt sample with dry procedure. The values of density at each gyration are corrected with the coefficient.

An example of gyratory compaction output data is given in the following

Table 3.3: gyratory compaction output data including density (corrected), voids and densification

Gyrations (N)	**Measured height**	**Density**	**Air voids**	**VMA Voids in the Mineral Aggregate**	**VFB Voids Filled with Bitumen**	**% G_{mm}**
	(mm)	*(g/cm³)*	*(%)*	*(%)*	*(%)*	*(%)*
1	132.2	1.936	22.0	32.6	32.5	78.0
10	119.9	2.135	14.0	24.6	43.0	86.0
50	111.4	2.297	7.5	18.1	58.6	92.5
100	108.4	2.362	4.9	15.5	68.5	95.1
120	107.6	2.378	4.3	14.9	71.4	95.7
200	106.0	2.416	2.7	13.3	79.6	97.3

Fig. 3.12: gyratory compaction curves of one of the asphalt mixtures investigated in the present research



The gyratory compaction has become nowadays the most common practice to compact asphalt materials in laboratory producing a densification of the material similar to that produced by roller compactor in the field [53] [54].

The properties of asphalt samples cored on site and of asphalt samples of the same material produced in laboratory with different compaction methods were measured and compared by Von Quintus (1991) and Button (2006). Volumetric and mechanical properties of laboratory and field samples were compared. Values measured on field cores were used as benchmark. Results in Table 3.4 show that gyratory compactor produces asphalt specimen with similar properties to the paved material. The research highlights a relevant difference between laboratory and field compaction.

Table 3.4: properties of asphalt materials measured on field cores and sample produced in laboratory with different compaction methods

Laboratory compaction method	**Similarity between properties measured on field and laboratory samples (%)**	
	Von Quintus et al.	*Button et al.*
Gyratory compactor	63	73
Rolling wheel compactor (tyre)	49	64
Kneading compactor	52	64
Vibratory kneading compactor	41	NA
Marshall hammer	35	50

3.4.2 Roller compaction

The laboratory roller compaction methods simulate the compaction process (kneading action and vertical load) to which bituminous mixtures are subjected on site. Parallelepipedal asphalt samples (slabs) can be prepared under controlled compaction energy or until a specified density is reached with the following methods:

- Pneumatic tyre
- Steel roller
- Sliding plates

Bituminous slabs are used directly for subsequent testing i.e. wheel tracking tests or can be sawn or cored to prepare test specimen for indirect tensile or bending fatigue tests.

A large number of laboratory roller compactors are available on the market. On laboratory devices, a section of metallic cylinder is able to move back and forth on the mixture in the mould. Alternatively, the mould can be placed on a table that moves back and forth beneath the steel roller, which applies a vertical load on the asphalt slab. The compaction is provided by the combined vertical load and shear force caused by the friction between the surface of the asphalt slab and the metallic roller.

It's opinion of the author that some drawbacks may be found on laboratory roller compactors on the market. These issues may affect the quality of the compaction, resulting in significant differences between target and actual physical properties of asphalt mixtures (density and void content). Typically, lower and more scattered values of density are obtained with laboratory roller compactors than those with gyratory compactor on the same material. This difference is caused by the number of boundary conditions that are involved in rolling compaction. With respect to gyratory compaction, preparation procedures take considerably larger amount of time because of the size of the slabs, the time required to fill the mould with asphalt (including weighing) and the time for positioning the sample into the compaction slot. During preparation, the temperature of the mixture considerably decreases (almost 10°C in the first minute out of the oven). If the temperature of the sample lowers below the optimal compaction temperature, the laboratory roller compactor would not be able to give the necessary compaction effort to reach the target density. The European standard EN 12697-33 indicates the optimal compaction temperature as a function of bitumen grade [55]. Values are reported in Table 3.5.

Table 3.5: compaction temperature for asphalt slab compacted with laboratory roller compactor as function of bitumen grade

Bitumen grade EN 12591	Compaction temperature in °C
20/30	180±5
35/50 and 30/45	160±5
40/60 and 50/70	150±5

70/100	140±5
100/150	135±5
160/220 and 250/330	130±5

There are two main critical aspects in the compaction process of automatic laboratory roller compactors: the first concerns the shape of the cylinder, which is commonly a section of a full cylinder and hence is not able to reproduce the kneading action caused by the passage of the steel roller on the asphalt surface; the second concerns the system that produces the vertical load. Pneumatic systems may produce asymmetric load of the cylinder on the specimen resulting in a not uniform compaction of the slab. However, most recent devices are equipped with electromechanical systems to limit this issue.

3.4.3 Elletipi roller compactor

In the Elletipi roller compactor, a steel cylinder moves continuously back and forth on the slab surface, simulating the passage of a real scale roller compactor on site.

The steel cylinder is 295 mm width, 300 mm diameter and 30 kg weight. It moves horizontally at a speed of 4 km/h on a rail and it’s thrusted by a 750W electric motor. The cylinder is forced to move back and forth on a single horizontal direction as a carriage. The amplitude of the horizontal motion of the cylinder is regulated by two lever hinge interlock operated safety switches, which gradually change the direction of cylinder’s motion. The switches are activated by an aluminium bar at the passage of the cylinder. The position of the switches control the movement of the cylinder and it plays an important role for the good quality of compaction.

A second electric motor moves the roller vertically, producing a pressure on the slab surface variable from 300 kN/m^2 to 400 kN/m^2. During compaction, the operator moves the cylinder downwards or upwards until the specified height of the sample is reached.

The loose and hot asphalt is put into an aluminium mould 300 mm width, 150 mm height and variable length up to 1200 mm. The geometry of the mould is adjustable and hence suitable to produce asphalt slabs with different dimensions.

The Elletipi roller compactor is equipped with a heating coil placed below the mould and an infrared lamp which is mounted on the frame with the cylinder and the two electric motors. The coil heats the base of the mould keeping the asphalt sample at the same temperature during compaction. The infrared lamp heats the surface of the cylinder avoiding thermal shocks between the cold steel and the hot asphalt.

An accurate control of temperature is a key factor for good compaction. If temperature of asphalt sample decreases too much, the target density (and hence void content) may not be reached. The longer the time needed for compacting the material, the greater the temperature drop, the poorer the quality of the compaction.

The most significant drop of temperature of asphalt slabs occurs in the following phases:

1. The loose asphalt is taken out of the oven and put into a steel pan and weighed before being placed in the mould

2. The asphalt is put in the aluminium mould of the roller compactor from the steel pan and roughly levelled in order to obtain an approximately plane surface

Asphalt may loose from 10°C to 30°C from when it's removed from the oven to the beginning of the compaction depending on the time necessary to the operator for completing the preparation procedures.

The Elletipi roller compactor has two main advantages compared with other devices on the market:

- The field compaction is accurately simulated by the Elletipi device which uses a steel cylinder that continuously moves back and forth. Other devices uses only sections of cylinder with limited movement (vertical and rotation)
- The Elletipi device is equipped with a heating coil and an infrared lamp which help maintaining the asphalt sample at constant temperature during the compaction and avoiding thermal shocks

Fig. 3.13: main components of the Elletipi roller compactor

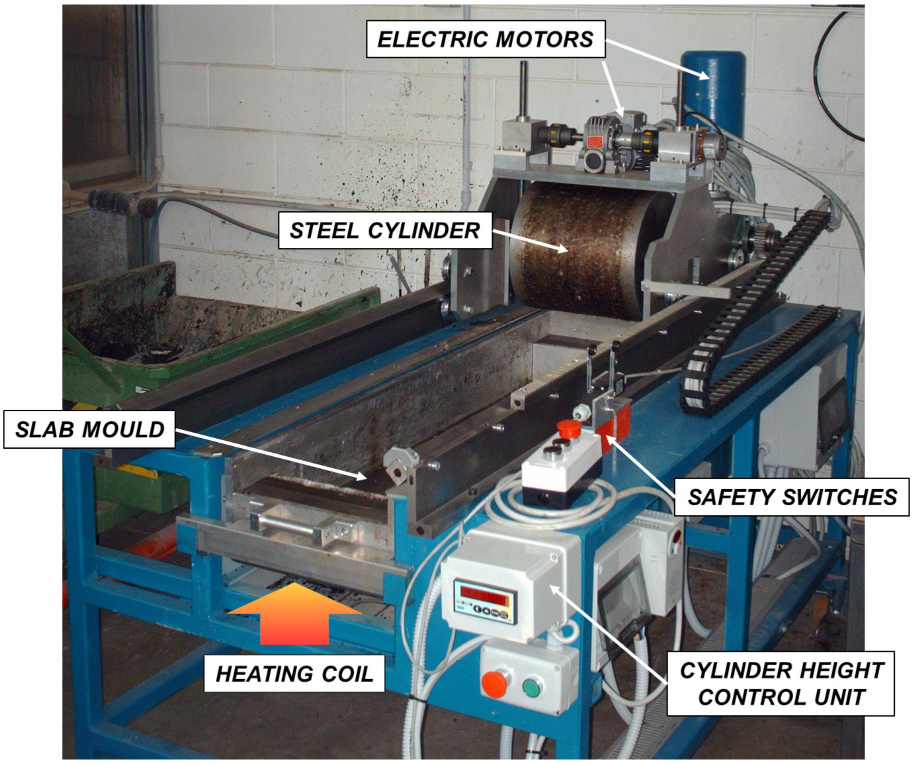


Fig. 3.14: detail of the two electric engines assembled on the metallic frame together with the steel cylinder

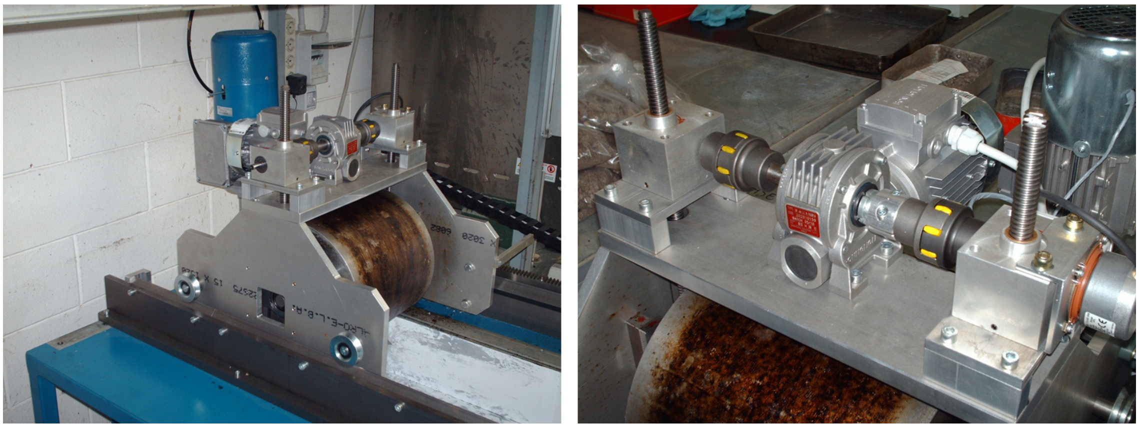


Fig. 3.15: compaction of an asphalt slab with the Elletipi roller compactor

3.5 Fatigue resistance of asphalt mixtures

3.5.1 Test methods

Laboratory test procedures for measuring the fatigue response of asphalt paving mixtures are commonly classified into categories dependent upon the mode of loading [56]:

1. **Simple flexure**: simply supported asphalt beams are subjected to pulsating or sinusoidal load or deflection in the following geometrical configurations:
    - Three-point bending (three-point bending, 3PB)
    - Four-point bending (four-point bending, 4PB)
    - Rotating cantilever beam flexure
    - Trapezoidal cantilever beam flexure (two-point bending, 2PB)
    - Rectangular cantilever beam flexure
2. **Supported flexure**: direct relationship between fatigue life and strain/stress is developed by loading beams or slabs that are supported in various ways to directly simulate in situ modes of loading and sometimes to simulate a more representative stress state. The test simulates the behaviour of an asphalt layer resting on an elastic foundation or subgrade

3. **Direct axial** (tension-compression, T/C): a cylindrical specimen is subjected to a pulsating or sinusoidal uniaxial load with or without stress reversal
4. **Diametral** (ITFT): the diametral fatigue test called also *Indirect Tensile Fatigue Test* is conducted by loading a cylindrical specimen with a repeated compressive load which acts parallel to and along the vertical diametral plain. The loading configuration develops a reasonably uniform state of stress in the central part of the specimen. The test is mostly run under stress controlled mode of loading, usually a haversine load pulse
5. **Triaxial**: the specimen is subjected to sinusoidal vertical load (z axis) with a constant confining pressure (x-y plain) applied
6. **Fracture mechanics**: the theoretical approach of fracture mechanics postulate that the fatigue life of asphalt mixtures can be described in three phases: crack initiation, crack propagation and failure. Crack propagation is defined by the Paris law:

$$\frac{dc}{dN} = AK^B$$

Where c is the crack length, N number of load applications, A and B material constants, K stress intensity factor which depend on specimen geometry, test configuration, boundary and load. The aim of the test is to calculate parameters A and B by measuring the crack length c with direct (LVDT, digital imaging) or indirect (beam compliance or stiffness) methods. The linear model may be substituted with more accurate (and complex) non-linear models [57] [58] [59].Tests are similar to those already described.
7. **Wheel tracking**: a wheel with a specified load passes backwards and forwards onto an asphalt slab simulating the action of a wheel on a pavement. Wheel tracking test can be divided into two categories: laboratory-scale and full-scale. Displacements and cracks are measured with transducers and strain gauges.

Conventionally, bending tests are supposed to represent the repeated blending forces caused in the pavement by the passage of vehicles, while tensile tests represent the tensile force induced at the base of the pavement by this bending. Simple flexure and diametral tests are the most common testing procedures used by testing laboratories for routine investigations.

Recently, a classification of laboratory tests into two main categories was proposed [60]:

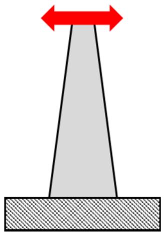
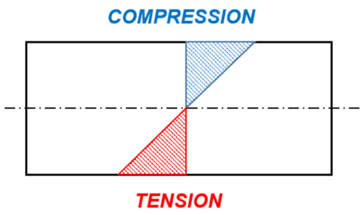
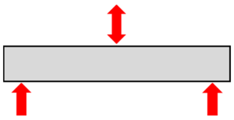
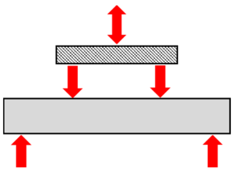
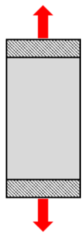
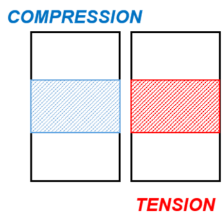
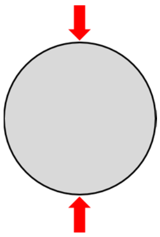
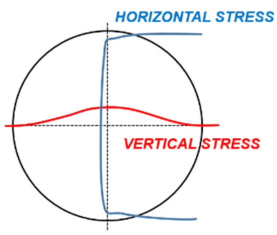
1. Homogeneous (tension-compression, shearing test, co-axial shearing)
2. Non-homogeneous (simple and flexure, diametral)

Homogeneity refers to the distribution of stresses and strains within the tested asphalt sample. Homogeneous tests give direct access to the stresses and strains, and therefore to the constitutive law (whether viscoelastic or not). Non-homogeneous tests call for postulating a constitutive law first and taking into account the geometry of the specimen calculations to obtain the parameters of the constitutive law [61].

3.5.2 Test configurations

Simple flexure (2PB, 3PB and 4PB), indirect tensile and tension-compression tests are the most common in laboratory practice. Other methodologies may be suitable for research purposes. However, the increased complexity of a test method lead to issues in terms of repeatability and accuracy of results. A resume of the test configurations, including geometry, load and stress/strain distribution, is given in Table 3.6.

Table 3.6: summary of test configuration (geometry, stress distribution and load) for most commonly used test methods

Test method	Test geometry	Stress/strain distribution	Loading
Simple flexure 2PB			• Haversine • Sine • Uniaxial • Non–homogeneous
Simple flexure 3PB		Same for all flexural tests	• Haversine • Sine • Uniaxial • Non–homogeneous
Simple flexure 4PB		Same for all flexural tests	• Haversine • Sine • Uniaxial • Non–homogeneous
Direct axial loading T/C			• Pulse • Sine • Uniaxial • homogeneous
Diametral loading ITFT			• Pulse • Biaxial • Non-homogeneous

The choice of the fatigue test method has a significant impact on results. Each method requires different procedures and equipment necessary to carry out the test. A list of procedures in common with all test methods can be summarized as follows:

1. Specimen preparation
    - Mix design (if needed)

- Mixing
- Compaction
- Sawing

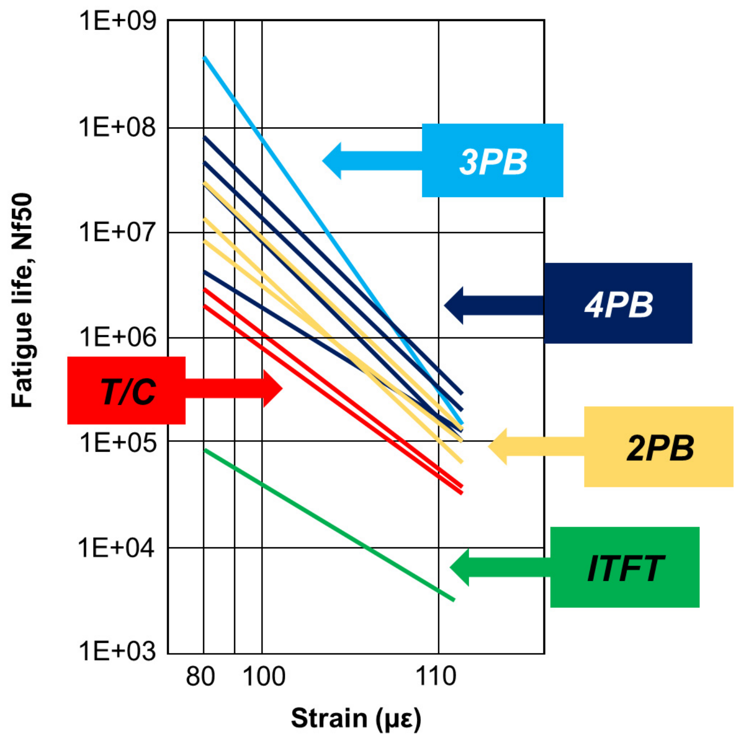
2. Preparation of test equipment
    - Calibration of load cell and transducers
    - Test set-up
3. Testing

Fatigue behaviour of asphalt mixes is very sensitive to boundary and loading condition [62]. This extreme sensitivity of fatigue tests is the key issue of the importance of the test method. All the phases previously listed contribute to define different boundary conditions for different methods. Even for the same test configuration, operations may differ when performed by different technicians and may be subjected to relatively low repeatability. Studies [62] [60] have shown that the fatigue life of a bituminous mix tested under different conditions changes with the test method. Di Benedetto et al. proposed the following considerations about the effect of test method on fatigue life:

- Simple flexure beam tests seem dependent of the kind of test and on the size of the sample
- No link can be found between fatigue lives of stress control and strain control tests
- Fatigue lives are different following the considered test method

These conclusions derives from an inter-laboratory fatigue test campaign organized by the RILEM 182 PEB Technical Committee. Results of the experimental program are presented in Fig. 3.16, where the effects of the test method on calculated fatigue life of the same asphalt mixture are evident.

Fig. 3.16: effect of test method on fatigue life of the same asphalt mixture



3.5.3 Type of loading

The shape of the applied cyclic loading is very important in the analysis of fatigue test results [63]. In laboratory tests, fatigue response has been shown to be a function of mode of loading [56]. In order to have a representative loading condition in a fatigue test it is necessary to understand how the loading is applied to a pavement structure.

Fig. 3.17 represents a pavement element showing the stresses which occurs in it when a wheel load is approaching. In this simplified representation, stresses are considered to be uniaxial. In real cases, they act three-dimensionally. Stresses and hence strains changes with time as the wheel passes [64] over the surface course of the pavement. The shape of the longitudinal and transversal strain measured by strain gauges located at the base of asphalt layers of a flexible pavement is depicted in Fig. 3.18.

Fig. 3.17: stress distribution induced by a moving wheel in a flexible pavement, and strain distribution calculated at the bottom of the asphalt layer with software Alize LCPC

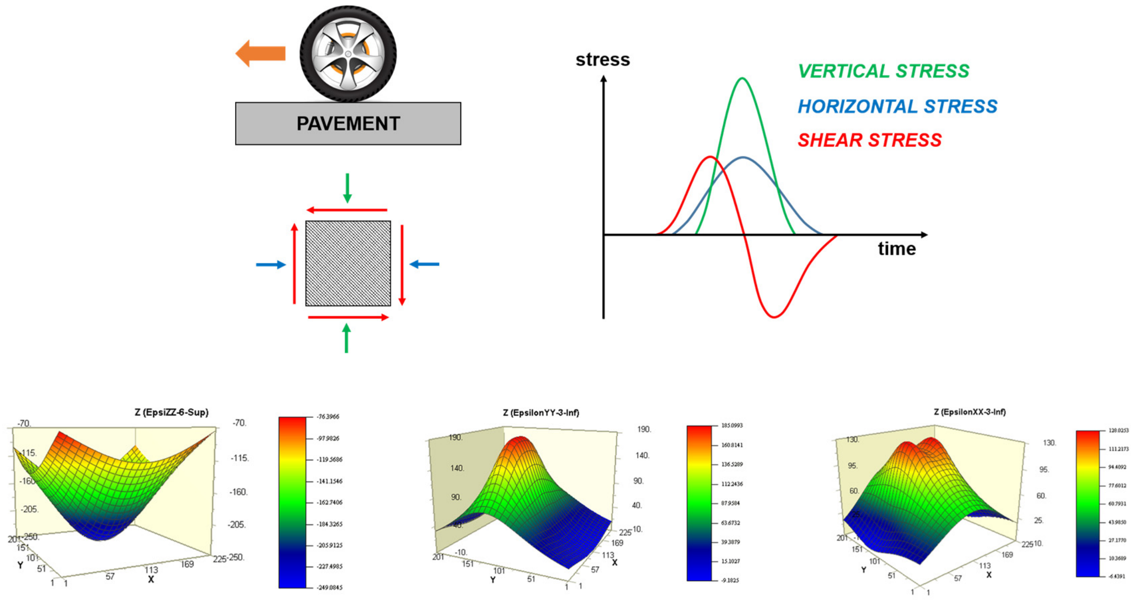
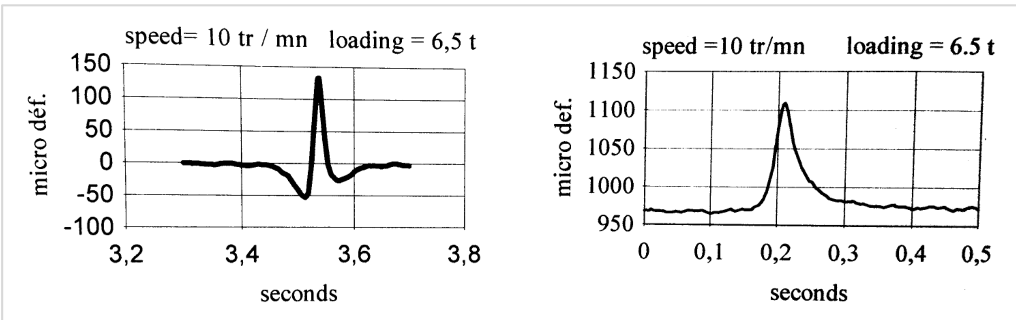


Fig. 3.18: shape of longitudinal (left) and transversal (right) strain measured with strain gauges bonded to the base of a bituminous layer, de La Roche et al, 1993



Strains and stresses which develop within the pavement are dependent upon the vehicle speed, depth below the pavement surface and wheel, axle and suspension configuration. In laboratory, the applied load shall be able to simulate the loading process occurring in field. The main type of load in used in laboratory are characterized by:

- Mode of loading
- Wave shape (also referred as loading pattern)
- Rest Periods

3.5.3.1 Mode of loading

The mode of loading refers to the manner in which stress or strain levels are permitted to vary during repetitive loading [56]. Two main modes of loading are identified [65]:

- Stress controlled: the load (or stress) is maintained constant
- Strain controlled: the displacement (strain) is maintained constant

It shall be pointed out that only load or displacement can be measured and controlled by the testing equipment. Stress and strain are calculated depending on the test configuration.

Fig. 3.19: representation of stress and strain controlled mode of loading (Epps and Monismith, 1971)

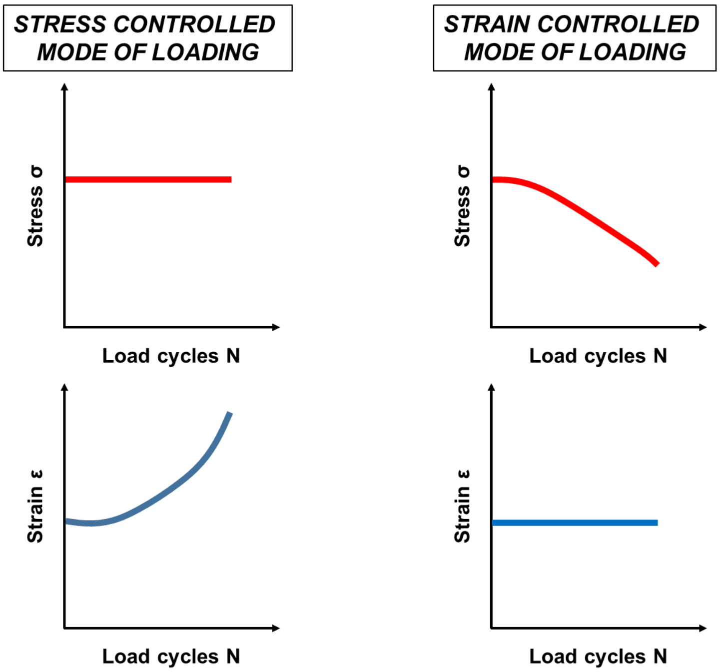


Fig. 3.20: plot of stress and strain (permanent and recoverable) versus load cycle for a stress controlled indirect tensile fatigue test on asphalt specimen (400 kPa stress and pulsating load)

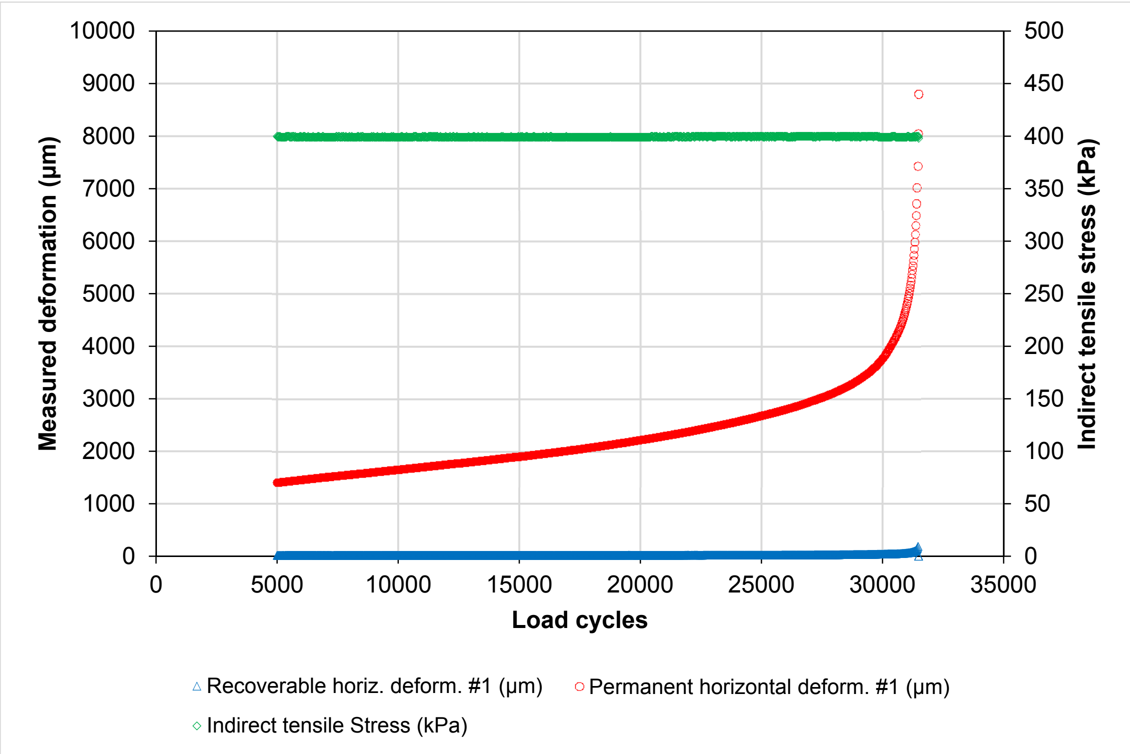
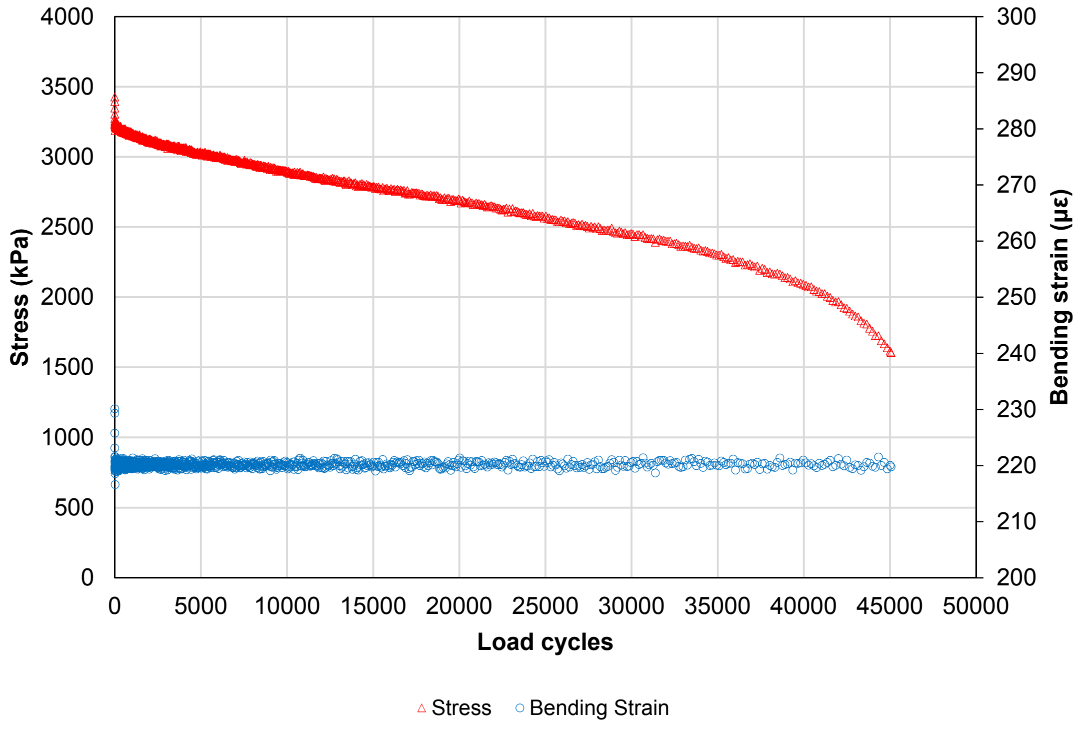


Fig. 3.21: plot of strain and stress versus load cycle for a strain-controlled 2PB fatigue test on asphalt specimen (220 µε at 25 Hz)



In the controlled stress loading, the strain decreases until failure of specimen. In the controlled strain test, the stress (load) decreases with increased number of load repetitions since the specimen is gradually damaged.

Stress and strain controlled modes of loading produce different results in terms of cycles to failure for the same asphalt mixture. In stress-controlled tests, the higher the stiffness of a mixture, the longer the measured fatigue life. In strain-controlled tests, the higher the stiffness, the shorter the fatigue life. For the same asphalt mixture, i.e. same stiffness, strain-controlled tests produce longer fatigue lives than stress-controlled tests.

Brown [66] explained this in terms of mechanism of failure which occurs during fatigue testing. Cracks initiate in points of the specimen subjected to high levels of stress, then cracks propagate during the test. The mode of loading has great influence on the crack propagation, since the higher the stress level, the faster the propagation. In a stress controlled test, the crack propagation is very rapid since the stress is constant for the entire test duration. In a strain-controlled test, the stress reduces as the crack propagates, therefore this mode of loading leads to longer fatigue lives.

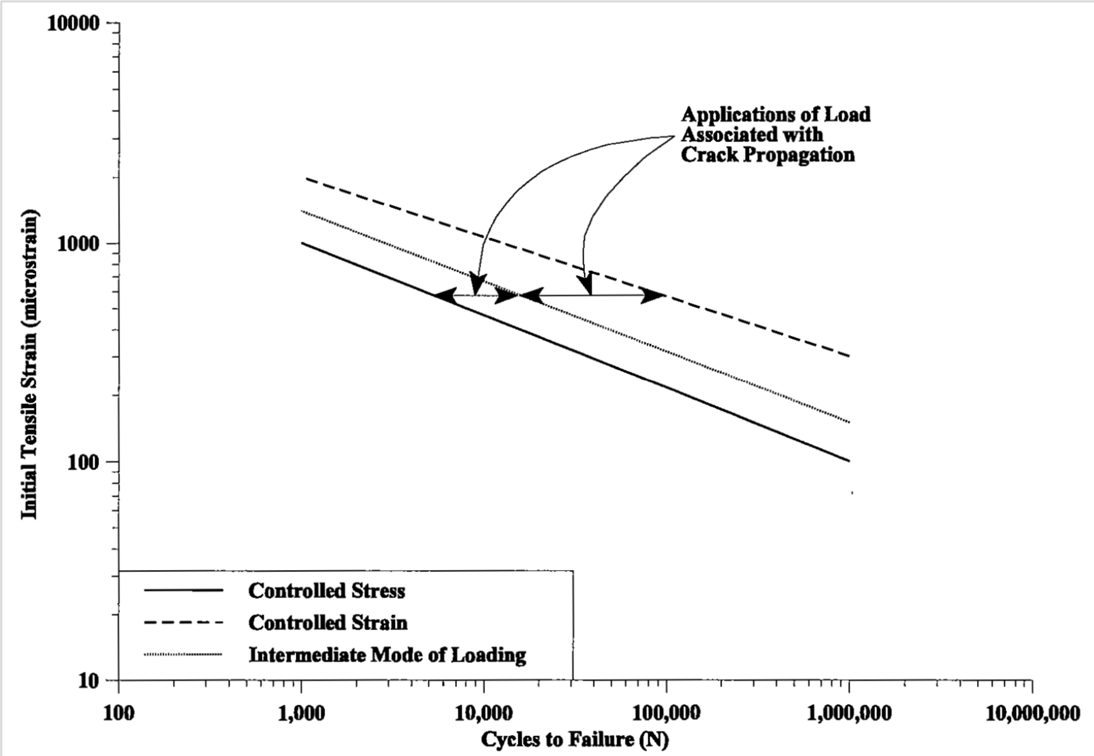
It is generally held [60] [66]that:

- A thin pavement (< 6 cm) is loaded at a constant strain, since it is the strain of the more flexible lower layer that controls the response of the upper layer. Thin surfacings are essentially moved by the lower structural layers which results in a strain controlled situation [67]

- A thick pavement (> 15 cm) is loaded at constant stress. The pavement as a whole system and layers with considerable structural significance (thick and stiff) are subjected to a stress controlled loading

The impact of test mode on the calculated fatigue life of the same material is illustrated in Fig. 3.22, where an intermediate mode of loading is also included [67].

Fig. 3.22: effect of mode of loading on the evaluation of fatigue life of the same asphalt mixture (Read, 1996)



A comparative evaluation of the two strain and stress-controlled modes of loading is given in the following Table 3.7 (Rao Tangella, 1990).

Table 3.7: comparative evaluation of controlled-stress and controlled-strain mode of loading (Rao Tangella, 1990)

Variables	Controlled-stress (load)	Controlled-strain (deflection)
Thickness of asphalt layer	Thick asphalt layers (> 15 cm)	Thin asphalt layers (< 6 cm)
Definition of failure	Well defined: fracture of the specimen	Arbitrary: reduction to a specified percent of the initial value of a parameter (commonly the complex modulus)
Scatter in test data	Less	More
Required number of samples	Smaller	Larger
Magnitude of fatigue life	Shorter	Longer
Effect of mixture variables	More sensitive	Less sensitive
Rate of energy dissipation	Faster	Slower

Rate of crack propagation	Faster than in-situ	More representative of in-situ conditions
Beneficial effects of rest periods	Greater beneficial effect	Lesser beneficial effect

3.5.3.2 Loading pattern

The wave shape or loading pattern has a great influence on the measured fatigue life of an asphalt material. It is characterised by:

- Waveform
- Frequency
- Duration (e.g. time of a load pulse)

The *waveform* of a load impulse affects the fatigue life. Waveform can be: sinusoidal, square, triangular, haversine, approximately haversine cyclic loading (pulse). The relationship between fatigue life and waveform was presented by Raithby and Sterling [68]. For the same stress/strain amplitude, the fatigue life is shorter if greater energy is imposed on a tested specimen during one loading cycle. The energy is proportional to the area of the stress/strain versus time curve.
Comparing haversine and sinusoidal waveform, Irwin [69] concluded that the first loading gives longer fatigue lives than the second, as indicated in Fig. 3.23. In the sinusoidal loading, stress reversal occurs in the specimen and both sides are alternately subjected to tension and compression, therefore cracks may develop and propagate from two sides of the specimen instead of one side in the haversine loading.

The loading waveform should approximate that which is experienced in the field. It has been found [68] that the sinusoidal is the closest waveform to that experienced in the field.

Load duration and *frequency* affect fatigue life as well. The shorter the duration time at the same frequency (same rest period between two consecutive load pulses), the longer the fatigue life. Similarly, the higher the loading frequency, the longer the fatigue life. The relationship between load duration and fatigue life reported in Fig. 3.24 from [70].

The laboratory fatigue life of asphalt materials appears to be shorter than the field fatigue life. Laboratory results require shift factors that may range from 3 to 300 to match actual fatigue cracking. This means that important factors are not properly accounted for in classic fatigue criteria. The difference between laboratory and field results may be attributed to loading conditions, including *rest periods* [71].

A rest period is the time between two consecutive applications of wheel loads and are very important as they allow time for crack to heal and stresses and strain to relax due to viscous flow of the bitumen [67]. Rest periods in laboratory are generally constant, whereas in the field are different for different roads and for different times of the day as they are dependent upon the volume of traffic.

Researchers [63] found that rest periods significantly increase the fatigue life of asphalt mixtures, indicating potential healing properties of asphalt mixtures.

The definition of healing given by Bhasin [72] is reported: “the growth of microcrack is associated with the creation of new fracture surfaces. A precursor to the growth of a microcrack is damage in the vicinity of the crack tip. This damage is associated with the deformation and rearrangement of molecules in the failure zone. The mechanism of self-healing can therefore be regarded as a reversal of these processes. More specifically, self-healing entails reversal of crack opening followed by reversal of the microdamage that occyred in the failure zone”.

At a larger scale, healing may correspond to an increase (recovery) of stiffness/complex modulus that occurs during the rest periods of laboratory fatigue tests.

Laboratory tests can include rest period in order to better simulate loading conditions in the field. Daniel and Kim [73] concluded that when specimen are subjected to rest periods, an increase in dynamic modulus of elasticity is observed, which can be attributed to microcrack healing of the material. Various studies performed on this subject lead to the following results:

- There is an increase of life between the continuous and discontinuous tests, and the test temperature and the duration of the resting time are factors of influence [74]
- The higher the temperature or the longer the resting time, the larger the healing [68] [73]

Several results show that beyond a certain limit, resting periods yield no additional increase of life. It is often assumed that a coefficient of 10 between the duration of resting period and the loading pulse yields the optimal increase of life.

Fig. 3.23: effect of haversine and full wave sine waveforms on fatigue life of the same asphalt mixture (Irwin, 1977)

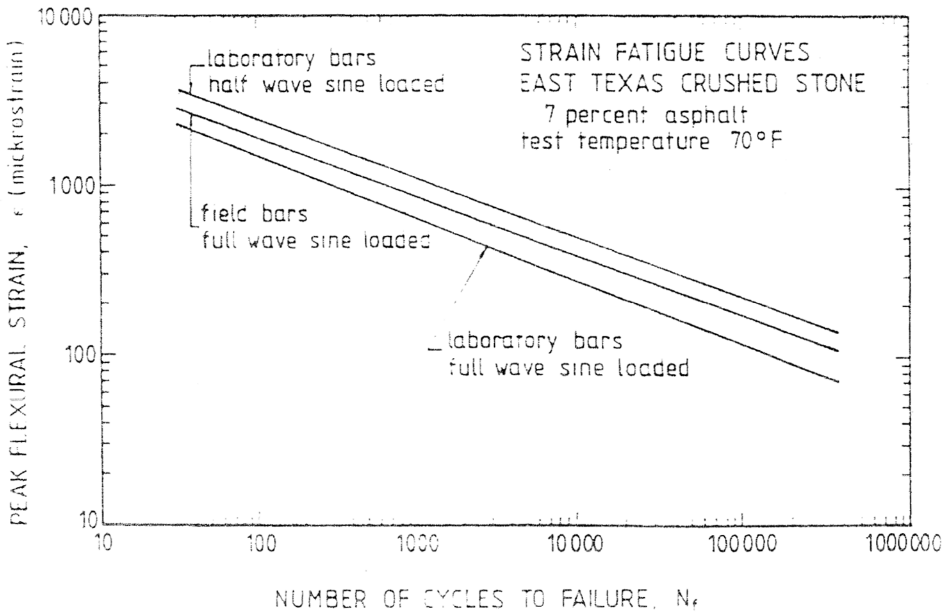
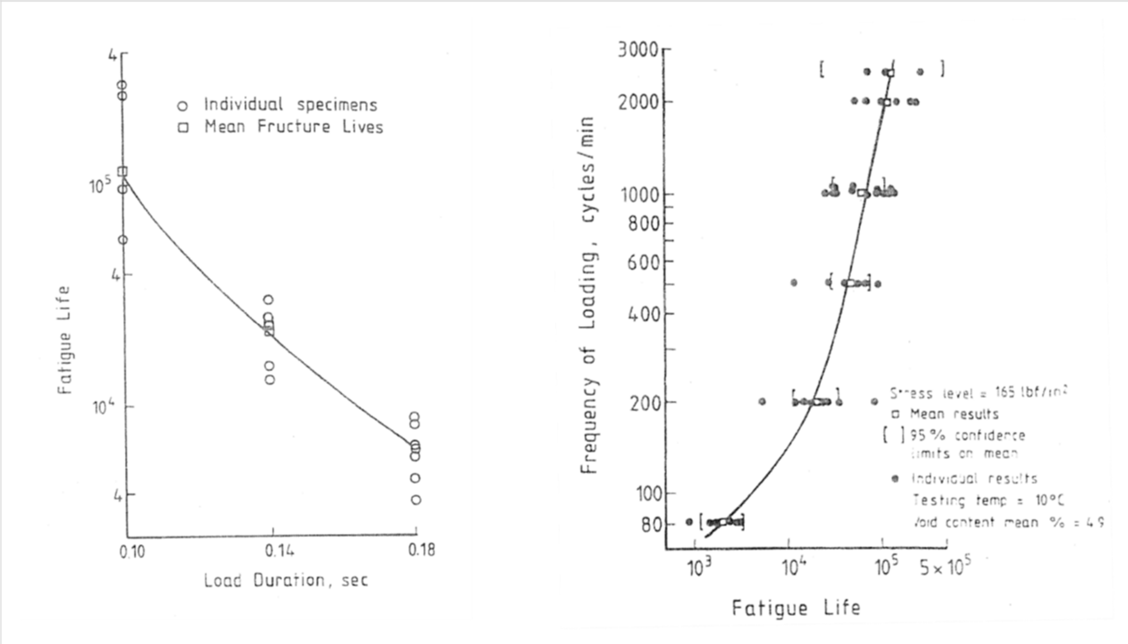


Fig. 3.24: effect of load duration and loading frequency on measured fatigue life of asphalt mixtures (Porter and Kennedy, 1975)



3.5.4 Fatigue law

The fatigue relationship of a bituminous material is assessed with any of the previously listed fatigue tests. The fatigue law consists in a relationship between the measured number of cycles to failure and the specified load (stress or strain depending on the test mode). The curve which represents this relationship is the material's Wöhler curve (Wöhler, 1852).

For controlled stress mode of loading, failure corresponds to the physical failure of the specimen. From the crack initiation, the duration of crack propagation is relatively short before to reach the physical failure, then the point of crack initiation is very well defined.

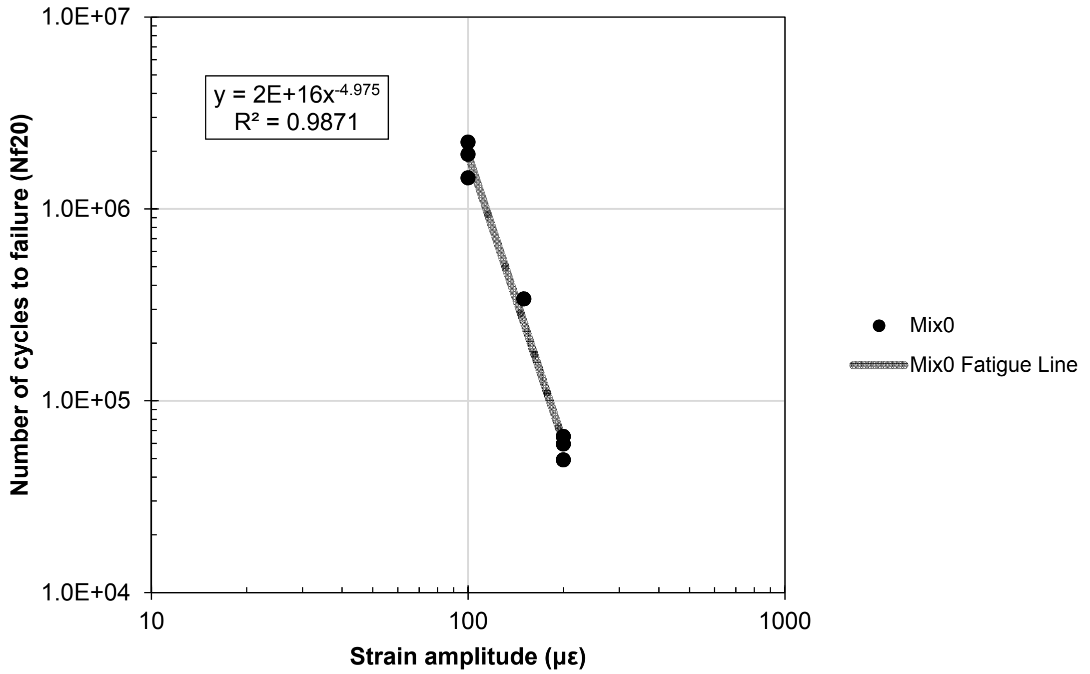
For controlled strain mode, the crack propagation phase may take long time before to break the specimen. Complete failure is not commonly reached when performing these tests, therefore an arbitrary definition of failure has to be made. The point of failure is normally defined as a reduction of the initial stiffness of 50%. Results are expressed with the following equations (fatigue law):

$$N_F = A\epsilon^B$$
$$N_F = C\sigma^D$$

The aim of laboratory fatigue testing is to collect enough data to in order to define a relationship between the specified test condition (initial stress or strain) and the measured fatigue life expressed as number of cycles to failure. A number of fatigue test is made for each test condition to determine the fatigue line of the investigated material. The two coefficients A and B (or C and D) of the previous equation are determined interpolating the measured data. For controlled-strain tests, three initial strain levels are defined and a number of asphalt specimen is tested at each strain level.

The number of specimen tested to fatigue depends on the availability of the material, budget limitations (fatigue tests are quite expensive compared with standard characterization tests) and specifications of standard references or technical documents. The EN 12697-24 specifies the minimum number of samples to test to fatigue for each test method.
The Wöhler line is a straight line in log-log coordinates. An example of fatigue curve for an asphalt material is given in Fig. 3.25.

Fig. 3.25: representation of the Wöhler fatigue curve of one of the asphalt materials tested in the present research. Coefficients A and B of the fatigue law are determined by interpolation of measured data



The traditional fatigue law should be intended as an empirical relationship between testing conditions (initial stress or strain) and number of cycles to failure.

3.5.5 Advantages and disadvantages of different fatigue test methods

Advantages and disadvantages of each test method have been discussed by Rao Tangella [56], Judycki [64] and Read [67] and are reported below.

Table 3.8: comparison of advantages and disadvantages of different fatigue test methods

Test method	Advantages	Disadvantages
Simple flexure	1. Methods are well known and widespread 2. The measured fundamental properties can be used for mixture evaluation and pavement design 3. Results can be used directly in structural design of pavements (i.e. elastic modulus), with appropriate shift factor	1. A shift factor is required for comparing laboratory and in situ behaviour of the material 2. The stress state is uniaxial unlike that within the pavement 3. In the three-point bending configuration, initiation of failure in a region of uniform tensile stress is not possible

	4. Failure of the specimen is initiated in a region of relatively uniform stress, especially in the four-point bending configuration	4. In the 3PB configuration the clamping effect adversely influences the accuracy of the method. If the clamping stress becomes too high, due to overtightening of the clamps, it is possible for failure to be initiated in the clamped region of the specimen
Direct axial loading	1. Stress/strain can be easily determined 2. Homogeneous stress distribution (direct access to constitutive law)	1. The test does not well duplicate the condition of stress and strain in the field 2. Heavy influence of surface conditions of the specimen at interface (sawing, gluing)
Diametral loading	1. Test is simple to conduct 2. The equipment can be used for other tests as the stiffness modulus or indirect tensile strength tests 3. The pulsating load can be applied with sufficient accuracy by a pneumatic actuator, which is less expensive and requires less maintenance than hydraulic actuator 4. Filed cores can be tested 5. Biaxial state of stress similar to the stress state occurring in situ 6. Sample preparation is easier than simple flexure tests (compaction of asphalt mixture in laboratory may be performed with gyratory compactor with limited sawing operations)	1. Permanent deformation occurs during testing. Therefore, the amount of load that actually contributes to the fatigue damage of the specimen cannot be accurately determined 2. Stress concentrations and distributions at the points of load application are very different compared to those occurring in situ 3. The magnitude of stress and strains in the diametral test are not generally measured but are calculated from elastic theory whereas in the other methods strains are directly derived from displacements 4. The fatigue life determined from this test has been found to differ significantly from that determined from other methods. Generally, shorter fatigue lives are found with this method
Supported flexure	1. Good simulation of field conditions 2. The support of the specimen reduces the effect of minor imperfections in the specimen	1. More time consuming than any other fatigue test 2. Test equipment is more complex compared to simple flexure
Fracture mechanics	1. The test provides the possibility of analysing the crack propagation of asphaltic layers over cracked bases (reflective cracking) 2. Testing time is short compared with other methods 3. Well established theoretical basis	1. Crack initiation is not explained by the test 2. The validity of the Paris law may not be fully questionable at high temperatures
Wheel tracking	1. Better simulation of field conditions than any other tests 2. Both crack initiation and propagation can be monitored 3. Special pavement structures can be tested	1. Extremely expensive 2. Large scattering in results

Simple flexure tests have similar stress distribution within the specimen. Differences exist amongst flexural tests in terms of stress distribution. A simple indication of stress and strain distribution in different flexural tests is reported in the following figures which represent a simulation of flexural testing with Abaqus.

Fig. 3.26: simulation of 2PB fatigue test and representation of strain developing in the specimen during the upward-downward loading

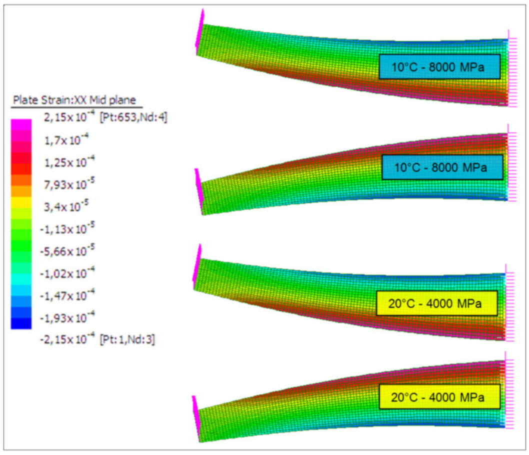


Fig. 3.27: simulation of 3PB fatigue test and representation of stress developing in the specimen during the upward-downward loading

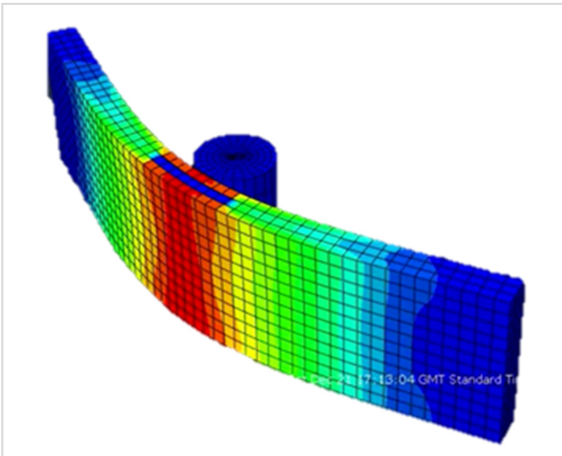
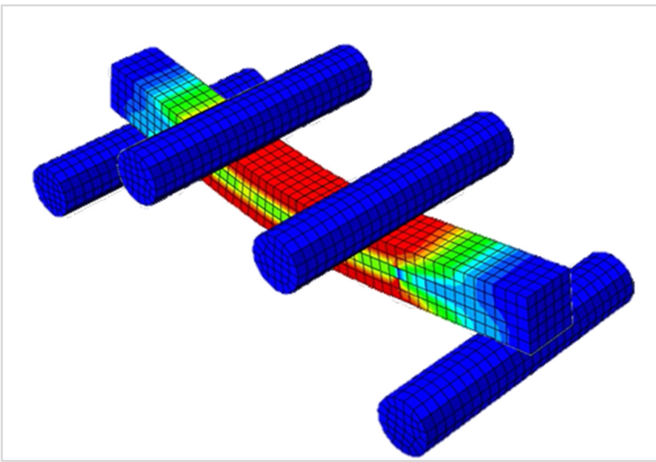


Fig. 3.28: simulation of 4PB fatigue test and representation of stress developing in the specimen during the upward-downward loading



The FEM analysis shows that 2PB and 4PB have a reasonably homogenous stress distribution in regions where failure is expected (red zones in Fig. 3.26, Fig. 3.27 and Fig. 3.28). The 3PB configuration has a limited region in which strains and stresses are homogeneous. Therefore, 2PB or 4PB test configuration should be preferred to the 3PB.

3.6 Four-point bending flexural fatigue

3.6.1 Test principle

The European Standard EN 12697-24:2012 annex D [75] specifies a method to characterise the behaviour of bituminous mixtures under fatigue loading using four-point-bending test equipment in which the inner and outer clamps are symmetrically placed and slender rectangular shaped specimens (prismatic beams) are used. The prismatic beams are subjected to four-point periodic bending with free rotation and translation at all load and reaction points. The bending is realised by loading the two inner load points (inner clamps) in the vertical direction, perpendicular to the longitudinal axis of the beam. The vertical position of the end-bearings (outer clamps) is fixed. This load configuration creates a constant moment, and hence a constant strain, between the two inner clamps. The applied load is sinusoidal. During the test, the load required to bend the specimen, the deflection and the phase lag between these two signals are measured as a function of time. Using these measurements, the fatigue characteristics of the material tested are determined.

The tensile strains (and stresses) that develop in the bottom region of the specimen during the fatigue test are similar to those in the bottom part of asphalt layers when a pavement is routinely subjected to traffic loading.

3.6.2 Testing equipment

The fatigue testing equipment is composed of the following elements:

- Loading system
- Four-point bending jig
- Data acquisition system
- Software (user interface)

3.6.2.1 Loading system (Universal Testing Machine)

The loading system is composed of the following elements:

- Servo-hydraulic universal testing machine (UTM-25): a 25kN capacity dynamic servo-hydraulic universal testing machine with interchangeable transducers, jigs, fixtures and other accessories, capable of applying sinusoidal loading frequencies up to 60Hz to asphalt specimen
- Hydraulic power supply: the unit controls the servo-hydraulic actuator
- Environmental chamber

Technical specification of each component are reported in tables below.

Table 3.9: UTM 25 technical specifications

UTM-25 Specifications	
Load Capacity	+/- 25kN Static, +/- 25kN Dynamic
Frequency	Up to 70Hz
Load Cell Low profile/Pancake type	+/-30kN capacity
Actuator Type	Double acting high precision labyrinth bearing or tie-rod actuator
Stroke	+/-25mm
In-built Displacement Transducer	50mm
Adjustable Cross-Head	Motorised
Crosshead Clamping	Hydraulic
Vertical Space	800mm
Space Between Columns	450mm
Size (HxWxD)	1,800x660x560mm
Weight	285kg (approx.)

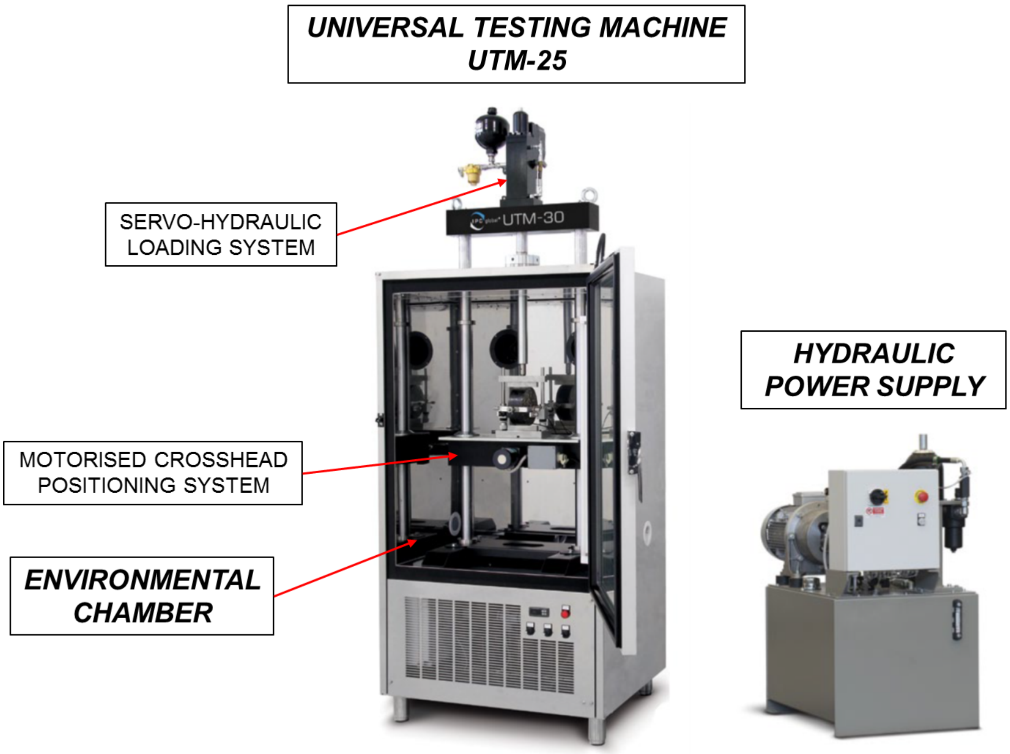
Table 3.10: hydraulic power supply specifications

HPS Specifications	
Flow Rate	5 litres/min
High Pressure	210 Bar
Low Pressure	50 Bar
Remote Starting	Available
Cooling System	Air-cooling
Mains Power	2.2kW single phase, 208-240V 50/60Hz
Oil Tank Capacity	28 litres
Size (HxWxD)	800x600x500mm
Weight	105kg (without oil)

Table 3.11: Environmental chamber technical specifications

Environmental chamber specifications	
Standard Range	-25˚C to +60°C
Extended Range	-50˚C to +80˚C
Temperature Accuracy	+/- 0.2˚C at centre of chamber near test specimen
Extended range maximum cooling rate	-30˚C/hour w/out ramp control Extended Range Programmable
Temperature Gradient	-20˚C/hour with ramp control

Fig. 3.29: loading system used for the four-point bending fatigue tests



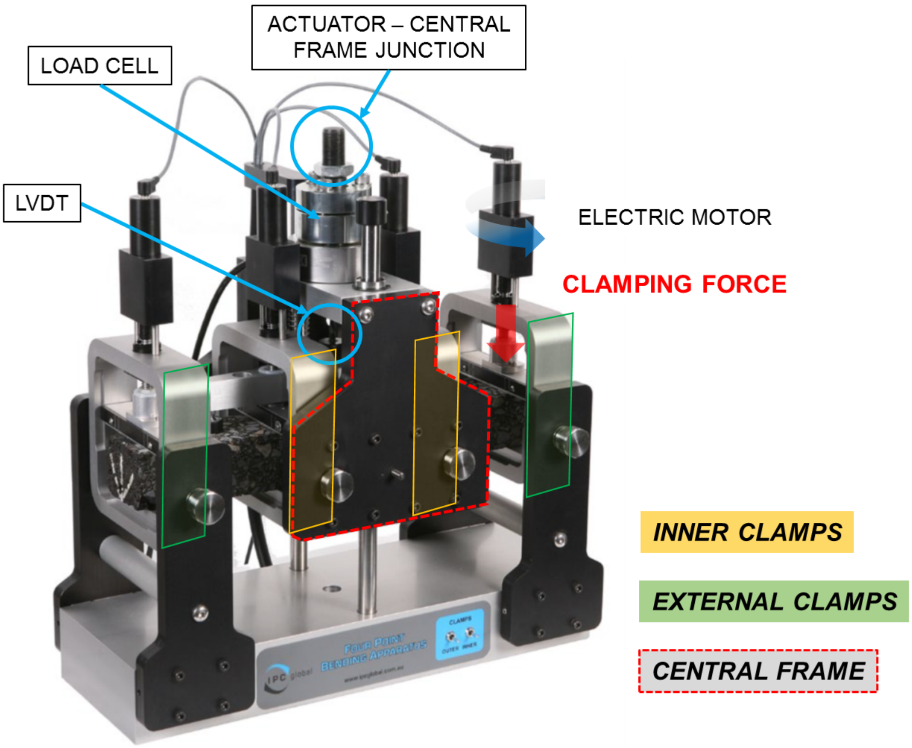
3.6.2.2 *Four-point bending jig*

The four-point bend jig is designed to subject an asphalt specimen to four point bending with backlash free rotation and horizontal translation of all load and reaction points.

The jig is composed of four clamping devices, or clamps. A clamping device is a device capable of clamping a specimen (beam) in the bending frame in order to provide horizontal translation and rotation freedom at all supports [75]. The outer and inner clamps shall be designed to permit rotation freedom and horizontal movements of the specimen within the clamps.

The external clamps are free to rotate on the horizontal axis and to move horizontally (x axis), acting as external support to the beam. The electrically motorized clamping system ensure the correct placement of the beam for the entire duration of the test. The internal clamps act as internal support similarly to external clamps. A central frame where the internal clamps are fixed permit the load transfer from the actuator to the specimen. Here is also located the housing of the Linear Variable Differential Transducer (LVDT) that measures the horizontal displacement at the centre of the asphalt beam. A representation of the 4PB components is given in Fig. 3.30.

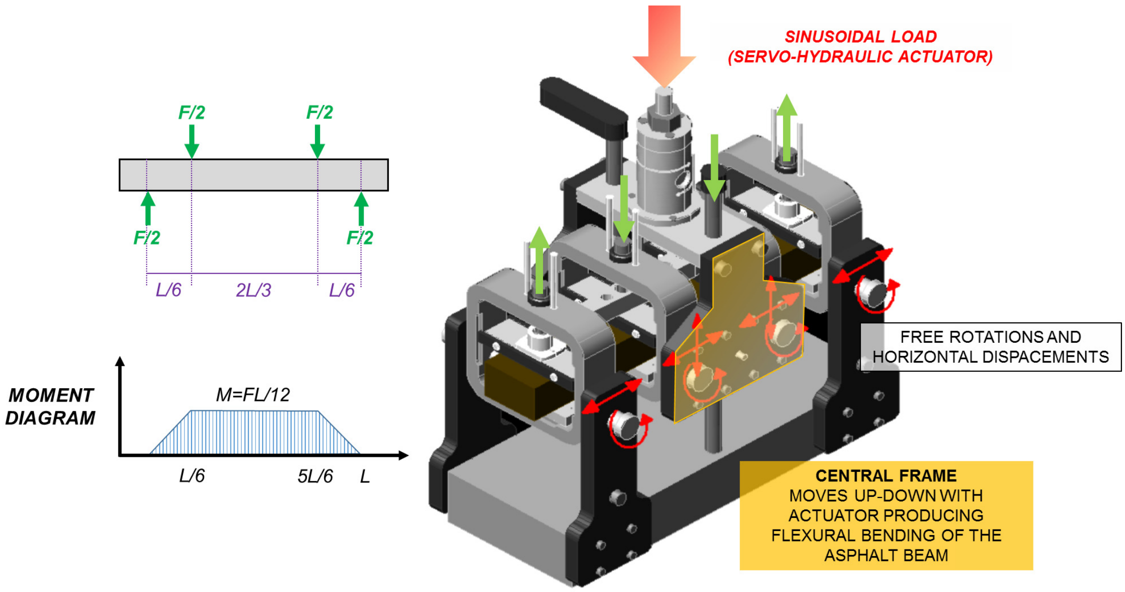
Fig. 3.30: components of the four-point bending jig



The four-point bending jig has two functions:

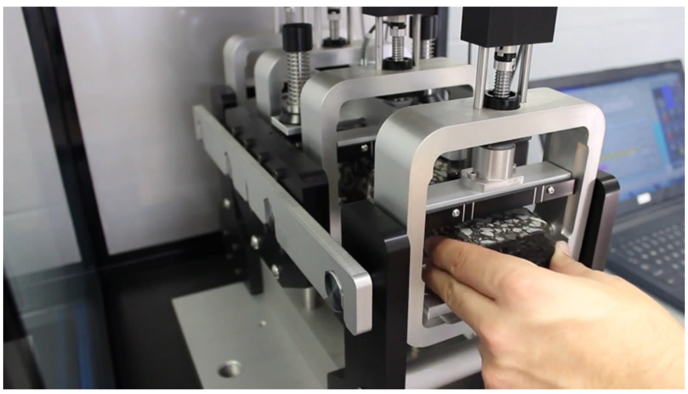
1. Ensures the correct placement of the asphalt beam recreating a simply-supported beam loaded in symmetric four-point bending. The specimen is laterally positioned by hand using etched lines as a visual guide for two specimen sizes, nominally 50 and 63.5 mm in width. Vertical clamping of the specimen is achieved by servo-motor driven ball screws which are operated continuously during the test to take up the compliance of the specimen at the clamping surfaces
2. Transmits the sinusoidal load from the actuator to the specimen through a vertical movement of the central frame. The two inner supports, which are part of the central frame, are subjected to the sinusoidal load of the hydraulic actuator. The supports are paired together and hence forced to move simultaneously upwards and downwards. This vertical movement tows the asphalt specimen and creates a constant moment within the sample between the two inner supports

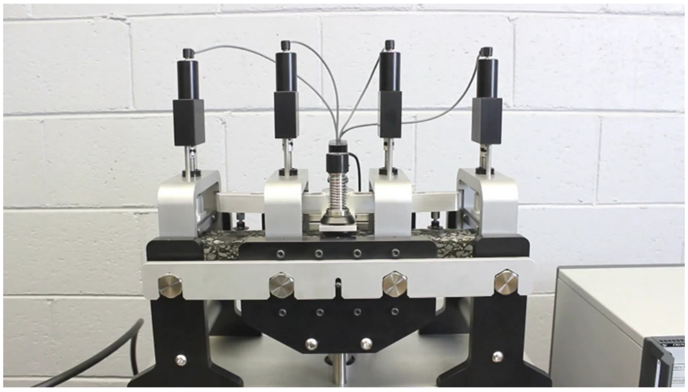
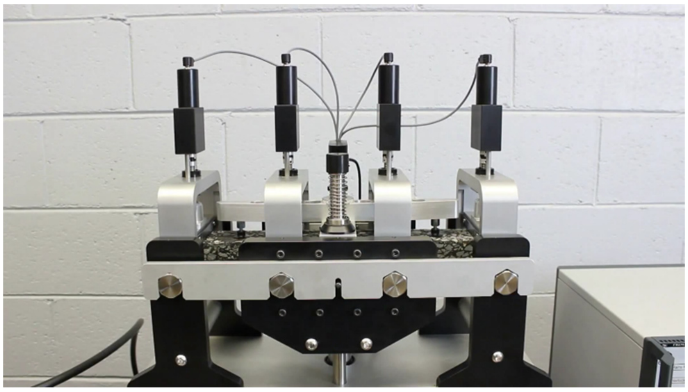
Fig. 3.31: four-point bending jig. The load frame reproduces the simply supported beam loaded in symmetric four-point bending, and transmits the sinusoidal loading from the actuator to the specimen though a vertical oscillation of the central frame



The correct placement of the asphalt specimen is a key factor in fatigue tests. First, good placement ensure that the static scheme of four supports beam is respected. In this case, all further calculations i.e. stress, strain and modulus, rely on realistic assumptions. Second, incorrect positioning of the specimen may cause excessive vibrations and hence unwanted motion of the asphalt beam. This will cause a significant scattering in single test results and lower repeatability of test series as well, because of the high sensitivity of LVDT (precision at 0.01 mm). Therefore, incorrect positioning of the specimen may frustrate accurate mixing and compaction procedures resulting in inaccurate and unprecise test results.

Table 3.12: procedure for positioning the asphalt beam into the four-point bending jig

	Clamps are completely lifted and the specimen is slid into the jig. The specimen is laterally positioned by hand using etched lines as a visual guide. A lateral aluminium bar is used to ensure horizontal alignment of beam support and hence of the beam.

	Once the specimen is placed in the correct position, clamping devices are ready to be activated, lowering the four horizontal plates that clamps the asphalt beam.
	In this position, clamps are completely lowered and horizontal plates lean on the upper surface of the beam. It's important to keep electric motors activated for ensuring am effective clamping during the test.
	The alignment with etched lines is verified after lowering the clamps on the beam. If the specimen is correctly placed, the lateral aluminium bar can be removed.

3.6.2.3 Data acquisition system IMACS

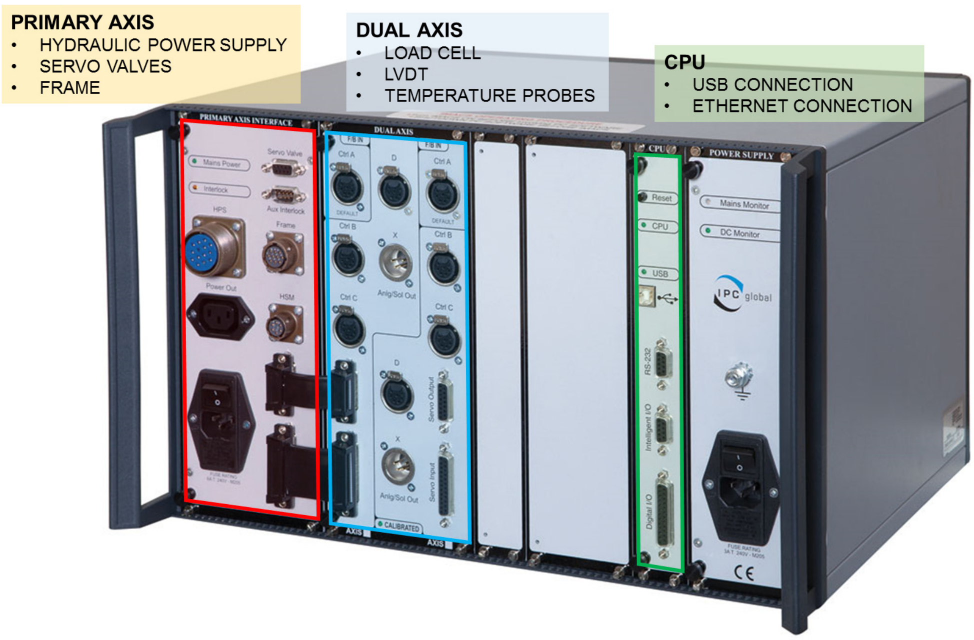
The Integrated Multi Axis Controls System (IMACS) is used to control the equipment i.e. LVDT, sensors, actuator, hydraulic power supply, and for the acquisition of test data. The external PC (user) and the machine hardware communicate through the IMACS. The unit convert the instruction given be the user through the software into electronic signals that are sent to the hardware components.

Table 3.13: specifications of the IMAC device

IMACS Specifications	
Configuration	Fully integrated
Real Time Digital Computer Control	32bit Processing
Acquisition Speeds	5kHz (simultaneous on all channels)

Data Over-Sampling	4x
Data Resolution	20bit effective auto-ranging data acquisition
Communication	USB 2.0 12MB/s
Ethernet	10/100MB/s
Firmware	Update Flash based
Analogue Inputs	Auto-calibrate on power up
Control	Up to 8 axis control
Acquisition	Up to 32 channels of data acquisition
Size	(HxWxD) 270x350x460mm
Weight	11kg
Mains Power	220-240V 50Hz, or 110-120V 60Hz

Fig. 3.32: IMACS unit for data acquisition and electronic control of the UTM

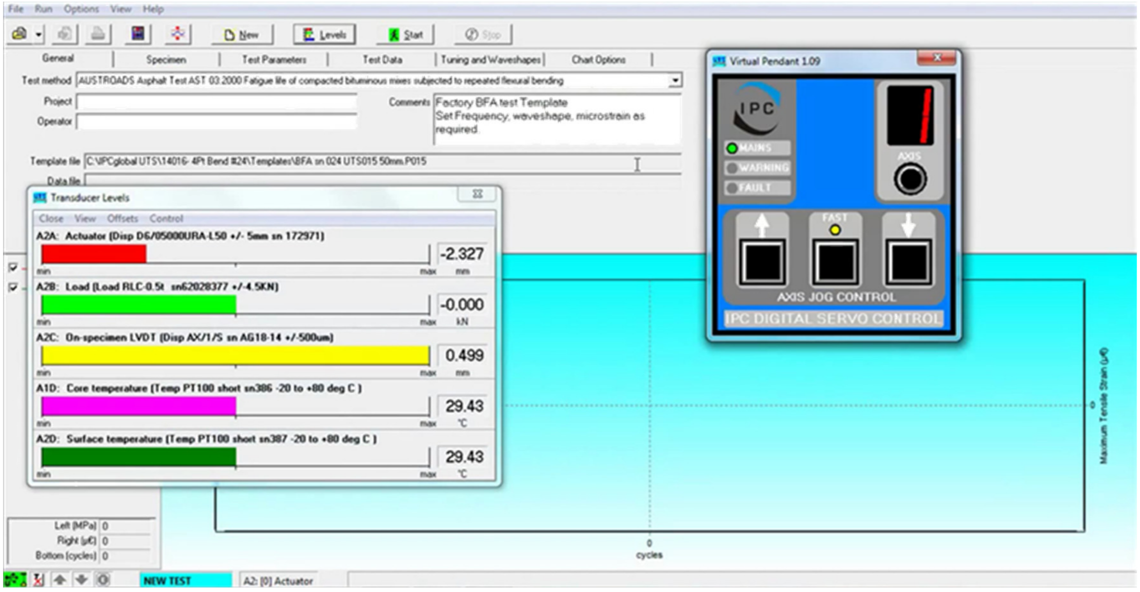


3.6.2.4 *Software UTS18*

The software Universal Testing System (UTS) 18 has been used in this research. The software is a user interface that allow the operator to properly set up and control the ongoing testing procedures.

A *virtual pendant* controls the position of the frame on which the 4PB jig is placed and the vertical movement of the hydraulic actuator. The signals of the active transducers (LVDT, temperature) are monitored and reported in the *transducer levels* window. The UTS18 also gives important information about loading, stiffness, hysteresis loop, measurements from transducers and sensors, calculated parameters as stress and strain.

Fig. 3.33: set up of a fatigue test with UTS18 user interface. Transducers levels and virtual pendant windows are showed



Under critical testing conditions, the hydraulic actuator may not be able to accurately apply a sinusoidal load. Low temperature (close to 0°C), high frequency (>40 Hz) and low strain level (<100 µε) can be considered as critical testing conditions. In these cases, the load signal recorded at each cycle shows significant noise and scattered trend.

The soundness of the sinusoidal load signal may be improved changing the “tuning parameters” of the machine commonly noted as proportional–integral–derivative controller (PID controller) through the UTS18. The PID controller has three principal control effects. The proportional (P) action gives a change in the input (manipulated variable) directly proportional to the control error. The integral (I) action gives a change in the input proportional to the integrated error, and its main purpose is to eliminate offset. The less commonly used derivative (D) action is used in some cases to speed up the response or to stabilize the system, and it gives a change in the input proportional to the derivative of the controlled variable. The overall controller output is the sum of the contributions from these three terms [76]. The UTS18 allow the operator to change PID enhancing the quality of the load shape.

Fig. 3.34: tuning and waveshapes control window of the UTS18

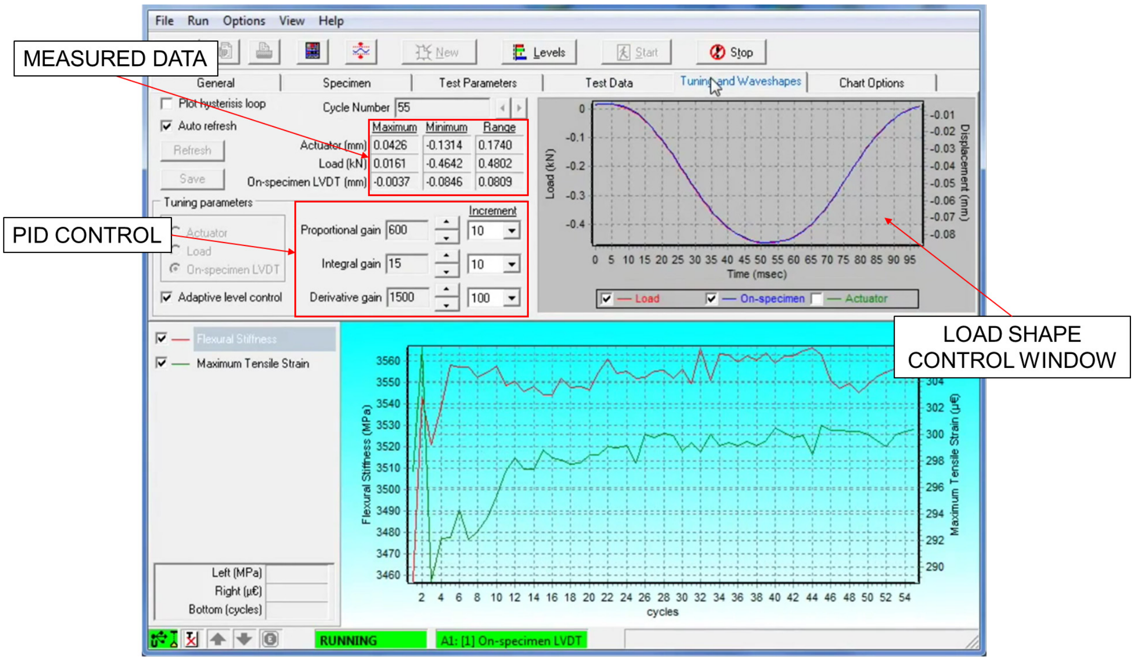
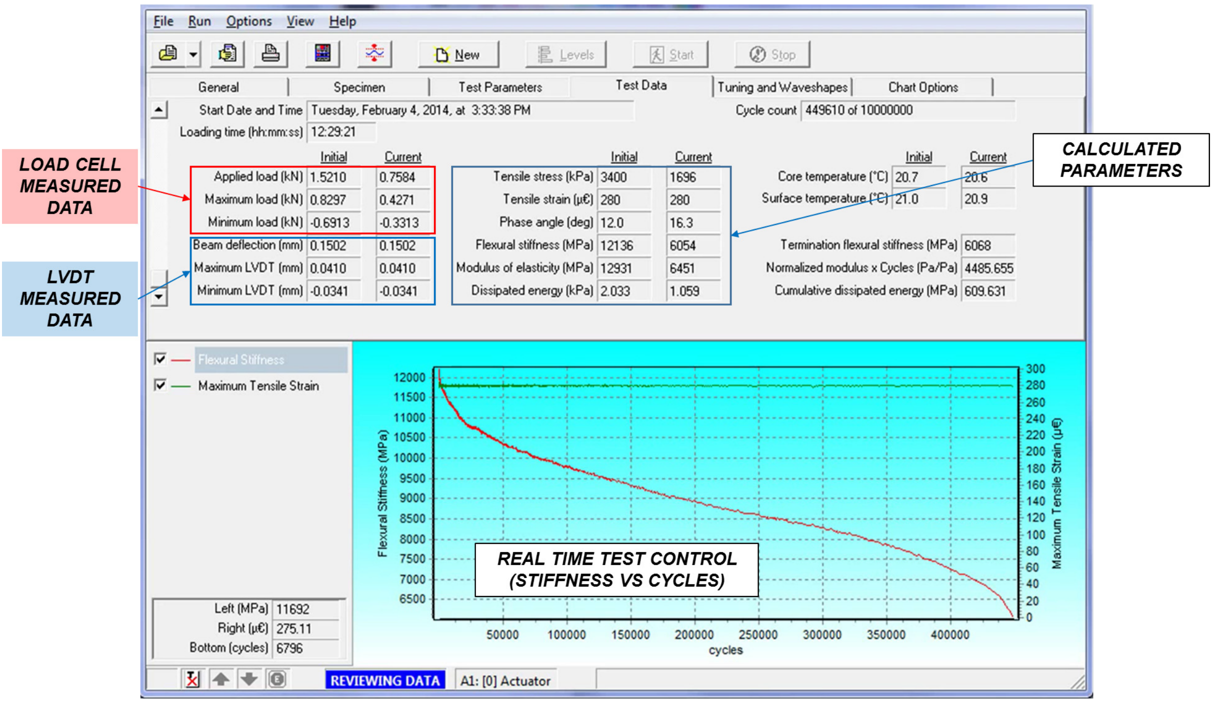


Fig. 3.35: test data control window of the UTS18. Information on measured and calculated parameters are given in real time

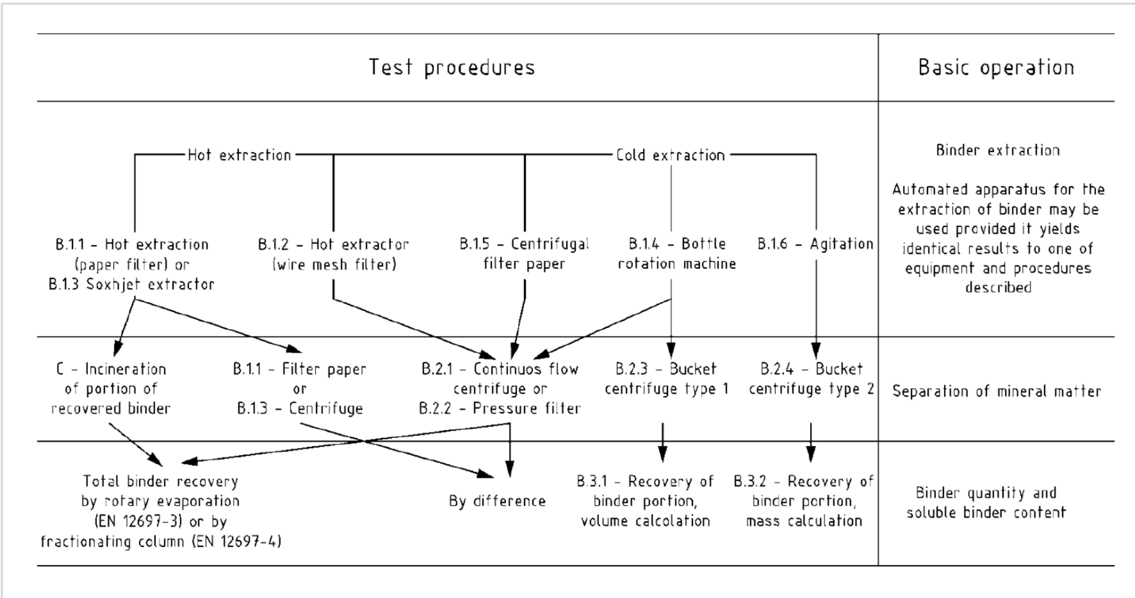


4 Review of laboratory testing on bitumen

4.1 Extraction and recovery

Accordingly to European standard EN 12697-1:2012, several test methods are available for determining the soluble binder content of asphalt mixtures. The test methods described in the European standard are suitable for quality control purposes during the production of plant mix and for checking compliance with a product specification [77]. Some test methods allow the recovery of extracted binder for further investigations. A summary of test procedures is reported in Fig. 4.1.

Fig. 4.1: test procedures for bitumen extraction described in the European standard EN 12697-1:2012



Two extraction procedures were used in this research:

1. Hot extraction, wire mesh filter method, separation of filler with continuous flow centrifuge
2. Cold extraction, centrifugal filter paper

4.1.1 Hot extraction with wire mesh filter method

This test method was used for determining the soluble binder content of samples of asphalt mixture. The soluble binder content is defined as proportion of extractable binder in an anhydrous sample determined by extracting the binder from the sample [77], whereas the insoluble binder content is the proportion of binder that adheres to the aggregate after extraction.

The testing procedure consists in dissolving the asphalt binder in a hot solvent (trichloroethylene) until no soluble binder is left adhering to the aggregate particles after the extraction. A sample of hot asphalt (140°C) is put into a dried and clean wire basket 63 μm mesh opening. The solvent (trichloroethylene) is poured into a glass jar and the basket is suspended in the jar by a supporting ring. A metallic condenser is put on top of the jar and all the apparatus is placed on a hot plate. The solvent evaporates and moves upwards to the cold metallic plate. Here, it condenses and drop on the asphalt sample in the wire basket, separating bitumen from aggregates. A blend of bitumen, solvent and filler with particle size smaller than the 63 μm mesh is gathered in the bottom of the jar. The filler, or ash, passing through the 63 μm basket has then to be separated from the bitumen/solvent solution using an appropriate centrifuge extractor.

The bitumen content is calculated by difference from the weight of extracted aggregates with the following formula:

$$b_a(\%) = 100 \cdot \frac{M_{mix} - M_{agg} - F}{M_{agg} - T + F}$$

$$b_c(\%) = 100 \cdot \frac{M_{mix} - M_{agg} - F}{M_{mix} - T + F}$$

Parameters b_a and b_c represent the soluble binder content as percentage by mass of aggregates or asphalt mixture, respectively. M_{mix} is the mass (g) of basket and asphalt sample, M_{agg} is the mass of basket and aggregates after extraction, F is the mass of the recovered filler (particles passing through the 63 μm mesh), T is the mass of clean and dry basket.

The procedure of separation of the mineral filler from the bitumen/solvent solution is the most awkward operation. The sources of error may be:

- Incorrect rotation of centrifuge apparatus resulting in a lower centrifugal acceleration (required $3 \cdot 10^4$ m/s^2)
- Control of the solvent flow rate of $100 \cdot 10^{-6}$ m^3/s. Higher flow rate may cause a loss of mineral matter resulting in overestimation of binder content
- Weighting procedures are not performed at environmental temperature causing measurement errors

Incorrect procedure or measurement errors may lead to significant errors in the determination of soluble binder content up to 1%.

4.1.2 Cold extraction with centrifugal filter paper

Cold extraction method was used in the present research for two purposes:

- Determine the bitumen content of asphalt specimen previously tested to fatigue
- Obtain a sample of the extracted bitumen for further investigations

The apparatus consists in a removable precision-machined rotor bowl housed in a cylindrical aluminium box rotating at speed up to 3600 rpm. The rotating unit is suspended on the base by four calibrated springs, which assure a perfect stability all over the test. The cover is precisely machined and fit with solvent resistant gasket to avoid leakages. Before performing the test, a quantity of 1200 g to 2000 g of loose asphalt is put at environmental temperature into a bowl with 250 ml of solvent (Dichloromethane CH_2Cl_2). Here, the sample decants for some hours. The content of the bowl is poured into the removable rotor bowl, a paper filter is applied on top of the bowl and the centrifuge is covered. Three washings with 150 ml of dichloromethane are performed. At the end of the test, a solution with bitumen and solvent is stored into special glass jars. The binder content is determined by difference from the weight of the asphalt mixture and the weight of extracted aggregates.

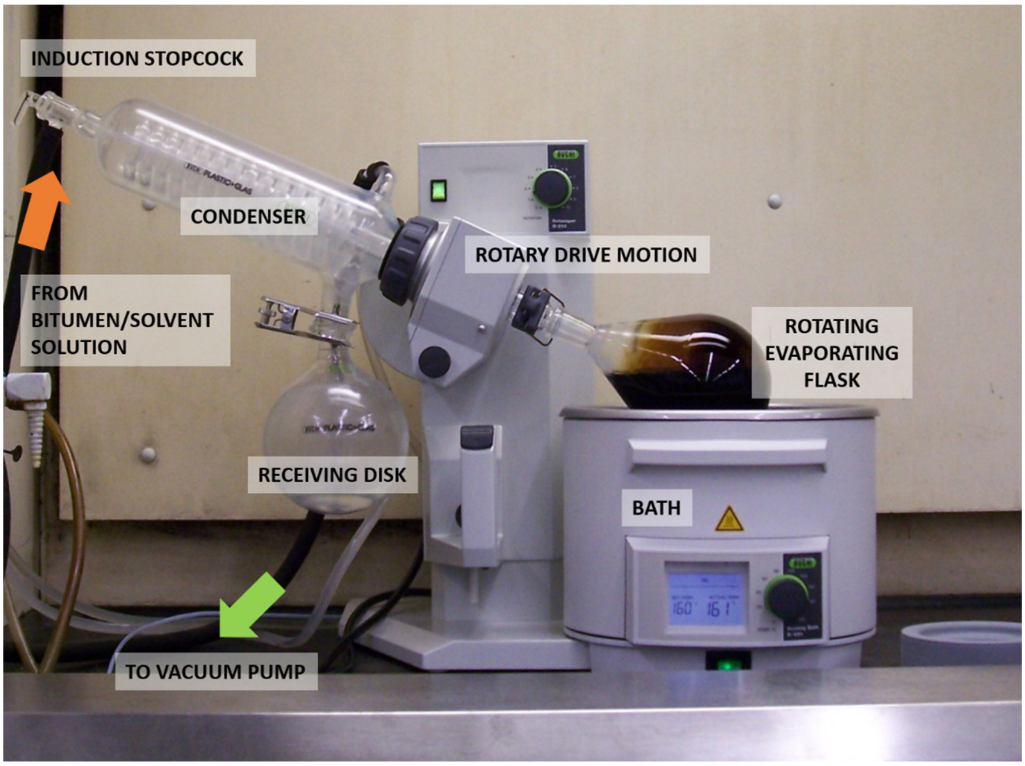
Testing methodologies for binder extraction are usually aimed to determine the soluble binder content of asphalt samples. In this research, the main purpose of the cold procedure is to separate bitumen from aggregates with minimal ageing effect of the solvent on properties of bitumen.

4.1.3 Bitumen recovery with rotary evaporator

The test is performed for the recovery of soluble bitumen from bituminous mixtures in a form suitable for further testing. The test can be undertaken on either loose or compacted asphalt materials. The procedure is suitable for the recovery of paving grade bitumens, for which materials this European Standard is the reference method [78].

During operation, the solution of solvent and bitumen from a prior extraction is distilled by partially immersing the rotating distillation flask of the rotary evaporator apparatus in a heating oil bath while the solution is subjected to high vacuum with fine regulation of pressure (up to ±0.1 kPa) according to EN 12697-3:2013.

Fig. 4.2: Typical rotary evaporator



The rotary evaporator was preferred to other recovery methodologies as Abson procedure for the following reasons:

- Repeatability: rotary evaporator is more repeatable than the Abson procedure. Researches show that physical properties of RAP bitumens recovered with Abson method exhibit high coefficient of variation (CV) ranging from 38% to 69% [10], whereas a CV of 12% was found for rotary evaporator method;
- Hardening of solvent on bitumen: the solvent hardens the asphalt binder. The Abson method was found to leave more solvent in the recovered binder than the rotary evaporator [79]. Greater hardening was observed with longer exposure periods and higher temperatures. Additional amount of solvent could be recovered with Abson through longer recovery times and higher temperatures, but this would result in greater solvent hardening. Experimental studies report that solvent hardening occurs to about the same degree in most solvents [80]. The use of rotary evaporator should reduce the hardening effect of solvent compared with the Abson.

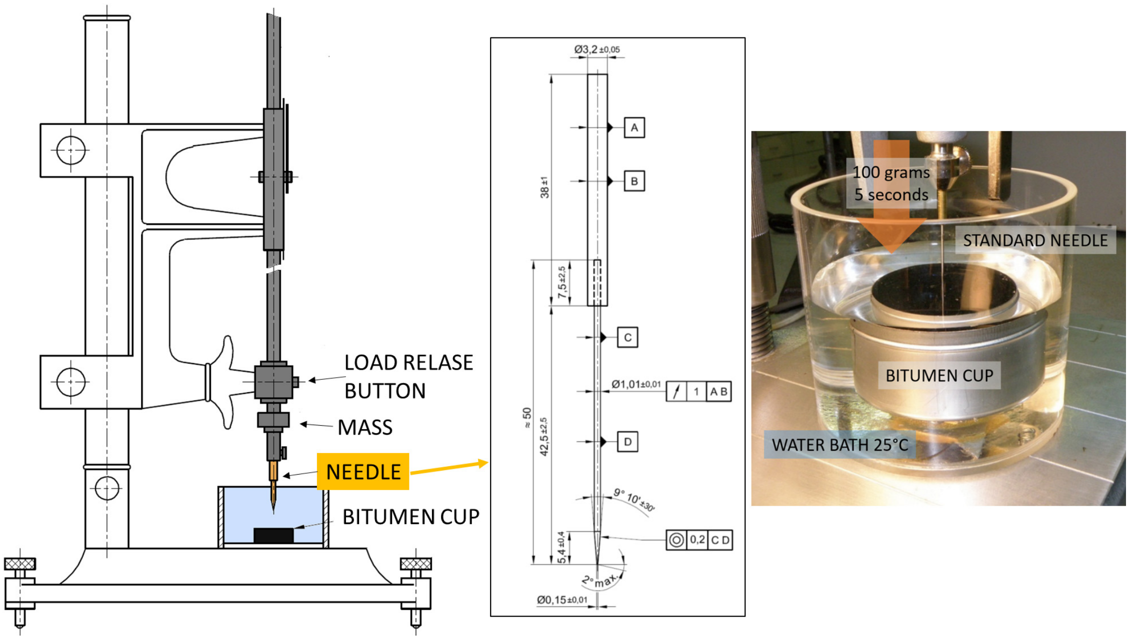
4.2 Standard specification tests

4.2.1 Penetration test

The penetration test is a method to measure the consistency of a bitumen at intermediate service temperatures. The penetration is defined as the distance (in decimillimetre) that a standard needle is allowed to penetrate vertically into a bituminous sample, under a known load of 100 g, at a fixed temperature of 25°C, for a known time of 5 seconds. The higher the penetration, the softer the bitumen and vice versa.

Penetration is the conventional measure upon which bitumen are classified into standard penetration grades. Although penetration is not a fundamental parameter of the material, it is possible to estimate important engineering properties from test results as viscosity and stiffness [81].

Fig. 4.3: penetration test apparatus **[82]** with geometrical specifications of standard needle and test configuration reported on the right



The test is performed accordingly to EN 1426:2007. The test procedure is simple, but every step shall be carried out with care in order to obtain accurate results. Needles must be checked regularly for straightness, profile and cleanliness (they shall be cleaned after each penetration). Because of the temperature-dependent nature of bitumen, temperature control is critical. For this reason, the bituminous sample shall be placed into a thermostatically controlled water in order to ensure homogeneity of temperature within the specimen. The tolerance is ± 0.1°C. Bituminous sample shall be heated above the softening point and placed into the penetration cup. Hence, bitumen shall be heated again for an hour at temperatures of 120-140°C in order to send air out of the sample.

Penetration is calculated from three individual measurements made under the specified testing conditions. Between each test, the needle is cleaned or replaced and the sample container is rotated so that a different portion of the sample is tested. For the most common bitumens (penetration ranging from 50 to 150 dmm), the three measures shall not differ more than 4. Repeatability and reproducibility for above mentioned testing conditions are specified as follows [82]:

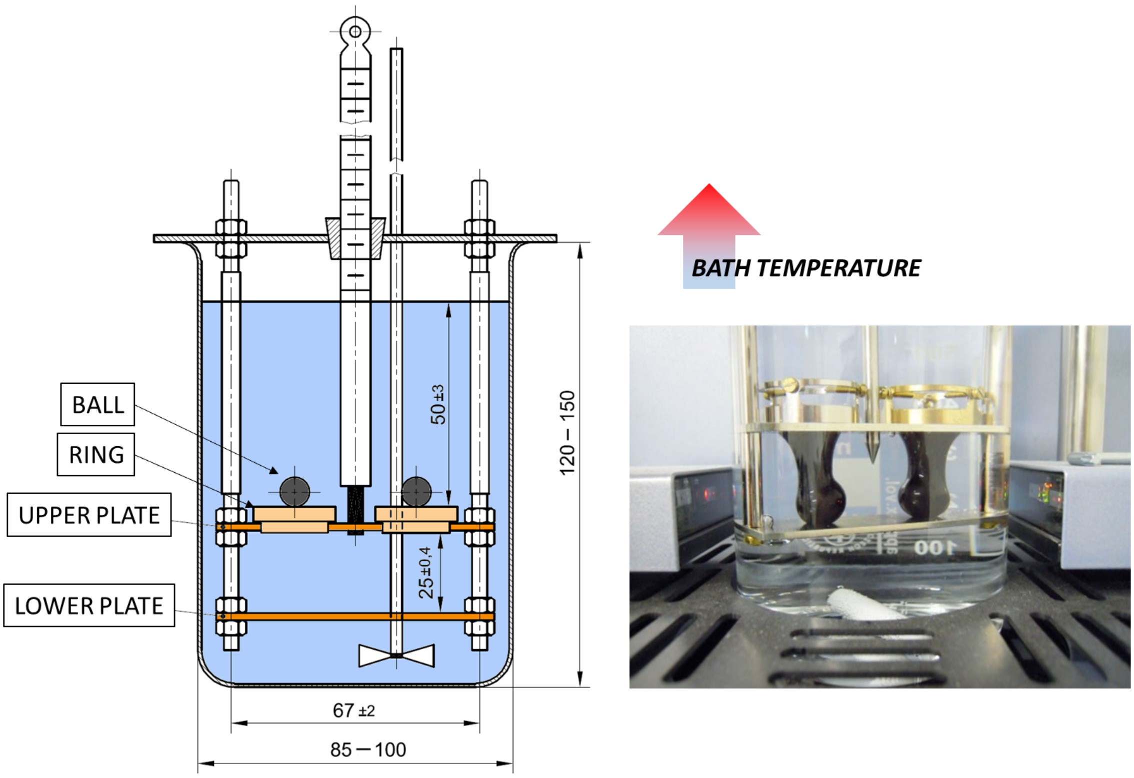
- Pen ≥ 50 dmm: repeatability (r) 4% of average; reproducibility (R) 6% of average
- Pen < 50 dmm: r = 2; R = 3

4.2.2 Softening point test

The softening point is a temperature at which the consistency of bitumen is between solid and liquid behaviour, and represent an equi-viscous temperature [81]. Therefore, the softening point measures the consistency of bitumen at elevated service temperatures.

In this test, performed according to EN 1427:2007, a 3.5 g steel ball is placed on a sample of bitumen contained in a brass ring that is then suspended in a water or glycerine bath. Water is only used when the softening point of the sample is below 80°C. The bath temperature is raised with a temperature gradient of 5°C/min [83]. Temperature is measured by a sensor placed in a middle position. A magnetic stirrer with adjustable speed assures temperature uniformity in the bath. The bitumen softens and deforms slowly with ball through the ring. At the moment the bitumen and the ball touch the 25 mm base plate below the ring, temperature of the bath is recorded.

Fig. 4.4: determination of softening point with ring and ball apparatus. Instrumentation and test principle reported on the right



The temperature is recorded for each bitumen/ball, then the softening point is determined as the average between the two measures reported to the nearest 0.2°C. The test shall be repeated if the two temperatures differ more than 2°C or if the ball breaks the bitumen film before touching the lower steel plate.

Repeatability and reproducibility of softening point measures are expressed in °C for liquid of the bath and bitumen type, as reported in Table 14.

Table 14: repeatability and reproducibility limits reported in EN 1427:2007 for bitumens with softening point below 80°C

Fluid in the bath	Bitumen type	Repeatability r (°C)	Reproducibility R (°C)

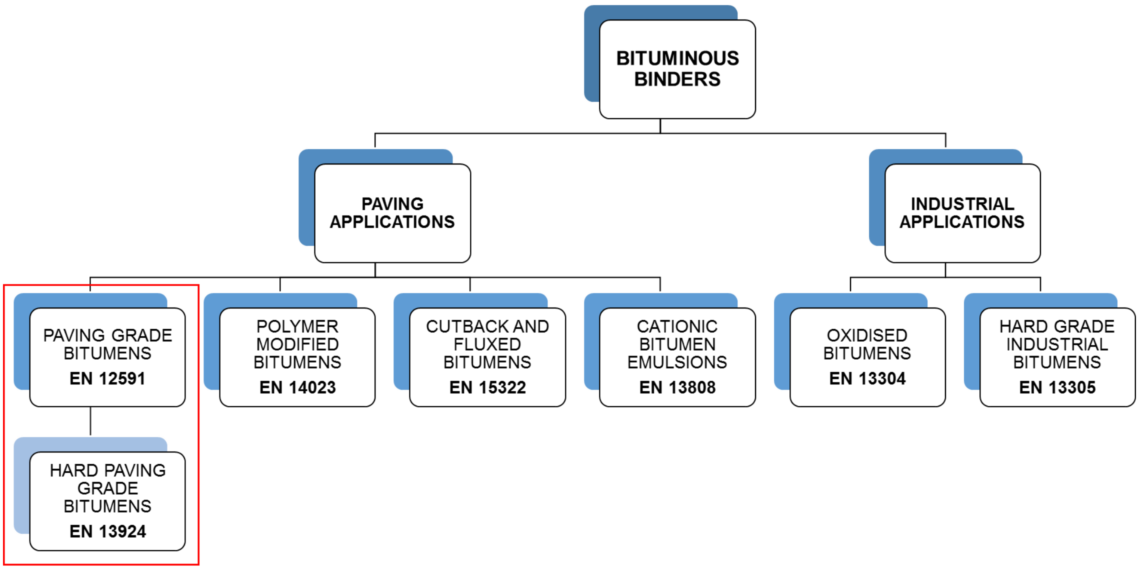
Water	Unmodified	1	2
Water	Modified	1.5	3.5
Glycerine	Aged	1.5	5.5

Statistical data of repeatability and reproducibility for bitumens with softening point greater than 80°C are not available.

4.3 European specifications for paving grade bitumens

Several types of bitumens for different applications are available on the market as paving grade bitumens, polymer modified bitumens (PMB), multigrade bitumens, bitumen emulsions and fluxed bitumens among the others. Since physical properties and applications may differ significantly from a bitumen type to another, properties and relevant test methods are reported in different European Standards for each bitumen type, as indicated in Fig. 4.5.

Fig. 4.5: European Standard references for different types of bitumens and applications



Paving grade bitumens used in road construction are classified into grades according to the penetration value measured at 25°C, with reference to the European Standard EN 12591. The pen-graded system provides a means to classify bitumens with the empirically-based test methods reported in Table 15 [84] [85].

Table 15: properties and related test methods for paving grade bitumens according to requirements of European Standards EN 12591:2009 "Bitumen and bituminous binders-specification for paving grade bitumens"

Test parameter	Requirement	Test method
Penetration at 25°C	Consistency of bitumen at intermediate temperatures	EN 1426
Softening point	Consistency of bitumen at high temperatures	EN 1427
Fraass breaking point	Brittleness of bitumen	EN 12592
Resistance to hardening		
Rolling Thin Film Oven Tests (RTFO T)	Short term ageing of bitumen	EN 12607-1
Retained penetration	Durability – consistency at intermediate temperatures after ageing	EN 1426
Increase in softening point	Durability – consistency at high temperatures after ageing	EN 1427

Other properties that may be specified for paving grade bitumens are: change of mass (EN 12607-1 or -3), flash point (EN ISO 2592) and solubility (EN 12592). Viscosity specifications may be required as well.

Testing procedures are simple and affordable, and results can be correlated with pavement performances thanks to an ample case history. The pen-grade is the most used system for classifying bitumens in Italy and in Europe as well.

Designation is by penetration range only, e.g. a 50/70 paving grade bitumen would be a binder having a minimum penetration value of 50 decimillimetre and a maximum penetration value of 70 decimillimetre at 25°C.

The reason for performing penetration tests and hence classifying bitumens referring to a temperature of 25°C is simple. At the intermediate temperature of 25°C, the bitumen exhibits the characteristics of both viscous liquids and elastic solids. When heated, bitumen acts as a lubricant, allowing the aggregate to be mixed, coated and tightly compacted to form a smooth and dense surface. After cooling, the asphalt acts as the glue to hold the aggregates together in a solid matrix. In this finished state, the behaviour of the bitumen is viscoelastic, meaning it has both elastic and viscous characteristics, depending on temperature and mode of loading. Therefore, penetration at 25°C gives an empirical indication of the viscoelastic behaviour of the bitumen.

The softest penetration grade bitumen is moderately firm at room temperature. At this temperature, a gentle pressure indents the surface of the sample. The hardest pen-graded bitumen is of a consistency that permits only a slight thumb print under firm pressure when the material is at room temperature [38].

As reported in the EN 12591, paving grade bitumens are divided into three categories [84]:

a. Paving grade bitumens specified by penetration at 25°C (from 20 dmm to 330 dmm) and softening point
b. Paving grade bitumens specified by penetration at 25°C (from 250 dmm to 900 dmm), penetration at 15°C and dynamic viscosity at 60°C
c. Paving grade bitumens specified by cinematic viscosity at 60°C (soft bitumens)

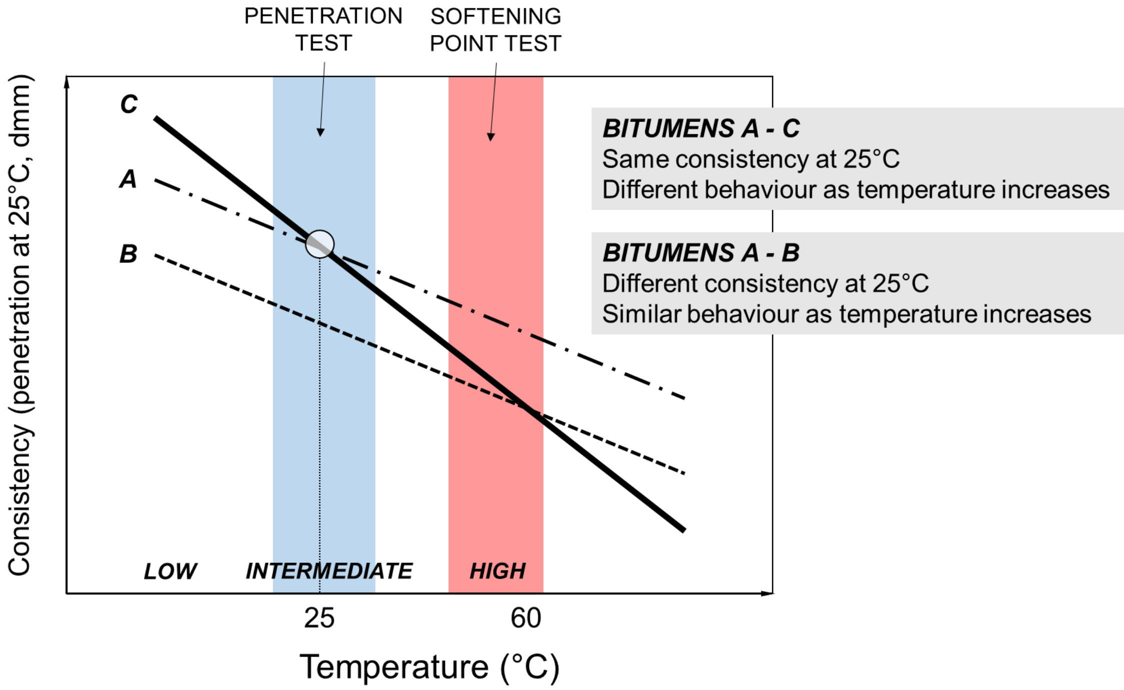
The paving grade bitumens at point "a" are used for the majority of road applications. Reference values of empirical test parameters for the most common paving grade bitumens used in Italy are reported in Table 16.

Table 16: typical values of penetration, softening point and Fraas for different types of penetration graded bitumens, EN 12591

Test parameter	Unit	Standard	Grade designation			
			35/50	*50/70*	*70/100*	*160/220*
Penetration at 25°C	0.1 mm	EN 1426	35-50	50-70	70-100	160-220
Softening point	°C	EN 1427	50-58	46-54	43-51	35-43
Fraass breaking point	°C	EN 12593	-5	-8	-10	-15
Retained penetration after hardening, minimum	%	EN 1426	53	50	46	37
Softening point after hardening, minimum	%	EN 1427	52	48	45	37

Because of the empirical nature of standard specification tests, the relationship between the test result and the actual performance may not be always accurate, depending on the quantity and the quality of measured data [38]. As conceptually illustrated in Fig. 4.6, the actual performance could be significantly different for bitumens with the same paving grade. For this reason, additional tests are recommended when the mechanical behaviour of bitumens under specified conditions need detailed investigation.

Fig. 4.6: different performances of bitumen with the same paving grade i.e. same penetration at 25°C



Thus, the penetration-graded system measures stiffness at an intermediate and a high temperature, which respectively correspond to 25°C and to the softening point of the bitumen. The stiffness of the asphalt binder at different temperatures cannot be directly measured and may only be inferred from the available data. In addition to the previous considerations, short term ageing effects are captured through the use of the RTFOT. However, there is no provision to conduct long term ageing of bitumens to simulate what happens to the asphalt stiffness after being in a pavement for a number of years [38]. Therefore, as already noted, in addition to the principal tests for defining grade of bitumen, several other tests may be required to determine specific properties.

4.4 Viscosity measurements with rotational viscometer

4.4.1 Scope and principle

Rotational viscometer is used to determine the viscosity of bitumens at application temperatures and at a given shear rate.

Viscosity is determined by measuring the torque required to maintain a constant rotational speed of a cylindrical spindle that is submerged in a bituminous sample at a constant temperature. The torque is directly related to the asphalt binder viscosity, which is calculated automatically by the viscometer. The viscometer can also be used to determine viscosity versus temperature charts for determining optimal mixing and compaction temperatures.

In this research, a Brookfield DV-II + Pro Extra rotational viscometer was used for viscosity measurements. The test was performed with reference to the EN 13702 "Determination of dynamic viscosity of modified bitumen – Part 2: coaxial cylinders method". Although the method has been developed for modified binders, it is also suitable for other binders [86].

4.4.2 Testing procedure

A sample of bitumen is poured into a preheated beaker. In this phase of testing, temperatures shall be such that the bitumen has a fluid consistency. Overheating of samples may cause unnecessary short-term ageing of bitumen. The amount of bitumen used is 8 to 12 grams depending on type and size of the spindle. The beaker containing the bitumen sample is placed in the preheated thermosel system operating at the proper test temperature. The preheated spindle is lowered into the sample, and the bitumen is ready to test when the temperature stabilizes.

The rotational viscosity test is started by activating the motor so that the spindle turns at a specified rate. As the liquid resists the rotation, the torque is measured by a spring that winds up while rotating inside the instrument. A digital viscometer has an electronic sensing system that reads the spring deflection. The measurement range of a rotational viscometer (in centipoise or milliPascal seconds) is determined by the rotational speed of the spindle, the size and shape of the spindle, the container the spindle is rotating in, and the full scale torque of the calibrated spring. A waiting period of 15 minutes may be needed to obtain a stable reading of viscosity [38].

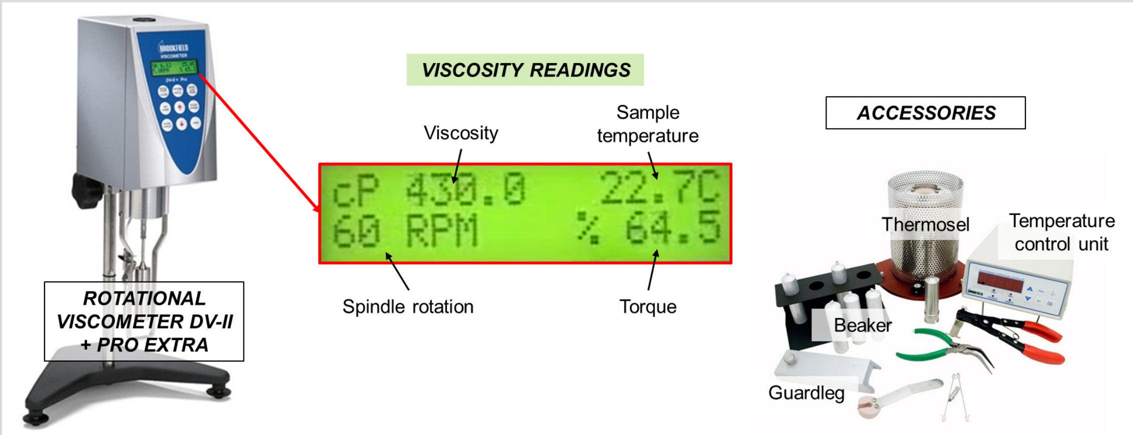
During the test, the viscosity reading and the percentage of torque can be observed on the digital display. Data are considered to be on-scale for torques 10–80% of full-scale range (FSR). FSR corresponds to 100%. As the percentage of torque is less than 10% or greater than 80%, test conditions shall be changed: spindle rotation shall be reduced if the torque increases above 80%, or increased if the torque reduces below 10%. If adjusting the spindle rotation would be ineffective, a different size spindle may be required. The digital output of some rotational viscometer is in units of centipoise (cP). The conversion of cP to the international unit Pascal-seconds (Pa·s) is given by the equation 1000 cP = 1 Pa·s.

Specifications of the Brookfield viscometer and a representation of the testing apparatus are reported in the following Table 17 and Fig. 4.7.

Table 17: specifications of the Brookfield DV-II + Pro Extra rotational viscometer used in this research

Spindle rotation range	01 - 200 rpm
Rotation control	0.01 rpm increments from 0.01 to 0.99 rpm
	0.1 rpm increments from 1.0 to 200 rpm
Temperature Sensing Range	-100˚C to 300˚C (-148˚F to 572˚F)
Viscosity Accuracy	±1.0% of full scale range
Viscosity Repeatability	±0.2%
Temperature Accuracy	±1°C | -100°C to +149°C
	±2°C | +150°C to +300°C

Fig. 4.7: representation of the rotational viscometer and accessories used for viscosity measurements



4.4.3 Measured and calculated parameters

When performing viscosity tests with rotational viscometer in controlled shear rate conditions, rotational speed of the spindle is set and the torque applied by the viscometer to the bituminous sample is measured as raw data. Rheological parameters $\dot{\gamma}$, shear rate (s^{-1}), and τ, shear stress (Pa), are calculated accordingly to the two-plates model. Hence, viscosity is calculated for ideally viscous or Newtonian fluids at a constant temperature with the following equation:

$$\eta = \frac{\tau}{\dot{\gamma}} \; [Pa \cdot s]$$

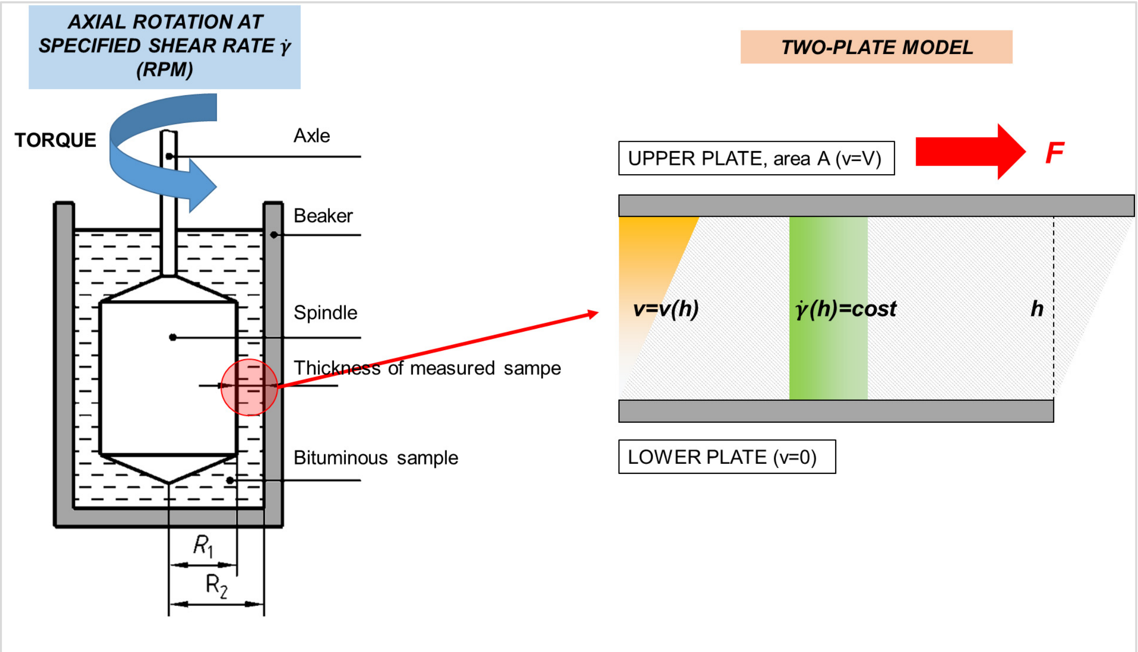
In the two-plate model, a fluid (bitumen) is sandwiched between two parallel plates, with area "A", at distance "h" one to the other. The fluid is assumed to show perfect adhesion to both plates with no wall-slip effects and it's assumed to flow in laminar conditions. If the upper plates moves with speed v with the lower plate being stationary (v=0), and the flow speed of the fluid linearly increases with distance h from the lower plate towards the upper plate, then the rheological parameters can be written as:

$$\dot{\gamma} = \frac{dv}{dh} = \frac{v}{h} \; [s^{-1}]$$

$$\tau = \frac{F}{A} \; [Pa]$$

A representation of the parameters and raw data measured during the test is given in Fig. 4.8.

Fig. 4.8: representation of the two-plates mathematical model used to calculate rheological parameters and dynamic viscosity of investigated fluid (bitumen) with rotational viscometer



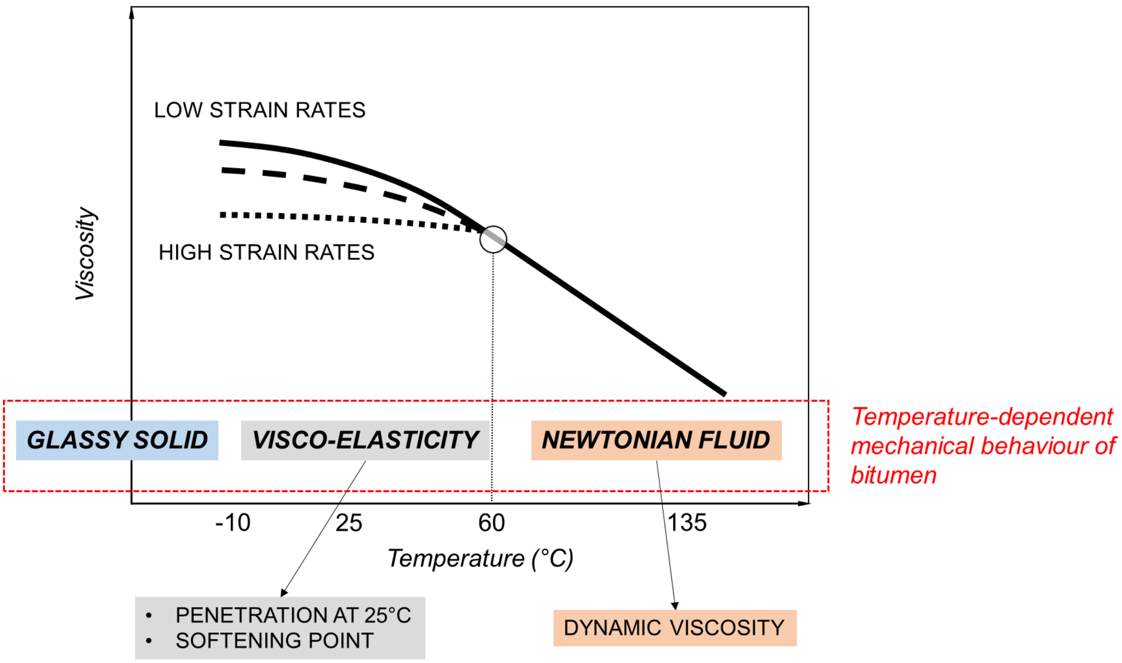
Parameters calculated with the two-plate model are valid under the assumption of Newtonian fluid, or ideally viscous. Bitumen is a thermoplastic material that behaves as a viscoelastic material. The viscoelastic behaviour can result in the bitumen behaving either as an elastic solid or a viscous fluid, depending on temperature and time of loading [87]. The viscoelastic response of bitumen may be generally divided into three zones of behaviour:

- Low temperatures (high loading frequencies), bitumen behaves as a glassy solid
- Intermediate temperatures, bitumen undergoes a gradual transition from glassy to fluid behaviour
- High temperatures (low loading frequencies), bitumen behaves as a Newtonian fluid [87]

With reference to viscosity tests, at very low shear rate almost all bitumens exhibit Newtonian behaviour. Non-Newtonian behaviour appears as the shear rate increases. Generally, unmodified bitumens exhibit Newtonian behaviour at temperatures greater than approximately 60°C. Polymer modified binders tend to be shear susceptible at 60°C [87].

The temperature-dependent rheological behaviour of bitumen is illustrated in Fig. 4.9.

Fig. 4.9: temperature-dependent rheological behaviour of bitumen. The figure highlights the three different zones of behaviour in which bitumen may be divided



Therefore, viscosity measurements made with rotational viscometer on unmodified binders shall be considered valid for temperatures from 60°C to greater values. An important practical implication of the validity of the Newtonian fluid assumption is that the values of dynamic viscosity of Newtonian fluids, as bitumen at high temperatures, are independent of the degree and duration of the shear load applied.

With reference to considerations made in previous sections on penetration at 25°C and softening point, it can be concluded that empirical test are unsuitable to describe the viscoelastic behaviour of bitumen, as can be qualitatively noted from Fig. 4.9. Viscosity testing, although more fundamental method for determining the rheological performance of a bitumen, does not provide information on the time dependence of bitumen in the temperature range below 60°C. These measurements shall be completed with a dynamic mechanical analysis when a detailed investigation on mechanical response of bitumen is required.

4.5 Dynamic mechanical analysis with DSR (Dynamic Shear Rheometer)

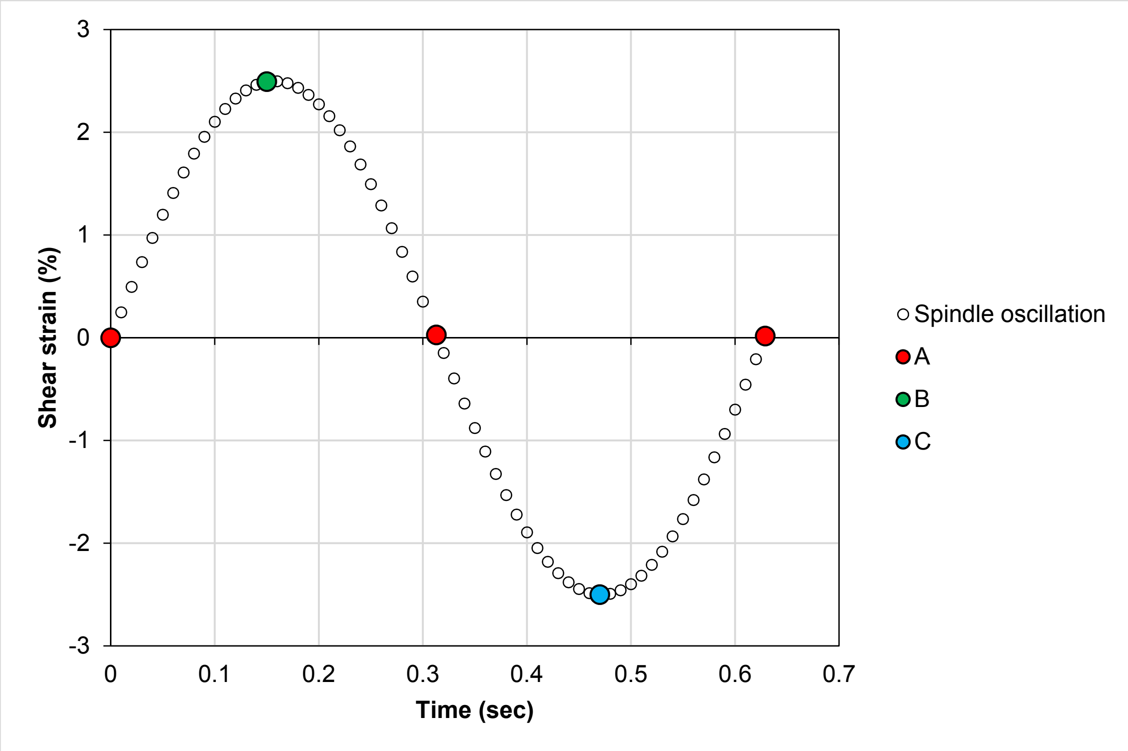
4.5.1 Scope and principle

Oscillatory tests are used to investigate all kinds of viscoelastic materials, as bitumen. This mode of testing is also referred to as dynamic mechanical analysis (DMA) [88]. Dynamic Shear Rheometer is an oscillatory testing device used to measure the rheological characteristics of bitumen.

This type of test applies an oscillatory shear force to a bitumen sample sandwiched between a spindle and a base plate. The spindle is allowed to rotate during the test, while the base plate is fixed. Following the scheme reported in Fig. 4.10, the spindle moves from the centre line, represented by point A, to point B, then reverses direction passing through its original position to point C and then returns to point A. This oscillation represents one cycle and it's continuously repeated during the test.

The European Standard reference used for DSR tests is the EN 14770:2005 Bitumen and bituminous binders - Determination of complex shear modulus and phase angle - Dynamic Shear Rheometer (DSR).

Fig. 4.10: plot of shear strain vs. time during a load cycle of a DSR strain-controlled test



Test can produce accurate calculation of the rheological parameters when the following conditions are met:

- Adhesion of the sample to both plates without slipping effects
- Homogeneity of deformation throughout the shear gap

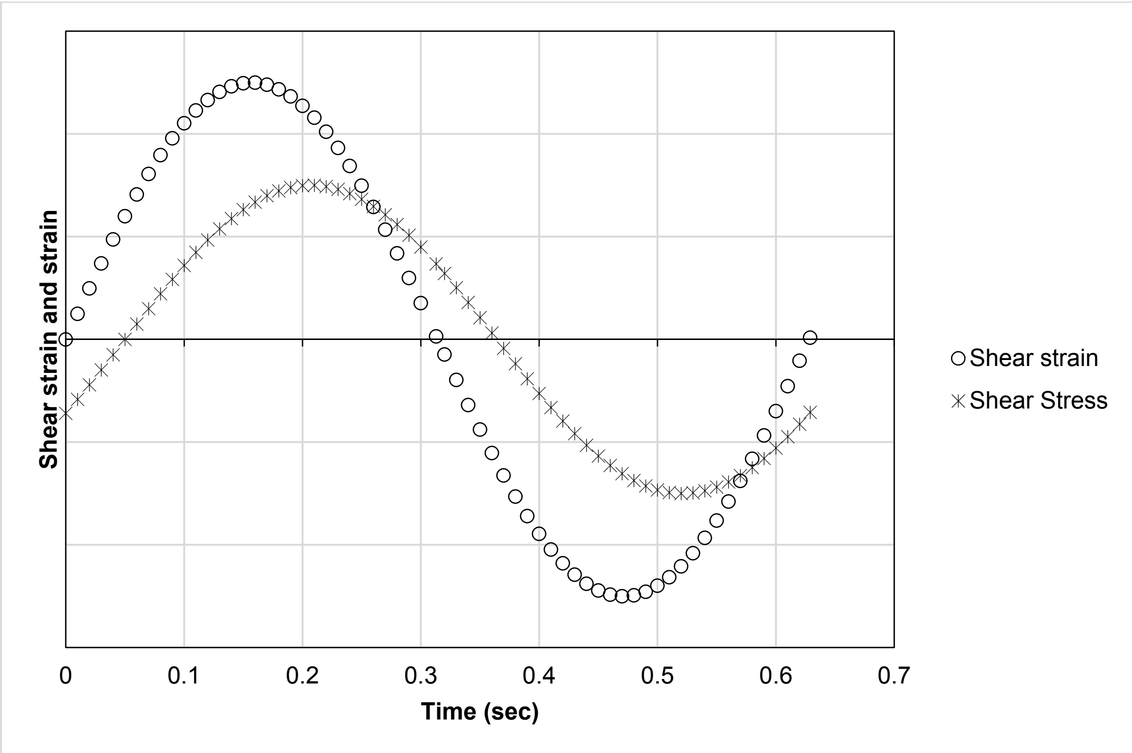
DSR tests can be carried out in controlled stress or controlled strain modes. In the controlled strain mode, which has been used in this research, the spindle rotates with specified amplitude γ_A and angular frequency ω (rad/s) and the torque necessary to maintain the oscillation is measured. The equations of shear stress and strain are:

$$\gamma(t) = \gamma_A \cdot \sin \omega t$$
$$\tau(t) = \tau_A \cdot \sin(\omega t - \delta)$$

The parameter δ is the phase shift angle in degrees between the preset (strain) and the measure (stress). The phase angle may range from 0° to 90° indicating pure elastic or pure viscous behaviour, respectively.

In the controlled stress mode, a fixed torque is applied to the spindle and a specified magnitude shear stress is applied to the bitumen sample. The resultant spindle rotation is measured, from which the magnitude of shear strain is calculated. Because the applied stress level is fixed, the distance the plate moves on its oscillatory path may vary between cycles.

Fig. 4.11: shear stress and strain vs. time measured during a cycle of DSR test. A phase lag can be seen between strain and shear (phase angle δ)



4.5.2 Measured and calculated parameters

When performing controlled shear strain tests, spindle rotation φ (deflection angle) is set and the resultant torque T is measured by the DSR. For controlled shear stress tests, T is set and φ is measured. Rheological parameters γ (strain) and τ (shear stress) are determined using the following formulas:

$$\tau(t) = \frac{2T(t)}{\pi r^3}$$

$$\varphi(t) = \frac{\varphi(t) \cdot r}{h}$$

Where h is the height of the specimen and r is the radius of the plate.

Bitumen is a viscoelastic material with both elastic solid (deformation due to loading is recoverable) and viscous liquid (deformation due to loading is non-recoverable) properties. DSR tests can be used to characterize viscous and elastic behaviour of bitumens by measuring the complex shear modulus G* and the phase angle δ.

The complex modulus G* is calculated as the ratio of the peak shear stress to peak shear strain measured within a test cycle, in both mode of testing. The "G" notation used for the complex shear modulus is to distinguish it from E*, which is the complex modulus measured in tension or flexure. G is also marked with a star as parameter calculated from harmonic-periodic processes. G* is comprised of elastic and viscous components defined as storage modulus G' and loss modulus G'', respectively.

The storage modulus G' is a measure of the deformation energy stored by the sample during the shear process [88]. The stored energy allows the material to restore its original shape after being loaded. Thus, G' represents the elastic behaviour of the material.

The loss modulus G'' represent the amount of loading energy transformed in heat by the material. Heating occurs due to the relative motion between molecules and particles of the material, which causes frictional heat (viscous heating). A part of the energy may heat up the sample, and another part may be lost in the form of heat to the surrounding environment. The energy loss indicates an irreversible deformation behaviour of the material.

Storage and loss components are linked to the complex modulus by the phase angle δ, which is the phase lag between the shear stress and shear strain responses measured at each cycle during the test, as reported in Fig. 4.11. The phase angle δ is often reported as tgδ (loss factor), which is defined as the ratio between the viscous and the elastic components of the complex modulus.

$$tg\delta = \frac{G''}{G'}$$

Since δ may range from 0° to 90°, tgδ may consequently range from 0 to ∞. When tgδ is equal to 1, the bitumen reaches the balance between elastic and viscous component of complex modulus. The temperature at which this viscoelastic transition occurs is called cross-over temperature.

All parameters reported above can be represented on a complex plane (Argand-Gauss diagram) where the storage modulus is the real axis (horizontal) and the loss modulus is the imaginary axis (vertical). Under normal conditions, the complex modulus G* is defined by a vector with magnitude |G*| and direction δ anti-clockwise from the horizontal axis. The complex modulus can be written as:

$$G^* = G' + iG'' = |G^*| \cdot (\cos\delta + i\sin\delta) = |G^*| \cdot e^{i\delta}$$

$$|G^*| = \sqrt{(G')^2 + (G'')^2}$$

The magnitude of the complex modulus G* corresponds to its absolute value.

4.5.3 Types of DSR testing

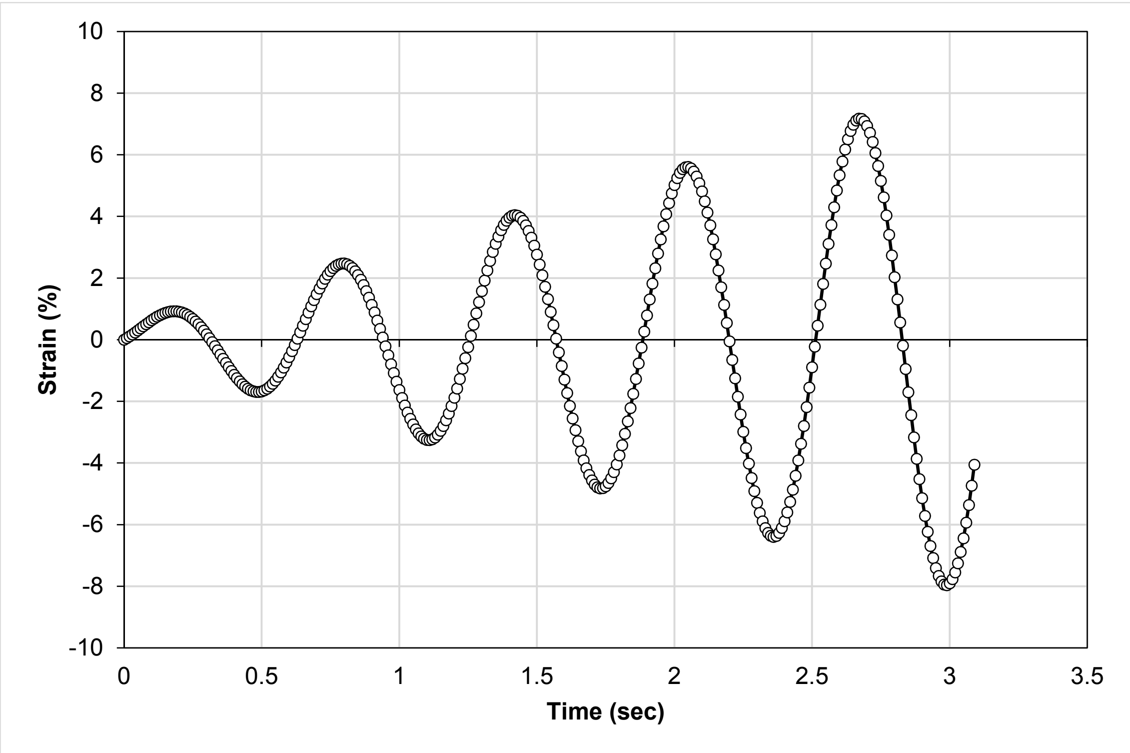
The following testing procedure are briefly described:

- Amplitude sweep tests
- Frequency sweep tests
- Dynamic thermo-mechanical analysis (temperature sweep tests)

4.5.4 Amplitude sweep tests

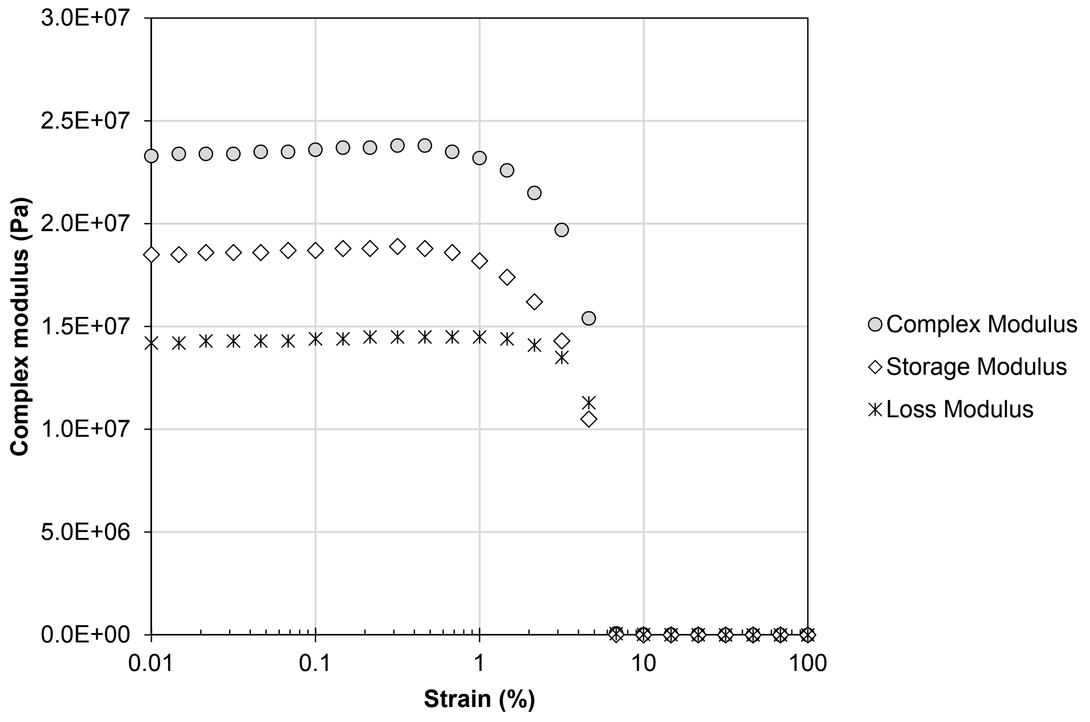
During amplitude sweep test, the bituminous sample is subjected to a sinusoidal load with constant angular frequency of 10 rad/s (frequency of 1.6Hz) and increased amplitude with time at constant temperature of 10°C. Load shape is plotted in Fig. 4.12.

Fig. 4.12: sinusoidal loading applied to a bituminous sample subjected to an amplitude sweep test with controlled shear strain. Amplitude increases with time, angular frequency has a constant value of 10 rad/s



The amplitude sweep testing procedure was carried out for determining the linear viscoelastic (LVE) domain on each bituminous sample. Linear viscoelasticity refers to a region of behaviour of bitumen in which the dynamic shear modulus is independent of shear strain or stress [87]. The accurate evaluation of the relationships between molecular structure and viscoelastic behaviour requires that experiments, such as rheological measurements, are conducted in the LVE region [89]. In the amplitude sweep test, increasing cyclic levels of stress and strain are applied at a constant frequency. The limit of the LVE domain is defined as the point at which the dynamic viscoelastic function G* deviates by 10% from a constant (plateau) value, as reported in Fig. 4.13.

Fig. 4.13: plot of complex, storage and loss modulus curves versus strain amplitude during amplitude sweep test on bituminous sample B0



There is a clear distinction between the linear and non-linear regions as the complex modulus results show a very small decrease (almost constant value) with increasing stress level in the linear region, whereas in the non-linear region the complex modulus of the bitumen decreases with increasing stress level [90]. Similar behaviour can be observed for the storage modulus G' and loss modulus G". Fig. 4.13 shows that within the linear range of response, values of storage modulus and loss modulus are independent of the applied stress amplitudes. In the non-linear range, however, the moduli decrease with increasing stress.

4.5.5 Frequency sweep tests

Frequency sweep tests are oscillatory tests performed at variable frequencies, keeping amplitude (strain or stress) and temperature at a constant value [88].

Frequency sweeps are used to investigate time-dependent deformation behaviour, since frequency is the inverse value of time. The higher the frequency, the more solid-like response of the material. Conversely, lowering the frequency results in a more fluid-like response of the material. At high frequencies, the storage modulus G' and the magnitude of complex modulus |G*| increase, indicating and increased elastic behaviour of the bitumen. At low frequency, the viscous behaviour is dominant, hence G''>G'.

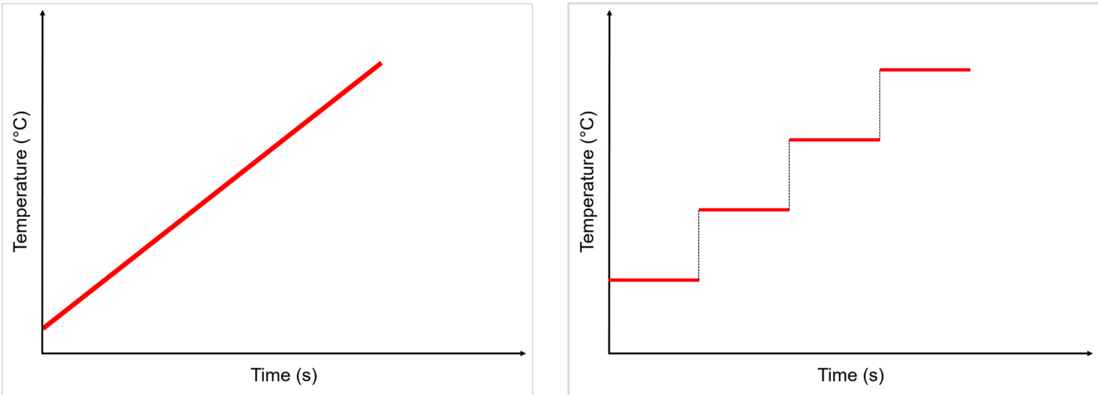
The frequency, or frequency range shall be chosen so as to be within the linear region of the test specimen over the temperature range selected. For the frequency range, a minimum of two frequency decades shall be specified with any individual frequency being at ± 10 % of the set value [91].

4.5.6 Dynamic thermo-mechanical analysis, DTMA

DTMA is performed setting a temperature variation at constant dynamic-mechanical shear conditions during the test with DSR. Temperature may vary with two different profiles:

- Ramp function, with constant heating (or cooling) rate with time, ranging from 1°C/min to 5°C/min. This procedure was used for the testing activity with DSR of this research, and it is called "dynamic temperature sweep";
- Step-and-hold function, in the form of several discrete temperature steps, each one with a constant temperature for a defined period of time

Fig. 4.14: from left to right, ramp (dynamic temperature sweep) and step-by-step pre-set temperature profiles used for dynamic thermos-mechanical analysis of bitumens



The procedure to control the heating (or cooling) rate shall be slow enough to temperature equilibration throughout the bituminous sample. The required and measured temperature shall have no significant deviation and time lag during the testing operations.

The DTMA can be used to investigate chemically stable materials. The aim of the test is to determine the influence of temperature on physical properties like phase changes or other structural modifications occurring within the material in the LVE domain [88].

4.5.7 Equivalency of frequency sweep and DTMA tests

For thermo-rheologically simple materials, loading frequency and temperature have a comparable effect on the rheological behaviour of the material. Exposing the sample to high temperatures or low frequency shearing result in the same softening effect on the material. Conversely, exposing the sample to low temperatures or high frequency shearing result in increasing elasticity of the material. Therefore, the effects of high temperatures – low strain rate and low temperature – high strain rate are similar.

The equivalency of the two testing procedures applies to thermo-rheologically simple materials, for which the time-temperature superposition principle is valid. In order to achieve appropriate results, the measuring temperature shall show enough distance to the glass transition temperature T_g ranging from -10°C to -30°C for bitumens.

4.5.8 Testing procedure and conditions

The testing procedure followed in the present research is summarized below:

1. Setting up of DSR instrumentation
2. Sample preparation (according to EN 14770:2005)
3. Thermal conditioning of the sample
4. Amplitude sweep pre-test
5. Calculation of test strain amplitude
6. Preparation of a new sample of the same investigated bitumen
7. Dynamic Thermo-Mechanical Analysis (Temperature sweep test)

Testing parameters for amplitude sweep and temperature sweep tests are reported in Table 18 as well as specifications of samples. The use of 8 mm parallel plates with 2 mm gap width is recommended when the magnitude of G* is expected to be within the range of 0.1 – 30 MPa [92].

Table 18: DSR test conditions for rheological investigations on bituminous samples with amplitude sweep and temperature sweep tests

Sample	
Specimen height	2 mm
Specimen weight	10 mg
DSR plate	
Plate diameter	8 mm
Gap width at test start	2 mm
Amplitude sweep test (pre-test)	
Mode of loading:	Strain control mode (sinusoidal)
Amplitude gamma	0.01 – 100 % log; |Slope| = 6 Pt. / dec
Angular Frequency omega	10 rad/s
Number of Data Points	25
Test temperature	10°C
Temperature sweep test	
Mode of loading:	Strain control mode (sinusoidal)
Time Setting	181 Meas. Pts.
Meas. Pt. Duration	0.16 min
Amplitude gamma	2.50%
Angular Frequency omega	10 rad/s
Start temperature	0°C

End temperature	90°C
Rate	Linear ramp 2.99 °C per min

4.5.9 Factors affecting DSR measurements

The accuracy of DSR measurements in terms of repeatability and reproducibility may be affected by affected by the following factors:

- Temperature control
- DSR torque limitations
- Handling and sample preparation

Temperature control and machine limitations will be briefly discussed below. Errors that may come from specimen preparation are related to the operator, therefore they will not be discussed.

Bitumen is a material with significant thermal susceptibility and reduced thermal conductivity. Therefore, adequate temperature control during DSR testing is essential in order to obtain accurate results. Two aspects shall be considered during testing activity:

- The whole bituminous sample should be at the same temperature
- Temperature is accurately controlled, with very small deviation between Pre-set and actual binder temperatures [87]

The conditioning system of the DSR shall be able to maintain a constant and uniform temperature within the bitumen sample, avoiding to produce temperature gradients, which may lead to a reduction in the precision and the accuracy of measured data [87]. Temperature gradients are strongly related to sample geometry [93], in particular gap width: the higher the gap, the higher the magnitude of the gradient that may develop within the specimen. The temperature control unit of the DSR used for the present research is a Peltier heating system with heating elements above and below the sample. Such thermal conditioning system should be able to significantly reduce temperature gradients. Temperature is electronically controlled by the instrument with a precision of 0.1°C.

The DSR has some operational limitations related to the upper and lower limits of torque or angular deflection. Since the DSR records only two measurements, i.e. torque T and angular rotation φ, all other results (e.g., strain and G*) are calculated from these two measurements. Also, the instrument and the integrated software assume that the torque is entirely due to bitumen. Therefore, it is essential that the limits for these measurements are carefully monitored [90].

At low temperatures/high loading frequencies, DSR may not be capable of applying the sufficient torque to achieve the specified stress level. In these conditions, the stiffness of the sample may be equal to or greater than the stiffness of the drive-shaft, leading to lower measurements of the stiffness than actual value [94]. At high temperatures/low loading frequencies, the DSR may not be able to apply a small enough torque to achieve target strain amplitude [87].

The solution to these issues can be summarized as follows: when maximum torque limit is reached (low temperature/high frequency), plate diameter can be reduced, leading to an increase of the applied shear stress; when minimum torque limit is exceeded (high temperature/low frequency), plate diameter can be increased with a decrease of the applied shear stress.

5 Experimental programme

5.1 Fear of the RAP?

The percentage of RAP that can be incorporated into asphalt mixtures depends on production process (plant type, production temperature, mixing time, and discharge temperature), paving technology and permitted emissions as well as RAP source and properties. These factors affect the interaction between RAP and virgin bitumens and consequently impact the performance of asphalt mixtures [5]. The maximum amount of reclaimed asphalt is mainly limited by the production technology. Considering hot in plant recycling, most conventional drum plants can accommodate 50% RAP, whereas the percentage of reusable RAP in batch plant ranges from 10 to 30% [6]. These limitations have been overcome thanks to multiple technologies readily available for production of up to 100% recycled hot mix asphalt [7].

The most of agencies and public administrations insert restrictions on RAP percentages ranging from 10 to 30% in their regulations due to concerns for pavement performances and production technologies. Uncertainties concern the interaction between RAP binder and virgin binder. Inaccurate assumptions on the effects of interaction might create problems both in mix design and pavement performance [2] [5], leading to mixtures that might be subjected to premature failure to cracking, ravelling, moisture damage and rutting.

Upper limits are frequently placed on the total amount of RAP that can be used in specific applications due to concerns about ability to obtain specific mix properties or about performances, especially in terms of durability, rutting, cracking and surface friction [7].

In current practice, RAP binder properties are not routinely determined because either RAP contents are kept below 25 percent or because the additional costs of determining the RAP binder properties and the softer grade of virgin binder resulting from the blending analysis diminish the feasibility of using RAP contents above the 25 percent threshold.

These restrictions specified in technical regulations together with a lack of processing (i.e., variability of RAP) and lack of RAP availability are factors that prevent the use of high quantity of RAP into asphalt mixtures (North Carolina Department of Transportation NCDOT, 2007).

In a 2009 NCDOT survey [3], participants were asked to identify major concerns and obstacles that limit or preclude the use of RAP in HMA. The two concerns cited most often regarded the quality of the blended virgin and RAP binder qualities, especially for high RAP mixes and polymer modified binders, and stiffening of the mix from high RAP quantities and resulting cracking performance. High RAP may affect binder properties resulting in an “overly stiff” mix that may experience low-temperature cracking. There was also concern that an overly stiff mix may not be as resilient and may crack prematurely for pavements experiencing high deflections.

Furthermore, several agencies are concerned that the use of RAP with polymer modified binders may reduce the quality of the polymer-modified virgin binder.

Summarizing, concern in the use of RAP is due to **uncertainty** related to the material, in terms of physical properties as gradation, binder content and density, and in terms of performances and durability of RAP mixes.

5.2 Objectives and research approach

This thesis focuses on the ***effects of RAP on performances and durability of new asphalt mixes produced incorporating RAP with a standard hot-in-plant process***. The effects of RAP have been investigated through an extensive laboratory testing campaign on asphalt mixtures and on bitumen recovered from the same mixes. The laboratory testing activity of the present research aims to investigate performances and durability of asphalt mixtures produced incorporating different percentages of RAP.

The objectives of the present experimental programme are listed below:

7. Investigate the effects of RAP on **durability** of asphalt mixtures. In detail, the research investigates the effect of increasing percentages of RAP (sampled from the same source) on performances of the same type of asphalt mixture. Therefore, RAP content is the only significant variable of the analysis
8. Define the **threshold value of RAP** that can be incorporated into a bituminous mix without compromising its long-term performances
9. Determine the correct **procedure** for incorporating RAP into mixtures produced in laboratory

The main goal of the research is to answer to simple questions that often rise in the day to day practice, as: "*how much RAP can be added into a specific mix?*" or "*how does the presence of RAP affects performances of the asphalt mixture?*"

The experimental programme of the research was conducted using only standardized test procedures according to **European standards**. This for two reasons.

First, test developed apart from the standards (thus with no standard procedure to follow) requires to be validated in terms of procedures, data processing and results. Repeatability and reproducibility are key issues when dealing with the high variability of RAP. Further uncertainty caused by test procedure is not required and desired neither.

Second, the use of standardized tests allows to develop a sound methodology that can be used by everyone who needs to test mixtures containing reclaimed asphalt. A characterization procedure can be proposed standing on solid bases of European standard tests

5.3 Experimental programme

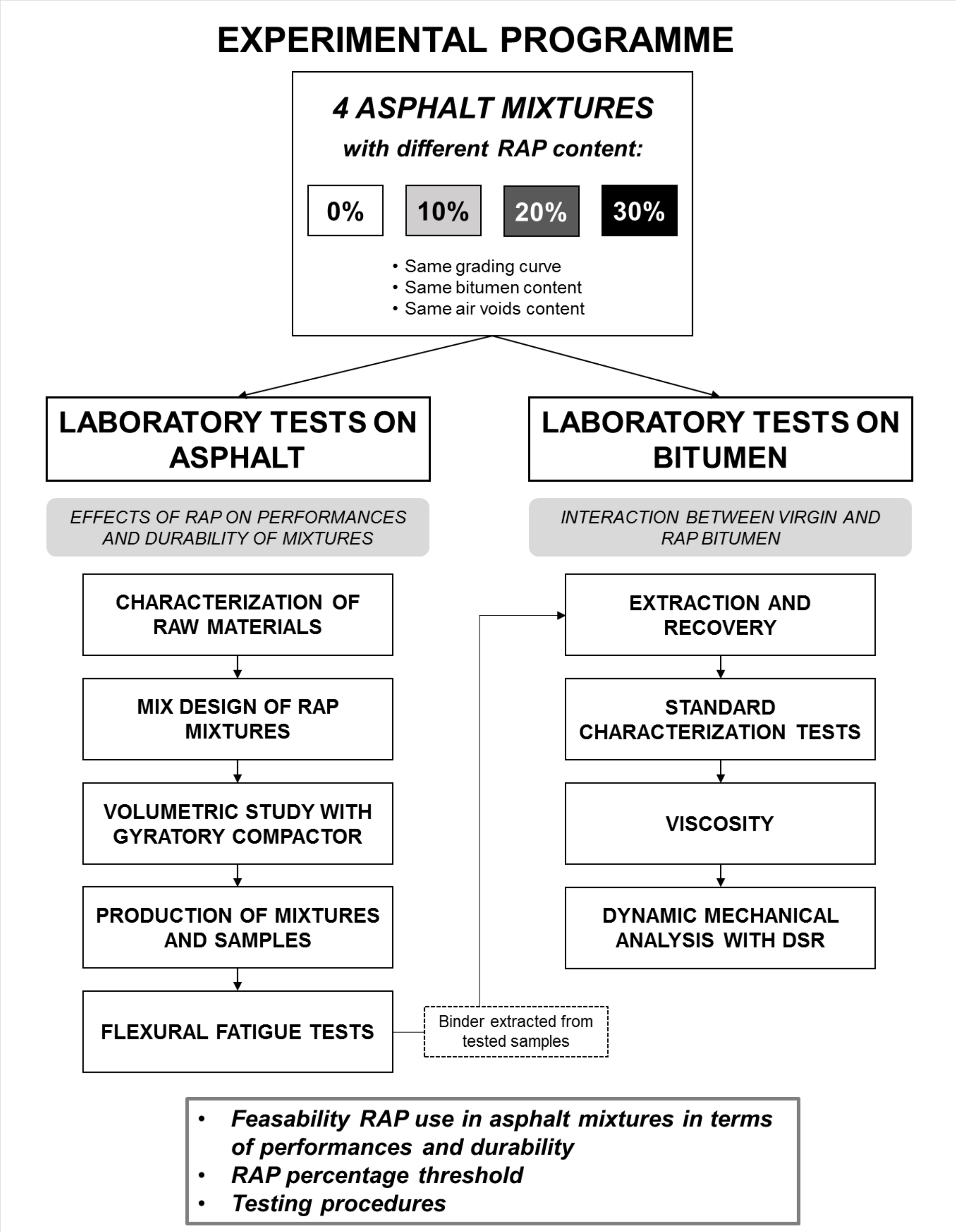
The research is divided in two phases: in the first phase, the effects of RAP on asphalt mixtures are investigated. A specific typology of asphalt mixture is produced with different RAP percentages of 10%, 20% and 30% by mass of aggregates. Asphalt materials are characterized and compared in terms of resistance to fatigue, stiffness and volumetric properties.

The second phase of the research is aimed to investigate the effects of RAP on asphalt binder as a composite blend of neat and RAP bitumen. Bituminous samples are recovered from asphalt samples tested in phase 1; then, they're characterized and compared with standard and dynamic tests.

The objective of the research is to comprehensively characterize RAP mixtures in terms of performances of asphalt mixtures and bitumens. Studies [95] show a strong correlation between rheological properties of bitumen and asphalt mix, especially at low strain level. Furthermore, the response of bitumen to laboratory tests is used as reference for results of lab tests on asphalt mixes. For this reason, both bitumen and asphalt mixture properties are investigated in this research.

The underlying idea of the research is to investigate a common asphalt mixture produced with standard materials, i.e. natural limestone aggregates and unmodified 50/70 pen grade bitumen, and with properties that are consistent with the most common technical specifications. RAP has been incorporated into new mixes simulating the hot-in-plant recycling process. Recent surveys show that 61% of available RAP is used in hot-in-plant recycling. The percentages of 10, 20 and 30% of RAP being incorporated into the mixes reflect the most common usage of RAP into traditional batch asphalt plants in Italy i.e. with no specific retrofitting to handle high quantity of reclaimed asphalt. The choice of common materials and well-established procedures was made to give the wider validity to research findings and to make the testing procedure accessible and reproducible to other laboratories or asphalt plants.

Fig. 5.1: flow-chart of the research experimental programme



5.4 Experimental programme on asphalt mixtures

In this first phase of the research, a specific typology of asphalt mixture is produced with different RAP percentages of 10%, 20% and 30% by weight of the mix. Asphalt materials are characterized and compared in terms of resistance to fatigue, stiffness and volumetric properties. Asphalt mixes are classified as Mix0 (control mixture), Mix1 (control+10% RAP), Mix2 (control+20% RAP) and Mix3 (control+30% RAP).

Results from laboratory tests are compared to evaluate the effect of the RAP percentage on the durability of asphalt mixtures. The experimental programme is reported below in Table 5.1.

Table 5.1: resume of the experimental program for the first phase of the research: laboratory testing on asphalt mixtures

Activity	Objectives
Sampling	• Collect representative samples for further testing
Characterization of materials with laboratory testing	• Determine properties of raw materials (aggregates, RAP and bitumen) on representative samples. The feedstocks of each material are characterized measuring aggregate gradation, apparent density and binder content (RAP)
Mix design with gyratory compactor (preliminary investigation on asphalt mixtures) of four asphalt mixes with increasing RAP content	• Define design parameters of the mixture as: bitumen and air voids content, density, grading • Validate the production laboratory operations verifying that design parameters are obtained • Produce similar sample in terms of grading, binder content, density and air voids, with the RAP content as unique significant variable
Production of prismatic samples	• Produce samples with similar characteristics of density and air voids
Flexural fatigue testing with four-point bending device	• Investigate the effect of RAP on long term performances of RAP mixes • Use different approaches from Wohler i.e. the “dissipated energy ratio” approach to study fatigue resistance of asphalt mixtures

The aim of this first phase of the research is to evaluate how the presence of RAP affects the mechanical behaviour of asphalt mixtures.

In order to reach this goal, it’s important to limit the effects of secondary variables as, for example, aggregate grading, density, binder and air void content, on test results. When analysing test results there should be no concern if results depend on the RAP content or on other properties. A significant effort was put to produce asphalt samples with similar physical and volumetric characteristics. No additives, fibres or rejuvenators were added to mixtures in order to keep results clean from these external and uncontrolled factors. With similar properties, asphalt mixtures differ only by the RAP content, which is then the only significant variable of the analysis.

Limiting the variables affecting test results to only the RAP content is one of the most challenging issue of the research.

There are many processes made of little steps which singularly contribute to produce sound test result: sampling, characterization of raw materials, production and compaction of mixes and laboratory testing. Some types of data treatment may be source of errors as well. If all the procedures are carried out correctly, RAP content can be considered as the most significant variable that affects test results.

5.4.1 Sampling of raw materials in plant

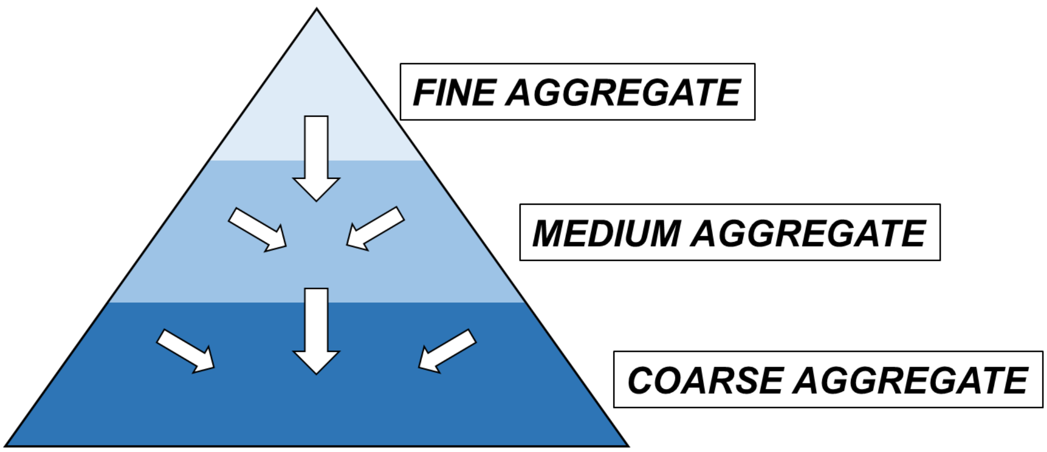
Aggregates and RAP were supplied by the asphalt plant of S.A.P.A.B.A in Bologna. Virgin aggregates are routinely extracted from the quarry, crushed, sieved and stored in conic stockpiles. Stockpiles are classified on the basis of the type and of the minimum “d” and maximum “D” particle size of the stored material, as indicated in European standards (d/D). The following materials were sampled:

- Limestone gravel 12/20
- Limestone gravel 6/10
- Limestone gravel 4/6
- Sand 0/4
- Filler: by-product obtained from muds used in leather production processes. This filler, which is called “Plastofill”, is routinely used by S.A.P.A.B.A for producing asphalt mixtures
- Processed (screened) RAP 0/8
- Processed (screened) RAP 8/12

Sampling of aggregate and RAP from stockpiles aims to obtain representative portions of the stored material to be used for further testing activity in laboratory. In other words, the sampled material shall have the same properties of the stockpiled material.

Segregation may occur within the stockpiles, compromising the representativeness of the sampled material. Segregation is caused by gravity and consists in an undesired separation of coarse and fine particles of the same aggregate class within the stockpile. In order to limit segregation of aggregates, stockpiles are frequently reshuffled by excavators. This ensures consistency of aggregate gradation and hence quality of produced asphalt mixtures. A schematic composition of a cone shaped stockpile of segregated aggregates is given in the following Fig. 5.2 [96].

Fig. 5.2: schematic representation of segregation of aggregates in a cone-shaped stockpile



Aggregate and RAP were sampled at the S.A.P.A.B.A plant with a loader from the conic stockpiles. In order to avoid possible segregation issues, aggregate and RAP were sampled from the interior of the stockpile and not from near the surface.

There's a difference in sampling material from RAP or virgin aggregate stockpiles: virgin aggregate stockpiles consist of homogeneous material supplied from a known source and processed i.e. screened before being stockpiled. Properties of stockpiled materials as grading and density generally have limited variability; RAP stockpiles are made of asphalt milled from different sources i.e. different types of road, resulting in great variability in the composition of stockpiled material. Consequently, RAP being stockpiled presents significant variability of its properties as bitumen content and type, grading, ageing of RAP bitumen and type of RAP aggregate.

Stockpiled aggregates are subjected to scheduled tests carried out by the plant laboratory to check aggregate gradation and particle density. Test results over the last two years show a good consistency of aggregate gradation, with a sound correspondence between the classification of stockpiles and the actual particle size.

Aggregates and RAP were sampled from stockpiles according to methods described in the EN 932-1 and EN 12697-26. The European standards use the following terminology:

a. *Batch*: a production quantity, a delivery quantity, a partial delivery quantity (railway wagon-load, lorry- load, ship's cargo) or a stockpile produced at one time under conditions that are presumed uniform
b. *Increment*: a quantity of material taken from a batch by one operation of the sampling apparatus
c. *Bulk sample*: an aggregation of the sampling increment
d. *Representative sample*: a bulk sample created by taking sampling increments according to a sampling plan, which makes it likely that the quality of this sample corresponds to that of the batch
e. *Laboratory sample*: a reduced sampled derived from a bulk sample for laboratory testing

Aggregate stockpiles and aggregate size of limestone gravels and sand are reported in Fig. 5.3. Drainage pipes are present at each stockpile to avoid water stagnation and humidity of aggregates as indicated in Fig. 5.4.

Fig. 5.3: aggregate stockpiles in S.A.P.A.B.A plant

Fig. 5.4: disposition of stockpiles in plant and drainage pipes used to reduce the humidity of aggregates

5.4.2 Characterization of raw materials

Raw materials sampled in plant were carried to laboratory and stored in stockpiles, or feedstocks as noted in the EN 12697-27. The objectives of characterization testing are:

a. Determine properties of stockpiled material as aggregate grading, apparent density and binder content of RAP. Raw materials are used in further mix design process, thus measures shall be accurate and representative of the actual properties of stockpiles
b. Measure properties on *representative samples*, therefore measure representative properties for each stockpiled material

Objective at point b is of primary importance for further steps of testing, in particular for the mix design. An accurate preliminary characterization of raw materials is necessary to limit variability and to ensure consistency of asphalt mixtures produced in lab.

Laboratory stockpiles composed of material sampled at the plant are called “feedstock” from now, in order to differentiate plant and laboratory stockpiles. With reference to terminology of the European standard, feedstocks can be considered as bulk samples i.e. an aggregation of single increments sampled from plant stockpiles.

Since the importance of obtaining sound and representative results, the characterization is composed of two steps:

1. Sampling of representative specimen from feedstock according to EN 932-1 and EN 12697-27
2. Testing of properties of raw materials

Laboratory samples are obtained sampling 10 increments from different positions of the feedstock, with a shovel. A 30 kg bulk sample is composed from the single increments. The bulk sample is then reduced by quartering to form the laboratory sample being tested. The mass of laboratory sample depends on the test to be performed ranging from 1 to 2.5 kilograms.

A list of characterization tests is given in Table 5.2.

Table 5.2: list characterization tests on raw materials

Characterization test	Raw material
• Particle size distribution (EN 933-1 and EN 12697-2) • Apparent density of aggregate EN 1097-6	Limestone gravel 10/20 Limestone gravel 6/10 Limestone gravel 4/6 Sand 0/4 Filler RAP 0/8 RAP 8/12
• Binder content (EN 12697-1, hot-extraction with solvent)	RAP 0/8 RAP 8/12

The hot-extraction method with solvent and wired mesh was preferred to ignition method, because this last one tends to overestimate the binder content of asphalt mixtures when not calibrated on the basis of other extraction methods. Furthermore, ignition hoven may cause change in aggregate gradation of the mixture [13].

For each aggregate feedstock, 10 samples were tested. The same number of samples was tested for determining the soluble binder content of each RAP fraction.

5.4.2.1 Physical properties of aggregates

Aggregate grading and apparent density were measured. Results of laboratory tests on aggregates were compared to results of quality control activity on the same aggregates of the plant of S.A.P.A.B.A. The quality control has been carried out for two years under the supervision of the author, therefore properties of raw materials are well known.

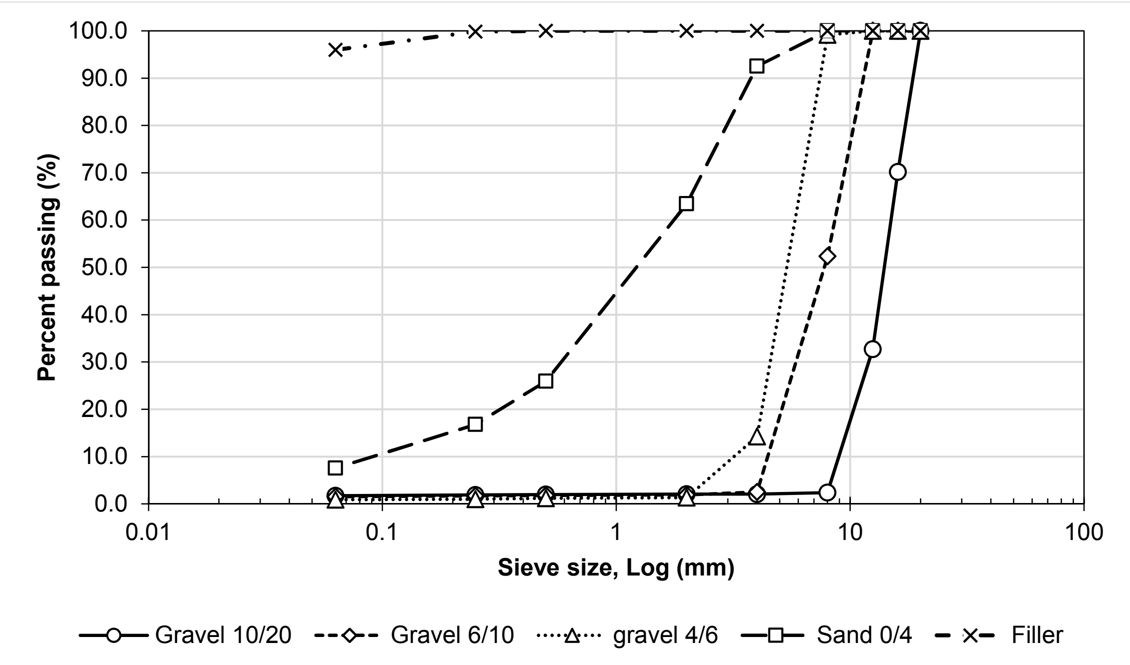
Aggregate grading was determined using 9 square-meshed sieves with the following sizes: 20, 16, 12.5, 8, 4, 2, 0.5, 0.25 and 0.063 millimetres. The EN standard ISO 3310-2 specifies that sieves with aperture size of 4 mm and above shall be perforated plate square hole test sieves. Below that size they shall be woven wire test sieves. An electromagnetic shaker with timer was used to ensure adequate mechanical action on sieves.

Apparent density was determined for each aggregate fraction, including filler, according to the volumetric procedure described in the European standard EN 1097-6. Results are reported below.

Table 5.3: physical properties of aggregates, grading and apparent density

Determination of particle size distribution of aggregates, EN 933-1					
Sieve size (mm)	*Gravel 10/20*	*Gravel 6/10*	*gravel 4/6*	*Sand 0/4*	*Filler*
20	100.0	100.0	100.0	100.0	100.0
16	70.2	100.0	100.0	100.0	100.0
12.5	32.7	100.0	100.0	100.0	100.0
8	2.4	52.4	99.2	100.0	100.0
4	2.1	2.5	14.3	92.6	100.0
2	2.1	1.9	1.3	63.4	100.0
0.5	2.0	1.8	1.2	25.9	100.0
0.25	1.9	1.7	1.0	16.8	99.9
0.063	1.7	1.5	0.9	7.6	96.0
Determination of apparent particle density, EN 1097-6					
Apparent particle density (g/cm^3)	2.693	2.710	2.708	2.702	2.701

Fig. 5.5: grading curves of aggregate fractions



5.4.2.2 Physical properties and bitumen content of RAP

RAP was tested before (unprocessed) and after (processed) being extracted, thus two grading curves were obtained for each sampled material. The grading curves of unprocessed and extracted RAP give detailed information about the material, in particular about the amount of fines in the RAP. Both curves can be used in the mix design. The use of unprocessed curve rather than extracted curve depends on the type of asphalt mixture to be produced. The interaction between virgin and recovered binder determines which of the two grading curves should be used in mix design.

If the RAP binder is supposed to blend with the virgin binder, then the curve of the extracted RAP shall be used. Bitumen blending occurs in hot mix asphalt, when aggregates, bitumen and RAP are mixed together at high temperatures, allowing the RAP bitumen to (partially) melt and being blended with the virgin bitumen. Because of this, RAP aggregates are considered to be uncoated.

If the RAP binder is not expected to blend with virgin bitumen, the curve of unprocessed RAP shall be used. This is the case of cold mixes produced at ambient temperature with RAP, virgin aggregates, cement or lime as active filler and bituminous emulsion or foamed bitumen as asphalt binder. For these mixtures, two aspects should be considered: first, production temperatures are low, not allowing the RAP bitumen to melt and hence being blended with the virgin bitumen. Only a limited amount of recovered bitumen is removed from RAP aggregates during mechanical mixing. Second, cold mixes contain less bitumen compared to traditional hot mix asphalts. This reduces the possibility of blending between virgin and recovered bitumen.

These considerations are summarized in Table 5.4.

Table 5.4: recommended use of unprocessed or extracted RAP grading curve depending on asphalt mixture

Type of asphalt mixture	RAP grading curve for mix design
Hot mix asphalt	Extracted
Cold mix asphalt	Unprocessed

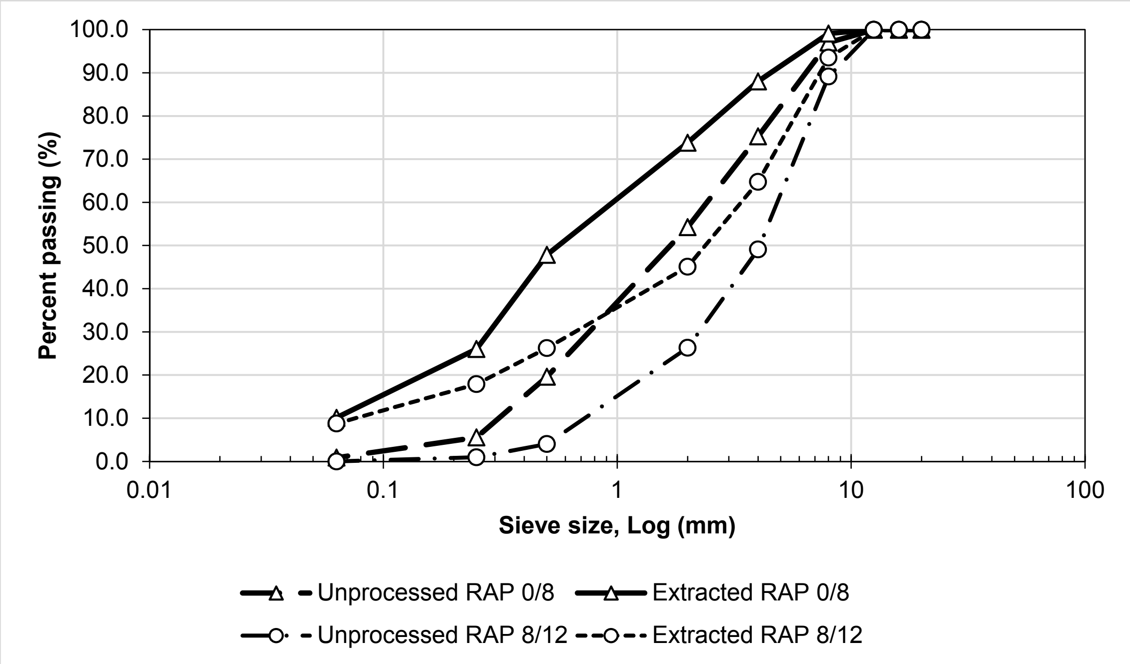
The apparent density was calculated on extracted RAP on both 0/8 and 8/12 fractions. Results of testing on physical properties of RAP are reported below.

Fig. 5.6: physical properties of RAP fractions, grading and apparent density

Determination of particle size distribution of asphalt mixtures, EN 12697-2				
Sieve size (mm)	*Unprocessed RAP 0/8*	*Extracted RAP 0/8*	*Unprocessed RAP 8/12*	*Extracted RAP 8/12*
12.5	100.0	100.0	100.0	100.0
8	97.0	99.1	89.2	93.5
4	75.3	88.0	49.1	64.8
2	54.3	73.8	26.3	45.1
0.5	19.6	47.8	4.0	26.3

0.25	5.6	26.0	1.0	17.9
0.063	0.9	10.1	0.0	8.8
Determination of apparent particle density, EN 1097-6				
Apparent particle density (g/cm³)	2.684		2.696	

Fig. 5.7: grading curves of unprocessed and extracted RAP 0/8 and 8/12



The bitumen content of RAP samples was determined with the wire mesh hot extraction method, according to the procedure described in the EN 12697-1. Tetrachloroethylene was used as solvent. The mineral matter was separated from the bitumen-solvent solution with a centrifuge, which also allow the recovery of the solvent.

The RAP 0/8 fraction is a mixture with significant quantity of fines, and can be considered as a coarse sand. Attention shall be paid during wire mesh hot extraction with fine materials because smaller particles may occlude the 0.063 millimetres wire mesh, leading to an ineffective washing of particles and hence to a potential underestimation of bitumen content. This undesired side effect can be avoided stirring the material in the basket with a putty knife every 30 minutes during the extraction.

Results of extraction and apparent density obtained from 10 laboratory samples are reported below.

Table 5.5: average bitumen content of RAP 0/8 and 8/12 laboratory samples

Determination of soluble binder content, hot extractor method (wired mesh), filler recovery with continuous flow centrifuge, EN 12697-1:2012		
B_c, binder content by weight of the mixture (%)	*RAP 0/8*	*RAP 8/12*
Average (10 samples)	4.47	4.43
Standard deviation	0.24	0.25
Minimum	4.12	4.08
Maximum	4.88	4.91

Table 5.6: average apparent density of RAP 0/8 and 8/12 laboratory samples

Determination of apparent particle density of reclaimed asphalt randomly sampled from stockpiles, EN 1097-6:2013		
ρ_a, apparent density of aggregates (g/cm^3)	*RAP 0/8*	*RAP 8/12*
Average (10 samples)	2.685	2.695
Standard deviation	0.010	0.006
Minimum	2.672	2.687
Maximum	2.708	2.707

5.4.3 Mix design

5.4.3.1 Introduction to mix design methods

The first mix design procedure was developed in the United States by Clifford Richardson. He published his procedures in “The Modern Asphalt Pavement” in 1905, recognizing the importance of material selection, especially the characteristics of fine aggregate, and the importance of air voids.

The Marshall procedure was developed by Bruce Marshall in 1939, and it was adopted by the US Army Corps of Engineers during World War 2 for the design of military airfields. The Marshall procedure is based on the Marshall impact compactor, which is used to produce lab samples, and the Marshall test, which is used to assess mechanical properties of the mixture. The goal of the Marshall method was to have a mix design procedure that required simple portable laboratory equipment. The Marshall method still remains one of the most used methods for mix design and quality control thanks to consolidate, affordable and simple procedure. Unfortunately, the Marshall mix design approach does not allow to investigate in detail the volumetric properties of mixtures, because of the strong empirical nature of this approach. This limits the range of materials that can be accurately investigated in laboratory, especially non-conventional asphalt mixtures produced incorporating reclaimed asphalt or artificial aggregates into the mix. However, the Marshall method can be used as a simple and affordable procedure to quality control of asphalt mixtures.

In the 1930s, Francis Hveem developed a mix design method based on the concept of kneading compaction. With this compaction, laboratory mixes would be more representative of field-produced mixes that are being compacted by steel and pneumatic-tired rollers. In addition to developing a kneading compactor, Hveem recognized the need to have mechanical test that would evaluate the performance of the mix.

In 1987, a five-year Federal Highway Administration study to improve the procedures for the performances of asphalt pavements was conducted [38]. The result was the Superpave system. This system was developed to give pavement engineers and contractors the tools they needed to improve the performances of asphalt pavements subjected to extremes of *traffic loading* and *temperature*. The Superpave mix design procedure consists of:

a. Selecting bitumen and aggregates that meet the design criteria
b. Developing several aggregate trial blends
c. Blending bitumen with the aggregate trial blends
d. Compacting the specimens with gyratory compactor
e. Analysing the volumetric properties of the mixture
f. Selecting the best aggregate blend as the design aggregate structure based on the desired *performance criteria* for the mixture

The Superpave method introduces traffic loading and service temperature as input variables of the mix design. This is a brand new approach that was not present in Marshall and Hveem methods. In the Superpave approach, engineers can select asphalt mixtures on the basis of certain target performances investigated in laboratory, which include: resistance to permanent deformation, resistance to fatigue, low temperature cracking, moisture resistance, durability, skid resistance and workability. The final goal of the mix design is to select a mixture that will achieve a balance among all the desired properties (performance criteria).

5.4.3.2 Standard mix design

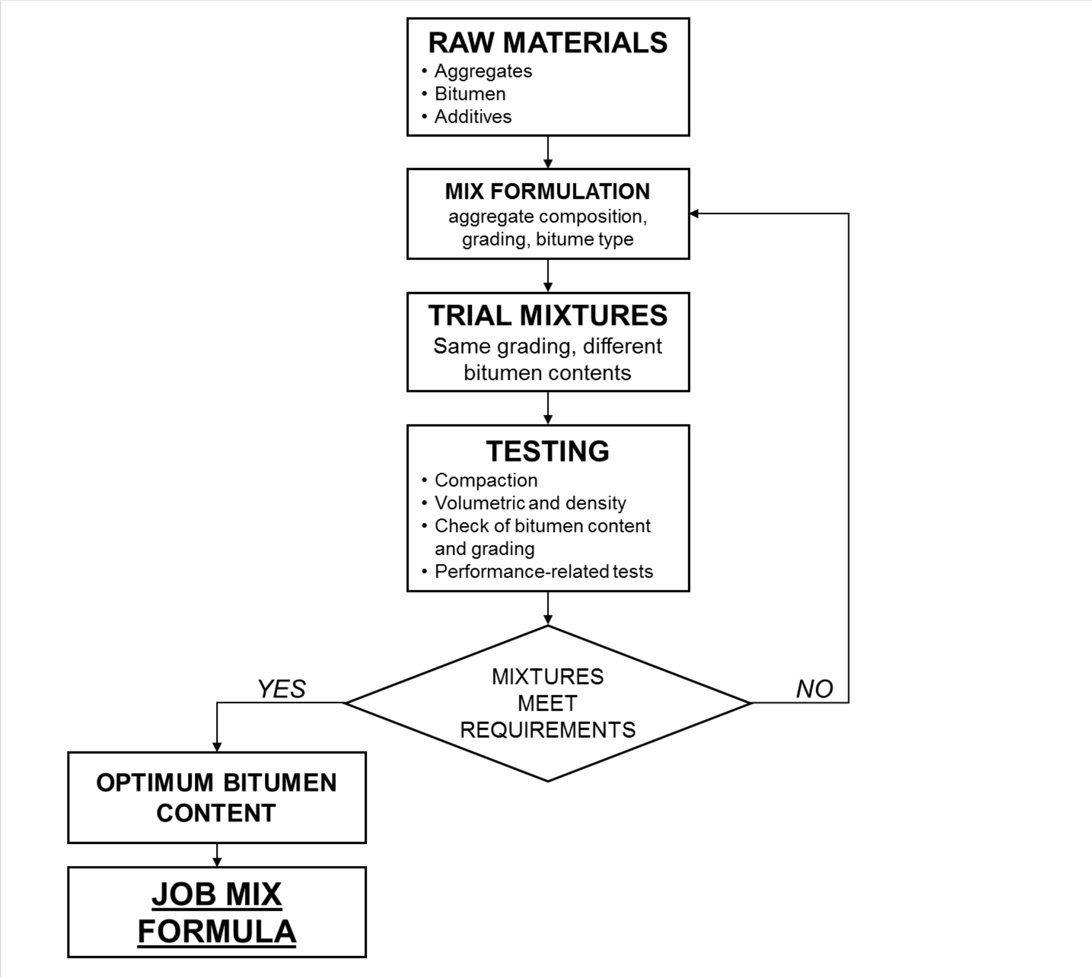
The main goal of the mix design of hot-mix asphalts is to produce asphalt mixtures through blending of available raw materials, usually aggregates and bitumen, in order to obtain the mix composition that better fit both requirements and desired performances.

The most important aspect in mix design is the determination of the optimum bitumen content of the mixture. Bitumen type and quantity have great influence on mechanical properties of a mix. Experience shows that not necessarily the higher the binder content, the better the mechanical performances. For each mix, there’s a bitumen percentage that gives the best results for a specific performance-related laboratory parameter i.e. indirect tensile strength, Marshall stability, resistance to fatigue or rutting resistance. The optimum percentage depends on the target performance investigated, meaning that the optimum bitumen content for fatigue resistance may differ from the optimum binder content for rutting resistance. Usually, the plot of a performance-related parameter versus binder content is a bell shaped curve with a maximum value within a specific bitumen content range.

Regardless of the design procedure used, the design of hot bituminous mixtures consists of the following steps (Fig. 5.8):

1. **Raw materials**: selection of available natural or artificial aggregates including RAP, bitumen and additives previously characterized
2. **Mix formulation**: design of aggregate composition in the mixture, mix grading, choice of bitumen type (when possible)
3. **Trial mixture**: basing on the mix formulation, at least three mixtures with different bitumen content are produced. Specimen subjected to further laboratory testing can be compacted with different methods depending on mix design procedure used
4. **Testing**: laboratory tests are performed on the investigated mixtures for determining volumetric and performance-related properties. The mixture that better fit required specification is chosen on the basis of laboratory test results. If results are unsatisfactory, the mix formulation should be modified, changing aggregate composition, grading, bitumen type and content
5. **Optimum bitumen content**: the bitumen content that provides best laboratory test results is chosen as the optimum bitumen content. The optimum content does not necessarily correspond to one of the investigated binder contents. It may be interpolated from available results
6. **Job mix formula**: results are summarized and mixtures are defined for aggregate composition (percent of each aggregate class by mass of the aggregate mixture), bitumen content (percent by mass of the aggregates). It's a common practice to define the quantity of filler as percentage by mass of the other aggregate class, because it's more familiar to plant operators

Fig. 5.8: flow-chart of standard mix design procedure



In addition to optimum bitumen content, mix design of RAP mixtures usually aims to determine the final grade of the bituminous blend that results blending virgin and RAP bitumen. The desired final grade of bitumen is obtained adjusting the grade of the virgin binder on the basis of quantity and rheological properties of the RAP bitumen.

For example, the stiffer the aged binder, the softer the neat binder necessary to obtain the desired grade of the composite blend. Similarly, the higher the RAP incorporated in the mixture, and hence the RAP binder content, the softer the virgin binder grade.

The effect of adding a soft asphalt can be determined by measuring the physical properties of the recovered bitumen. The asphalt binder is tested is tested to determine penetration, viscosity or rheological parameters, for example complex modulus G*. The laboratory parameter is plotted versus the percentage of RAP binder. The curve is noted as blending chart. This analysis should be verified by blending various percentages of virgin and RAP binder and then testing the composite blend to determine the changes in the physical properties.

Thus, the approach of traditional mix design is to produce mixtures with optimized performances from available raw materials, including reclaimed asphalt.

Two aspects amongst others differentiate mix design procedures of traditional and RAP mixtures:

a. **Variability of RAP properties**: reclaimed asphalt is milled from different sites and usually stored in a unique stockpile at the plant. This results in variability of the stockpiled material. Mix design requires accurate characterization of virgin aggregates, RAP and bitumen properties. Insufficient information lead to inaccurate mix design and hence to uncontrolled properties of the asphalt mixture. This uncertainty is exactly what agency and constructors are afraid of, leading to limitation on RAP usage
b. **Presence of aged bitumen** in the reclaimed asphalt: neat and virgin binder interact when mixed together. The level of blending that occurs between the two bitumens has a fundamental importance in the physical and rheological properties of the composite blende, and hence on the mechanical behaviour of the RAP mixture. Unfortunately, there's not a direct methodology that allows a precise determination of the level of blending. However, the effects of neat-old bitumen interaction can be indirectly measured with performance-related laboratory tests i.e. resistance to fatigue, rutting resistance, indirect tensile test among the others. Studies [13] indicate that when not enough RAP binder is present, mixture properties are not altered. At high RAP contents (ranging from 15 to 30%) the differences become significant

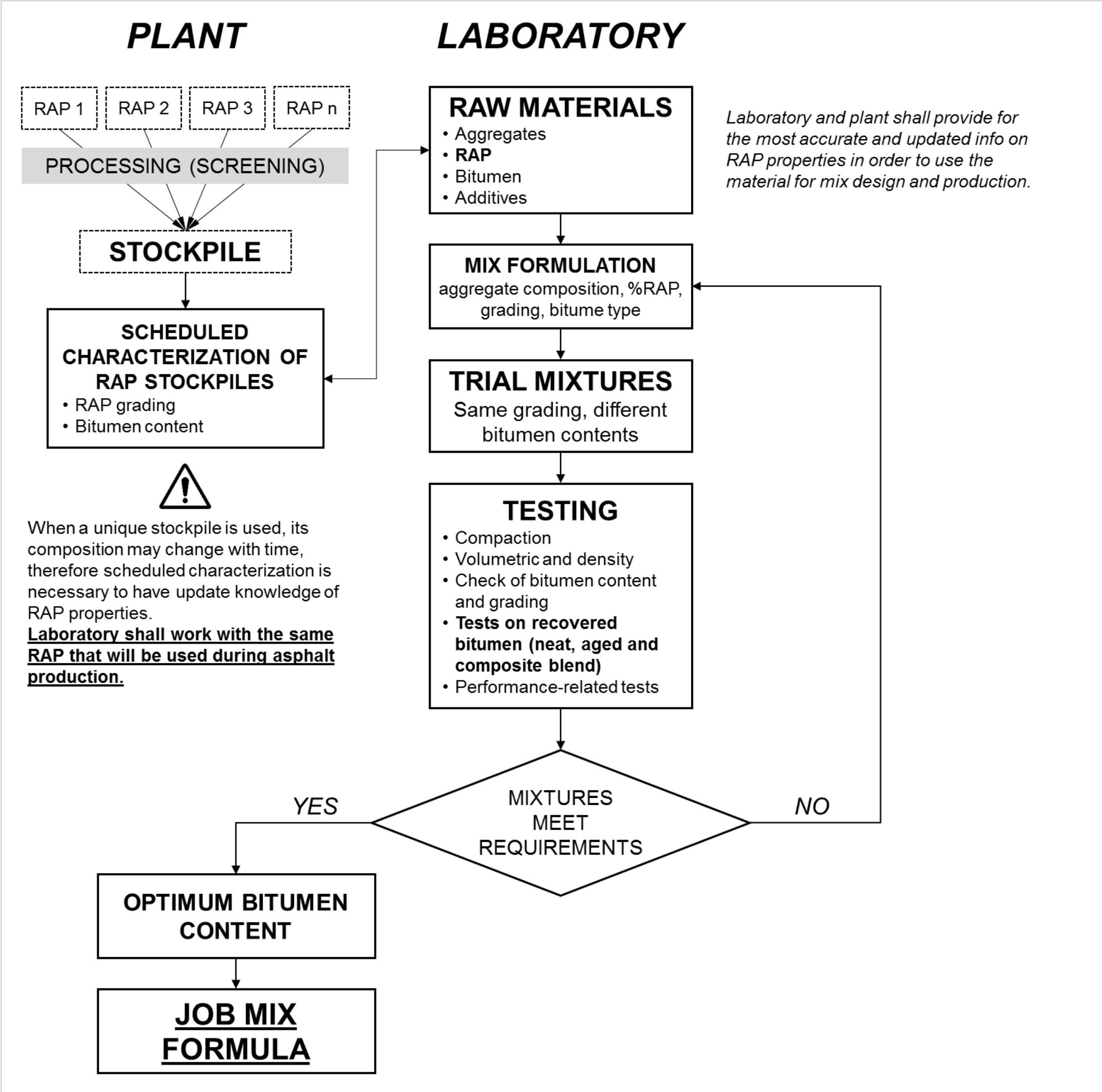
Mix design procedure presented before should be adjusted for RAP mixtures in order to take into account of the variability of RAP and of the presence of aged bitumen in the milled material. This requires an accurate characterization of stockpiles where RAP is taken from in order to produce mixes in plant. Also, laboratory tests shall be performed to investigate the effect of RAP on mixture's performance and composite bitumen grade. Mix design procedure for mixtures incorporating RAP is presented in Fig. 5.9.

Mix design procedures for traditional and RAP mixtures are similar. The goal is to determine the job mix formula that produces an asphalt mixture with desired performances. The job mix formula represents the best combination of raw materials for a specific use of the mixture considering traffic, temperature and environmental conditions to which the material will be subjected during its service life.

An important aspect of RAP mix design is the routine characterization of RAP stockpiles that should be carried out at the plant. This requires a minimum of laboratory equipment at the plant to perform grading and extraction of bitumen from RAP samples. Alternatively, an external laboratory may conduct characterization tests. In either cases, routine characterization is necessary but leads to additional production costs.

Despite raised production costs, characterization of RAP limits negative effects of RAP variability, leading to more consistent information on raw materials that can be used for more accurate mix design. Hence, properties of designed, produced and paved materials are under control and subjected to limited variation, helping to reduce uncertainty related to RAP mixtures.

Fig. 5.9: flow-chart of mix design procedure for asphalt mixture incorporating RAP



5.4.3.3 Research mix design

5.4.3.3.1 Objective and procedure

The objective of the mix design procedure used in the present research is to produce four version of the same asphalt mix, each with “reasonably” identical properties (grading, bitumen content, density, air voids) and different RAP content, precisely 0%, 10%, 20% and 30% by mass of aggregates. RAP mixes must be similar in order to make the RAP content the only significant variable affecting results of further testing. However, limited and controlled differences between mixes are inevitable and expected. Thus, the research mix design aims to determine the effects of RAP content on performance and durability of a specific asphalt mixture made with different RAP percentages. This approach is different from that of standard mix design, which aims to define the best job mix formula of an asphalt mix.

The idea is that RAP can substitute virgin material as aggregate fractions and bitumen. For example, the finest RAP fraction can be used instead of sand. The RAP 8/12 can be used instead of gravel 4/6 or 6/10. RAP aggregates may considerably reduce the amount of virgin aggregates if correctly incorporated into RAP mixtures i.e. with an accurate mix design. For RAP bitumen, which is the aged bitumen film that coats RAP aggregates, used as substitute of virgin bitumen, further considerations are needed about the interaction between neat and virgin bitumen.

The research mix design procedure proposed here relies on considerations summarized below:

- The 0% RAP (reference) mix formulation is known. The asphalt mixture is routinely produced by S.A.P.A.B.A and classified as AC 20 BINDER 50/70 (CE marking)
- Four mixtures are investigated with 5% air voids (by volume of the mix), 5% bitumen content (by mass of aggregates) and similar grading curve. Mixtures mainly differ in RAP percentages of 0% (control), 10%, 20% and 30% by mass of aggregates
- The bitumen content of the mixture is calculated considering the presence of RAP bitumen under the assumption of total blending between virgin and old bitumen. Thus, for a specific binder content, the higher the RAP content, the lower the virgin bitumen added
- RAP is used as “substitute” of virgin materials. Mixtures are produced with same grading, volumetric properties and bitumen content using less virgin material. The research programme will investigate performances of asphalt mixtures produced saving virgin materials
- Attention to operative details is necessary to ensure good quality of produced mixtures, which means concurrence of target and measured mixtures properties. Laboratory operations require care, precision and accuracy to obtain desired mixtures. Recommendations are included in the research mix design

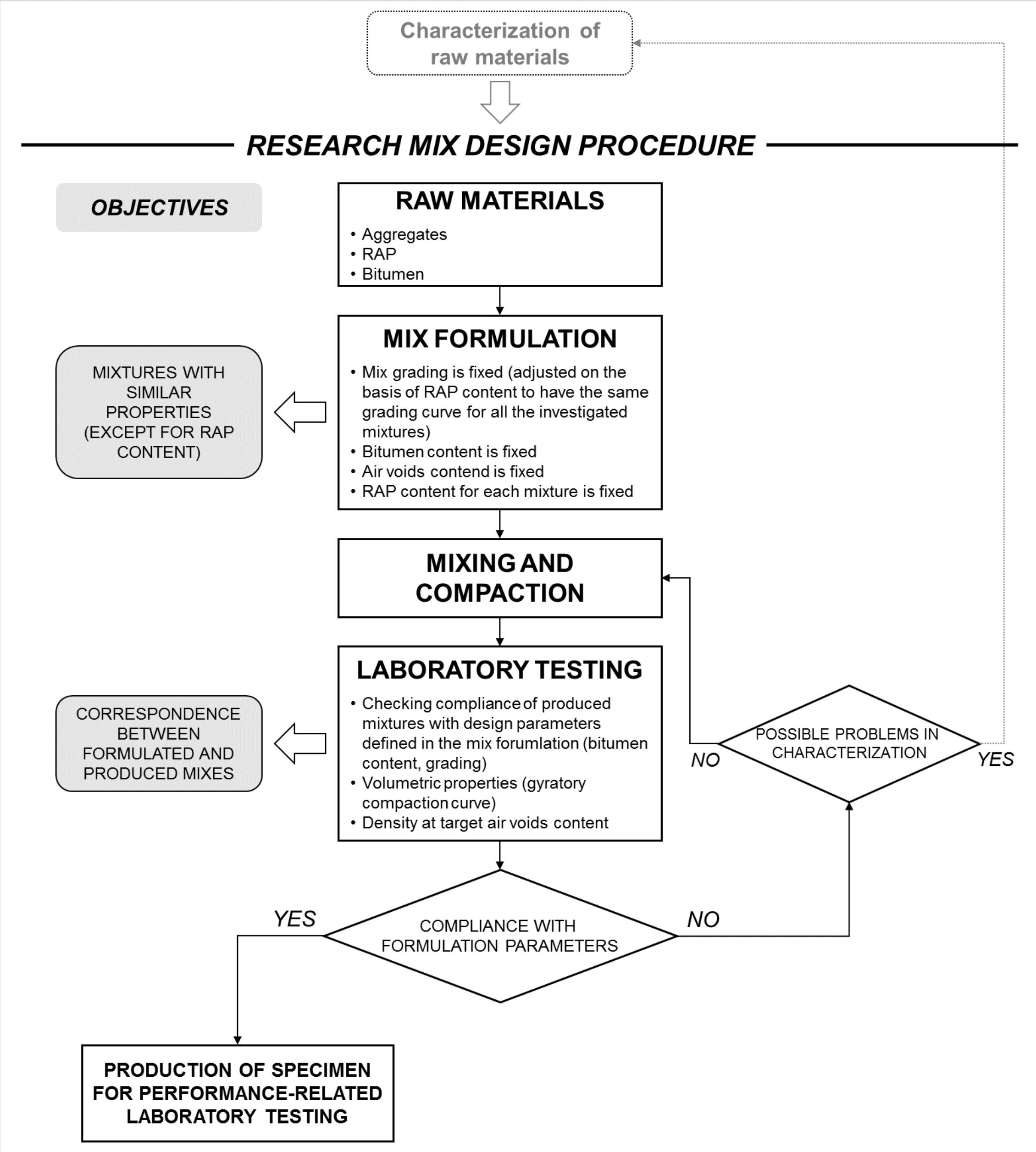
The research mix design procedure is composed of the following steps:

1. **Raw materials**: selection of aggregates, bitumen and RAP previously characterized (see 5.4.2 for details on characterization of raw materials)
2. **Mix formulation**: mix grading, bitumen and air voids content are fixed. Composition is adjusted on the basis of the RAP content to obtain the same grading curve for each mix. The same is done for bitumen content: the presence of old bitumen reduces the amount of virgin binder needed
3. **Mixing and compaction**: raw materials are mixed together. The resulting loose asphalt is compacted with gyratory compactor
4. **Laboratory testing**: the compliance of produced mixtures with parameters defined in the mix formulation is checked verifying grading curve and bitumen content. Volumetric properties are determined from the compaction curve. Density corresponding to design air voids content is determined (target density). This value of density (one for each mixture) will be used in further step of specimen production. At this point, if measured and design parameters correspond, the mix formulation and the production procedure are validated, thus the next phase of production of specimens for performance-related tests may start. If measured and design

parameters significantly differ, the produced material shall be discarded and the mixing repeated. Errors may come (but should not) from inaccurate characterization of raw materials. In this case, characterization shall be repeated, especially on RAP

The procedure flow-chart is reported in Fig. 5.10.

Fig. 5.10: flow-chart of research mix design procedure



5.4.3.3.2 Interaction between virgin and RAP bitumen

As mentioned before, the interaction between RAP and neat bitumen is a key issue of mix design of RAP mixtures. Three levels of blending are usually identified:

a. **Black rock**: there's no interaction between neat and aged bitumen. Consequently, RAP not significantly change the virgin binder properties and it can be considered as an aggregate. McDaniel [10] shows that RAP does not act like a black rock in hot asphalt mixtures. Black rock is more representative of asphalt cold mixes made with bituminous emulsion or foamed bitumen

b. **Total blending**: virgin and RAP bitumen fully blend together resulting in a composite bituminous blend with properties that are significantly affected by the presence of old bitumen. Many design procedures assumes that all the aged binder is fully available in the mixture and would effectively contribute to the blend. This means that the amount of virgin asphalt binder can be reduced by the full amount of asphalt binder in the RAP for the percentage specified [9]
c. **Actual practice**: it's an intermediate level of interaction between black rock and total blending. However, studies show that total blending and actual practice are not significantly different from each other [13]

One of the three levels of blending must be assumed for the mix design. The level of blending cannot be directly measured on bitumen extracted from RAP mixtures, because extraction process removes at least some of the RAP binder, whether it has actually combined with the new binder or not. Mixture tests sensitive to the binder properties may be used to resolve this issue.

On the basis of previous considerations, total blending between virgin and RAP bitumen is assumed. As important practical implication, the amount of virgin bitumen can be reduced because of the presence of RAP bitumen. The higher the RAP percentage, the lower the virgin binder required.

5.4.3.4 Mix formulation

Four asphalt mixtures are produced combining virgin aggregates, reclaimed asphalt and neat bitumen. Proportioning of virgin aggregates and RAP aims to produce mixtures with closest grading curve. The bitumen content is determined taking into account for the presence of aged binder into reclaimed asphalt.

Table 5.7: composition of the four investigated asphalt mixtures

Mixture	Virgin aggregates	RAP	Neat bitumen
Mix0 (control mix)	Gravel 10/20 Gravel 6/10 Gravel 4/6 Sand 0/4 Filler	No	Pen grade 50/70 (not modified)
Mix1	Same as Mix0	10% RAP 0/8 and 8/12	Pen grade 50/70 (not modified)
Mix2	Same as Mix0	20% RAP 0/8 and 8/12	Pen grade 50/70 (not modified)
Mix3	Same as Mix0	30% RAP 0/8 and 8/12	Pen grade 50/70 (not modified)

The upper limit of 30% RAP incorporated into the mixture was chosen on the basis of actual use of RAP in batch plants. These plants may use 10-20% of RAP and very rarely RAP content exceeds 40% [2]. Steam generated from the contact between heated aggregates and RAP, clogging of screens and elevators, need to overheat aggregates and increased mixing time are amongst the factor that limit the use of RAP in batch plants. Drum plants are able to incorporated greater amount of RAP. Most conventional drum plants can routinely accommodate for 50% RAP.

Proportions of raw materials are reported in Table 5.8 for the investigated mixtures.

Table 5.8: proportion of raw materials for the investigated RAP mixtures

RAP mixtures composition (% by mass of aggregates)				
Aggregates and RAP	*Mix0 (control mixture)*	*Mix1 (control+10% RAP)*	*Mix2 (control+20% RAP)*	*Mix3 (control+30% RAP)*
Gravel 10/20	25	26	26	23
Gravel 6/10	16	13	14	15
Gravel 4/6	16	16	14	15
Sand 0/4	40	33	24	15
Filler	3	2	2	2
RAP 0/8	0	5	8	14
RAP 8/12	0	5	12	16

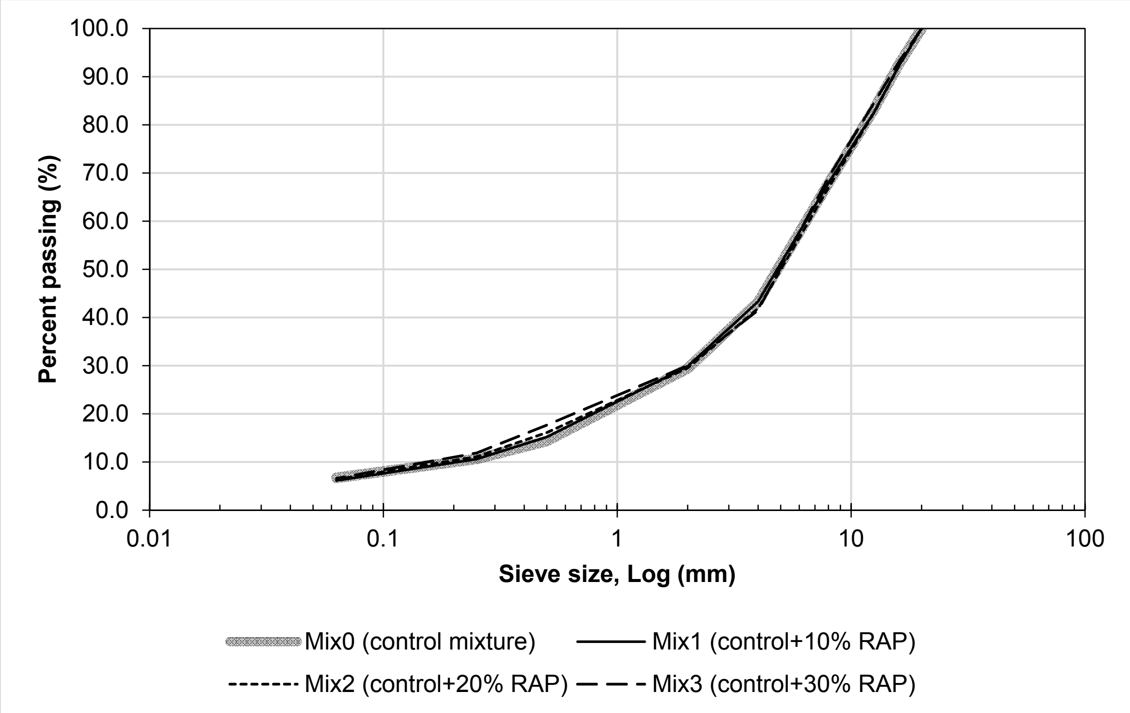
Percentages expressed in Table 5.8 refer to the aggregate blend, including RAP. The percent of RAP passing each sieve refers to the extracted material.

The percent passing of each aggregate and RAP fractions, obtained from characterization testing, are combined together following the composition above, and the grading curves of the four RAP mixtures are determined. Grading curves of RAP mixes are represented in Fig. 5.11.

Table 5.9: percent of material passing at each sieve for each investigated mixture

Determination of particle size distribution of aggregates, EN 933-1:2012				
Sieve size (mm)	*Percent of material passing the correspondent sieve (% by mass of the sample)*			
	Mix0 (control mixture)	*Mix1 (control+10% RAP)*	*Mix2 (control+20% RAP)*	*Mix3 (control+30% RAP)*
20	100.0	100.0	100.0	100.0
16	92.5	92.2	92.2	93.1
12.5	83.2	82.5	82.5	84.5
8	67.8	67.9	67.0	69.1
4	43.2	43.3	41.9	41.6
2	29.4	29.9	29.5	30.0
0.5	14.3	15.2	16.1	17.7
0.25	10.6	10.6	11.1	11.9
0.063	6.7	6.2	6.4	6.6

Fig. 5.11: grading curves of investigated mixtures



Increasing the RAP content causes an increase of the fines, in particular of the sand fraction. Grading curve of Mix3 shows a small shift towards upper values of percent passing in the sieve size range from 0.25 to 2 millimetres. Data reported in Table 5.9 show a decrease in virgin sand content, from 40% of Mix0 to 15% of Mix3, as the RAP content increases.

As confirmed by actual practice in plant, adding RAP leads to an increase in the sand fraction of the final mixture. This because milling modifies grading of asphalt, producing a material with higher fines/dust content than the pre-milled asphalt.

The high fines content in RAP forces conventional plants to limit the quantity of RAP dried in the drum. Dust and fine aggregates protects other aggregates in the drum from heating, acting like a screen, thus more heat and hence fuel is needed to reach the desired temperature. This causes an increase in fuel consumption and emissions.

Virgin bitumen is added considering the contribution of RAP bitumen. Thus, mixtures with higher RAP content require less virgin binder to reach the target 5% content (by mass of aggregates). The average bitumen content of RAP 0/8 and RAP 8/12 is 4.47% and 4.43% respectively, expressed by mass of the mixture (aggregate and bitumen, b_c) or similarly 4.68% and 4.64% expressed by mass of aggregate (b_a).

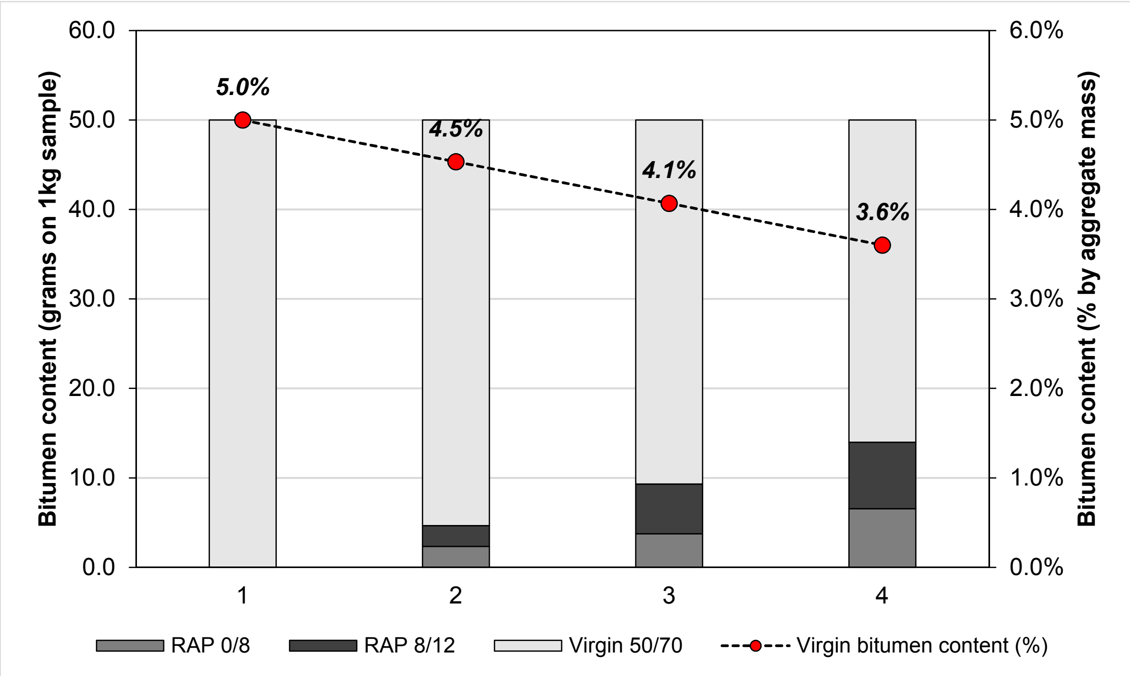
The quantity of virgin bitumen needed, b_{av}, is calculated from the design bitumen content, b_{aD} the RAP bitumen content of each RAP fraction, b_{aRAP} and the percentage of each RAP fraction incorporated into the mixture, R. The generic RAP fraction may be noted with letter "i". Percentages are calculated by aggregate mass, as indicated by subscript "a". Same results can be obtained calculating all percentages by mass of the mixtures, including the mass of bitumen. Required virgin binder can be calculated with the following formula:

$$b_{av} = b_{aD} - \sum R_i \cdot b_{aRAP,i} \ [\% \ by \ aggregate \ mass]$$

The amount of bitumen in grams can be calculated multiplying b_{av} by the mass of aggregates, (including RAP).

Increasing the RAP content up to 30%, virgin bitumen reduces from 5% to 3.6%. For example, considering 1 kilogram of mix, the quantity of RAP and virgin bitumen in the mixture vary as reported in Fig. 5.12.

Fig. 5.12: variation of virgin bitumen in the mixture with the RAP content in 1kg of mixture. Increasing RAP reduces the quantity of neat binder necessary to reach the target bitumen content



As mentioned before, it is assumed that 100% of the RAP bitumen blends with the neat binder. At the moment, there's no method to directly determine the interaction between binders. The effective level of blending occurring between virgin and old bitumen will be (indirectly) determined on the basis of performance-related test results on asphalt and bitumen samples.

5.4.4 Laboratory mixing

Raw materials were mixed together with a planetary mixer. The device has a five levels speed control system which can be adjusted depending on the quantity of fine aggregates in the mixture. The higher the content, the lower the initial mixing speed. During mixing, the bowl, where materials are poured and mixed, is placed on a heating coil, which helps the mix to maintain a constant temperature.

5.4.4.1 Preheating

Before being mixed together, materials are heated or dried at a specific temperature. Aggregates are *heated* at 180°C for 2 hours, RAP is *dried* at 110°C for 90 minutes and bitumen is heated for 2 hours at 165°C.

Heating materials prior to mixing will typically take from 2 to 4 hours, but the actual time required to heat aggregates and binders to reach the specified mixing temperature will vary considerably depending on the size and type of oven used, the amount of material being heated, and the properties of the aggregate. The oven should be set to a temperature about 10°C above the mixing temperature range. The actual temperature of the aggregates and binder should be checked prior to mixing and compaction with a properly calibrated electronic thermometer. Mixing bowls, mixing paddles/stirrers, and gyratory compaction moulds must also be heated to the same compaction temperature range prior to compacting specimens [48].

Experimental studies conducted by McDaniel [10] show that heating RAP at 110°C for 4 hours or at 150°C for 2 hours has no effect on RAP binder properties i.e. complex shear modulus G* measured with DSR. Higher temperatures and longer heating times have been shown to change the properties of some reclaimed asphalts. These results were gathered in the NCHRP manual "Recommended use of RAP in the Superpave mix design method", where a heating temperature of 110°C for a time of no more than 2 hours is recommended for sample sizes of 1 to 2 kg [13].

When heating bitumen, temperature shall be regulated on the basis of bitumen type. The harder the bitumen (with reference to pen grade system), the higher the temperature. Polymer modified bitumens also require higher temperature relative to that of traditional binders. Bitumen overheating may occur due to too high temperature or too long heating time or both, causing bitumen ageing. Some practical recommendation on bitumen heating: first, bitumen shall be put into the oven in a metallic can with a lid on top. The can should be covered and not sealed by the lid; second, bitumen can shall be placed far from heating source, for example heating coils, otherwise the sample may be overheated.

Preheating time and temperature are presented in Table 5.10.

Table 5.10: Temperature and time of preheating of raw materials

Raw material	Preheating temperature and time
Virgin aggregates	180°C, 4 hours
RAP	110°C, 90 minutes
Bitumen 50/70	165°C, 2 hours

5.4.4.2 Batching

Aggregates must be weighed after being dried. The same principle is valid for RAP. Reclaimed asphalt contains a certain amount of aged bitumen. Because of the assumption of total blending in the mix formulation, the mass of RAP added to the mixture shall not include the mass of aged bitumen. Therefore, the mass of unprocessed RAP (not being subjected to extraction) must be greater than the mass of extracted RAP required from mix formulation.

This is better explained with an example. Let's say we're preparing a mixture which requires 1 kg of RAP aggregate (extracted RAP), from mix formulation. RAP has 5% binder content by mass of aggregates (b_a). The mass of unprocessed RAP shall take into account of the presence of binder. First, the binder content is expressed as percentage by mass of the mixture (b_c):

$$b_c = 100 \cdot \frac{b_a}{1 + b_a} \ [\%]$$

Calculated binder content b_c is 4.76%. Then, the quantity of unprocessed RAP, in grams, necessary to obtain 1000 g of RAP aggregate is calculated with the following formula:

$$m_{RAP} = \frac{m_{RAP,agg}}{1 - b_c} \ [g]$$

The calculated mass of unprocessed RAP is 1050 g. This is the amount of material that shall be considered during batching.

It's important that raw materials are put in the bowl from the oven and mixed as quickly as possible, in order to limit temperature drops. This means that materials must be preferably weighed and prepared before being mixed together. Aggregates are first dried at 180°C for two hours, then the exact quantity of each class is put in separate pans and in the oven, ready to be poured in the bowl after four hours of heating. The same is done for RAP, which is first dried at 60°C for three hours, then put in the oven at 110°C for two hours in pans containing the exact quantity of material.

Bitumen is weighed directly in the bowl. The bowl with RAP and aggregates is placed on the balance, which is set to zero. Bitumen is carefully and slowly poured from the metallic can into the bowl, monitoring measures on the balance's display. Pouring the exact amount of bitumen into the bowl is essential to obtain consistent mixtures.

5.4.4.3 Mixing

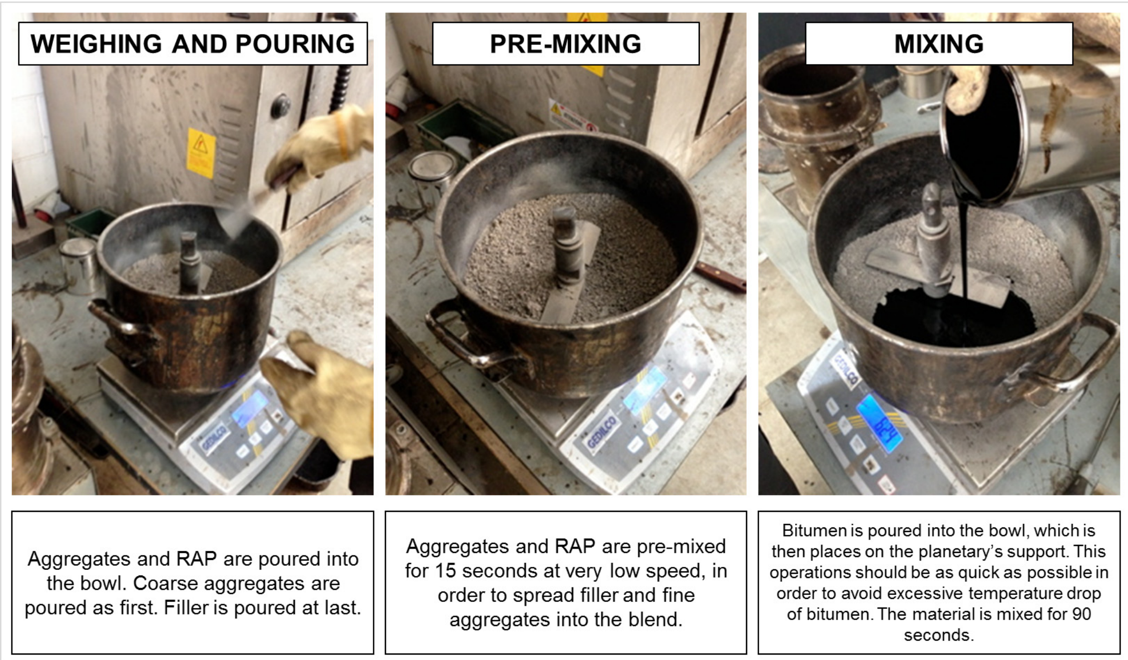
Mixing is a very important part of the study because raw materials are combined and mixed together following compositions defined in the job mix formula. A good mixing produces asphalt mixtures with desired bitumen content and aggregate grading. Together with compaction, mixing is also necessary to ensure the target volumetric properties and density of mixtures.

Mixing consists of the following steps:

1. **Weighing and pouring**: aggregates and RAP prepared as described in 5.4.4.2 are weighed and poured in the hot bowl, which is placed on a balance. Balance is set to zero after each aggregate class is weighed. Coarser aggregates are poured first, whereas filler is the last to be poured in the bowl

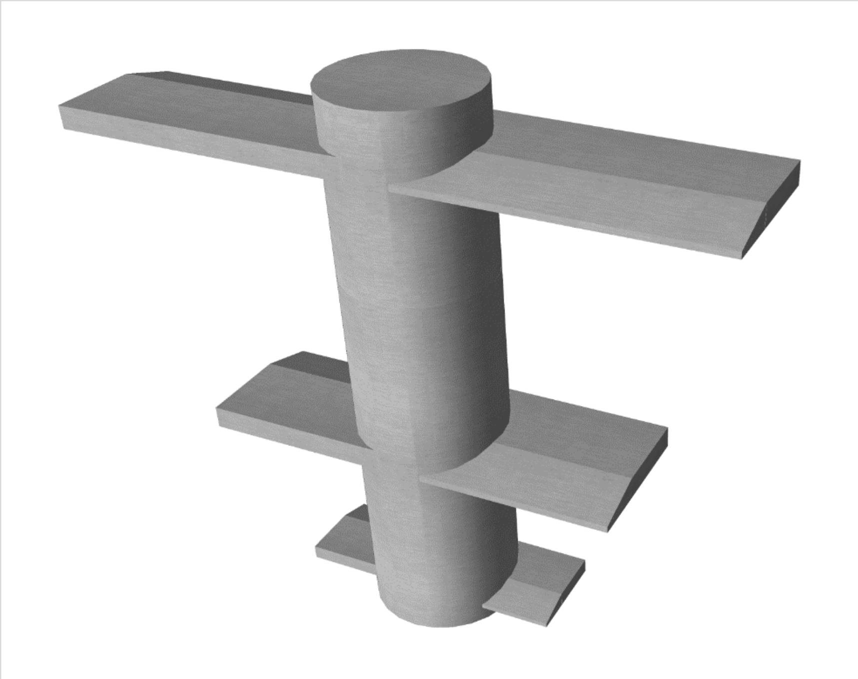
2. **Pre-mixing**: the blend of virgin and RAP aggregates is mixed with planetary at very low speed for 15 seconds. This is done to distribute the finest aggregates into the blend, which would be more difficult after having poured bitumen. The bowl is put into the hoven to recover temperature that has been lost during this first two steps
3. **Mixing**: after 10-15 minutes in the oven, the bowl is placed again on the balance. Bitumen is poured into the bowl in the desired quantity. The bowl is mounted on the planetary apparatus and the material is mixed at a specific speed for 90 seconds

Fig. 5.13: phases of mixing with laboratory planetary device



When mixing with a laboratory planetary device, segregation of fine aggregates may lead to inaccurate mixing and hence to differences in binder content and grading from design specifications. Segregation occurs between coarse aggregates and bituminous mastic, which is composed of fine aggregates and bitumen. Mastic tends to stuck on the bottom of the bowl if not adequately mixed by the whist. The shape of the whist is not specifically designed for mixing asphalt materials and its ineffectiveness may be a problem, especially when polymer modified bitumens are used. For this reason, a new aluminium screw was designed and produced. The screw is illustrated in Fig. 5.14.

Fig. 5.14: aluminium mixing screw for planetary device designed and produced for the research



5.4.5 Volumetric testing

5.4.5.1 Objectives and procedure

Volumetric properties of the mixtures are investigated with a gyratory compactor. The objectives of this step are:

a. Check the compliance of measured bitumen and air voids content with mix formulation values
b. Determine density (bulk density) of the mixture at 5±0.7% air void content from compaction curves

Test configuration is described in Table 5.11.

Table 5.11: configuration parameters of gyratory device and specimen info

Parameter	Value
Vertical pressure	600 kPa
Internal angle	21.8 mrad
Speed of rotation	30.6 gyrations/minute
Specimen diameter	150 mm
Specimen mass (target)	4500 g
Temperature	165°C

Fig. 5.15: gyratory compactor used for the present research

Mixtures are formulated with 5% bitumen and air void contents, the first expressed by mass of aggregates, the second by volume of the mixture. If all previous operations have been performed correctly, the same design values should be obtained from laboratory tests. Otherwise, mixing or characterization of raw materials should be repeated.

Determining density at 5±0.5% V_m is necessary to calculate the amount of loose hot asphalt being put into a defined prismatic volume (asphalt slab) of the roller compactor.

The volumetric testing is carried out according to European standards. For each asphalt mix, three cylindrical specimen are produced with gyratory compactor according to EN 12697-31. Air void content and density are measured with reference to EN 12697-5, EN 12697-6 and EN 12697-8.

The key steps of the procedure are summarized below:

1. The soluble binder content of a portion of loose asphalt is determined according to UNI EN 12697-1. Then, on the extracted material, the apparent density of aggregate is calculated (EN 1097-6). Parameters b_c, the bitumen content by mass of the mixture, and ρ_A are determined. The maximum density of the asphalt mix ρ_M is calculated with the following formula [47]:

$$\rho_M = \frac{100}{\frac{P_A}{\rho_A} + \frac{b_c}{\rho_{bin}}} \ [kg/m^3]$$

Where P_A is the percentage of aggregate by mass of the mixture (P_A+b_c = 100%) and ρ_{bin} is the bitumen density (approximately 1020-1030 kg/m^3)

2. The remaining portion of loose asphalt is compacted with the gyratory device. Three samples of 4.5 kg each are produced at a specific N_{max}. In this case, N_{max} is set to 200 gyrations
3. The gyratory compactor software calculates geometric density $\rho_{b,m}$ of the specimen at each cycle. Density is calculated from the mass of the sample, weighed just before to start compacting, the diameter (150 mm in this case) and the specimen's height measured at each gyration. Bulk density calculated by dimensions, as noted in the EN 12697-6, is usually smaller than the actual density of the specimen, thus leading to greater air void content. The volume of the specimen is slightly less than the volume of a smooth-sided cylinder because of surface irregularities. For this reason, geometric density measured at each cycle is multiplied by a correction factor "c" calculated as ratio of the dry bulk density of the specimen compacted at N_{max} and the geometric density of the specimen at N_{max} as indicated in the following formula:

$$c = \frac{\rho_{b,dry}(N_{max})}{\rho_{b,m}(N_{max})} \; [-]$$

4. The geometric density at generic gyration N, $\rho_{b,m}(N)$, is corrected with factor "c" and the corrected bulk density $\rho_b(N)$ is calculated
5. The percent of maximum density reached at each gyration is calculated from the corrected bulk density as follows:

$$\%\rho_M(N) = \frac{\rho_b(N)}{\rho_M} \; [\%]$$

6. The compaction curve, obtained plotting the number of gyrations N (Log) versus the percentage of maximum density $\%\rho_m(N)$ at each cycle, is plotted for each sample
7. From compaction curves, bulk density at 5±0.7% air void content is measured. This value of density will be used in the next step of production of asphalt slabs with roller compactor

Results of laboratory tests and gyratory compaction are reported below.

5.4.5.2 Results

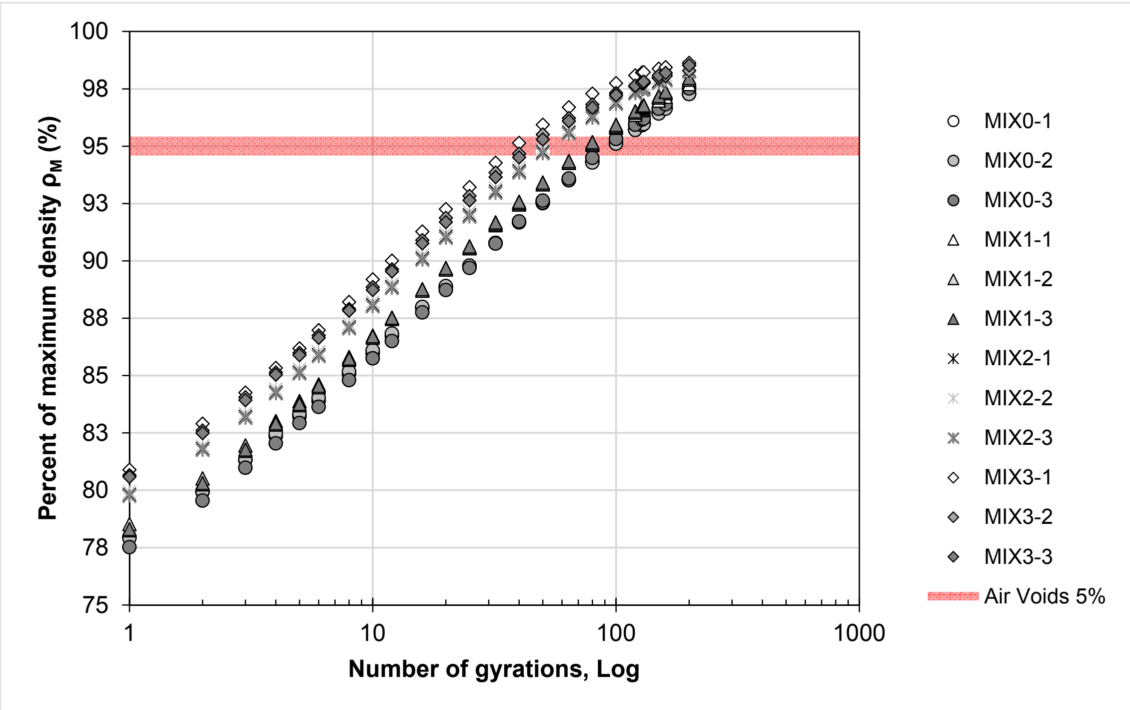
Results of binder extraction show a bitumen content slightly below target value of 5% (by mass of aggregates) for all mixes. The average binder content of Mix0 is 4.29%, 0.7% below the target. The small standard deviation, despite the few data available; indicates good consistence of the mixing procedure. Mix0 was the first mixture to be produced, thus unfamiliarity with operations might be the cause of this variance.

Pouring of bitumen into the bowl is a very delicate operation. Previous similar laboratory works have shown that part of the bitumen clings to the bowl during mixing, reducing the effective bitumen content of the mixture. In order to avoid this issue, a slightly larger amount of bitumen (approximately 3-5% of the target quantity) is poured into the bowl during mixing performed in the next step of the research of asphalt slab production.

Table 5.12: results of bitumen extraction on asphalt mixes

Bitumen content of cylindrical samples produced with gyratory compactor, b_a (%) EN 12697-1				
Sample	*Mix0*	*Mix1*	*Mix2*	*Mix3*
1	4.38	4.65	4.77	4.54
2	4.29	4.73	4.63	4.63
3	4.20	4.57	4.72	4.72
Average	4.29	4.65	4.71	4.63
St. Dev.	0.09	0.08	0.07	0.09

Fig. 5.16: compaction curves of asphalt mixes Mix0, Mix1, Mix2 and Mix3 compacted with gyratory device. Red line indicates the target 5% void content (95% of ρ_M)



Compaction curves of asphalt mixes are reported in Fig. 5.16. Slope of compaction curves "c" is calculated in the range of 10 and 100 gyrations with the following formula:

$$c = \frac{\%\rho_M(100) - \%\rho_M(10)}{90}$$

Values of compaction curves slope are reported in Table 5.13.

Table 5.13: compaction curves slope

Sample	Mix0	Mix1	Mix2	Mix3
1	0.102	0.101	0.098	0.095
2	0.100	0.102	0.098	0.094
3	0.106	0.102	0.098	0.094
Average	0.103	0.102	0.098	0.094

From volumetric testing results, the following considerations are made:

a. Compaction curves of Mix0 samples show similar values of $\%\rho_M$. The same can be noted for Mix1 and Mix2. Samples of Mix3 exhibits slightly larger variation of $\%\rho_M$. Densification appears similar for the compacted samples of each mixture, indicating consistent volumetric properties of each production
b. Parallelism of compaction curves reported in Table 5.13 indicates a good control between mixes in relation to aggregate structure. Researches [14] show that compaction slope, unlike mixture performance, is not sensitive to asphalt binder content. Paired mixtures with the same aggregate structure, but asphalt contents different by 1.0%, may have similar compaction slopes, but different mix properties
c. At a specific number of gyrations, compaction curves shift upward to greater values of $\%\rho_M$ as the RAP content increases. Thus, mixes with higher RAP percentage require a smaller compaction effort to reach the target void content. Target air void content (95% ρ_M) is reached at the following gyrations: for Mix0 N is between 80 and 100; for Mix1, N is 80; for Mix2, N is 50; for Mix3, N is between 40 and 50. This may be explained by the increase in sand fraction of the asphalt mix caused by the increased RAP content. Percentage of retained aggregate at sieves 2 mm, 0.5 mm and 0.25 mm increases with RAP content, as reported in Table 5.9 and illustrated in Fig. 5.11. The increase in sand fraction is due to the grading of RAP 0/8. Percent of material passing at 2 mm sieves is 73.8% indicates a material with high content of fines, as expected when dealing with RAP

To conclude, volumetric testing show mixtures with similar compaction curves. This indicate good control of the production process that can be therefore repeated in the next step of the research. Pouring of bitumen shall be made adding a slightly larger amount of bitumen. Increasing the RAP content, densification curves shift upwards, indicating that the higher the RAP content, the smaller the compaction effort required to reach the target air void content.

Target density values used to produce asphalt slabs in the further step of the research are determined from volumetric tests. For each mix, lower and upper density limits are determined corresponding to air void content closest to 5%. The slab target density at 5% air void content is calculated as average of lower and upper limits.

Table 5.14: target density calculated from volumetric testing with gyratory compactor

Mixture	**Lower limits**		**Upper limits**		**Slab target density at 5% air void content (g/cm^3)**
	Density (g/cm^3)	Air voids (%)	Density (g/cm^3)	Air voids (%)	
Mix0	2.341	5.7	2.367	4.7	2.354
Mix1	2.354	4.9	2.357	4.8	2.356
Mix2	2.348	5.3	2.354	5.1	2.351
Mix3	2.344	5.5	2.369	4.5	2.357

5.4.6 Production of prismatic samples

The objective of this research step is to produce prismatic asphalt samples with similar bitumen content, density and in compliance with other specifications laid down in the mix formulation. Asphalt beams will be further subjected to flexural fatigue testing to investigate their performances and durability.

Mixes are produced with the same lab mixing procedure described above and validated by volumetric test results. As specified in 5.4.5.2, a slightly larger amount of virgin bitumen 50/70 is poured into the bowl to compensate bitumen that clings to the bowl during mixing.

In this step, loose asphalt is compacted with a laboratory roller compactor designed and realized for this research. Previous lab works have shown that the device is able to simulate the actual roller compaction on site. The roller compactor is capable to maintain asphalt temperature with two heating systems, one mounted below the device, heating the loose asphalt, and one mounted on the steel roller, which avoid temperature shocks caused by the contact between the cold roller and the hot asphalt surface. The device has been described in detail in the previous chapter.

First, asphalt mixtures are produced following the procedure described in previous sections. Raw materials are heated in the oven at different temperature, weighed and blended together with a lab planetary mixer. The produced loose asphalt is put in the oven to recover temperature drop that may occur during mixing. Once asphalt is at the desired compaction temperature of 165°C, it’s quickly put into the roller compactor lodge. Asphalt slabs are produced compacting material to the desired density. Slab target density was determined in volumetric testing.

Two parameters determines the density of the slab: the quantity of loose asphalt put into the lodge and the height of the slab at the end of compaction. Dimensions of slab base are defined and cannot be modified. The slab height is defined on the basis of sawing. For example, if we want to obtain two rows of prismatic samples from an asphalt slab, the slab height shall be defined as sum of two prismatic samples height plus the quantity of asphalt material lost during sawing, which is approximately 5 mm for each cut and correspond to the thickness of the saw. The slab height is monitored during compaction on a display which shows the vertical position of the steel roller. The roller moves upwards and downwards into a predefined range. The lower height is defined prior to compaction so that to obtain the desired slab height.

Asphalt slab are 300x450x130 millimetres, prismatic samples are 60x400x50 (width-length-height). Slab dimensions allow trimming of each prismatic sample.

After being compacted, slabs rest for 24 hours until room temperature is reached. Then, slabs are cut with a circular saw into 8 prismatic samples.

Table 5.15: dimensions of asphalt slabs and prismatic samples

Dimensions	Asphalt slab	Prismatic sample
Width (mm)	300	60
Length (mm)	450	400
Height (mm)	130	50

Dimensions and bulk density of each prismatic sample is measured. Density indicates if specimen have been produced correctly. Prismatic samples should have similar density and binder content in order to compare performance-related properties in a statistically significant manner. In fact, comparing fatigue resistance or stiffness of samples with significantly different density may lead to erroneous conclusions.

For example, let's say we have two prismatic samples of the same asphalt mixture, one with considerably lower density than the other. Fatigue resistance of the low density sample is expected to be lower than the other one because of the higher air void content. In this case, result of fatigue test is affected by the production process. The density variance means that the production has not been carried out correctly. It's important that results are dependent on material properties i.e. RAP content and not on other variables as production process.

Soundness of density data can be measured verifying repeatability of test results as indicated in the European standards. Repeatability of density and air void content is calculated according to EN 12697-6 and EN 12697-8, respectively.

Density results are reported in Table 5.16, where measured maximum density is included. Air voids calculated from measured densities are reported in Table 5.17.

Table 5.16: results of bulk density measurements on prismatic samples obtained from asphalt slab

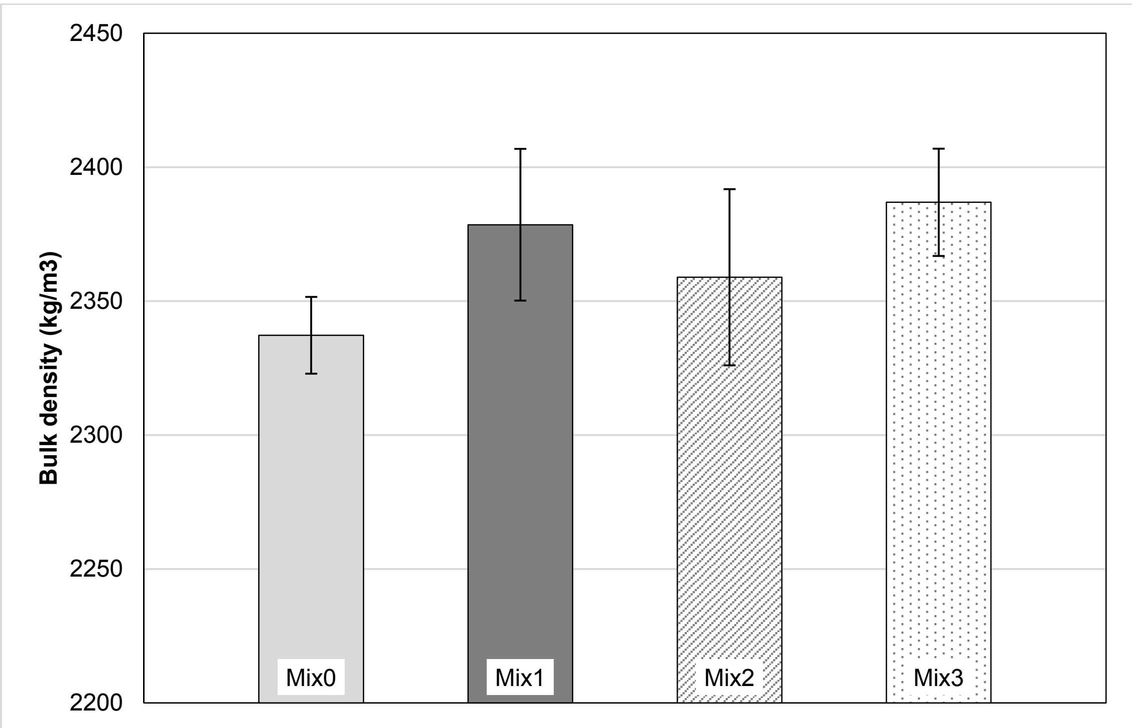
Determination of bulk density (expressed in kg/m^3) of prismatic asphalt specimen, EN 12697-6, Procedure A: Bulk density - Dry				
Sample ID	*Mix0*	*Mix1*	*Mix2*	*Mix3*
1	2332	2401	2401	2378
2	2359	2359	2346	2409
3	2326	2342	2397	2401
4	2349	2403	2322	2369
5	2321	2364	2352	2409
6	2346	2399	2316	2362
7	2327	2412	2388	2401
8	-	2347	2350	2367
Average	2337	2379	2359	2387

Standard deviation	14	28	33	20
Maximum density EN 12697-5 (kg/m^3)	2483	2477	2480	2480

Table 5.17: air void content calculated from measured bulk and maximum density for each prismatic sample

Determination of air void content (% of the mix volume) of prismatic specimen, EN 12697-8				
Sample ID	*Mix0*	*Mix1*	*Mix2*	*Mix3*
1	6.1	3.1	3.2	4.1
2	5.0	4.8	5.4	2.9
3	6.3	5.5	3.4	3.2
4	5.4	3.0	6.4	4.5
5	6.5	4.6	5.2	2.8
6	5.5	3.1	6.6	4.8
7	6.3	2.6	3.7	3.2
8		5.2	5.2	4.6
Average	5.9	4.0	4.9	3.8
Standard deviation	0.6	1.1	1.3	0.8

Fig. 5.17: average bulk density measured for each mixture



Compliance of density results with repeatability indicated in the European standard is determined first considering density measures of each sample into a mix group, i.e. for the 8 samples of Mix1, and then considering density of the entire specimen population. In the first case, repeatability of density measured on different samples of the same material is checked. In the second case, repeatability of density of the whole population is checked.

Repeatability check is made as described below:

a. Estimation of standard deviation s_R of measured data (s_R is an estimator of standard deviation σ_R)
b. Calculation of repeatability "r" according to European standard for the test method
c. Comparison of the ratio s_R/r with confidence interval as function of degrees of freedom v=N-1, where "N" is the number of valid measures

Assuming a normal distribution of the N density measures, average μ and standard deviation σ_R are estimated with the following formulas:

$$\mu: \ \bar{x} = \frac{1}{N}\sum_{i=1}^{N} x_i \quad [kg/m^3]$$

$$\sigma_R: \ s_R = \sqrt{\frac{\sum_{i=1}^{N}(x_i - \bar{x})^2}{N-1}} \quad [kg/m^3]$$

Repeatability is calculated according to EN 12697-6 with the formula:

$$r = 17 + 0.3 \cdot A \quad [kg/m^3]$$

Where parameter "A" indicates the percentage of aggregate in the mixture larger than 11.2 millimetres.

If the ratio s_R/r is included within confidence interval, results are validated and in compliance with European standard requirements. Confidence intervals depend on degrees of freedom of the distribution, as reported in Table 5.18.

Table 5.18: confidence interval for normal distribution as function of degrees of freedom

Degrees of freedom v = N-1	Confidence interval lower limits P = 0.025	Confidence interval upper limits P = 0.95
1	0.0316	2.241
2	0.160	1.921
3	0.268	1.765
4	0.348	1.669
5	0.408	1.602
6	0.454	1.551
7	0.491	1.512
8	0.522	1.480
9	0.548	1.454
10	0.570	1.431

11	0.589	1.412
15	0.646	1.354
20	0.692	1.307
25	0.724	1.275

Results of density repeatability results for each series of prismatic samples are reported in Table 5.19. Repeatability check is positive, since the ratio of estimated standard deviation and repeatability is comprised into the confidence interval. This means that each sample of a mix series is representative of the mixture behaviour. Thus, operations made for producing each series prismatic samples can be considered correct, since they gives consistent results. Density can be considered as an indicator of quality of laboratory processes. The lower the variability, the higher the repeatability, the better the quality of operations. Low variability of density means high repeatability of laboratory procedures, resulting in samples representative of mix properties. This conclusion lean on the very subtle distinction between testing the sample and testing the material. Results in Table 5.19 ensure that we're going to test the material and not only the specimen.

Table 5.19: repeatability of density measurements of each group series

Parameter	Mix0	Mix1	Mix2	Mix3
Average (kg/m^3)	2337	2379	2359	2387
Standard deviation s_R (kg/m^3)	14	28	33	20
r (kg/m^3)	23.5	23.8	23.8	23.0
s_R/r (-)	0.61	1.19	1.38	0.87
Degrees of freedom v	6	7	7	7
Confidence interval	0.454 – 1.551	0.491 – 1.512	0.491 – 1.512	0.491 – 1.512
Check result	Validated	Validated	Validated	Validated

Repeatability of density values of the entire population i.e. all prismatic samples is checked. Results reported in Table 5.20 show that density measures of all prismatic specimens comply with repeatability requirements of the EN 12697-6.

Table 5.20: repeatability of density measurements of all prismatic samples

Parameter	All samples
Average (kg/m^3)	2365
Standard deviation s_R (kg/m^3)	30
r (kg/m^3)	24.4
s_R/r (-)	1.24

Degrees of freedom v	31
Confidence interval	0.735 – 1.255
Check result	Validated

As observed before, compliance of repeatability means that testing procedure gives sound results, hence operations to produce all sample of all mixtures were correctly carried out.

To conclude, density was chosen to check accuracy of production process and homogeneity of samples as well. Analysis of test data show that density measures complies with repeatability specifications of EN 12697-6, indicating that specimens have similar properties. Therefore, results of further testing on different samples of different mixture can be compared.

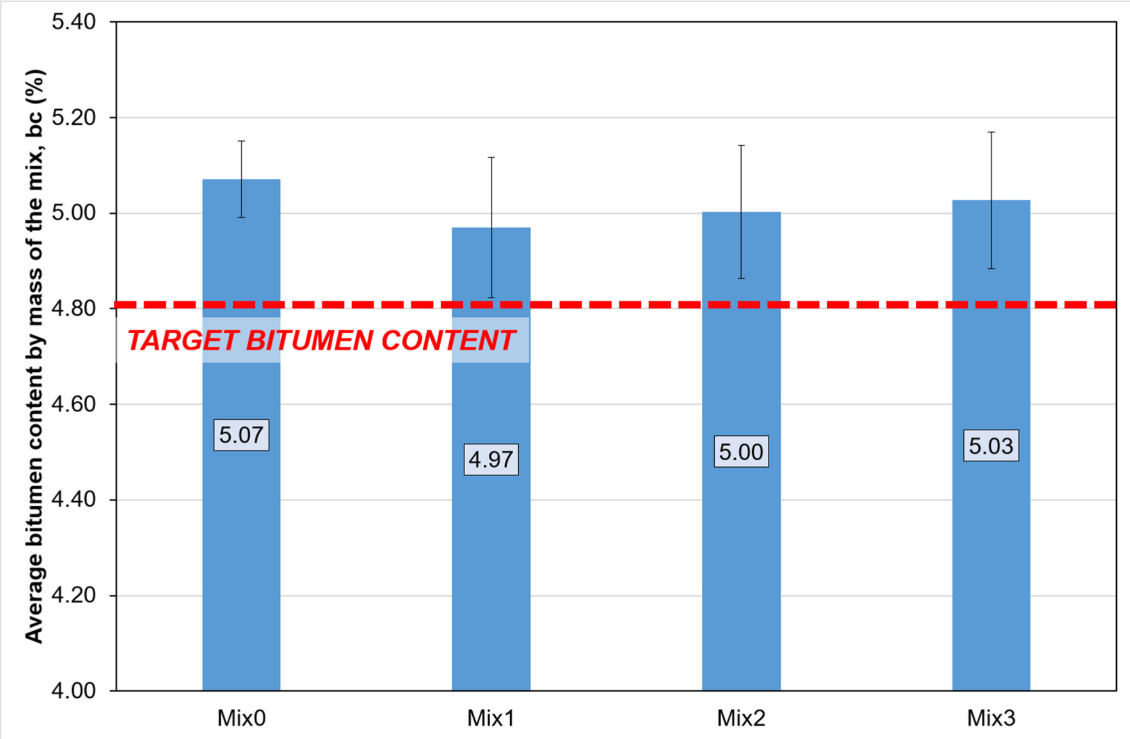
After being tested to flexural fatigue, the bitumen content of each prismatic sample has been determined according to EN 12697-1 using the hot extraction with wired mesh method. Results reported in Table 5.21 and Fig. 5.18 show that the bitumen content is slightly above the target value of 4.8% (by mixture mass, corresponding to 5% by aggregate mass) and, even more important, that the variation of binder content, expressed by the standard deviation, is exceptionally small.

As observed with regards to measured density, the low variability in bitumen content furtherly confirms the homogeneity of prismatic samples. Criteria for judging the acceptability of the binder content of bituminous mixtures determined by the EN 12697-1 are limited [77]. The repeatability r for binder content is 0.3 %, whereas the reproducibility R for binder content is 0.5 %. The variability of bitumen content is within the ranges indicated in the European standard.

Table 5.21: bitumen content by mass of the mix of prismatic samples, after being tested to flexural fatigue

Determination of soluble binder content of investigated mixtures according to EN 12697-1, hot-extraction with wired mesh, b_c (%)				
Specimen ID	*Mix0*	*Mix1*	*Mix2*	*Mix3*
1	5.13	4.81	5.12	5.26
2	5.07	5.10	5.03	4.94
3	5.21	4.77	4.76	
4	4.97	4.90	5.24	5.07
5	5.05	5.17	4.95	4.86
6	5.00	4.92	5.05	5.02
7		5.12	4.86	5.18
8			5.01	4.86
Average (%)	5.07	4.97	5.00	5.03
Standard Deviation (%)	0.08	0.15	0.14	0.14

Fig. 5.18: bitumen content of prismatic samples. Binder content has been determined after fatigue tests



5.4.7 Flexural fatigue testing

In the present research, flexural fatigue tests with four-point bending device have been performed in strain-controlled mode of loading. 7 prismatic samples were investigated at three different strain level of 100, 150 and 200 με. Details are given in Chapter 6 together with test results. In this section, considerations on laboratory tests are given.

Fatigue is a mechanism of failure that develops in a wide range of materials, which are subjected to small magnitude and repeated loading throughout a given period of time. Carpenter [97] has defined fatigue resistance of asphalt mixtures as the ability to withstand repeated bending without fractures, formed by cracks caused by repeated traffic loading.

During their service life, asphalt pavements undergo thousands or millions of loads due to the passage of cars and trucks. Each passage causes a micro-damage within the pavement structure and leads to more severe damage being accumulated in time. A pavement built with poor-quality materials or designed with inadequate bearing capacity has high chances to experience fatigue failure and to develop severe distress, which might compromise structural and functional properties of the infrastructure.

Loads, environmental conditions, materials and pavement structure determine the resistance to repeated loading. Loads, traffic typology and percentage of heavy vehicles in the traffic spectrum trigger fatigue mechanism. Environmental conditions as presence of water in sub-base or foundation layers and large seasonal temperature variations, not related to the material, contribute to accelerating the fatigue crack propagation. The choice of the materials for each pavement layer plays a significant role in durability of asphalt pavements, together with an accurate structural design of pavement structure. The thickness of each pavement layer is determined considering all the aforementioned loads and external factors together with road materials being paved.

The resistance of asphalt materials to repeated loading can be determined in laboratory with different test methods, which have been reviewed in chapter 3. Results of fatigue tests shall be handled and interpreted with great care, for many reasons.

Firstly, different test methods performed give different results in terms of number of cycles to failure of the same material. This means that fatigue resistance of asphalt mixtures, as it is measured in laboratory, is a highly test-dependent parameter that only partially depends on material properties. For example, indirect tensile test usually gives shorter number of cycles to failure relative to flexural fatigue test, despite are both performed on the same asphalt mixture [62].

Secondly, the number of cycles to failure measured in laboratory cannot be directly correlated to the actual service life of the pavement. Fatigue life on site is generally up to ten times larger than the laboratory fatigue life. This because laboratory and site conditions are significantly different in terms of mode of loading, resting periods, specimen geometry, environmental and atmospheric actions. Thus, laboratory testing methods are fairly able to capture the large variety of conditions present on site. Several attempts were made to make lab test more representative e.g. including resting periods in the test procedure, on order to better simulate a real series of passages. Resting periods are supposed to favour healing of asphalt materials, contributing to increase fatigue life of the material on site. However, studies does not show conclusive results at the moment, and a sound procedure able to include rest periods into current test standards has not been found yet.

According to Di Benedetto [60], several material-related factors affect fatigue life of asphalt mixtures:

- **Type of bitumen**: the harder the bitumen, the better the fatigue strength. Also, asphalt mixes made with modified bitumen often exhibit better fatigue results than those made with traditional bitumen
- **Bitumen content**: it’s considered to be the factor that influences the most fatigue results. There’s an optimal bitumen content below and above which fatigue life decreases. The optimal binder content varies with mixtures
- **Air void content**: as void content decreases, fatigue life results are less scattered and more consistent. Generally, fatigue life decrease as void content increases
- **Aggregates**: solidity and adherence properties affect the fatigue resistance of mixes
- **Filler content**: it’s related to bitumen content since it contributes creating the asphalt mastic within the mixture. The optimal filler content with respect to fatigue strength has been found to be between 7% and 9%. It seems that the filler/bitumen ratio determines the fatigue resistance of the asphalt mixture

All factors listed above influence results of laboratory fatigue tests. Manufacturing of asphalt samples has a significant impact on test results. The process involves several steps that should be accurately carried out in order to obtain homogeneous and hence samples that are representative of the investigated mix. Generally, test methods are highly sensitive to small variations as material properties, test set up, temperature and so on. Everything shall prepared with care in order to obtain a sound fatigue response.

Fatigue tests are complex, time consuming and require trained technicians to be performed and expert engineers to be properly interpreted. Specific and expensive equipment is also needed. Some methods require less effort in terms of time consumption and instrumentation. For example, the indirect tensile fatigue test is the most commonly performed fatigue test since it does not require special shaped specimen and uses the same equipment that can be used for other tests e.g. the indirect tensile stiffness. Some other methods are supposed to better represent the actual behaviour of asphalt mixtures in field, as flexural fatigue tests, in spite of the higher complexity of the test procedure and the specific equipment.

Also, fatigue tests are generally not conclusive. Thresholds and reference values of fatigue test results cannot be easily defined and hence are generally not included into technical guidelines. This means that fatigue test cannot be used for passed/not passed purposes with reference to quality control. For example, reference values can be defined for Marshall, indirect tensile and stiffness tests, making test results able to state if the material complies or not with required specifications. With reference to fatigue tests, most of the times results may not be compared to reference values, therefore the test provides only for a general indication on the material quality.

For these reasons, fatigue tests are not routinely performed in the majority of testing material laboratories, in Italy. The ITFT may be an exception. However, if properly conducted and interpreted, fatigue tests can give precious information on performances and durability of asphalt mixtures, since they're very sensitive to material properties.

The fatigue characteristics of asphalt mixes are usually expressed as relationships between the initial stress or strain and the number of load repetitions to failure determined by using repeated flexure, direct tension, or diametral tests performed at several stress or strain levels. The fatigue behaviour of a specific mix can be characterized by the slope and relative level of the stress or strain versus the number of load repetitions to failure and can be defined by a relationship of the following form:

$$N_f = a\left(\frac{1}{\varepsilon_0}\right)^b \cdot \left(\frac{1}{S_0}\right)^c$$

Where N_f is the number of cycles to failure, ε_0 is the tensile strain, S_0 is the initial mix stiffness, a, b and c are coefficient calculated through fatigue laboratory tests.

In the present research, fatigue tests have been performed with the following purposes:

1. The output parameter N_f, number of cycles to failure, is considered to be an indicator of durability of asphalt mixes
2. The parameter N_f cannot be considered, in any case, as an estimation of the fatigue life of asphalt materials on site
3. Fatigue results are used to compare long-term performances of RAP mixes
4. A standard fatigue test procedure according to EN 12697-24 has been carried out in order to make the process reproducible

5.5 Experimental programme on recovered bitumen

Bituminous samples are recovered from asphalt samples tested in phase 1 and further characterized and compared with standard and dynamic tests.

The study on recovered bitumen aims to investigate the effects of the interaction between virgin and RAP binder on physical and rheological properties of the composite bituminous blend. The objectives of the experimental study on bitumen are summarized as follows:

- Investigate the effects of the presence of RAP into asphalt mixtures on standard characteristics of bitumen as penetration grade and softening point
- Determine the effects of RAP binder on rheology of composite blends with neat binder
- Define threshold values for RAP content above which dynamic characterization tests are needed, in addition to standard characterization tests, in order to evaluate the effects of the interaction between neat and RAP binder
- Identify practical implications on the use of different RAP percentages into asphalt mixtures

Five bitumens are investigated. A sample of bitumen was recovered from each mixture, respectively B_0 from Mix0, B_1 from Mix1, B_2 from Mix2, B_3 from Mix3 and B_{RAP} from the unprocessed RAP. Bituminous samples B_1, B_2 and B_3 are considered being blends because of the presence of both neat and RAP binder.

It is assumed that he RAP percentage in the asphalt mixtures coincides with the percentage of RAP bitumen present in the final bituminous blend.

The experimental programme is reported in Table 5.22.

Table 5.22: resume of the experimental program for the second phase of the research: testing on recovered bitumen

Activity	Objectives
Extraction and recovery	• Determine the bitumen content of prismatic samples subjected to fatigue testing • Obtain bitumen samples for testing (recovered bitumen)
Standard characterization testing and determination of dynamic viscosity on recovered bitumen	• Determine effects or RAP percentage on properties of recovered bitumen throughout the analysis of standard parameters as penetration at 25°C and softening point

	• Investigate the effects of RAP bitumen on viscosity of bituminous blends. Determine if viscosity variations may lead to possible workability issues
Dynamic mechanical analysis with Dynamic Shear Rheometer (DSR)	• Investigate the effects of RAP bitumen on rheology of materials • Define a threshold RAP percentage above which RAP bitumen significantly affects properties of the final blend

5.5.1 Extraction and recovery

Scientific literature considers as "low" a RAP content in the mix below 10-20%. For these mixtures, tests on bitumen are not required since it's assumed there's not sufficient old and hardened RAP bitumen to change properties of the composite bituminous blend. When the RAP percentage is above 20%, the RAP content is considered "high". Thus, characterization tests on bitumen are required in order to assess properties of the composite blend of old and neat bitumen. However, the purpose of this research phase is to investigate the effects of interaction between virgin and RAP bitumen. For this reason, characterization tests are performed on each bitumen regardless of the RAP content.

The objective of extraction and recovery process is to obtain samples of bitumen in sufficient quantity for further testing and with the lesser modification caused by heating and interaction with solvent.

Studies [10] show that processes of extraction, recovery and solvent influence properties of bitumen at the completion of operations. Bitumen is recovered from prismatic specimens and unprocessed RAP using a centrifuge extractor according to the European standard EN 12697-1 annex B.1.5 for cold extractions methods, using dichloromethane (DCM) CH_2Cl_2 organic solvent. The centrifuge method was chosen because of its accuracy and limited amount of time required for testing. Furthermore, it's one of the most common methods used in private laboratories.

The solution of bitumen and DCM from prior extraction is distilled with rotary evaporator, according to EN 12697-3. Together with rotary evaporator, the Abson method was evaluated prior to choose the Rotavapor. Researches show that physical properties of asphalt binders recovered using the Abson or the Rotavapor recovery procedure varies greatly. This can be attributed to three factors [10]:

1. Solvent may alter physical properties of bitumen while in solution
2. Residual solvent may remain in the recovered bitumen at the completion of the recovery process, altering binder properties. Abson recovery procedure can leave enough residual solvent to produce significant softening of bitumen. Even 0.5% of residual solvent could result in a 50% decrease in viscosity
3. Bitumen is not completely extracted from aggregates, leaving strongly adsorbed material that may have significantly different bulk physical properties than the recovered bitumen

Experience suggests that solvent, extraction and recovery may change bitumen properties. This is something inevitable. Despite this change in properties caused by investigation methods, for the purpose of the research, which is focused on comparison between bitumen properties, it's important that all investigated materials change to the same extent. This can be obtained performing all the procedures in the same way.

Extraction and recovery procedure is summarized below:

1. Crumbling of prismatic samples: samples are heated and worked with a spatula to obtain loose asphalt. In this step, it's important to avoid ageing of the specimen with excessive temperature or heating time
2. A quantity of 2000 grams of loose asphalt is put into a clean bowl together with 250 millilitres of Dichloromethane (DCM). Bowl is capped with a tinfoil, asphalt and solvent decants for 3 hours at room temperature
3. Asphalt and solvent are poured into the clean rotating bowl of the cold centrifuge, which has a mass of 2622 g. The weight of the bowl shall be calibrated before running the test. A filter paper is put on top of the bowl and the centrifuge is covered
4. Three washes with 150 millilitres of DCM each are made until the flow leaving the centrifuge becomes transparent
5. During the test, the solution of bitumen and DCM is stored into a special glass jar
6. The bitumen content of the prismatic sample is calculated as difference of the sample before and after extraction
7. The solution with bitumen and solvent is treated with rotary evaporator (Rotavapor)

Results of extraction on prismatic samples are reported in Table 5.23.

Table 5.23: soluble binder content extracted and recovered from beams and unprocessed RAP

Specimen	Recovered binder sample	Binder content b_c (% on mixture mass)
Mix0	B_0	5.13
Mix1	B_1	4.92
Mix2	B_2	5.01
Mix3	B_3	5.07
Unprocessed RAP	B_{RAP}	4.87

Extraction tests performed on the first mix production reported in Table 5.12 showed a bitumen content slightly below target content of 5% by mass of aggregate. A larger amount of virgin 50/70 bitumen was added into the bowl to compensate binder clinging that occurs during mixing. Results in Table 5.23 show that binder content of tested prismatic samples exceed target 5% of 0.3% on average, whereas the bitumen content of the first production was 0.43% below target value, on average.

5.5.2 Standard characterization

Recovered bitumen is subjected to laboratory testing aimed to investigate the effects of RAP on properties of bituminous blends. Actually, only B1, B2 and B3 should be considered as true blends. B_0 and B_{RAP} represent the two opposite behaviour that the blends are expected to have.

First, bitumen is characterized with standard tests: penetration at 25°C and softening point according to European standards EN 1426 and EN 1427 respectively. The analysis of standard parameters obtained from routine lab tests could be very useful for a preliminary evaluation of the effects of RAP on the final blend. Values of penetration at 25°C and softening point are considered to be representative of bitumen consistency at intermediate (25°C) and high (softening point) temperatures. However, this is a simple indication since there's not a unique relationship between the two parameters.

Because of the presence of old and hardened bitumen from the RAP fraction, increasing the RAP content is expected to produce harder bitumen. The harder the bitumen, the lower the penetration at 25°C, the higher the softening point. Therefore, results showing this type of variation of penetration at 25°C and softening point can be considered indicative of an interaction between old and neat bitumen.

5.5.3 Viscosity

The more fundamental viscosity test is used to investigate consistency of bitumen, and hence workability of the asphalt mix, on a specific temperature range. A Brookfield rotational viscometer DV-II + Pro Extra is used for viscosity measurements. Viscosity is measured at 135°C, 150°C, 160°C and 170°C. The spindle's speed has no effect on measurements due to bitumen type (unmodified) and testing conditions (high temperatures), which make bitumen a Newtonian fluid for specified testing conditions. Specifications reported on technical documents refers to viscosity measurements at 135°C and 160°C. Measures at 150°C and 170°C are included in this research to investigate viscosity at intermediate and higher temperatures.

From viscosity tests, results will be reported as follows:

- Plot of dynamic viscosity versus test temperature for evaluating temperature susceptibility of bitumens
- Plot of dynamic viscosity versus RAP percentage for determining the effect of RAP content on viscosity and hence workability of the material
- Verify the assumption of 100% blending throughout the log-additive rule methodology

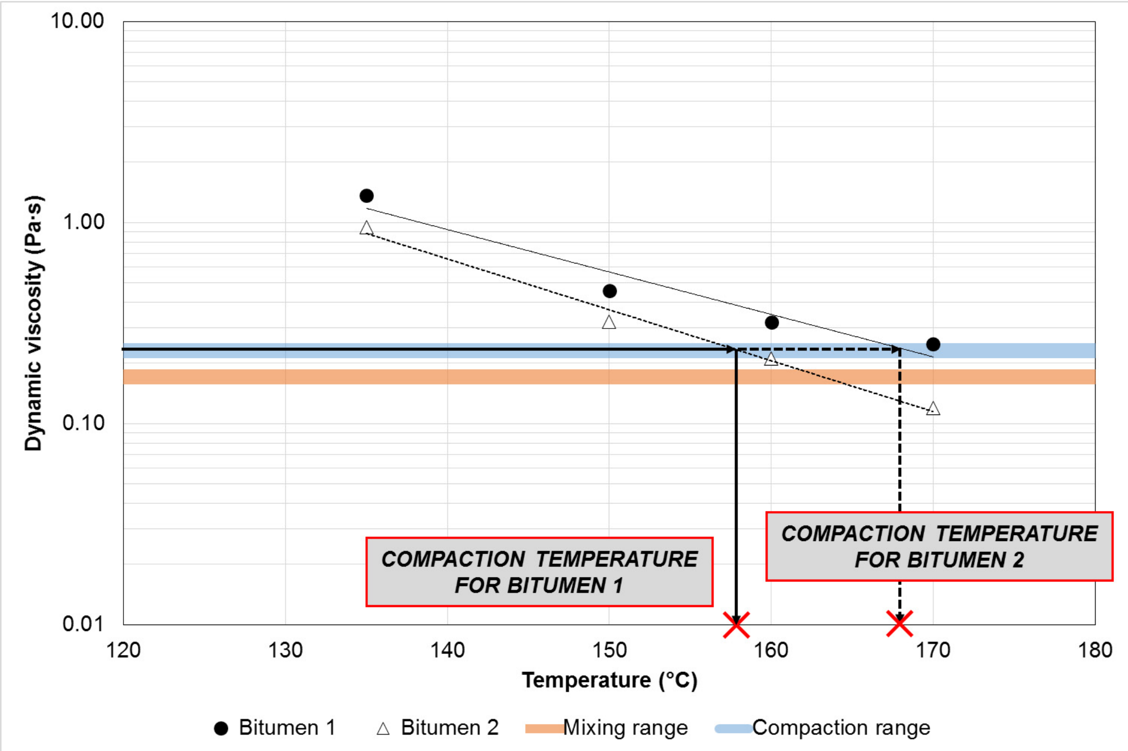
Studies and previous experiences have shown that bitumen viscosity increases with bitumen hardening. The higher the hardening, the higher the viscosity. This can be observed when bituminous samples are artificially aged in lab at different extent: the longer the samples are aged, the more they're hardened, the higher the viscosity. Similarly, different percentages of hardened bitumen blended with virgin binder are expected to increase the viscosity of the final bitumen.

Viscosity is a very important parameter, but not the only one, to evaluate workability of asphalt mixtures. Workability affects both handling and compaction of mixes. Satisfactory workability is important in obtaining the desired smoothness and density within a compacted pavement [98]. Pavements with insufficient compaction may experience significant performance problems primarily due to high air void content [99] [9]. If not properly compacted, the potential for permeability problems, as well as rate of oxidative aging of the binder, is increased considerably thereby reducing the life of the pavement.

Field practice have shown that increasing at random (without a lab mix design) RAP content into an asphalt mix may cause problems during paving and compaction of the material due to low workability of the paved mix. Asphalt exhibits insufficient flow and tends to be clumpy, leading to delays in paving operations. There are several methods to address this issue:

a. Since high RAP content causes an increase in viscosity of the final bitumen and hence a decrease in mix workability, the asphalt material can be heated to **higher temperature in order to decrease viscosity**. This is basically the idea on which the equi-viscosity criterion leans on. Temperature of mixing and paving are determined from viscosity testing on bitumen with rotational viscometer at 135°C and 165°C. Viscosity ranges are 0.17±0.02 for mixing and 0.28±0.03 for compaction. An example of determination of mixing temperature with equi-viscosity criterion is illustrated in Fig. 5.19. Viscosity measures at four different temperatures are reported for two bitumens. Bitumen 1 is a PMB, bitumen 2 is an unmodified pen grade 50/70. Mixing temperature of unmodified binder of 158°C is significantly lower than that for modified binder. The same can be observed for mixing temperature. Therefore, an asphalt mixture produced using the modified bitumen (bitumen 1) would require higher mixing and compaction temperatures than that produced with the unmodified bitumen (bitumen 2). Polymer-modified binder requires much higher temperature to attain the same viscosity as unmodified binder. According to the equi-viscosity criterion, temperatures obtained for mixing and compaction are close to 170°C or even higher. This is not reasonable based on practical experience

Fig. 5.19: use of viscosity test results for determining mixing and compaction temperatures



b. Excessive hardening of bitumen can be avoided using a **softer bitumen** when RAP percentages are high i.e. higher than 20%. If the desired penetration grade of the final bituminous blend is 50/70, a soft virgin bitumen as the pen graded 70/100 may be used to compensate the higher viscosity of the RAP bitumen. Generally, the softer the virgin bitumen, the higher the RAP percentage that can be incorporated in the mixture at a specific pen grade

Unfortunately, a single value of viscosity is not sufficient to define bituminous materials. As specified in previous chapter, parameters calculated with this test method are valid under the assumption of Newtonian or ideally viscous fluid. For unmodified bitumens, this condition occurs when bitumen temperature is above 60°C. Viscosity test does not provide useful information on bitumen properties below 60°C. In the temperature range below 60°C, a rheological analysis with DSR is necessary.

5.5.4 Dynamic mechanical analysis with DSR

Dynamic mechanical analysis aims to investigate the rheological behaviour of bituminous materials. In detail, rheological tests investigate the effects of RAP content on stiffness and viscoelastic properties of bitumen. As for previous tests, bituminous sample B_0 and B_{RAP} are considered to be representative of the two components virgin bitumen and RAP bitumen, respectively. B_1, B_2 and B_3 represent three blends of virgin binder with different percentages of RAP bitumen.

The following results will be reported:

- Complex shear modulus G* (magnitude or norm |G*|) isochronal plot (G* master-curves)
- Phase angle δ isochronal plot (phase angle master-curves)
- Black diagram, plot of complex modulus G* (magnitude |G*|) versus phase angle
- Cole-Cole diagram, plot of Storage modulus G' versus Loss modulus G''

The complex modulus and phase angle master curves give information on the rheological behaviour of bitumen at different temperatures. Definition of rheological response over the temperature domain is essential in order to understand the viscoelastic behaviour of bitumen, which is strongly dependent on temperature and mode of loading.

The Black diagram is a graph of the magnitude of the complex modulus |G*| versus the phase angle. Temperature and frequency are eliminated from the plot, which allows all the dynamic data to be represented in one plot without the need to perform time-temperature superposition manipulation of raw data. A smooth curve indicates correct time-temperature equivalency, while a disjointed curve indicates the breakdown on the TTSP and the presence of a high wax content bitumen, a high asphaltenes structured bitumen or a highly polymer modified bitumen [87].

The Cole-Cole diagram provides a means of representing the viscoelastic balance of bitumen without incorporating temperature or frequency as one of the axes.

The analysis is conducted using a Dynamic Shear Rheometer (DSR) in temperature-sweep mode. Disk-shaped bituminous samples are prepared according to EN 14770 with 8 mm diameter, 2 mm height (or gap) and 10 mg weight. Bitumen is heated until it becomes fluid and can be poured into the mould, then it's cooled at room temperature. Amplitude sweep pre-test is performed on bituminous samples for determining the strain amplitude of further temperature-sweep test. The linear viscoelastic (LVE) domain of the material is determined by measuring values of complex shear modulus G* in a strain amplitude range from 0.01% to 100% of the machine's torque. Within the LVE range, the complex shear modulus G* is independent of applied shear strain i.e. it's independent of the mode of loading. The limit of the LVE domain is defined as the point at which the dynamic viscoelastic function G* deviates by 10% from a constant (plateau) value. During AS test, the shear stress is applied with constant frequency of 10 rad/sec at constant temperature of 10°C. Shear stress and complex modulus versus shear strain are plotted in Fig. 5.20 and Fig. 5.21.

Fig. 5.20: shear stress versus shear strain during amplitude sweep pre-test

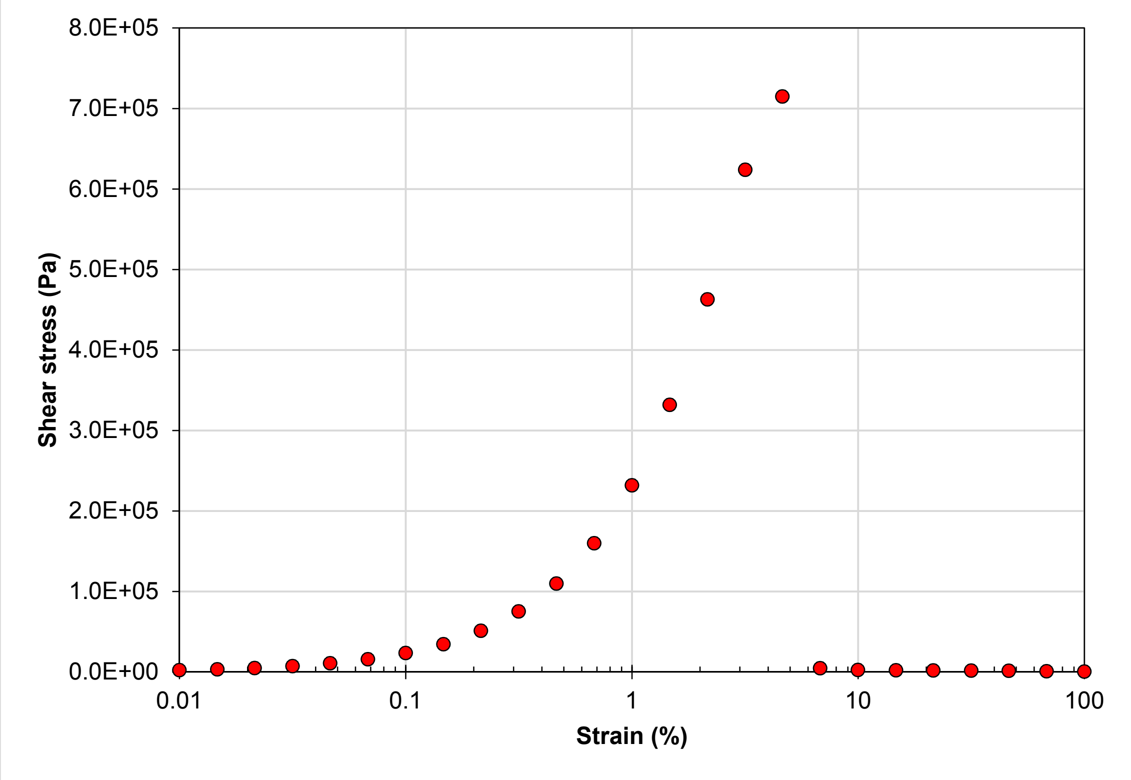
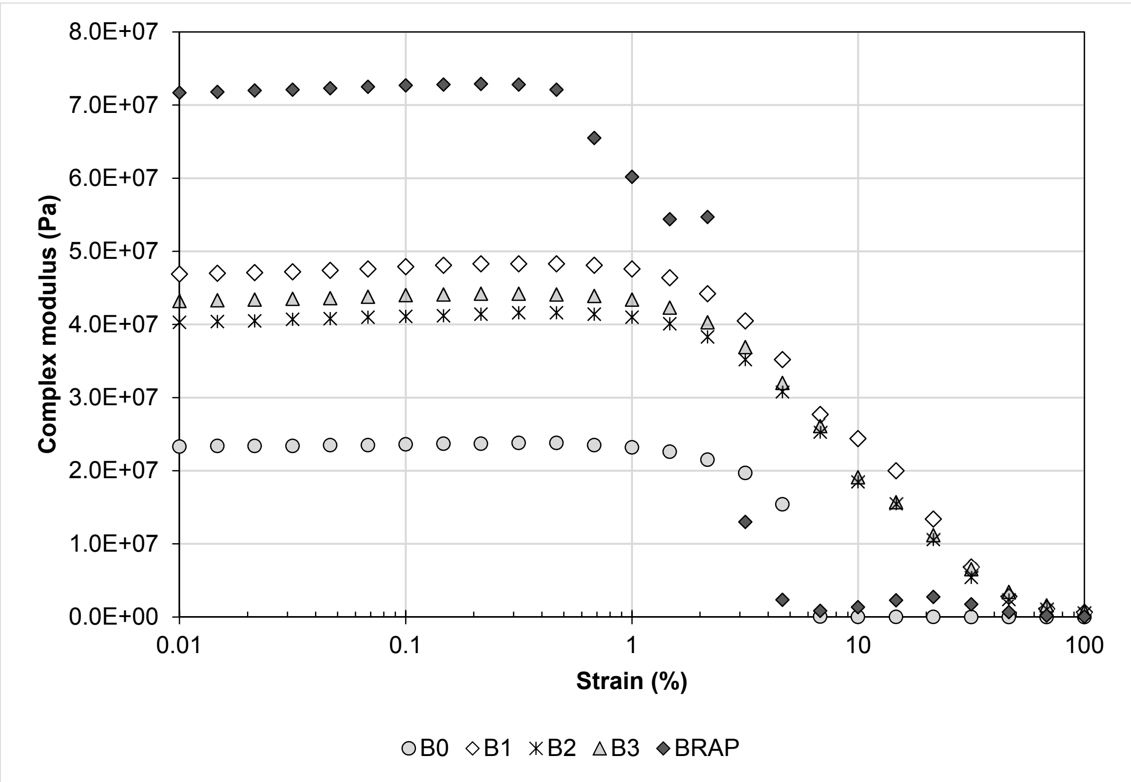


Fig. 5.21: complex modulus G* versus shear strain during amplitude sweep pre-test



Thermo-mechanical analysis (or temperature-sweep test) is performed on bituminous samples at strain amplitude of 2.5% as determined from amplitude sweep pre-test at constant frequency of 10 rad/sec. Temperature ranges from 0°C (initial temperature) to 90°C (end temperature) with constant heating rate of 2.99°C/minute. The temperature control unit of the DSR used for the present research is a Peltier heating system with heating elements above and below the sample. Such thermal conditioning system should be able to significantly reduce temperature gradients. Temperature is electronically controlled by the instrument with a precision of 0.1°C.

5.6 Annexes

5.6.1 Proportioning of aggregate and RAP in mix design

The proportion of aggregate and RAP being added into mixtures determines the grading curve of the mix. The investigated mixes are designed to have similar particle size distribution in spite of the different RAP content. A similar grading curve was obtained adjusting the amount of virgin aggregate against the RAP content.

The grading curve of extracted RAP must be considered in this process since the final product is a hot-mix asphalt. If the final product was a cold-mix, the grading curve of unprocessed RAP should have been used.

The procedure is illustrated in Fig. 5.22.

Fig. 5.22: calculation of the grading curve of mixes from the grading curves of aggregate and RAP fractions

Particle size distribution of each aggregate and rap fractions

Sieve size (mm)	*Percent passing (%)*						
	Gravel 10/20	***Gravel 6/10***	***Gravel 4/6***	***Sand 0/4***	***Filler***	***RAP 0/8***	***RAP 8/12***
31.5	100.0	100	100	100	100	100.0	100.0
20	100.0	100.0	100.0	100.0	100.0	100.0	100.0
16	70.2	100.0	100.0	100.0	100.0	100.0	100.0
12.5	32.7	100.0	100.0	100.0	100.0	100.0	100.0
8	2.4	52.4	99.2	100.0	100.0	99.1	93.5
4	2.1	2.5	14.3	92.6	100.0	88.0	64.8
2	2.1	1.9	1.3	63.4	100.0	73.8	45.1
0.5	2.0	1.8	1.2	25.9	100.0	47.8	26.3
0.25	1.9	1.7	1.0	16.8	99.9	26.0	17.9
0.063	1.7	1.5	0.9	7.6	96.0	10.1	8.8

EXTRACTED RAP GRADING

Grading curves of each component are combined proportionally to yellow-highlighted quantities to form the final mixture grading

	Gravel 10/20	**Gravel 6/10**	**Gravel 4/6**	**Sand 0/4**	**Filler**	**RAP 0/8**	**RAP 8/12**	**MIX 1 10% RAP**
	26	**13**	**16**	**33**	**2**	**5**	**5**	
Sieve size (mm)	*Percent passing (%)*							
31.5	26.0	13.0	16.0	33.0	2.0	5.0	5.0	***100.0***
20	26.0	13.0	16.0	33.0	2.0	5.0	5.0	***100.0***
16	18.2	13.0	16.0	33.0	2.0	5.0	5.0	***92.2***
12.5	8.5	13.0	16.0	33.0	2.0	5.0	5.0	***82.5***
8	0.6	6.8	15.9	33.0	2.0	5.0	4.7	***67.9***
4	0.5	0.3	2.3	30.5	2.0	4.4	3.2	***43.3***
2	0.5	0.2	0.2	20.9	2.0	3.7	2.3	***29.9***
0.5	0.5	0.2	0.2	8.6	2.0	2.4	1.3	***15.2***
0.25	0.5	0.2	0.2	5.5	2.0	1.3	0.9	***10.6***
0.063	0.4	0.2	0.1	2.5	1.9	0.5	0.4	***6.2***

MIXTURE GRADING

5.6.2 Compaction curves

The compaction curves of each mixtures are reported below.

Fig. 5.23: compaction curves of Mix0

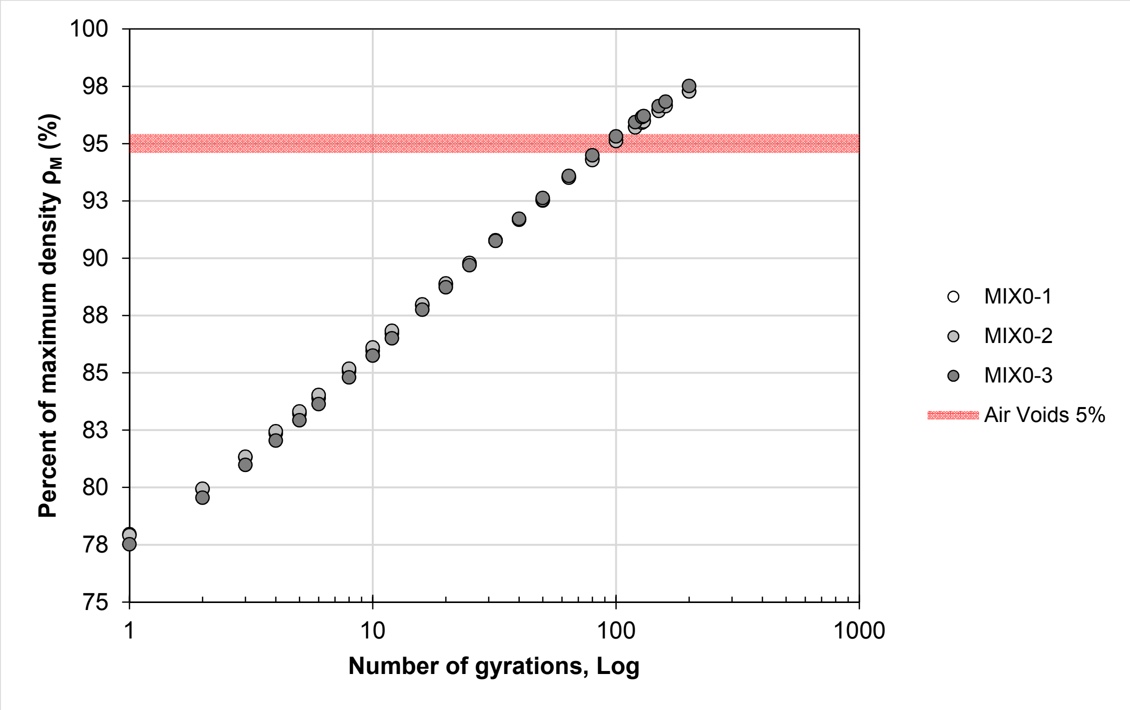


Fig. 5.24: compaction curves of Mix1

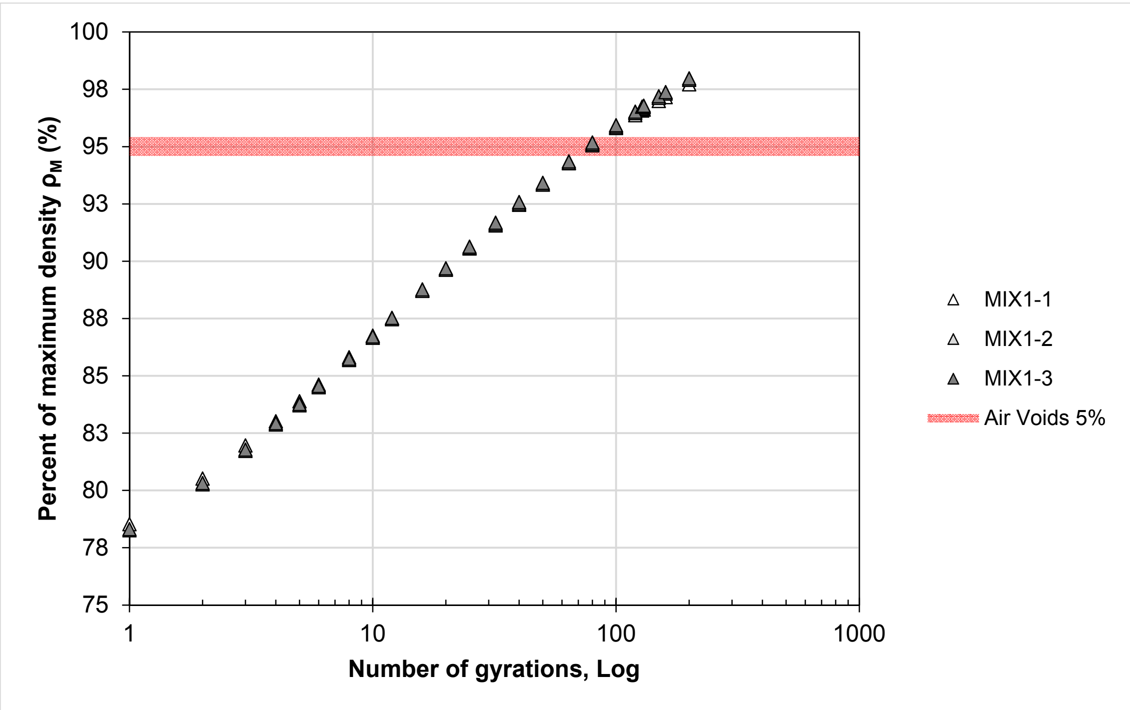


Fig. 5.25: compaction curves of Mix2

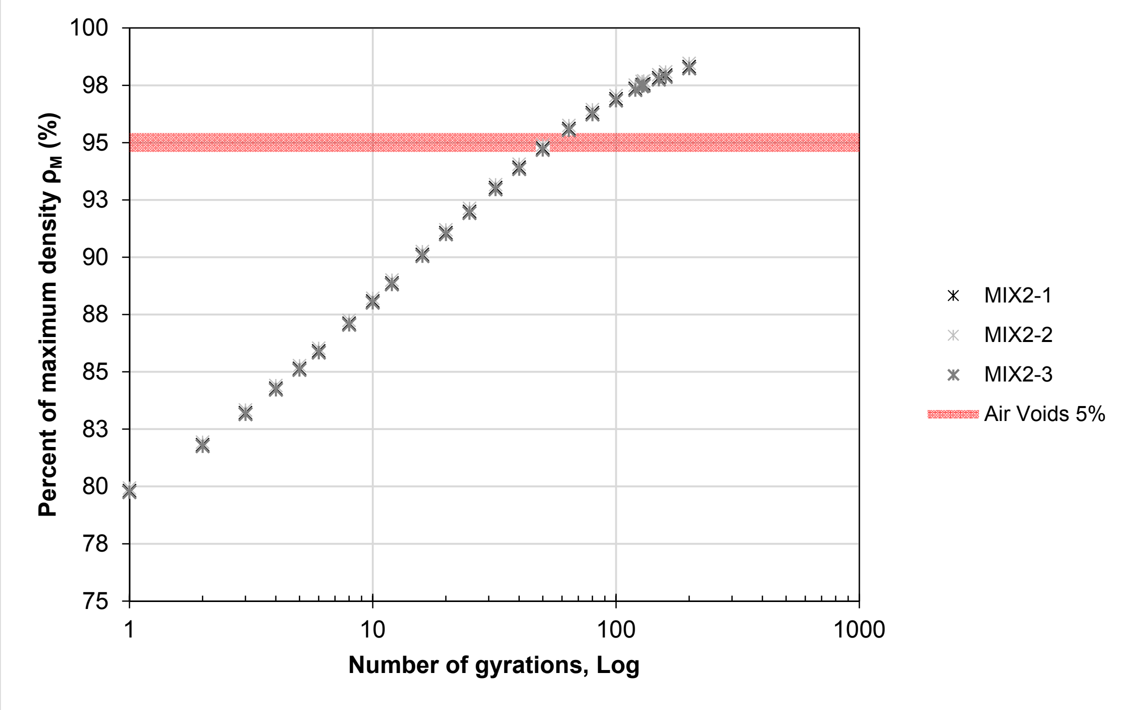
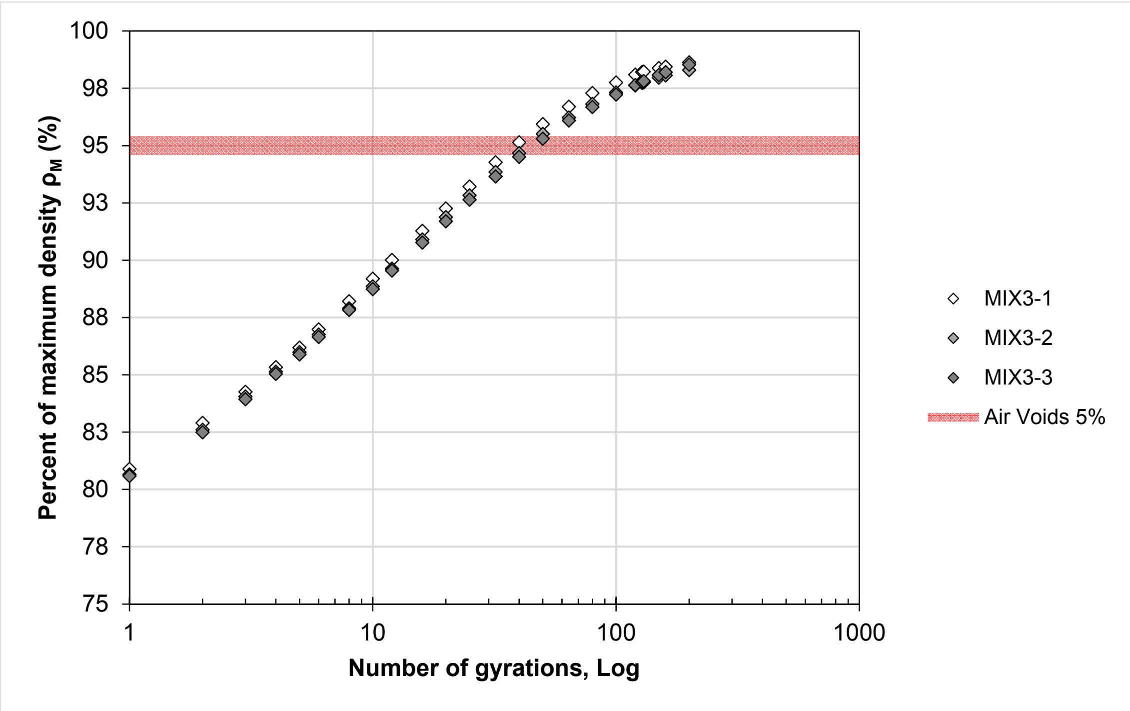


Fig. 5.26: compaction curves of Mix3



6 Results and discussion

6.1 Flexural fatigue tests on asphalt mixtures

Laboratory fatigue tests aim to evaluate the effect of RAP on durability of asphalt mixes. The simple flexure four-point bending test method was carried out on 7 prismatic samples for each mix accordingly to the European Standard EN 12697-24.

Flexural fatigue tests are performed in strain-control mode at three strain levels. A sinusoidal load is applied to prismatic samples with frequency of 25 Hz and amplitudes of 100, 150 and 200 με. Test are performed at 10°C. Prismatic specimens are thermally conditioned for 6 hours before being tested. Preparing testing equipment and setting up the specimen in the jig before to run the test is essential to limit temperature leaps that may occur by opening the conditioning chamber's door.

Strain-controlled fatigue tests generally don't lead specimen to physical failure; test termination shall be conventionally defined by choosing a failure criteria. The conventional failure criteria is defined as the number of load applications when the complex stiffness modulus has decrease to a specific extent its initial value [75]. In this research, failure criteria is the number of cycles to failure when the stiffness modulus reaches the 20% of its initial value, which is calculated at the 100th cycle. The N_{f20} criteria is chosen to investigate the response of mixes at high number of cycles and low strain level. This allow to study both the possible presence of an endurance limit and the effects of RAP on the fatigue response of mixes in critical conditions.

Details on fatigue test configuration are summarized in Table 6.1.

Table 6.1: fatigue test configuration

Fatigue test parameters	
Testing configuration:	Flexural four-point bending
Loading mode:	Strain (On-specimen LVDT)
Strain amplitude (μm/m):	100, 150, 200
Frequency (Hz):	25
Initial values are calculated at cycle number:	100
Maximum number of test cycles:	10,000,000
Termination modulus (% of initial value)	20
Cycles saved per logarithmic decade	5
Test temperature (°C)	10

Depending on the type of analysis on raw data, different outputs can be obtained from fatigue tests. The following have been considered in the present research:

- Fatigue curves ε-N_f (imposed strain – cycles to failure) based on traditional Wöhler approach

- Black diagram δ-|E*| (phase angle – complex modulus norm)
- Fatigue curves PV-N_f (Plateau value – cycles to failure)based on energetic approach (Ratio of Dissipated Energy Change, RDEC)

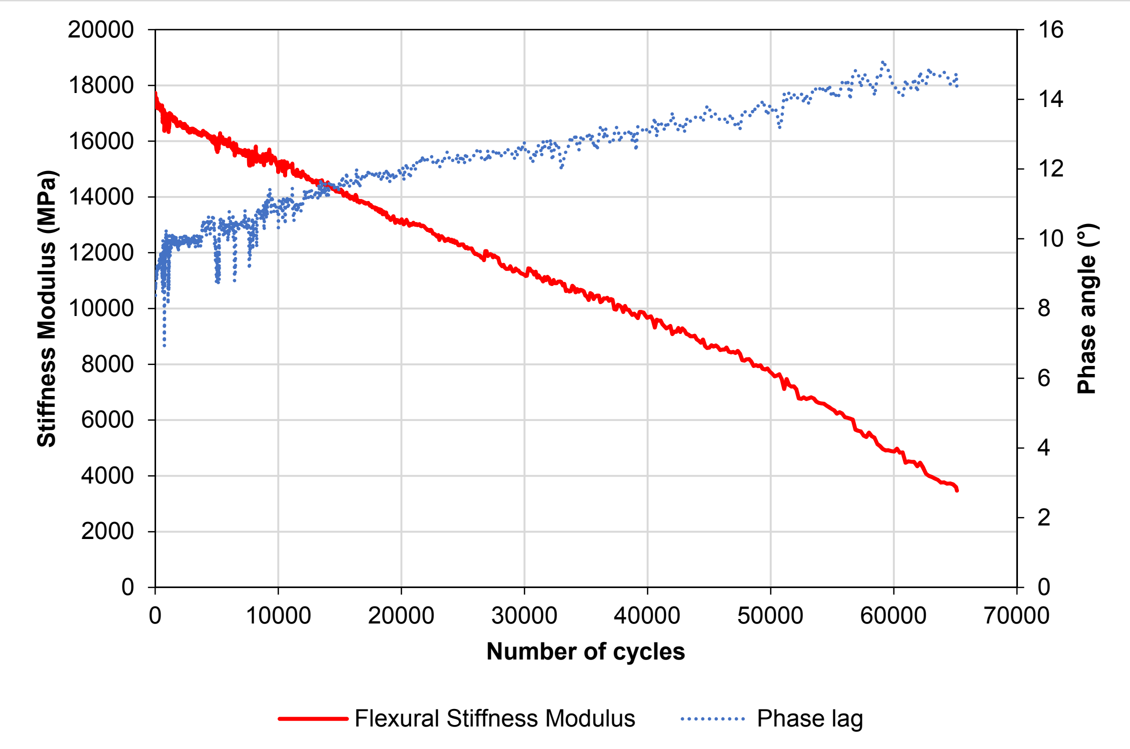
6.1.1 Fatigue curves ε – N_f

August Wöhler was the first to study fatigue and to propose an empirical approach. Between 1852 and 1870, Wöhler studied the progressive failure of railway axles. He constructed a test rig, which subjected two railway axles simultaneously to a rotating bending test. Wöhler plotted the nominal stress versus the number of cycles to failure, which has become known as the S-N diagram. Each curve is still referred to as a Wöhler fatigue line and the S-N method is still the most widely used today [100].

At each cycle, the vertical displacement and applied force are measured by the LVDT and the load cell. Measures are used to calculate stress σ, strain ε, stiffness modulus S, phase lag δ, dissipated energy and complex modulus |E*|.

The user interface UTS-18 automatically plots the S-N curve during the test together with the phase angle. The stiffness curve provides information on the quality of data and the fatigue response of the asphalt samples being tested. A scattered curve may indicate an incorrect set up of the prismatic sample into the jig. Scattering generally becomes noticeable in the end of the curve as fatigue cracks start to propagate into the material. An example of the S-N curve measured during the test is given in Fig. 6.1 for the prismatic sample of Mix3 (30% RAP) tested at 200 με.

Fig. 6.1: example of S-N and phase angle-N curves automatically plotted by the UTS-18 during the test



The number of cycles to failure at N_{f20} at the imposed strain level is reported in Table 6.2 for each sample, together with the stiffness modulus at the 100^{th} cycle and at test termination.

Table 6.2: number of cycles to failure at N_{f20} resulting from flexural fatigue tests on RAP mixes

Mix	*% RAP*	*Specimen ID*	*Strain amplitude (με)*	*Cycles to failure at Nf20*	*Flexural stiffness at 100th cycle (MPa)*	*Flexural stiffness at test termination (MPa)*
Mix0	0	2	100	2.2E+06	14603	6410
Mix0	0	4	100	1.5E+06	17717	3525
Mix0	0	5	100	1.9E+06	17351	3470
Mix0	0	6	150	3.4E+05	15031	2897
Mix0	0	1	200	4.9E+04	13317	2503
Mix0	0	3	200	6.5E+04	17447	3471
Mix0	0	7	200	5.9E+04	17127	3425
Mix1	10	2	100	3.1E+06	15830	3494
Mix1	10	4	100	1.9E+06	18090	4639
Mix1	10	5	100	2.6E+06	17789	3558
Mix1	10	1	150	3.9E+05	16684	3656
Mix1	10	3	200	7.9E+04	15542	3999
Mix1	10	6	200	6.9E+04	17866	3472
Mix1	10	7	200	7.5E+04	16546	3309
Mix2	20	1	100	1.3E+06	18044	2957
Mix2	20	2	100	1.7E+06	16938	3358
Mix2	20	6	100	1.7E+06	18156	3631
Mix2	20	3	150	3.6E+05	18796	3753
Mix2	20	4	200	6.4E+04	17087	3397
Mix2	20	5	200	5.4E+04	18813	4630
Mix2	20	7	200	6.3E+04	17968	3594
Mix3	30	2	100	4.3E+06	19734	5456
Mix3	30	7	100	6.1E+06	21735	4316
Mix3	30	6	100	5.1E+06	19867	3973
Mix3	30	5	150	4.3E+05	20430	4061
Mix3	30	1	200	2.5E+05	18105	3550
Mix3	30	4	200	7.4E+04	18943	3785
Mix3	30	8	200	1.82E+05	20789	4158

On the basis of the results representing the length of life $N_{i,j,k}$ for the chosen failure criteria j and the set of test conditions k for the specimen i, the fatigue line shall be drawn by making a linear regression between the natural logarithms of $N_{i,j,k}$ and the natural logarithms of the initial strain amplitude (strain amplitude at the 100^{th} cycle) [75]. The shape of the fatigue line is expressed in the following formula:

$$\ln N_{i,j,k} = q + p \cdot \ln \varepsilon_{i,j,k}$$

From the measured data, the following parameters are calculated and reported in Table 6.3:

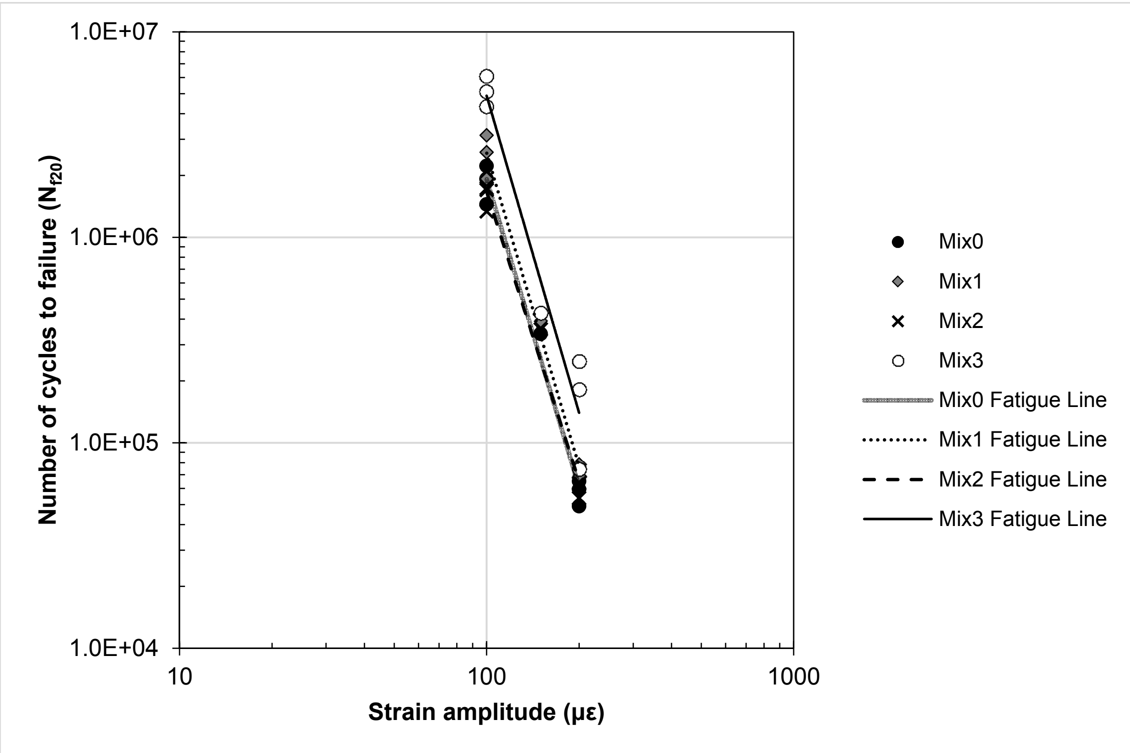
- Estimation of the slope p
- Estimation of q
- Coefficient of determination R^2
- Estimation of the initial strain for the chosen failure criteria at which a fatigue life of 10^6 can be expected for the given set of test conditions ε_6

Table 6.3: fatigue lines parameters

Mixture	Fatigue line coefficient q	Fatigue line coefficient p	R^2	Q, strain at 10^6 cycles (ε_6)
Mix0	37.37	-4.98	0.98	114
Mix1	38.09	-5.06	0.99	121
Mix2	35.89	-4.68	0.98	112
Mix3	39.01	-5.12	0.95	137

For each mix, the fatigue line is determined by plotting the strain level on the x-axis and the number of cycles to failure, measured according to the N_{f20} criteria, on the y-axis. The Wöhler fatigue curves are plotted in a single Log-Log chart for all the four mixes and are illustrated in Fig. 6.2.

Fig. 6.2: fatigue curves (Wöhler) of investigated asphalt mixes on Log-Log chart



The fatigue curves plotted above define the response of the four asphalt mixes to flexural fatigue testing, giving and estimation on the durability of materials. As mentioned in the previous Chapter 5, the number of cycles to failure measured in laboratory does not correspond to the actual fatigue life of the material in field. A comparative study should be carried out in order to effectively correlate laboratory and field performances in terms of cycles to failure. In spite of this, laboratory tests give precious information on the effects of RAP content on the long-term performances of investigated mixes. On the basis of fatigue test results, the following considerations are proposed:

i. Fatigue curves appear to be sensitive to RAP content. This is evident from the vertical shift of the fatigue lines: as the RAP content increases, the curves tend to shift towards higher number of cycles to failure

j. The RAP mix made with 30% RAP (Mix3) shows the highest number of cycles to failure at each strain level, relative to other mixes. The Mix3 fatigue line can be clearly distinguished from the other curves. This suggest that the asphalt mix with 30% RAP may have better long-term performances than other mixes made with less or without reclaimed asphalt

k. Despite Mix3 with 30% RAP showing the longer fatigue life, possible brittleness issues should be clarified and investigated by testing the rheological behaviour of bitumen recovered from Mix3 samples. Consideration on fatigue life of Mix3 at the previous point are encouraging but need of more evidence to be conclusive

l. Fatigue lines of mixes Mix0, Mix1, Mix2 and Mix3 are parallel, with reasonable approximation. Coefficients “p” of fatigue curves (slope) range from -4.68 (Mix2) to -5.12 (Mix3) showing very good consistence. Hence, three considerations can be made:

    i. The fatigue response is similar for all the investigated mixtures

    ii. The value of coefficient “p” slightly increases as the RAP content increases, with the exception of Mix2 (see Table 6.3). Since the class of bitumen affects the slope of the fatigue curves (the harder the bitumen, the higher the p coefficient) [60], it can be concluded that the presence of reclaimed asphalt contributes stiffening the neat binder, resulting in an harder blend of neat and virgin bitumen

    iii. A strong effort was put to produce mixtures with similar properties i.e. grading, void and bitumen content, with RAP content being the unique (most significant) variable to affect test results. Consistence of slope coefficients indicates an accurate and precise manufacturing process capable to produce mixes that mainly differ for RAP content and hence bitumen type

m. In terms of durability, mixes made incorporating intermediate to high RAP contents (up to 20% [10]) appear to perform as the same mix made only with virgin materials. A RAP content up to 20% by mass of aggregate into the mix doesn’t seem to negatively affect the fatigue response of the investigated asphalt mixtures, since the fatigue lines of Mix0, Mix1 and Mix2 are almost superimposed

n. The number of cycles to failure measured from fatigue test on Mix3 samples are more scattered than those of the other mixes, as can be seen from the lower R^2 value

6.1.2 Effects of RAP percentage on fatigue life of mixtures

In the next figures, the average number of cycles to failure measured from fatigue tests is plotted for each mixture both for 100 and 200 με tests (see Fig. 6.3 and Fig. 6.4). This representation of data aims to furtherly clarify the effect of RAP content on the laboratory fatigue life of mixes.

Fig. 6.3: average number of cycles to failure at 100 με for the investigated mixtures

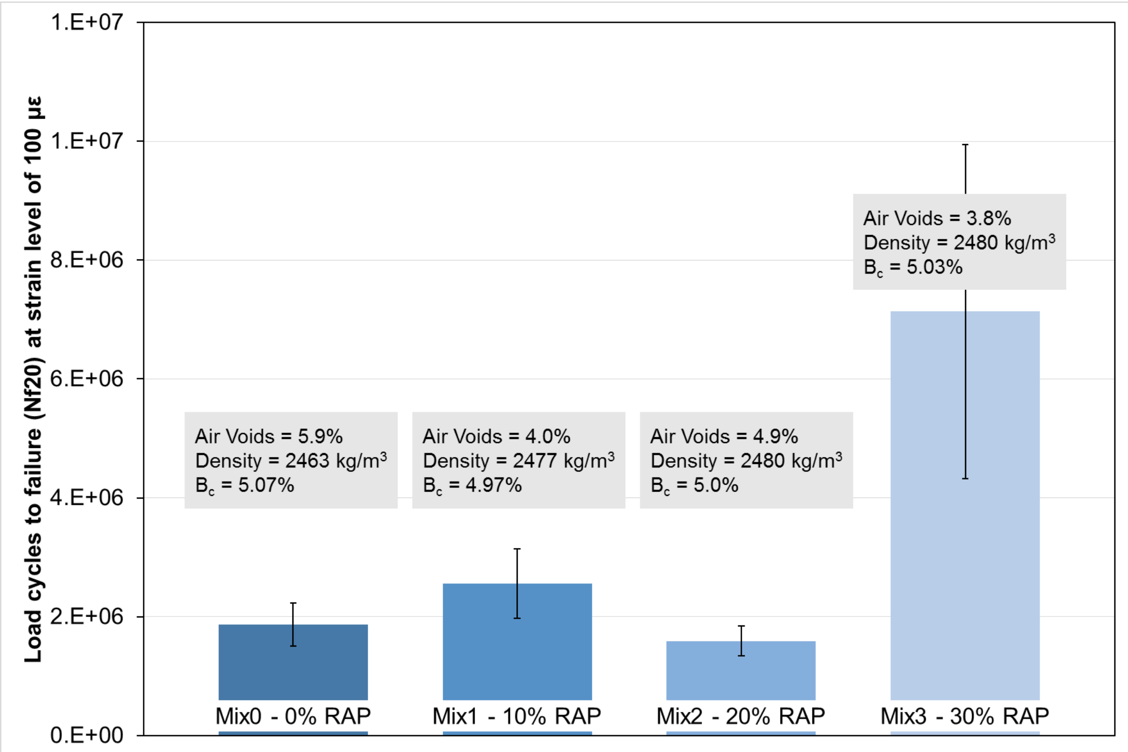
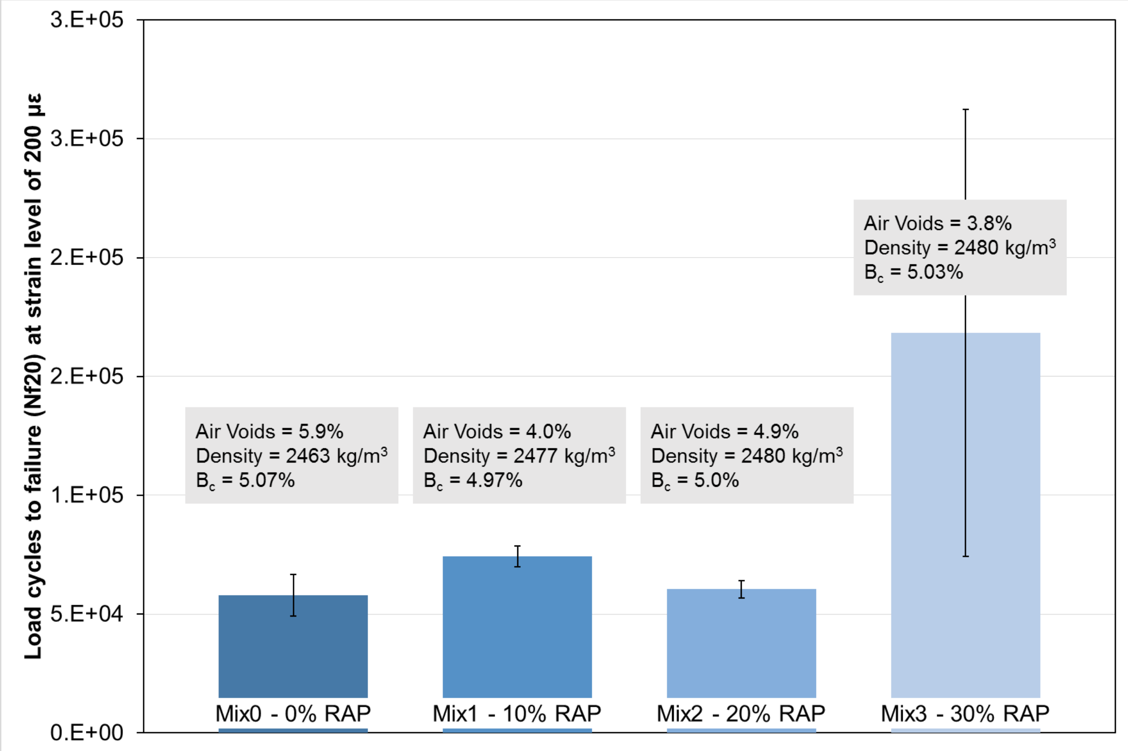


Fig. 6.4: average number of cycles to failure at 200 με for the investigated mixtures



From figures above we can observe that:

o. Mix0 (reference mix with no RAP), Mix1 (10% RAP) and Mix2 (20% RAP) exhibit similar fatigue performances both at 100 and 200 με

p. The number of cycles to failure measured for Mix0, Mix1 and Mix2 is consistent both at 100 and 200 με. In particular, variability of test results at 200 με is very low, whereas it's slightly higher at 100 με. This is generally due to a higher sensitivity of low-strain fatigue testing to small variations that may occur during the test

q. Adding up to 20% of RAP seems to have no negative effects on the durability of asphalt mixtures. Therefore, 20% of RAP being added to a mix can be considered as a threshold value for the investigated mixes

r. Mix3 (30% RAP) shows higher number of cycles to failure and more scattered results. This may indicate a variability of the material, in particular of its properties as void content

s. The RAP content affects fatigue life more than other material properties as e.g. air void content. For example, the difference in air voids between Mix0 and Mix1 is the same as between Mix0 and Mix3. With similar void and bitumen contents but higher RAP percentage, Mix3 shows higher number of cycles to failure relative to Mix1 both at 100 and 200 με

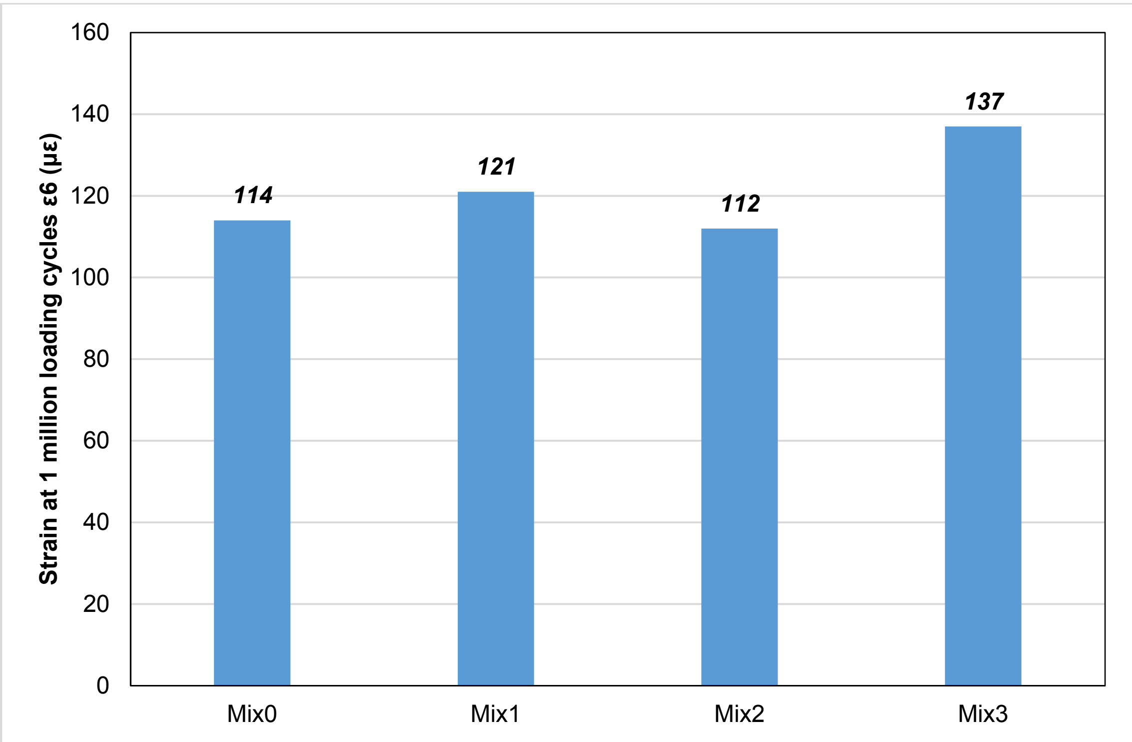
t. With reference to the previous point "e", it can be concluded that the interaction between virgin and RAP bitumen significantly affects the fatigue performance of the investigated mixes. The presence of less than 20% of RAP (by mass of aggregates into the mix) being incorporated into the new mixes doesn't seem to produce a significant interaction between neat and old binder. This furtherly confirms observation at point "c"

More detailed information about the interaction between new and old bitumen are necessary and will be provided with laboratory testing on recovered bitumen presented in 0.

The parameter ε_6 has been calculated for each mixture from fatigue lines. It represents the strain the material experiences after 1 million of load cycles, based on measured data. Thus, it can be considered as an estimator of the fatigue life of investigated mixes.

Results in Fig. 6.5 show the same trend observed for the number of cycles to failure. Fatigue performances of Mix0, Mix1 and Mix2 appear to be very similar, with ε_6 ranging from 112 to 121 με. Mix3 exhibits the higher ε_6 indicating the longer fatigue life relative to other mixes.

Fig. 6.5: ε6 calculated for each mixture

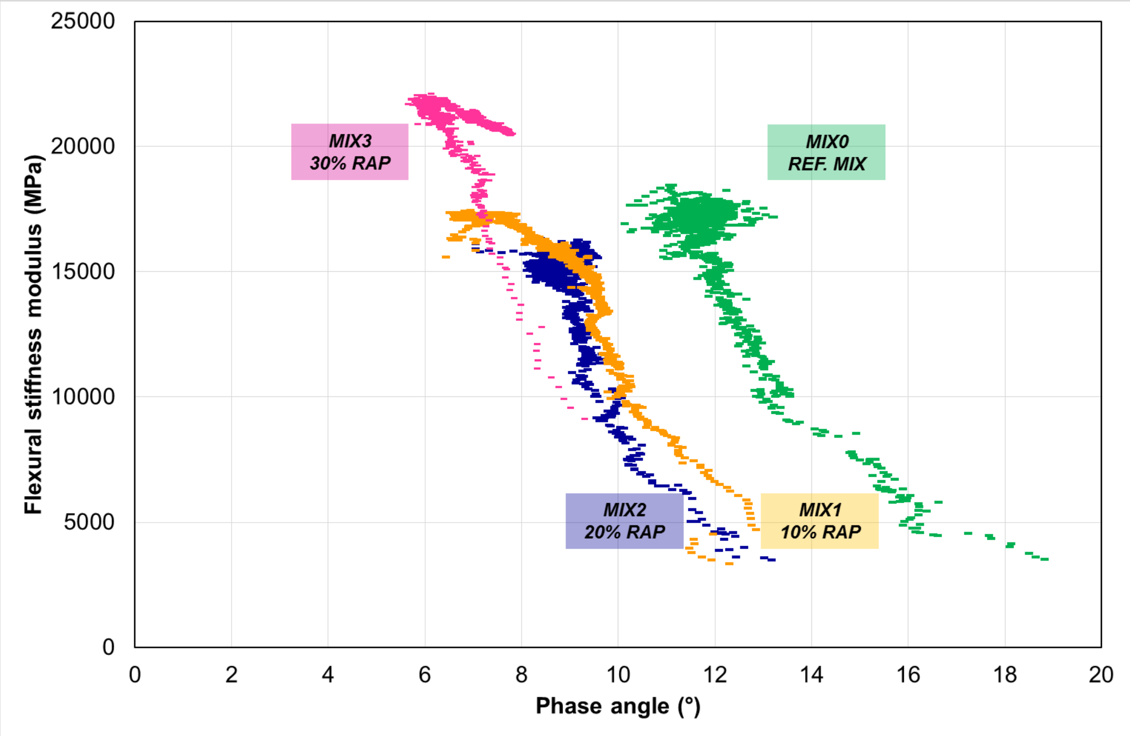


6.1.3 Black diagram

A Black diagram is a graph of the magnitude (norm) of the complex modulus, |E*|, versus the phase angle δ, obtained from a dynamic test [87]. In this case, the Black diagram is plotted using the flexural stiffness modulus, S, measured during the fatigue tests, instead of the norm of complex modulus. The frequency and the number of cycles to failure are therefore eliminated from the plot, which allows all the dynamic data to be presented in one plot.

Samples of the same mixture exhibit similar S- δ curves. For this reason, the curves of four samples tested at 100 με are reported in the following Black diagram (Fig. 6.6).

Fig. 6.6: Black diagram of investigated asphalt mixes



The Black diagram gives information about the viscoelastic response of mixes being tested in terms of stiffness modulus and phase angle. As the test carries out, the stiffness modulus decreases, indicating a reduced capacity of the material to withstand the external load, and the phase angle increases, which means that the ability of the material to store and release the load energy is being reduced.

The S-δ curves shift towards lower values of the phase angle as the RAP content increases. This indicates that the behaviour of the mixes becomes more elastic increasing RAP content. In fact, the phase angle represents the ratio between the elastic E_1 (real) and viscous E_2 (imaginary) components of the complex modulus. As the phase angle decreases, the balance between E_2 and E_1 changes in favour of the elastic part, indicating and increased elastic response of the asphalt mix.

With reference to Mix0 and Mix3, the increased RAP content produces both a shift towards lower values of phase angle and higher values of flexural stiffness modulus, which indicates a more elastic and stiffer behaviour of the mix with 30% RAP relative to the reference mix with no RAP.

Mix1 and Mix2 show a behaviour that is in the middle between the reference mix and Mix3. The shift towards higher phase angles appears to be proportional to the RAP content.

In conclusion:

u. Increasing RAP content leads to an increased elastic response of the mixes in proportion to the RAP content

v. With 30% RAP content, the mixture exhibits hardening relative to other mixes with lower RAP percentages. The increased stiffness is not significant when the RAP content is below or equal to 20%. The hardening effect of RAP will be furtherly analysed in order to evaluate possible brittleness of the mix

6.1.4 Energy methods to analyse fatigue test results

6.1.4.1 Background

Energy is dissipated in asphalt mixtures during loading and relaxation because the material behaves substantially in a viscoelastic manner at ambient temperatures [101].

When a generic load is applied to a material, stresses and strains develop within the material as consequence of the load application. Elastic materials can completely restore their initial stress-strain level after the load is removed. For these materials, the energetic balance is zero, since the same amount of energy enters into and exit from the system. Therefore, elastic materials are capable to store a certain amount of load energy and then to completely release the same amount of energy. This happens both for linear and non-linear elastic materials.

On the other hand, viscoelastic materials dissipate part of the load energy in the form of heat due to internal friction. This means that part of the load energy is stored and released, whereas another part is lost and used to increase the internal temperature of the material. The more viscous the material, the higher the energy dissipated. Asphalt behaves as a viscoelastic material. The explanation also recall the terminology generally used to define the two “storage” and “loss” components of complex modulus.

If we plot the response of an elastic material to an external load in terms of stress and strain, or equivalently load and displacement, the result is a straight line or a curve, depending on the linear of non-linear type of elasticity. If we do the same for a viscoelastic material, the result is a loop that is commonly known as hysteresis loop (ὑστέρησις, from the ancient Greek word meaning “deficiency”, “lag”).

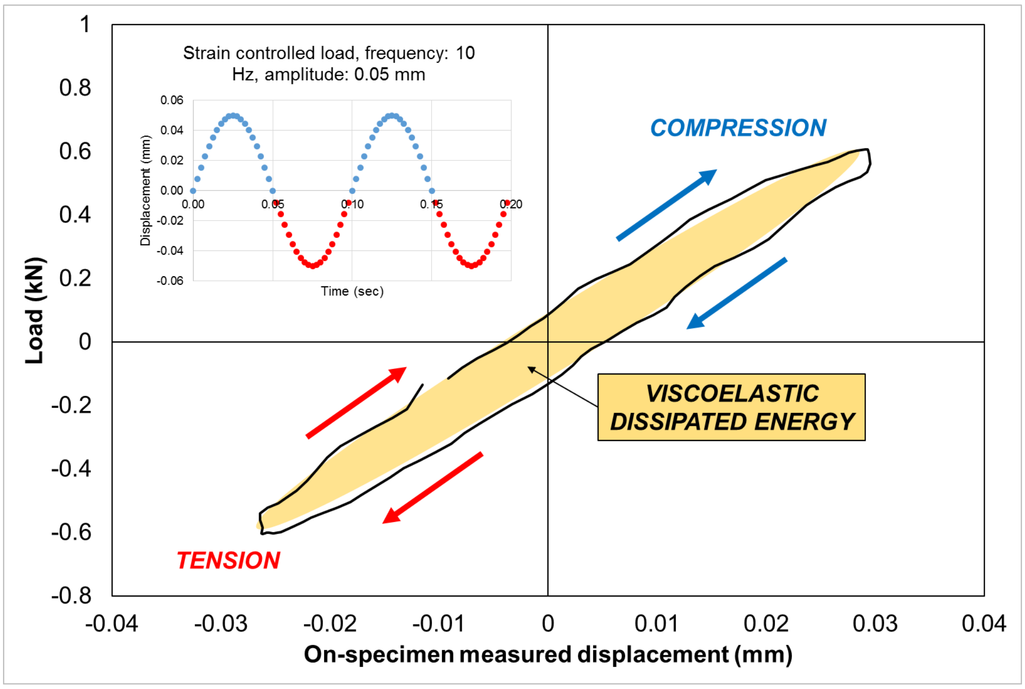
Hysteresis defines the property of viscoleastic materials to dissipate a portion of the external load energy they are subjected to during a load cycle. If a viscous material, with phase angle δ, is subjected to a repeated sinusoidal load, the dissipated energy i.e. area of the hysteresis loop at the generic i-cycle can be calculated with the following expression:

$$w_i = \pi \cdot \varepsilon_i \cdot \sigma_i \cdot \sin \delta_i$$

Where w_i, ε_i, σ_i, δ_i are the dissipated energy, strain, stress and phase lag measured at load cycle i. The area of the hysteresis loop is calculated from the integral of σ and ε on time domain. According to EN 12697-24 for flexural four-point bending fatigue tests, the dissipated energy is defined as the dissipated viscous energy in the beam per unit volume and per cycle, in kilojoules per cubic metre (kJ/m^3).

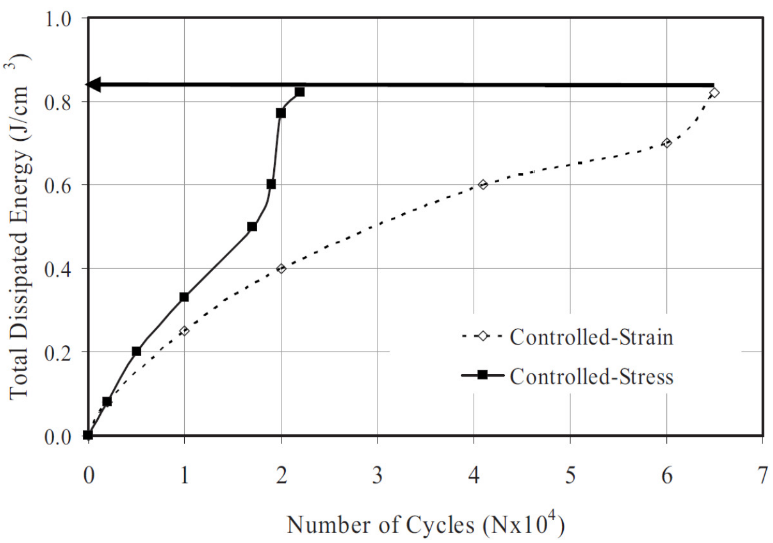
An example of hysteresis loop and hence of dissipated energy is given in Fig. 6.7. This case refers to a sinusoidal load cycle that one of the prismatic samples of Mix0 is subjected to.

Fig. 6.7: example of hysteresis loop measured during a loading cycle of fatigue test on Mix0



In 1972, Van Dijk [102] introduced the use of the dissipated energy to characterize fatigue cracking in asphalt mixtures. The dissipated energy was supposed to be constant irrespective of the mode of loading, as illustrated in Fig. 6.8.

Fig. 6.8: laboratory fatigue test results proposed by Van Dijk that suggest the dissipated energy is independent on mode of loading (Van Dijk, 1972)



The underlying assumption was the existence of a unique relationship between the fatigue life, expressed in number of cycles to failure, and the total dissipated energy measured during the test. The Wohler fatigue line is an empiric law that relates two different parameters measured from laboratory testing. Therefore, it should not be considered as a fundamental law of the material, but rather as a representation of the fatigue behaviour of a specific material under well-defined testing conditions. The dissipated energy approach was aimed to overcome this issue. In further studies, Van Dijk [103] proposed a fatigue law based on the total dissipated energy defined as:

$$w_{fat} = \sum_{i=1}^{N} w_i \quad [kJ/m^3]$$

Where w_{fat} is the total dissipated energy during the test due to fatigue damage of the material, while w_i is the dissipated energy at i-cycle. If we define Ψ as the ratio between initial and total dissipated energy, the fatigue law proposed by Van Dijk can be written as:

$$N_f = \left(\frac{\pi \cdot S_0 \cdot \sin\delta_0}{A \cdot \psi}\right)^{\frac{1}{z-1}} \cdot {\varepsilon_0}^{\frac{2}{z-1}}$$

With ε_0, S_0, and δ_0 strain, stiffness modulus and phase angle at test start. This relation is regarded by the authors as dependent on the mix design of the mixture but independent on the fatigue test method, of the temperature, of the loading mode and of the frequency [60]. The previous expression can be simplified in the following form:

$$w_{fat} = A{N_f}^{z} \quad [kJ/m^3]$$

The dissipated energy fatigue law was deducted from fatigue test data for 13 different types of asphalt mixtures in the context of the SHRP (Strategic Highway Research Program). If this law was valid, which means the relation is independent on aforementioned parameters, the total dissipated energy would be a material parameter as e.g. the stiffness modulus.

Unfortunately, there’s no definitive evidence to prove or disprove the validity of the Van Dijk’s law [60] [104]. However, considerations can be made about the meaning of the dissipated energy: the Van Dijk’s approach assumes that the entire amount of dissipated energy goes to damage the material. This assumption has been challenged by some authors [105] that consider the dissipated energy as a combination of two different processes of energy dissipation, which are:

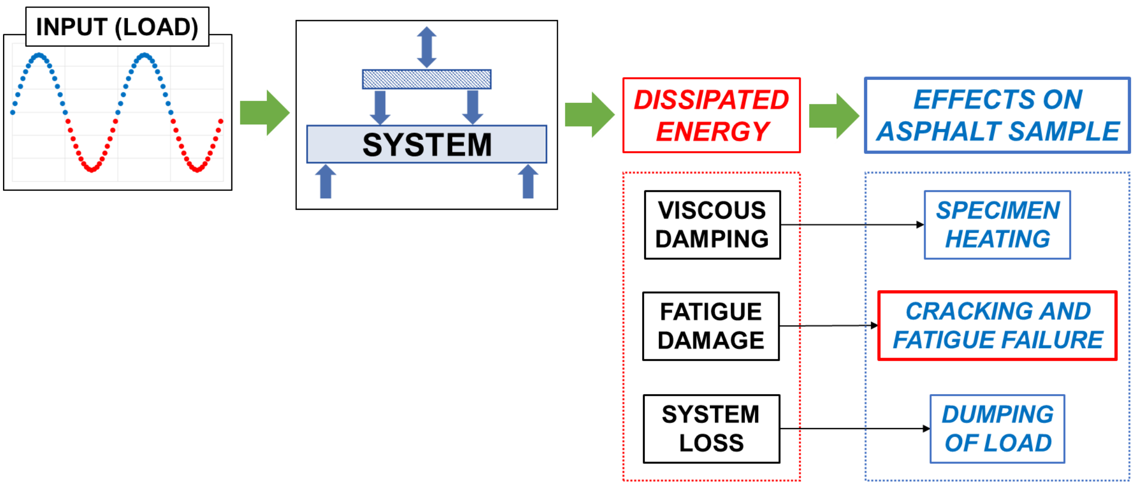
1. Viscoelastic damping and
2. Incremental damage to the material due to plastic deformation or crack formation (fatigue)

The dissipated energy does not just destroy bonds, but it also seems to heat the specimen due to internal viscous dissipation, with internal temperature of the material being increased [60].

In the European standard EN 12697-24, the energy dissipated due to system loss is also considered. System damping is caused by the inertia of the testing equipment moving masses being subjected to the repeated action of the external load. It can play a role in the interpretation of the data, particularly if the test frequency ω_0 is close to the first resonance frequency of the test equipment. The influence of system losses can only be determined by calibration measurements using an elastic material with a known Young's modulus.

In conclusion, a certain amount of energy is dissipated during each load cycle. The energy is dissipated in different forms i.e. viscous damping, fatigue damage and system loss. Of these, only the energy dissipated due to fatigue damage causes cracking and failure in the specimen. The idea is illustrated in Fig. 6.9. Therefore, **only a certain amount of the dissipated energy per cycle can be related to the fatigue damage**.

Fig. 6.9: types of energy dissipated during a fatigue test



The Van Dijk model considers the entire amount of dissipated energy per cycle being responsible for the fatigue damage. When a viscoelastic material is tested to fatigue, the damping effect becomes relevant if compared to an elastic material, which should not be affected by stress-strain lag. Thus, viscous damping should be distinguished from the actual fatigue damage in order to define a sound material parameter.

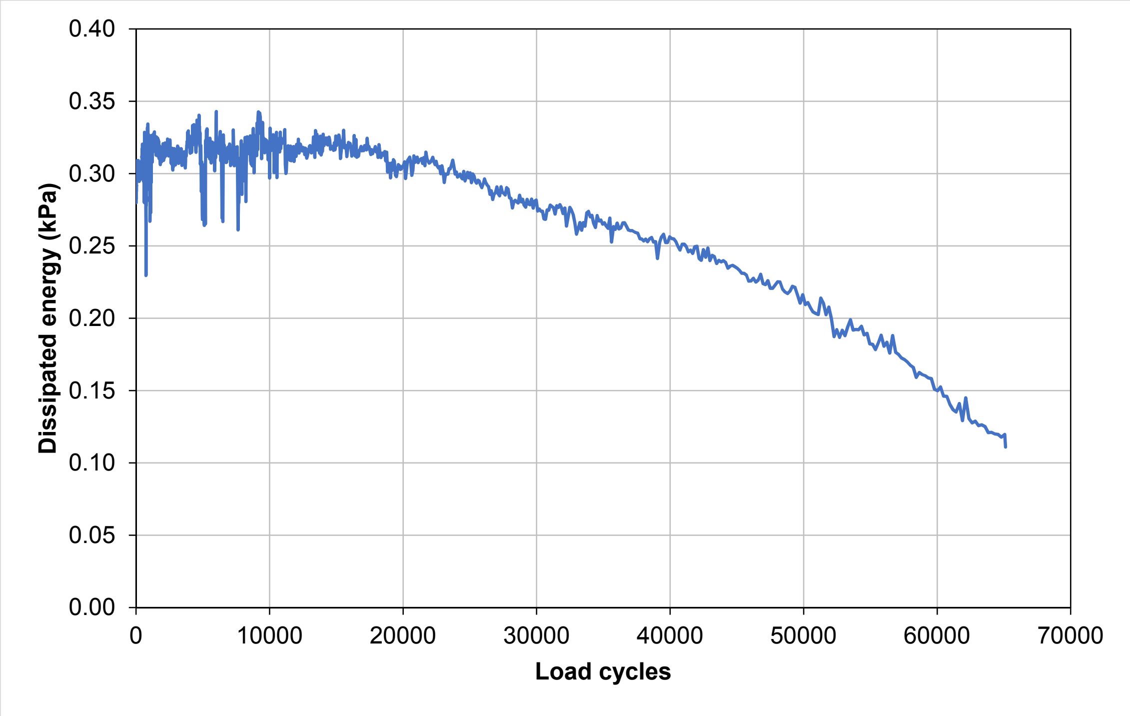
6.1.4.2 Change in dissipated energy

An attempt to define a fundamental fatigue law using a material based parameter was made with the change in dissipated energy or ratio of dissipated energy approach [106] [107] [108], originally proposed by Ghuzlan and Carpenter.

The central concept of the energy approach is the energy fatigue curve, which is based on two key elements, namely the plateau value (PV) and the number of cycles to failure. The plateau value represents the constant value of the percentage of dissipated energy that produces damage to the material under cyclic loading [106].

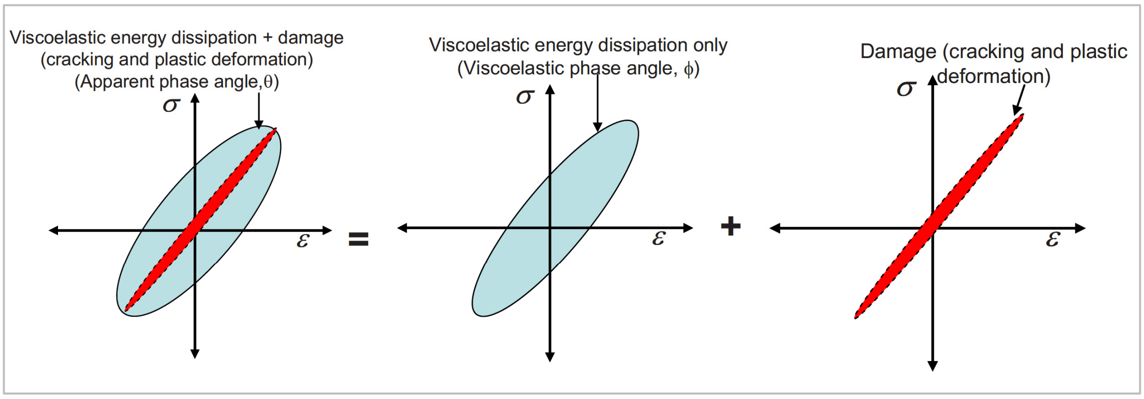
The plateau value is calculated from the dissipated energy, which is measured at each load cycle. The dissipated energy of one of the investigated samples (Mix0 at 200 με) is plotted versus the number of cycles in Fig. 6.10.

Fig. 6.10: example of dissipated energy during a fatigue test (4PB fatigue test on Mix0 at 200 με)



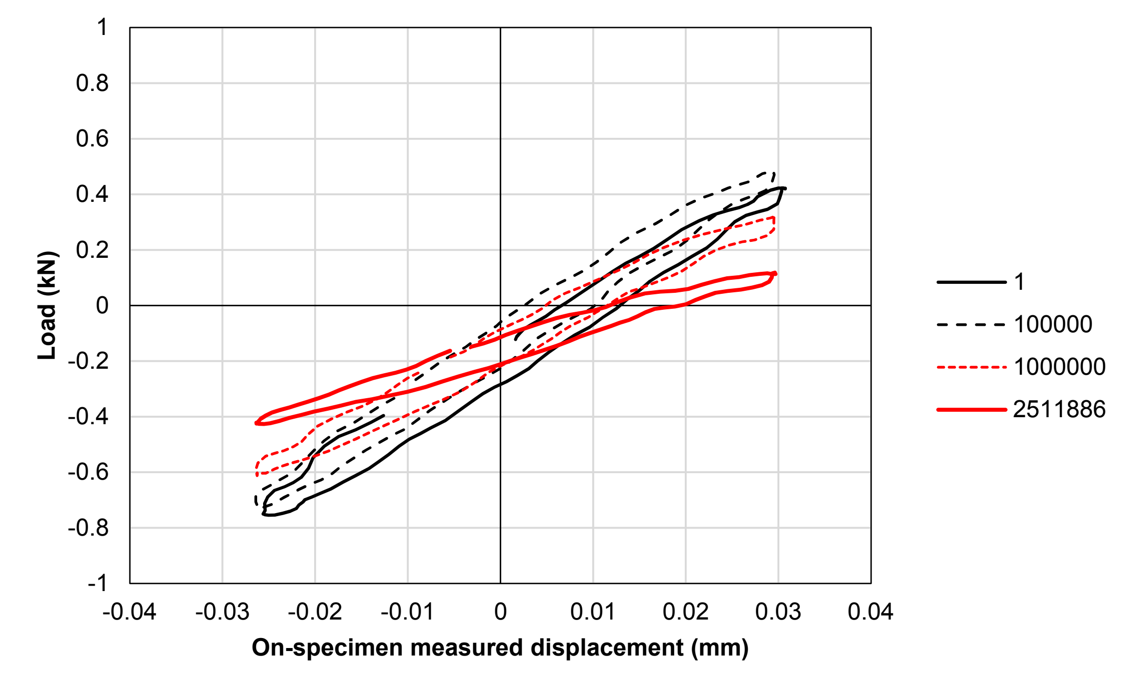
If we represent the dissipated energy on the ε-σ chart illustrated in Fig. 6.11, the light blue area represents the viscoelastic energy dissipation measured at one cycle, whereas the red area is the energy dissipated by fatigue damage at the same cycle [104].

Fig. 6.11: illustration of the hysteresis loop of a fatigue test load cycle. The two components of dissipated energy, viscoelastic damping (light blue) and fatigue damage (red) are clearly defined (Bhasin et al, 2009)



The hysteresis loop rotates clockwise during the test due to the increased fatigue damage. Fatigue affects the flexural stiffness with rigidity being reduced by crack propagation and heating of the sample (see Fig. 6.12).

Fig. 6.12: hysteresis loop rotates clockwise during fatigue test. Number of cycles are reported on the right from test start to termination for Mix0 tested at 100 με



The change in dissipated energy measures the amount of energy that is dissipated from a reference cycle "a" and a further cycle "b". This can be represented by the parameter Ratio of Dissipated Energy Change, RDEC [107], defined by the following equation:

$$RDEC_a = \frac{|w_a - w_b|}{w_a \cdot (b - a)}$$

RDEC is the absolute value of the difference between the dissipated energy at the generic cycle *"a"* (w_a) and at a successive cycle *"b"* (w_b) normalized by the initial dissipated energy and the number of cycles between "a" and "b". In other words, RDEC represents a variation of dissipated energy between two generic cycles of a fatigue test.

Under certain assumptions, the RDEC can be considered an indicator of the fatigue damage for the material. The assumptions are:

Hyp. 1. The energy dissipated at each cycle is the sum of two components:

a. Energy dissipated at cycle i due to viscoelastic damping, w_i^η
b. Energy dissipated at cycle i due to fatigue damage, w_i^ζ

$$w_i = w_i^\eta + w_i^\zeta$$

Hyp. 2. The energy dissipated due to viscoelastic damping w_i^{η} doesn't significantly change between two generic load cycles, therefore it can be considered constant during the test:

$$w_i^{\eta} \approx w_{i+1}^{\eta} = w^{\eta}$$

Hyp. 3. At a generic cycle i, the energy dissipated due to viscoelastic damping is significantly larger than the energy dissipated due to fatigue damage:

$$w_i^{\eta} \gg w_i^{\zeta}$$

On the basis of the aforementioned assumptions, the RDEC can be written with reference to the viscoelastic damping and by fatigue damage from cycle "a" to cycle "b":

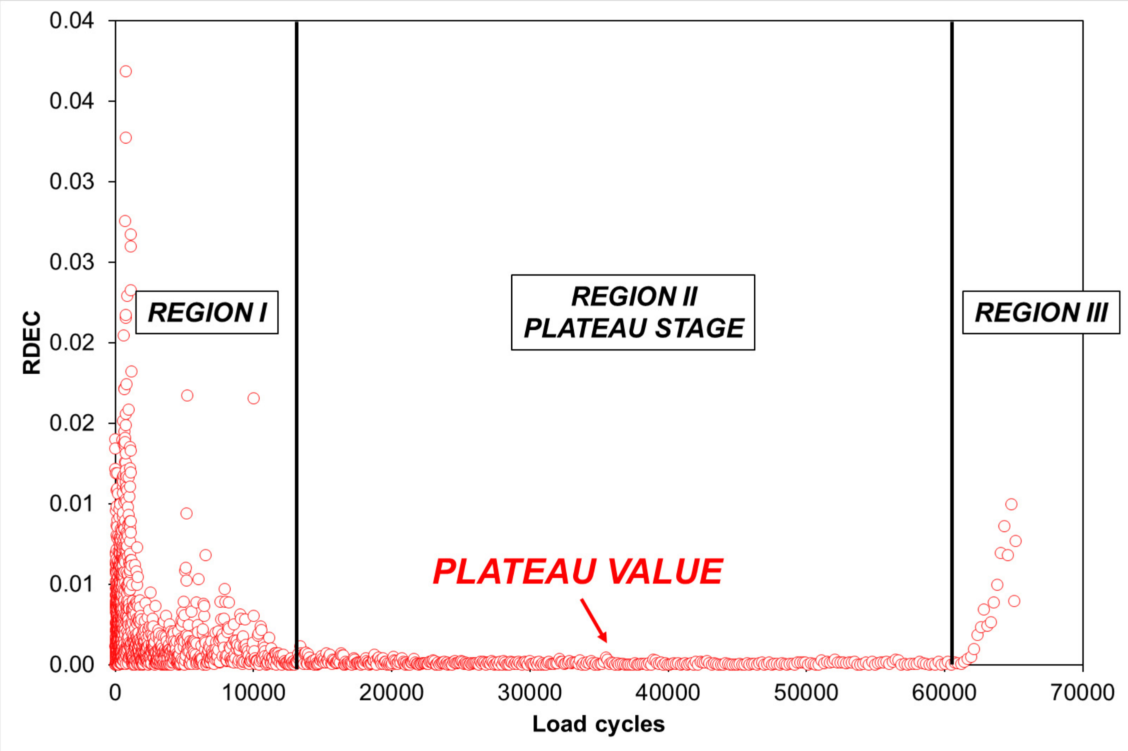
$$RDEC_a = \frac{\Delta w^{\zeta}}{w^{\eta} \cdot (b-a)}$$

Therefore, the RDEC is the amount of energy being dissipated by the material at a generic load cycle that goes to fatigue damage. This specific amount of energy causes cracks first to initiate and then to propagate within the sample. This overcomes the limitations of Van Dijk's model, where the parameter *w* (total dissipated energy) was related both to viscoelastic damping and fatigue damage.

The RDEC can be plotted versus the number of cycle to evaluate the change in dissipated energy due to fatigue damage [106] during the test. Ghuzlan and Carpenter identify three regions in the RDEC curve:

1. Region I represents a reorientation of the material due to the application of load. It's an initial phase with high scattering of measured data
2. Region II is an extended level plateau in the data plot. This plateau value represents a period where there is a constant percent of input energy being turned into damage. The value appears to be a mixture and load/strain input related value. In other words, in region II the entire amount of input energy is turned into fatigue damage
3. Region III represents ultimate failure in the mixture. The upturn in the curve indicates that more damage is being done per load cycle with each subsequent load cycle. This represents an unstable condition in the mixture, and ultimately the mixture has no load carrying capability. The failure point, defined by the onset of unstable damage accumulation, occurs for all fatigue modes, providing a means of producing fatigue curves relating the plateau value of the dissipated energy ratio to the number of loads to a "true failure" in the laboratory

Fig. 6.13: typical plot of RDEC versus number of load cycles with three behaviour zones. The example is taken from



The value of RDEC in Region II is used to calculate the Plateau Value, which is the constant value of dissipated energy that produces damage to the material under cyclic loading. The PV can be calculated from the expression of the RDEC considering an interval of 100 load cycles (i.e. b – a = 100) with the following expression:

$$PV = \frac{1 - \left(1 + \frac{100}{N_f}\right)^f}{100}$$

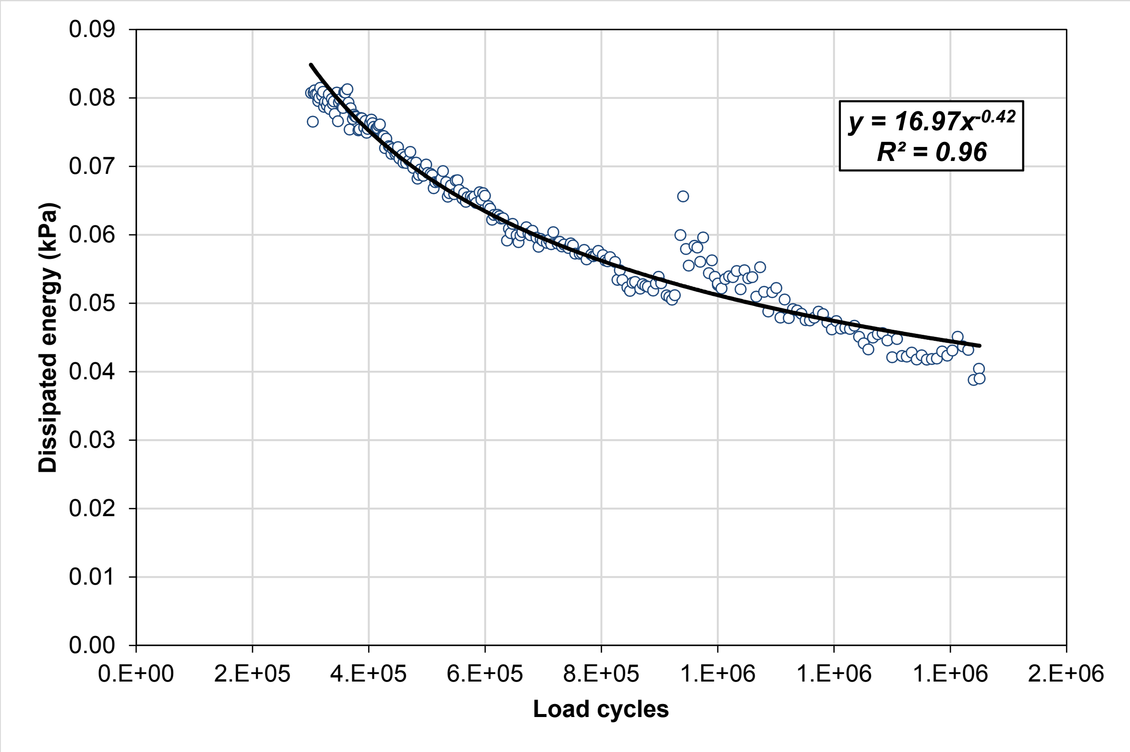
PV, the constant value of RDEC in stage II (Plateau Stage), is significant for evaluating HMA's fatigue behaviour because it represents a period where there is a relatively constant percent of input energy being turned into damage [109].

N_{f20} is the number of cycles to failure that corresponds to the conventional failure criteria at 20% of the initial flexural stiffness. The exponent "f" is calculated from the plot of dissipated energy versus number of cycles to failure. It is assumed that the regression equation of dissipated energy-loading cycle relationship follows a power law relationship [108]:

$$w(N) = A \cdot N^f$$

The key for the curve fitting process is to obtain a slope (in the power law relation plot) of the curve, f, which can best represent the original curve. In general, there are two rules for evaluating the goodness of the fitted curve: a high R-square value, and/or correct trend of the DE-N curve. An example is given in Fig. 6.14.

Fig. 6.14: determination of coefficient "f" of PV from the DE-N curve fitting with power-law equation



Once the PV is calculated from each fatigue test, the energy fatigue curves PV- N_f are plotted. In spite of the longer procedure required to determine the Plateau Value, the main advantages of RDEC approach relative to the Wöhler approach are [109]:

1. The unique fatigue relationship (PV- N_f) that it presents is independent of mixture type (bitumen grade, aggregate type, gradation, etc.), mode of loading (controlled strain or stress) and loading (test) conditions. It is therefore a fundamental relationship
2. The uniqueness of the PV- N_f relationship provides a way to study both fatigue endurance limit and healing, both of which have a great influence on asphalt material fatigue behaviour

6.1.5 Fatigue curves PV – N_f

Results of energy analysis are reported in Table 6.4 in terms of: number of cycles to failure at N_{f20}, total dissipated energy, coefficient "f" of the power law fitting of dissipated energy-load cycles curve, plateau value corresponding to the chosen failure criteria.

Table 6.4: Plateau Value PV and number of cycles to failure at N_{f20} resulting from flexural fatigue tests on RAP mixes

Mix	*% RAP*	*Sample ID*	*Strain level (με)*	*Cycles to failure at N_{f20}*	*Total dissipated energy (kJ/m³)*	*Power Law Slope N_{f20}*	*PV N_{f20}*
Mix0	0	2	100	2.2E+06	185	-0.322	1.44E-07
Mix0	0	4	100	1.5E+06	133	-0.172	1.19E-07
Mix0	0	5	100	1.9E+06	155	-0.197	1.02E-07
Mix0	0	6	150	3.4E+05	271	-0.388	1.14E-06
Mix0	0	1	200	4.9E+04	520	-0.270	5.48E-06
Mix0	0	3	200	6.5E+04	532	-0.403	6.18E-06
Mix0	0	7	200	5.9E+04	501	-0.272	4.57E-06
Mix1	10	2	100	3.1E+06	121	-0.270	8.59E-08
Mix1	10	4	100	1.9E+06	125	-0.285	1.48E-07
Mix1	10	5	100	2.6E+06	118	-0.201	7.73E-08
Mix1	10	1	150	3.9E+05	259	-0.476	1.21E-06
Mix1	10	3	200	7.9E+04	566	-0.360	4.58E-06
Mix1	10	6	200	6.9E+04	635	-0.369	5.34E-06
Mix1	10	7	200	7.5E+04	587	-0.308	4.10E-06
Mix2	20	1	100	1.3E+06	108	-0.180	1.34E-07
Mix2	20	2	100	1.7E+06	93	-0.140	8.21E-08
Mix2	20	6	100	1.7E+06	99	-0.184	1.06E-07
Mix2	20	3	150	3.6E+05	279	-0.256	7.15E-07
Mix2	20	4	200	6.4E+04	537	-0.176	2.74E-06
Mix2	20	5	200	5.4E+04	707	-0.310	5.74E-06
Mix2	20	7	200	6.3E+04	524	-0.278	4.37E-06
Mix3	30	2	100	4.3E+06	88	-0.142	3.28E-08
Mix3	30	7	100	6.1E+06	135	-0.136	2.24E-08
Mix3	30	6	100	5.1E+06	113	-0.183	3.55E-08
Mix3	30	5	150	4.3E+05	312	-0.192	4.47E-07
Mix3	30	1	200	2.5E+05	447	-0.300	1.20E-06
Mix3	30	4	200	7.4E+04	625	-0.183	2.46E-06
Mix3	30	8	200	1.82E+05	489	-0.331	1.82E-06

Energy fatigue curves are built similarly to traditional Wöhler fatigue curves. A linear regression is made between the natural logarithms of $N_{i,j,k}$ and $PV_{i,j,k}$ for the chosen failure criteria j, set of test conditions k and for the specimen i. The energy fatigue line has the following expression:

$$\ln PV_{i,j,k} = a + p \cdot \ln N_{i,j,k}$$

From the measured data, the following parameters are calculated and reported in Table 6.5:

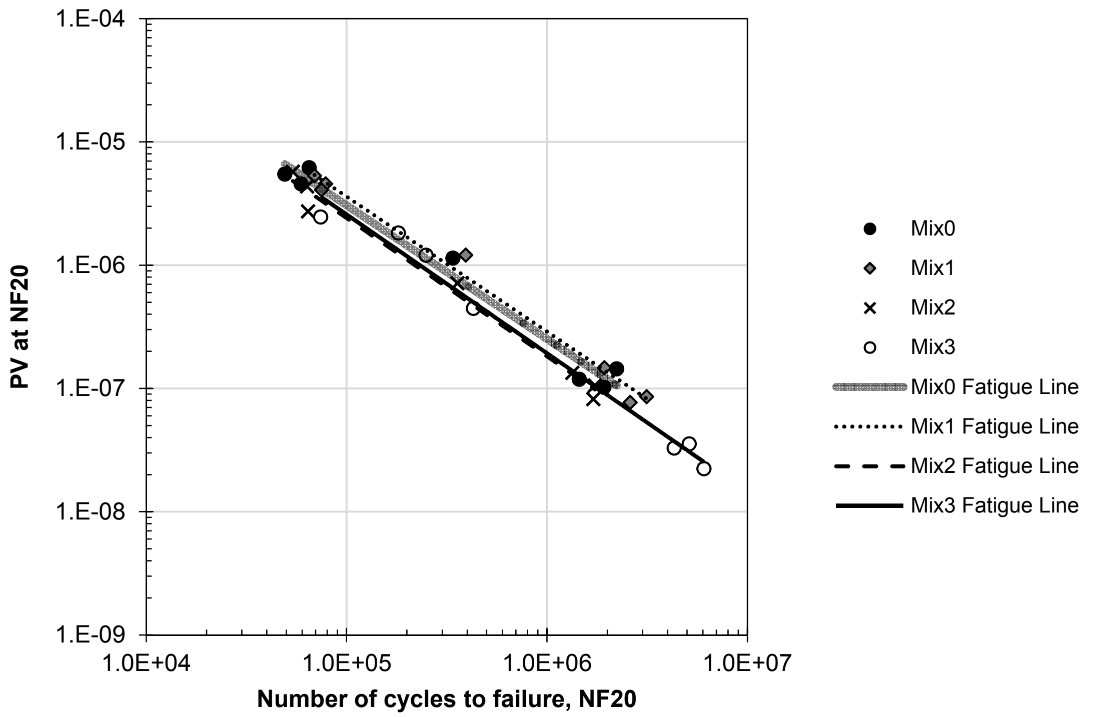
- Estimation of the slope p
- Estimation of q
- Coefficient of determination R^2

Table 6.5: energy fatigue lines parameters

Mixture	Fatigue line coefficient q	Fatigue line coefficient p	R^2
Mix0	0.07	-1.08	0.98
Mix1	0.05	-1.09	0.99
Mix2	0.06	-1.12	0.99
Mix3	0.06	-1.09	0.99

The energy fatigue line is obtained for each material by plotting the Plateau Value on the x-axis and the number of cycles to failure on the y-axis. Fatigue lines are represented on a Log-Log (base 10 logarithm) chart in Fig. 6.15.

Fig. 6.15: energy fatigue lines of investigated asphalt mixes plotted on Log-Log chart



Since the PV is supposed to be a fundamental parameter i.e. independent of mode of loading and testing conditions, all the Plateau Values and correspondent number of cycles to failure can be plotted on a single fatigue line [107] [108] [109]. This appears to be reasonable observing fatigue line coefficients “p” (line slopes) and “q”, which show almost equal values. The same parameters referred to a single energy fatigue lines are reported in Table 6.6

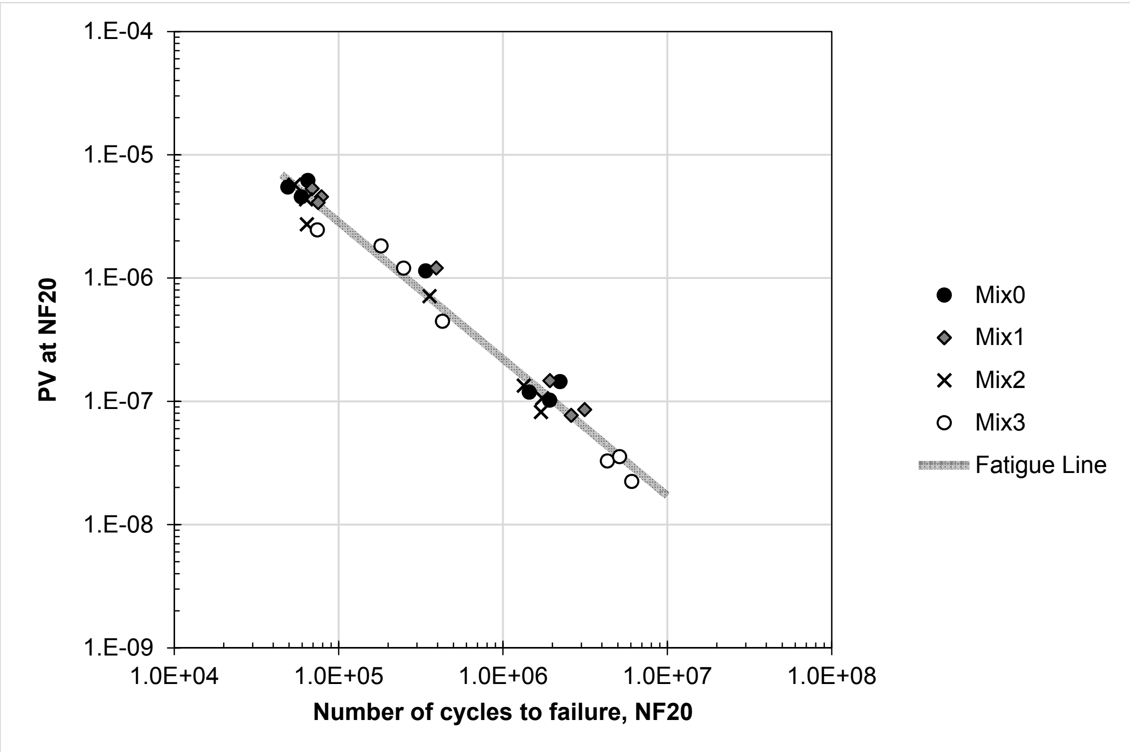
Table 6.6: unique energy fatigue line parameters

Mixture	Fatigue line coefficient q	Fatigue line coefficient p	R^2
Unique energy fatigue line (all mixes)	0.03	-1.11	0.98

The fatigue response of asphalt mixes is represented through the unique energy fatigue curve in Fig. 6.16. The PV represents the amount of energy that, under condition of load-dissipated energy equilibrium i.e. region II, is being turned into fatigue damage. The higher the PV, the higher the fatigue damage, hence the lower the fatigue life. The interpretation of energy and Wöhler fatigue lines is therefore the opposite. In energy fatigue line, the top-left corner of the plot represents high fatigue damage associated with low number of cycles to failure, whereas in the Wöhler fatigue curve, the same part of the curve indicates a condition of low strain and high number of cycles to failure.

The energy fatigue line allows to plot results obtained from different fatigue tests (i.e. mode of loading and test method) and different materials on a unique line. This representation may be convenient when a large number of mixtures are investigated and compared in terms of fatigue response. The good correlation between PV and N_f reported in Table 6.6 seems to conform with the idea of a unique energy fatigue line.

Fig. 6.16: unique energy fatigue line for all the investigated mixes on Log-Log chart



From the analysis of energy fatigue curves, the following considerations are proposed:

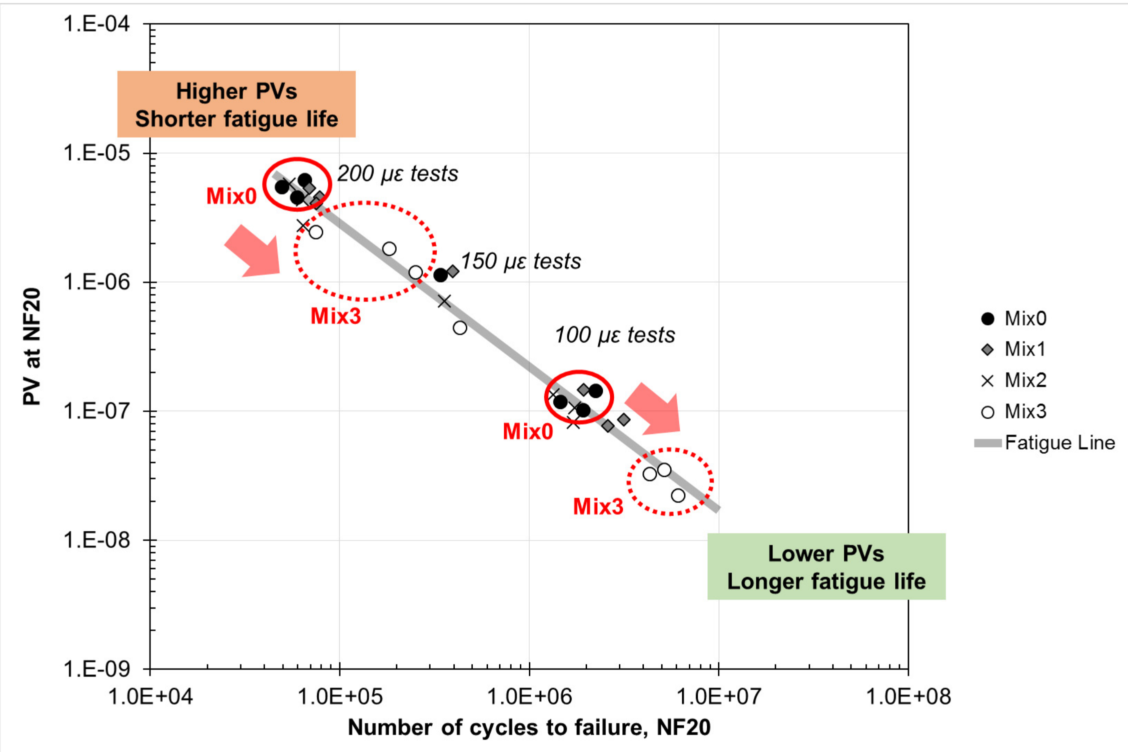
w. The Plateau Value (PV) is an indicator of fatigue damage occurring on asphalt mixtures that are subjected to cyclic loading. The PV refers to a phase of loading process where all the load energy is being turned into fatigue damage. The “plateau stage” corresponds to the region **II** on the RDEC-load cycles plot. All PVs are calculated with reference to the conventional failure criteria N_{f20}

x. The fatigue response of the four RAP mixes appears to be similar since fatigue lines are parallel. Slope coefficients “q” obtained from linear regression of natural logarithm of PV and N_f range from -1.08 to 1.12. The coefficient of variation R^2 is above 0.98

y. The higher the PV, the higher the fatigue damage, therefore the shorter the fatigue life of the material. PVs calculated at high strain levels, in this case at 200 με, are represented on the top-left side of the energy fatigue curve, whereas the low strain levels are plotted on the bottom-right side

z. The energy fatigue line should be interpreted differently from the Wöhler curve: the sliding of the points to the bottom-right corner of the chart indicates an increased fatigue life, since the PV i.e. the fatigue damage decreases and the correspondent number of cycles to failure increases (Fig. 6.17)

Fig. 6.17: interpretation of energy fatigue line



aa. Results show a similar fatigue resistance for Mix0, Mix1 and Mix2. Mix 3 shows lower values of PV and hence a better response to fatigue testing i.e. higher number of cycles to failure at all strain levels (Fig. 6.17)

6.1.6 Effects of RAP percentage on the dissipated energy

The influence of RAP content on the dissipated energy is evaluated by plotting the average Plateau Value versus the RAP percentage both for 100 and 200 με strain levels. Results are shown in Fig. 6.18 and Fig. 6.19.

Fig. 6.18: average number of cycles to failure at 100 με for the investigated mixtures

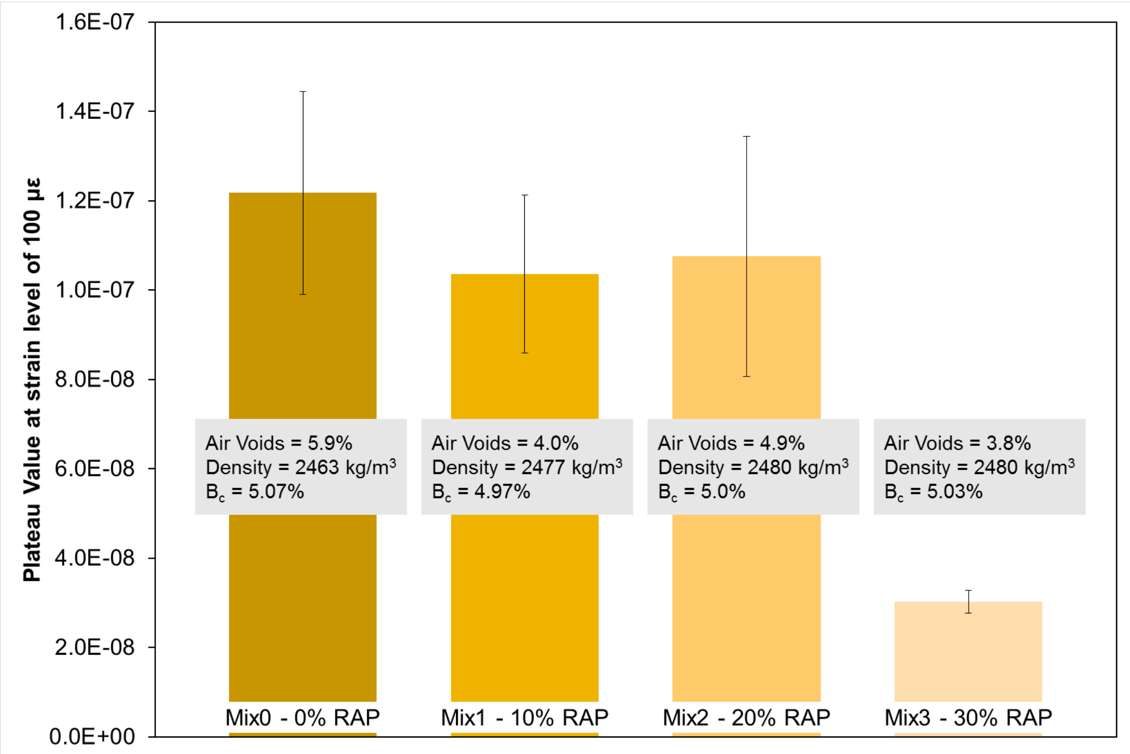
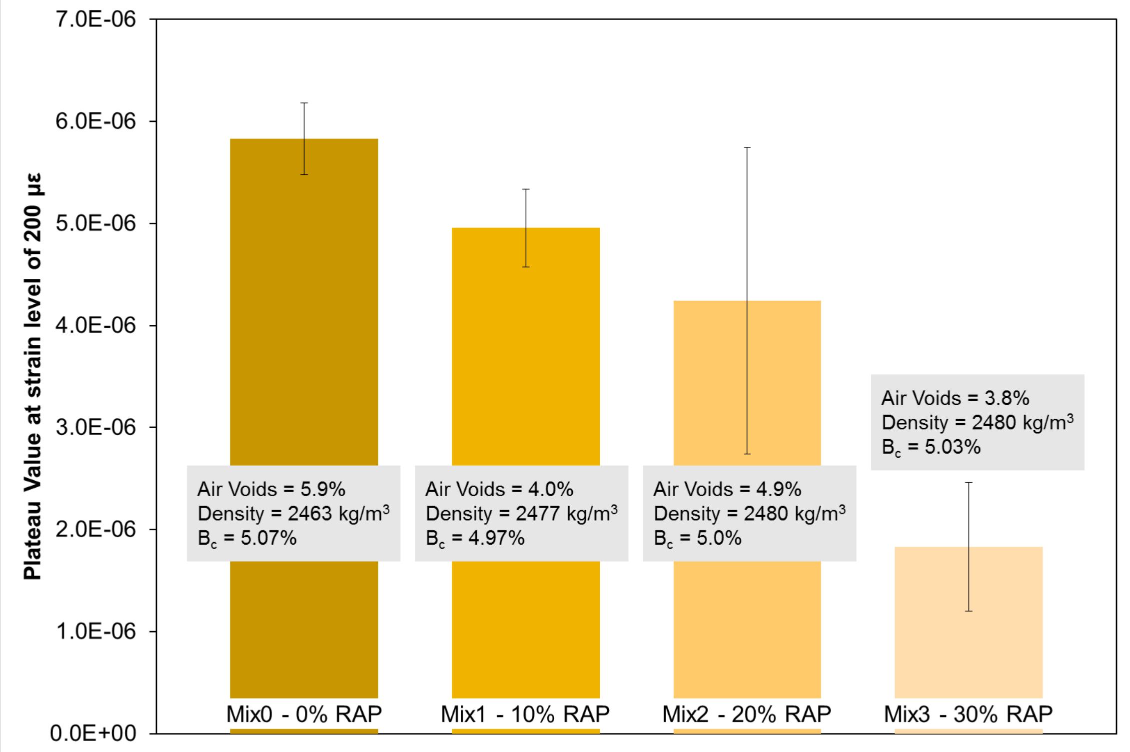


Fig. 6.19: average number of cycles to failure at 200 με for the investigated mixtures



From figures above the following considerations are proposed:

bb. Mix0, Mix1 (10% RAP) and Mix2 (20% RAP) show similar fatigue response both at 100 and 200 με, since they exhibit similar PVs

cc. Mix3, the mixture with 30% RAP, shows the longest number of cycles to failure, and hence the longest fatigue life, among the four mixes. In fact, PV of Mix3 is significantly lower than PV of other mixes, indicating that a lower fatigue damage occurs in the mixture during cyclic loading. On the basis of the energetic approach results, Mix3 has the lower amount of energy being turned into damage relative to other mixtures

dd. The variability of calculated Plateau Values is higher compared to the variability of N_f, which has been used in the traditional approach to evaluate the effects of RAP content on durability of RAP mixes. This may be due to the fact that the number of cycles to failure Nf is a measured parameter, whereas the PV is obtained after a process that involves assumptions, manual procedures and calculations, increasing the chance of inaccuracies

ee. The value of 20% of RAP being incorporated into asphalt mixes appears to be a threshold below which the effects of RAP content on fatigue life are negligible or of low extent

ff. The change in PV seems to be more sensitive to the RAP content rather than to material properties as void or bitumen content. On the basis of test results and with regards to the asphalt mixture being investigated in this research, the RAP content is the most impacting factor on durability of mixes

gg. The previous point “d” suggests that the interaction between virgin and RAP bitumen plays a significant role on the fatigue performance of asphalt mixes. Results of RDEC approach suggest that the presence of 20% or less of RAP into new mixes doesn’t produce a significant interaction between neat and old bitumen, as already observed from the traditional fatigue analysis

6.1.7 Considerations on the RDEC approach

The aim of the RDEC is to provide a fundamental law to describe fatigue behaviour of asphalt mixtures. The Plateau Value is considered to be a parameter that is independent from mode of loading, test method and even material properties i.e. grading curve or type of aggregate [109]. The unique fatigue line is presented as a mechanistic and causal relationship between the cause, the fundamental parameter PV, and the effect, the number of cycles to failure. On the other hand, the Wöhler equation is a phenomenological law: it describes a phenomenon i.e. the fatigue resistance of asphalt mixtures on the basis of how it reveals itself. The traditional Wohler approach doesn’t investigate the causal link between the fatigue life and its causes as the RDEC approach pretends to. It just provide a relationship between two measurable and measured parameters as strain (or stress) level and correspondent number of cycles to failure.

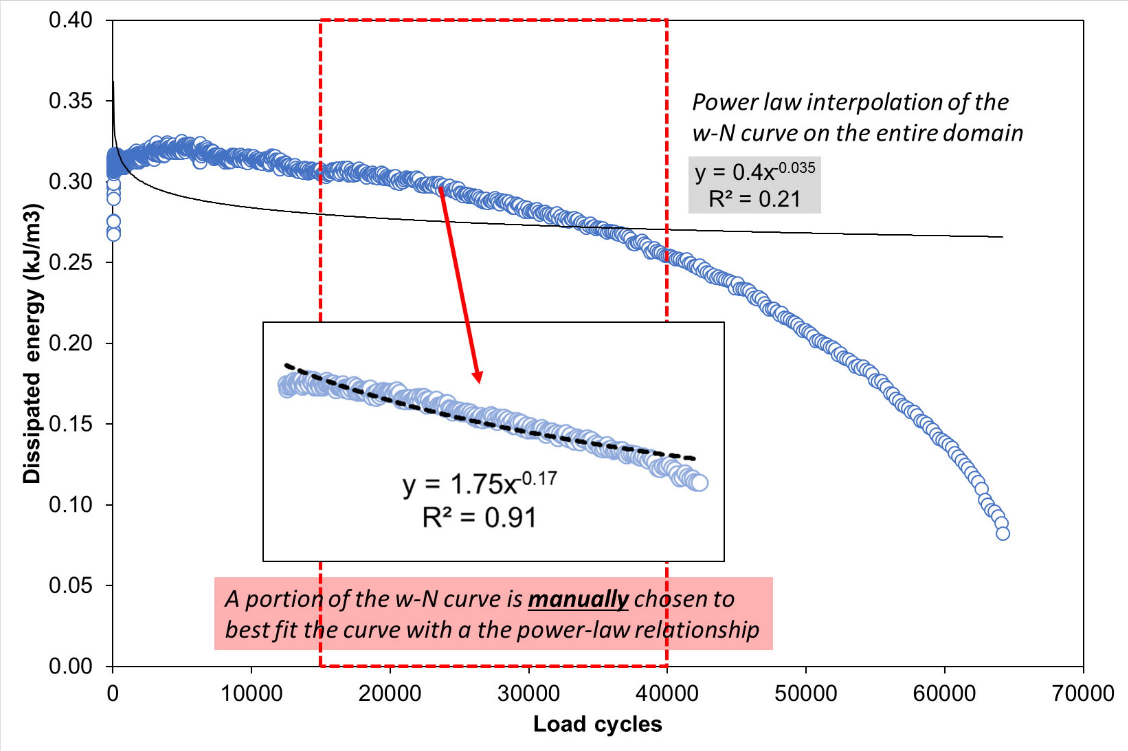
However, there are several doubts about the validity and the real benefit of the RDEC approach. Many of them have been encountered during the application of the energy method and are summarized below:

1. **Fitting of dissipated energy-load cycles (w-N) curves**: it is assumed that the w-N curve can be fitted with a power law equation. The x exponent is used to calculate the RDEC and the Plateau Value. In this research, none of the dissipated energy curves followed a power law trend. The authors of the RDEC approach [108] suggest to reduce the domain of the w-N curve until the power law fits the curve with high R^2 or with similar trending. This step is manually performed by the operator that treats data, therefore it’s arbitrary and with low reproducibility. An example of the procedure is given in Fig. 6.20. The figure refers to the fatigue test at 200 με on Mix2 with 20% RAP. It can be clearly observed that the dissipated energy curve has a trend that significantly differs from the power law, which cannot be used to fit w(N) on the entire domain. A portion of the load cycle domain should be manually chosen in order to obtain a good fitting. In this case, the domain is reduced from 0-60,000 to 15,500-40,000 load cycles. It can be concluded that:
    a. the coefficient “f” of the power law represents only a portion of the dissipated energy-load cycles curve
    b. An arbitrary and operator-dependent procedure is necessary to obtain a good fitting of the w-N curve with a power law equation, decreasing repeatability and reproducibility of the method. In other words, the parameter “f”, exponent of

the power law used to calculate the RDEC, depends on the interpretation of the operator

c. The manual fitting of w-N curves is a very time-consuming procedure

Fig. 6.20: manual procedure to fit the w-N curve with a power law equation



2. **Power law equation**: on the basis of available data, the power law equation doesn't seem to be the best equation to fit the dissipated energy curve. Consequently, the expression of the RDEC may be inaccurate
3. **Hypothesis on dissipated energy**: the expression of the RDEC and hence of the PV relies on three assumptions presented before. The hyp.1 says that the energy dissipated in a load cycle is sum of energy dissipated by viscoelastic damping and by fatigue damage. The first hypothesis is valid as long as rest periods are not introduced into the loading procedure. Rest periods are associated with healing, which is a form a self-recovery of the material i.e. bituminous mastic from fatigue damage. Healing should be considered into the energetic balance of the system. The hyp. 2 says that the energy dissipated by viscoelastic damping is constant during the test or at least in the plateau stage. However, the validation of this assumption is complicated since there are no methods to measure the internal heat produced within the sample by the internal friction. If the viscoelastic damping was not constant, the expression of RDEC would be:

$$RDEC = \frac{1}{b-a} \cdot \left(\frac{\Delta w^{\eta} + \Delta w^{\zeta}}{w_a^{\eta}} \right)$$

Therefore, the difference between viscoelastic damping from cycles “a” to “b” should be included into the RDEC expression. This means that the RDEC and hence the PV cannot be directly related to the fatigue damage only, but they include also the amount of energy dissipated by viscoelastic damping. Thus, the idea of the PV as a fundamental indicator of fatigue damage appears to be weak

4. **Variability of PV**: in addition to previous considerations, data reported in Fig. 6.18 and Fig. 6.19 show high variability (higher than that of parameter N_f), suggesting that PV is a less reliable indicator of fatigue damage compared to number of cycles to failure

On the basis of the points presented above, the RDEC approach seems to fail to determine a fundamental fatigue law for asphalt mixes. Furthermore, the time-consuming procedure and its variability, which is due to the presence of some operator-dependent steps in the procedure, make the RDEC approach less appropriate to evaluate the laboratory fatigue behaviour of asphalt mixtures compared to the traditional Wöhler approach. The latter has a consolidated literature, it’s standardized and, in the present research, gives the same information of the more complicated RDEC approach.

6.2 Tests on recovered bitumen

Results of fatigue tests have shown that the interaction between virgin and RAP bitumen is a key issue to understand the effects of RAP on durability of the investigated mixes. Data analysed both with Wöhler and energy approaches indicate that RAP content affects fatigue life more than other material properties as air void content and density. The increase in fatigue life of the mix made with 30% RAP pays attention to possible brittleness issues of the bituminous blend. Furthermore, the Black diagram reveals a hardening of mixes with RAP content being increased.

Tests on asphalt mixes provide only an indirect evaluation of the interaction. This explain the importance of investigating in detail the interaction between old and new bitumen with a specific laboratory testing campaign.

6.2.1 Standard characterization test results

Results in Table 6.7 show a decrease in penetration and an increase in softening point and viscosity as RAP content increases. This indicates a hardening of the composite blend caused by the presence of aged bitumen. Penetration, softening point and viscosity change in proportion to the RAP content in the mixture.

Parameters measured for B_0, B_1 and B_2 indicate a small hardening of the composite blend due to the presence of RAP binder. The hardening effect of the old bitumen on the composite blend becomes relevant as the RAP binder content is above 20% by weight of the mix.

The Penetration Index (PI) was calculated according to EN 12591 as an indicator of temperature susceptibility of the bitumen. The lower the value of PI, the higher the temperature susceptibility of the binder. PI reported in Table 6.7 indicates that B_0, B_1 and B_2 have identical temperature susceptibility, with B_3 and B_{RAP} having a higher PI value and, therefore, a lower temperature susceptibility. This result contradicts expectations of greater temperature susceptibility of blends with high RAP content. Researches [110] have shown that PI is poorly related to temperature susceptibility of bitumens, since temperature and time effects are confounded in the determination of PI.

Table 6.7: results of standard characterization tests on recovered binders

Bituminous sample	Penetration at 25°C (dmm)	Softening point (°C)	Dynamic viscosity at 160 °C (Pa·s)	Penetration Index
B_0	27	55	0.32	-1.3
B_1	25	56	0.33	-1.3
B_2	21	57	0.37	-1.3
B_3	16	63	0.53	-0.8
B_{RAP}	7	73	0.55	-0.5

6.2.2 Dynamic mechanical analysis with DSR

6.2.2.1 Dynamic viscosity test results

Viscosity values of bituminous blends for each test temperature are shown in Table 6.8.

Table 6.8: results of dynamic viscosity test on recovered binders

Bituminous samples	Dynamic Viscosity (Pa·s)			
	135°C	*150°C*	*160°C*	*170°C*
B_0	1.36	0.46	0.32	0.25
B_1	1.41	0.45	0.33	0.25
B_2	1.51	0.46	0.37	0.28
B_3	1.59	0.75	0.53	0.32
B_{RAP}	1.92	0.96	0.55	0.32

Fig. 6.21: dynamic viscosity of bituminous samples at test temperatures of 135, 150, 160 and 170°C

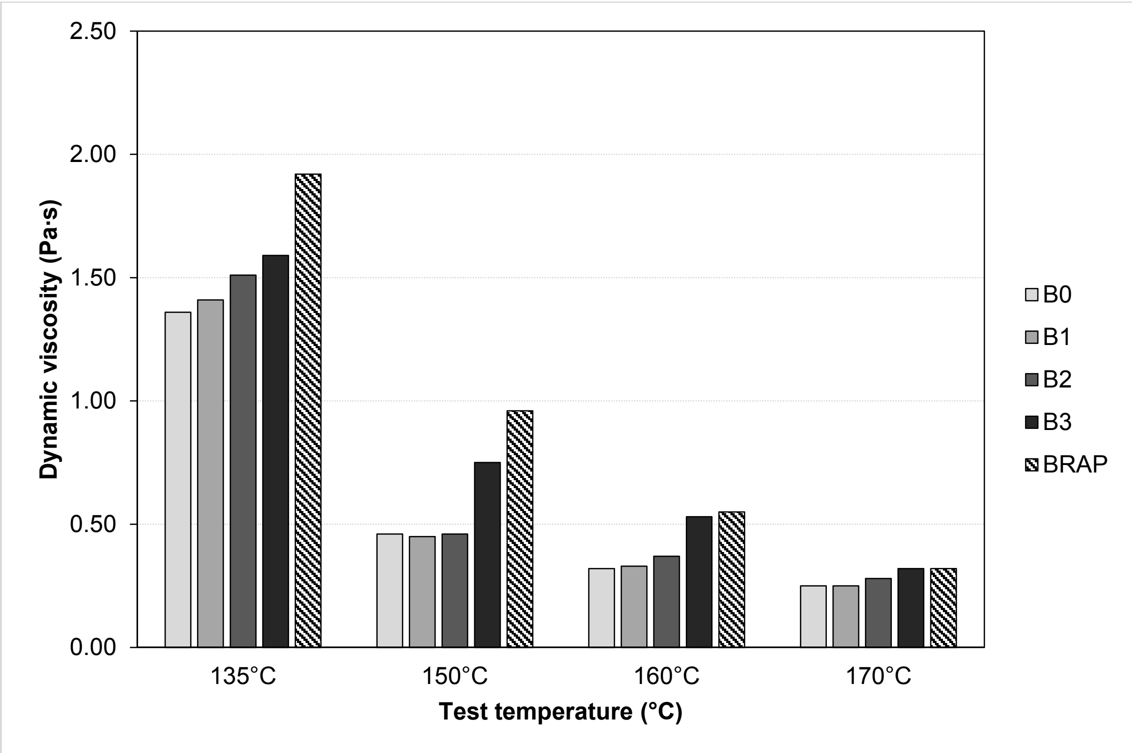
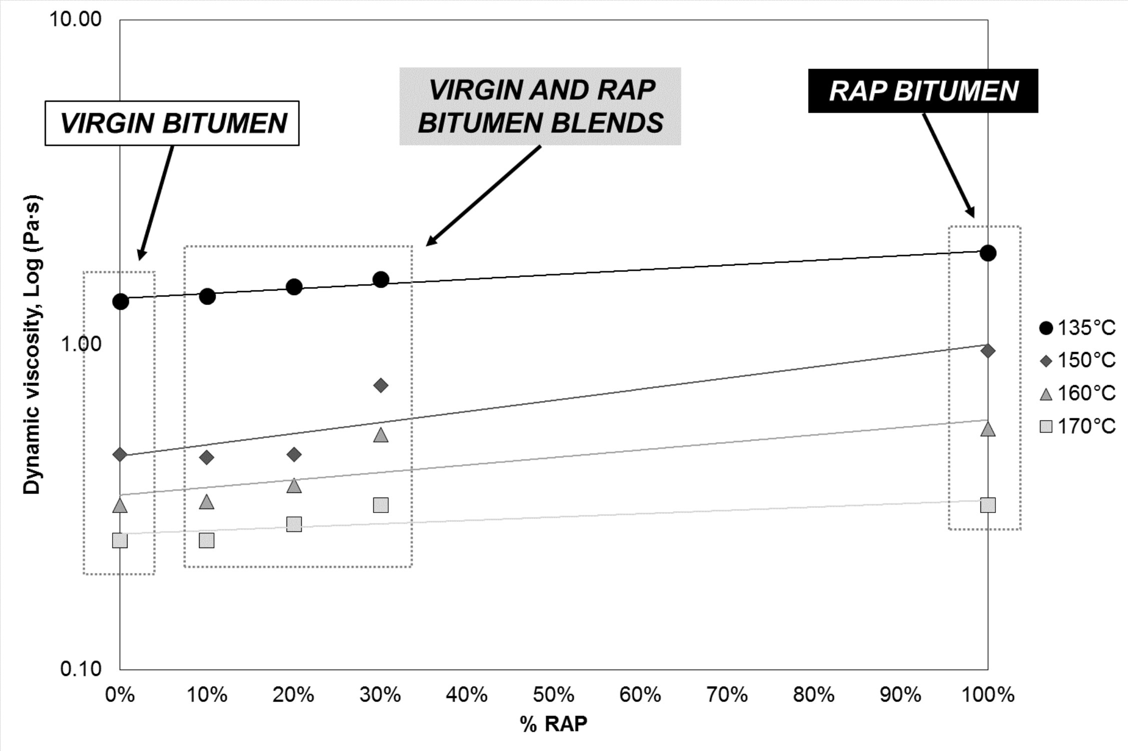


Fig. 6.22: dynamic viscosity as function of aged binder content



Viscosity of blends increases in proportion to RAP binder content (Fig. 6.21 and Fig. 6.22). The higher the RAP binder, the higher the viscosity of the blend. The presence of the more viscous old binder causes the composite blend to harden, resulting in viscosity being increased. This confirms the results obtained from penetration and softening point tests.

RAP binder has been exposed to ageing processes that have hardened the bitumen. Short term (ST) ageing, which takes place during mixing, transport and laying of bituminous materials, changes the chemical composition of the binder with significant impact on its rheological properties. During ST ageing, bitumen experiences increasing in viscosity caused by oxidation, volatilisation and exudation [2] [111]. During the long term (LT) ageing, bitumen progressively increases viscosity and stiffness due to oxidation, polymerisation, photo-oxidation of surface layers, thixotropy and syneresis [9]. The major part of asphalt ageing occurs during the short term [90]. The increase in viscosity of RAP bitumen is caused by several factors: ratio of resins to asphaltenes [87], increase in number of asphaltenes (between 5 to 20% by weight [90]), increase in molecular weight and polydispersity [111]. Therefore, the presence of RAP binder changes chemistry of the bituminous blend, resulting in a harder and consequently more viscous composite bitumen.

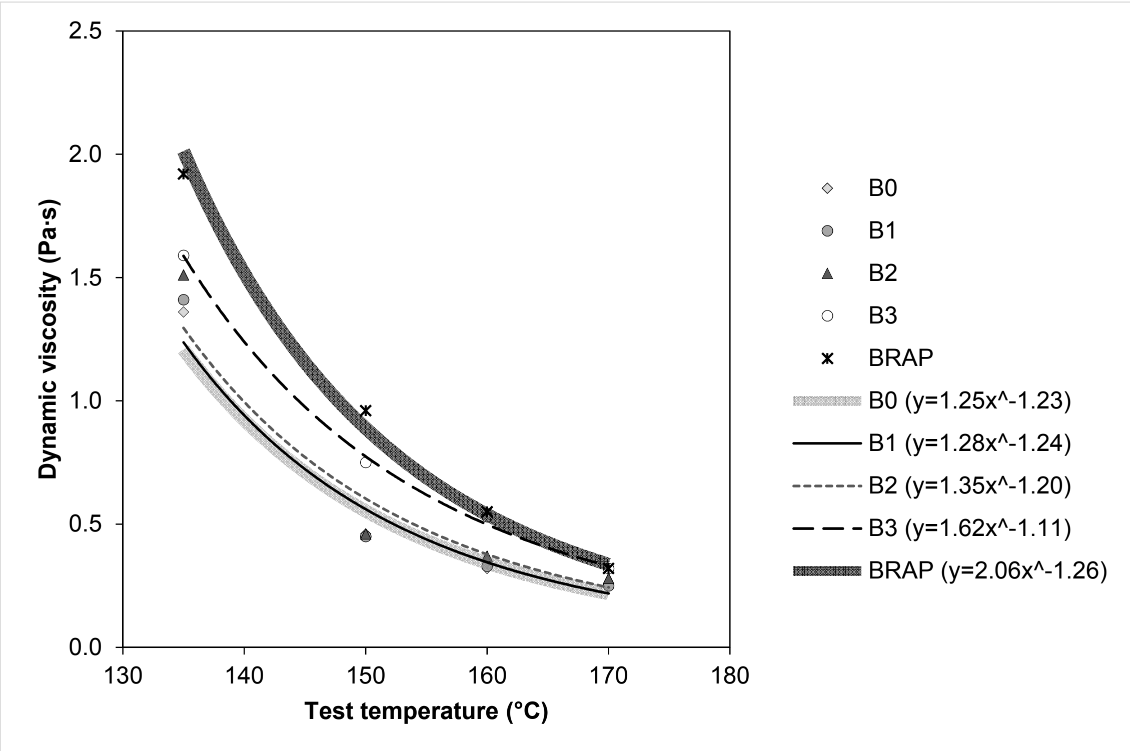
The values of viscosity of RAP blends are greater than typical viscosities of 50/70 bitumens. This is expected since penetration grade and softening point of the neat binder showed deviation from typical values of unmodified 50/70 binders (Table 6.1), indicating a harder bitumen compared with a traditional 50/70. Measured viscosities exceed recommended values of 0.17±0.02 and 0.23±0.03 Pa·s at 160°C reported in ASTM D 2493 respectively for optimal mixing and compaction of HMA with unmodified binders. Similar values of viscosity may be found on Polymer

Modified Binders (PMB), for which production and working temperatures range from 160 to 180°C. However, the presence of polymers significantly differentiates the rheological behaviour of modified and unmodified binders, allowing the firsts to be subjected to higher temperatures without experiencing ageing. Lowering the viscosity of RAP blends appears to be crucial to ensure performances and feasibility of RAP asphalt mixtures. An increase in production and working temperatures might cause the composite blend to harden, leading to a further increase in viscosity. Therefore, a softer neat bitumen or additives may be used for lowering the viscosity of RAP blends.

Results indicate no significant differences in viscosity between B_0 and B_1. Therefore, the presence of 10% or less of RAP (low RAP content [99]) into the mixture does not affect the viscosity of the composite blend. Viscosity significantly increases as RAP content exceed 20% (high RAP content [99]) at temperatures of 150 and 160 °C. This indicates a significant hardening effect on the blend caused by the aged binder as the RAP content is above 20%.

The temperature susceptibility of bituminous blends was represented investigating the trend of viscosity as function of temperature (Fig. 6.23). Because viscosity measurements is fitted with power-law trend, the x-coefficient of power-law can be used as indicator of temperature susceptibility of bitumens. Results show that the higher the RAP binder content, the higher the temperature susceptibility within the test temperature range. The trend of viscosity of RAP blends is between the trends of neat and RAP binders in proportion to RAP binder content in the blend. When the RAP content is above 20%, the bituminous blend experiences a significant shift towards higher values of viscosity. This indicates a hardening effect of RAP binder present in the mixture.

Fig. 6.23: temperature susceptibility of bituminous samples measured through viscosity tests



Viscosity test results can be used to verify the assumption of total blending between neat and RAP binders after being extracted with the procedure previously described. The procedure of extraction and recovery of bituminous blends is expected to produce 100% blending of RAP and virgin binder [2]. The assumption of total blending was verified using the log-additivity rule (LAR). LAR predicts viscosity of homologous polymer blends (HPB), a sub-class of PB with chemically identical polymers differing in molar mass. Viscosity of HPB proportionally depends on mass fraction (percentage by mass) of constituent polymers [90]. LAR is expressed as [112]:

$$\ln \eta_C(T) = \sum_i w_i \ln \eta_i(T)$$

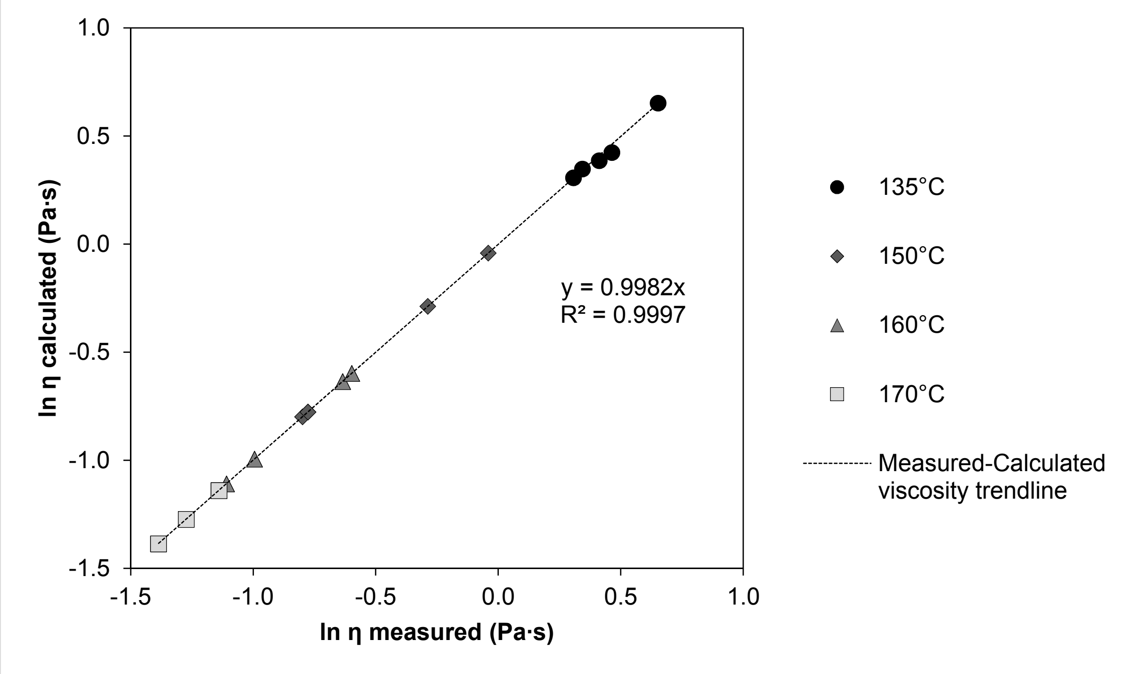
In which η_C is the viscosity of the composite blend at temperature T, w_i and η_i are the mass fraction and the viscosity of the component i at temperature T. Results in Table 6.8 show that viscosity of bitumen blends increases proportionally with the aged binder content in the composite blend, ranging from recovered neat binder (B_0) viscosity and RAP binder (B_{RAP}) viscosity. Similarly to HPB, the LAR was applied to bitumen blends. Viscosity was calculated with LAR for B_1, B_2 and B_3, then predicted and measured values were compared. Results are reported in Table 6.9.

Table 6.9: comparison between viscosity values of bituminous blends measured in laboratory and values calculated with LAR

Bituminous sample	Dynamic viscosity at 135°C		Dynamic viscosity at 150°C		Dynamic viscosity at 160°C		Dynamic viscosity at 170°C	
	Measured	*Calculated*	*Measured*	*Calculated*	*Measured*	*Calculated*	*Measured*	*Calculated*
B_0	0.307	0.307	-0.777	-0.777	-1.139	-1.139	-1.386	-1.386
B_1	0.344	0.348	-0.799	-0.799	-1.109	-1.109	-1.386	-1.386
B_2	0.412	0.387	-0.777	-0.777	-0.994	-0.994	-1.273	-1.273
B_3	0.464	0.424	-0.288	-0.288	-0.635	-0.635	-1.139	-1.139
B_{RAP}	0.652	0.652	-0.041	-0.041	-0.598	-0.598	-1.139	-1.139

A strong correlation was found between measured and calculated viscosities, as indicated by the linear relationship between $\ln(\eta_M)$ and $\ln(\eta_C)$ with slope of 0.99 and $R^2 = 0.99$ (Fig. 6.24). The viscosity of bitumen blend is proportionally affected by the content of RAP binder in the composite mix, showing a viscosity-concentration dependence in relation to the LAR. Investigated bituminous blends can be considered as homologous blends, validating the assumption of total blending between neat and RAP binders for the recovered bitumens.

Fig. 6.24: relationship between measured and LAR calculated viscosity of bituminous samples



6.2.2.2 Complex modulus isochronal plots (master curves)

An isochronal plot is a curve on a graph representing the behaviour of a system at a constant frequency or loading time. Curves of the complex modulus G* versus temperature are isochrones [113].

Master curves of complex modulus G* are reported in Fig. 6.25. Values of complex modulus (Table 6.10) increase in proportion to RAP content over the test temperature range, resulting in a vertical shift of G* curves. The increase in complex modulus indicates a hardening of the bitumen caused by the presence of the aged binder.

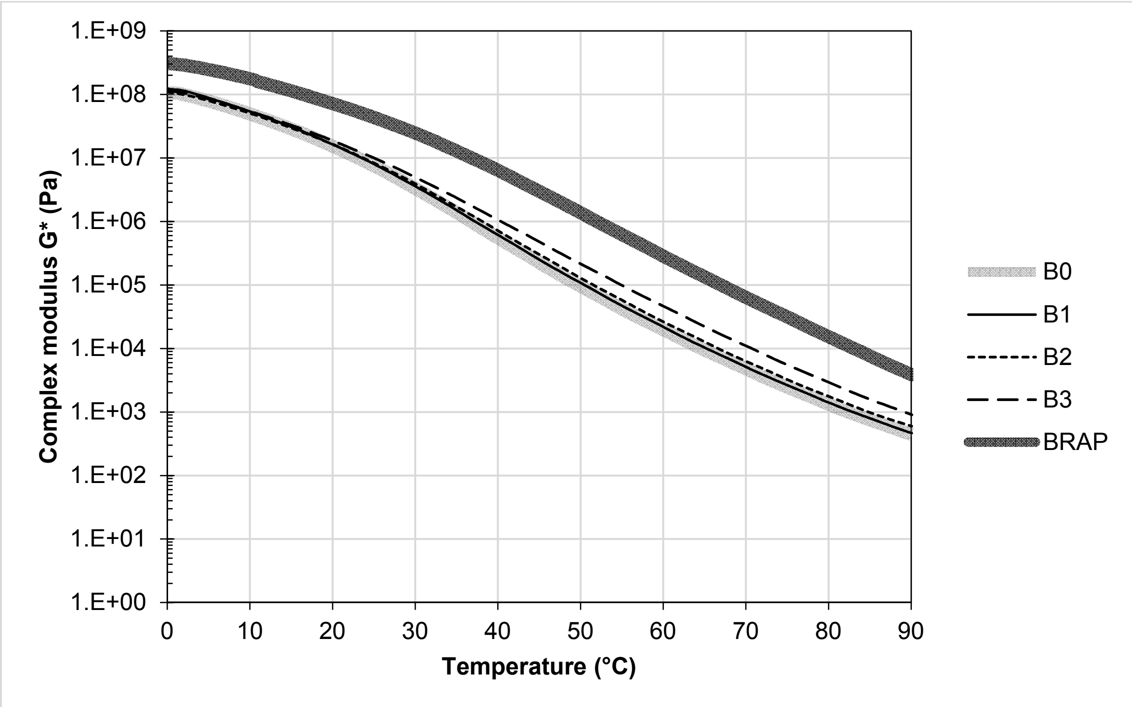
The hardening effect of aged binder on the composite blend is proportional to the RAP binder content in the mixture. When RAP content is equal or below 10%, the vertical shift is almost negligible. For RAP percentage equal or above 20%, the influence of recovered binder becomes more relevant, resulting in a significant vertical shift of G* curves. Therefore, adding small amounts of reclaimed asphalt into the mixture has very limited effects on binder stiffness.

The effects of the aged binder on the complex modulus vary over the test temperature range. At temperatures from 0°C to 20°C, curves of G* for B_0, B_1, B_2 and B_3 show the same trend and similar values, indicating a limited stiffening effect of the aged binder. This might suggest that the presence of even high percentages (greater than 20%) of RAP binder into the mixtures has small effect on the low-temperature cracking susceptibility of the blends. As temperature exceed 20°C, isochrones of B_1, B_2 and B_3 deviate from the reference curve of B_0 and shift towards higher values of G* (with a magnitude in proportion to the RAP binder content). The stiffening effect of the RAP binder on the complex modulus of the blends becomes more relevant when temperature exceed 20°C, becoming constant at a temperature of approximately 50°C.

Table 6.10: complex modulus values measured with DSR for each bitumen, reported at temperature steps of 10°C

Temperature (°C)	Complex Modulus |G*| (kPa)				
	B_0	*B_1*	*B_2*	*B_3*	*B_{RAP}*
0	104000	123000	110000	114000	308000
10	51600	56400	52800	56600	182000
20	16200	17200	17300	19600	74500
30	3550	3830	4200	5250	25900
40	592	658	772	1130	6790
50	103	116	138	229	1460
60	21.1	23.4	28.1	49.2	305.0
70	5.1	5.5	6.7	11.6	67.7
80	1.4	1.5	1.9	3.1	16.3
90	0.5	0.5	0.6	1.0	4.1

Fig. 6.25: isochronal curves of complex modulus G* of investigated mixtures



6.2.2.3 Phase angle isochronal plots

The phase angle δ is plotted in Fig. 6.26 as tgδ versus temperature. Values of δ are reported in Table 6.11. Tgδ is defined as the ratio between loss (G’’) and storage (G’) components of the complex modulus, therefore it can be considered as an indicator of the viscoelastic balance of the mixture.

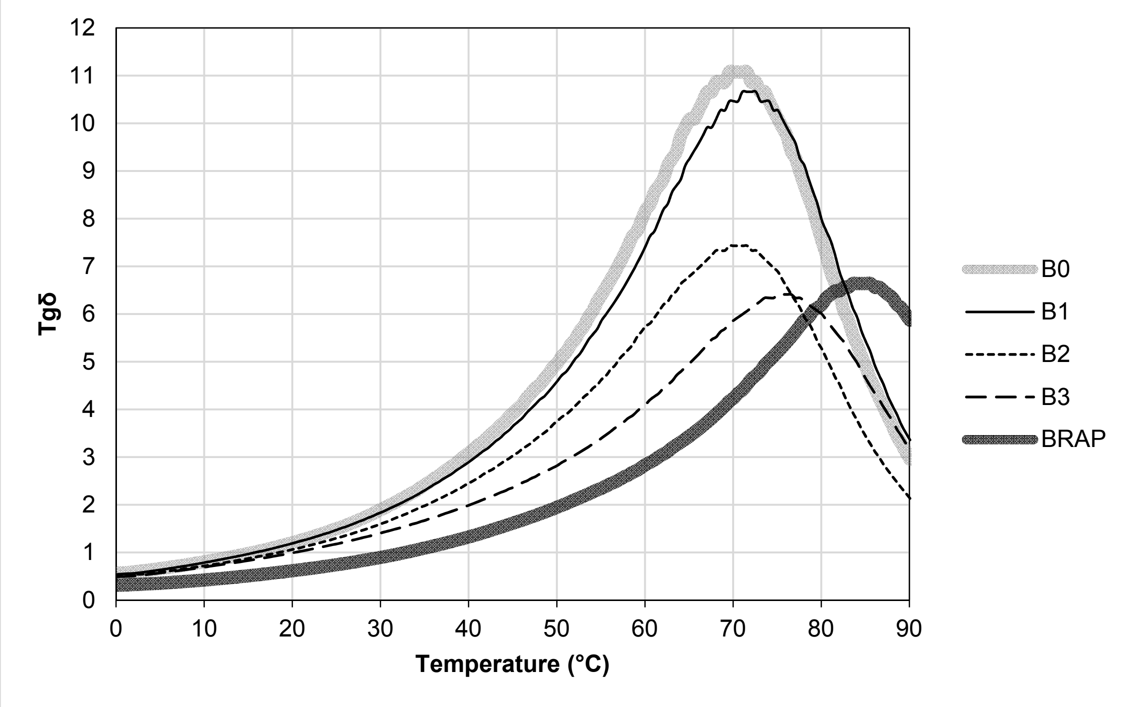
Results show that the phase angle decreases in proportion to RAP content in the mixture over the temperature range. Since tgδ is defined as G’’/G’, the decrease in the phase angle corresponds to an increase in the elastic behaviour of the bitumen caused by the presence of aged binder.

As observed for complex modulus isochrones, the magnitude of the decrease in tgδ and hence the increased elastic behaviour of the mixture is proportional to the aged binder content. When the aged binder content is equal or below 10%, there is a limited influence of the aged binder on rheological properties of the final blend. The effects of the presence of aged binder into the composite blend become more relevant as the RAP content raises to or exceed 20%. The curves of tgδ for B_2 and B_3 shift towards the B_{RAP} curve and lower values of tgδ, indicating a more elastic behaviour of the blends.

Table 6.11: phase angle values measured with DSR for each bitumen, reported at temperature steps of 10°C

Temperature (°C)	Phase Angle δ (°)				
	B_0	B_1	B_2	B_3	B_{RAP}
0	29	28	26	26	18
10	38	38	35	34	23
20	50	49	46	44	31
30	62	61	57	54	41
40	72	71	67	63	52
50	78	77	75	70	62
60	83	82	80	76	70
70	85	84	82	80	76
80	83	83	79	81	81
90	72	74	66	73	80

Fig. 6.26: plot of phase angle represented as tgδ versus test temperature



G' describes the amount of energy stored and elastically released during each oscillation of the DSR, while G'' describes the energy dissipation associated with viscous effects [90]. When the stored energy equals the dissipated energy, the mixture experiences a viscoelastic transition to a predominantly elastic or viscous behaviour. The equality of G' and G'' occurs as tgδ = 1 at the cross-over temperature. The cross over temperature allows to determine the temperature range where the mechanical response of the bituminous blend is predominantly elastic or viscous. The calculated cross-over temperatures are reported in Table 6.12.

Table 6.12: results of cross-over temperatures and corresponding complex moduli G* of investigated mixtures

Bituminous sample	Cross-over temperature (°C)	G* at cross-over temperature (MPa)
B_0	15.6	26.7
B_1	16.1	27.1
B_2	18.6	19.6
B_3	20.3	18.4
B_{RAP}	33.1	16.6

Results indicate that the cross-over temperature increases as the content of RAP binder into the blends increases. Therefore, an increase in the aged binder content leads to a widening of the temperature range where the mechanical response of the bitumen is predominantly elastic.

On the basis of the previous considerations, the plot of tgδ (Fig. 6.26) can be divided into three regions. A first region (low temperatures), corresponding to a temperature range of 0°C to cross-over temperature, where the mechanical response of bitumens is predominantly elastic. A second region (intermediate temperatures), from the cross-over temperature to approximately 70/80°C depending on RAP binder content, where the mechanical response is predominantly viscous and tends to become more viscous as temperature increases. A third region (high temperatures), corresponding to a temperature range of 70/80°C to 90°C (end of test), where the mechanical response is predominantly viscous but tends to become more elastic.

This behaviour at high temperatures can be also observed plotting the isochronal curves of storage and loss modulus, reported in Fig. 6.27 and Fig. 6.28. The curves of the storage modulus change trend at high temperatures, with tendency of G' to be horizontal, whereas the curves of the loss modulus continue decreasing over the entire temperature range. The plot of G' and G'' versus temperature indicates an increased elastic behaviour of the bitumen at high temperatures, as results from the trend of tgδ at high temperatures.

Fig. 6.27: isochronal curves of storage component of complex modulus G'

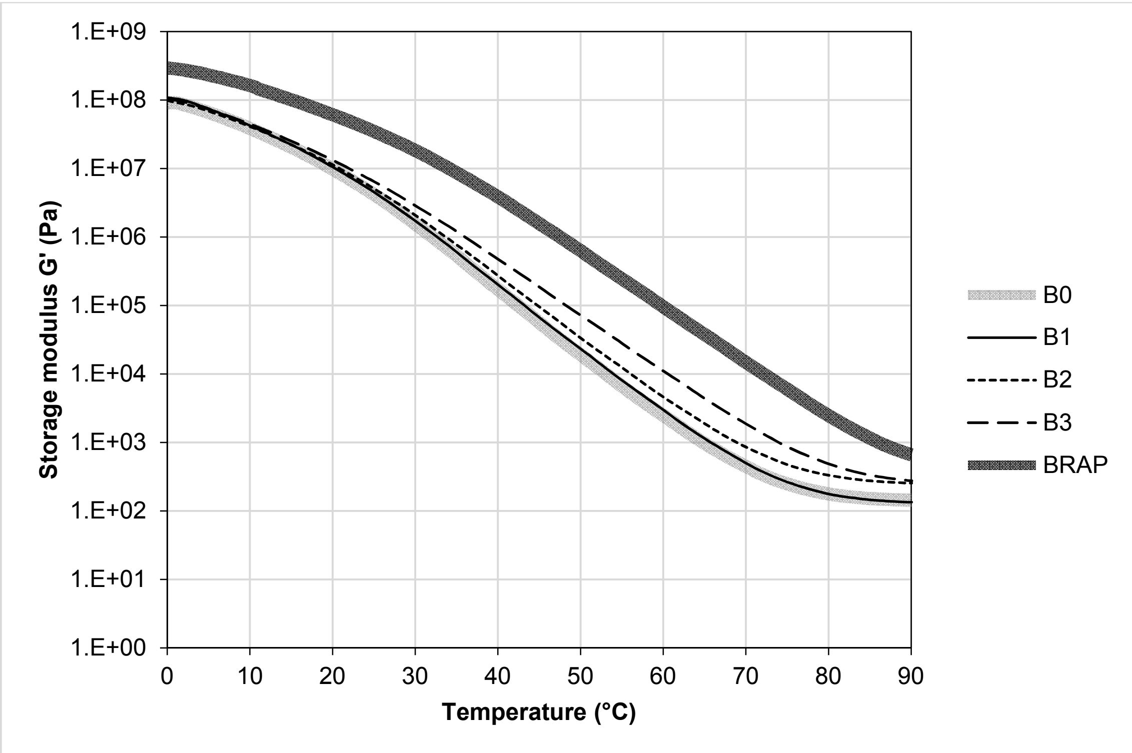
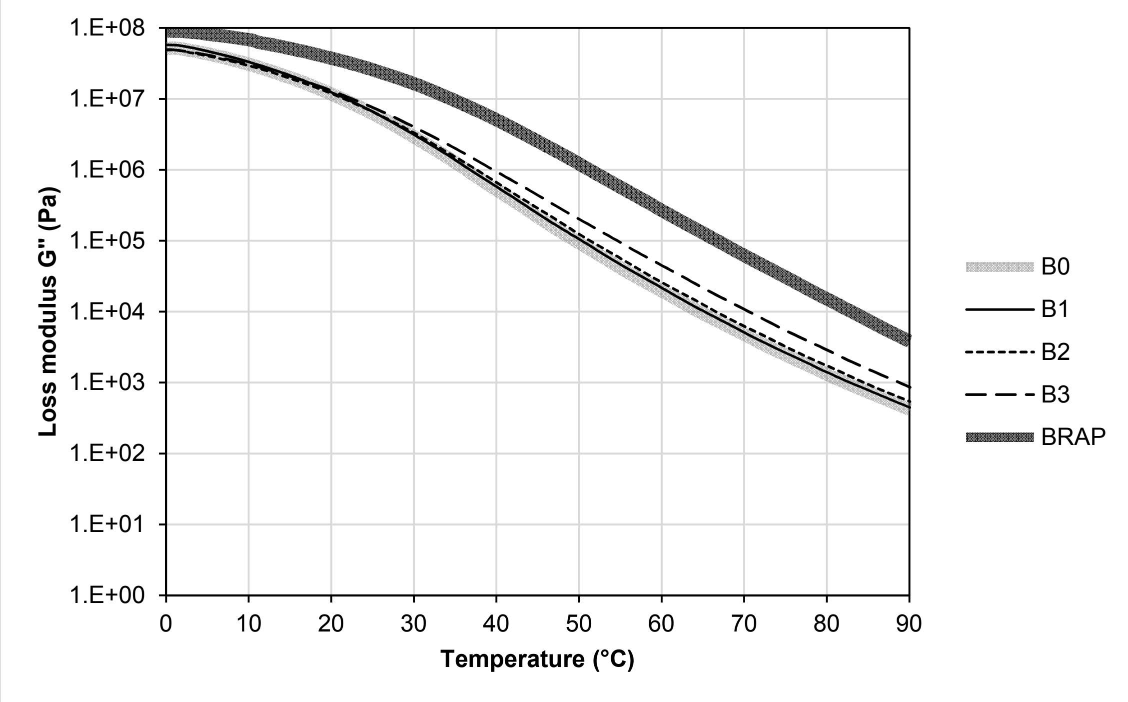


Fig. 6.28: isochronal curves of loss component of complex modulus G''



6.2.2.4 Black diagram

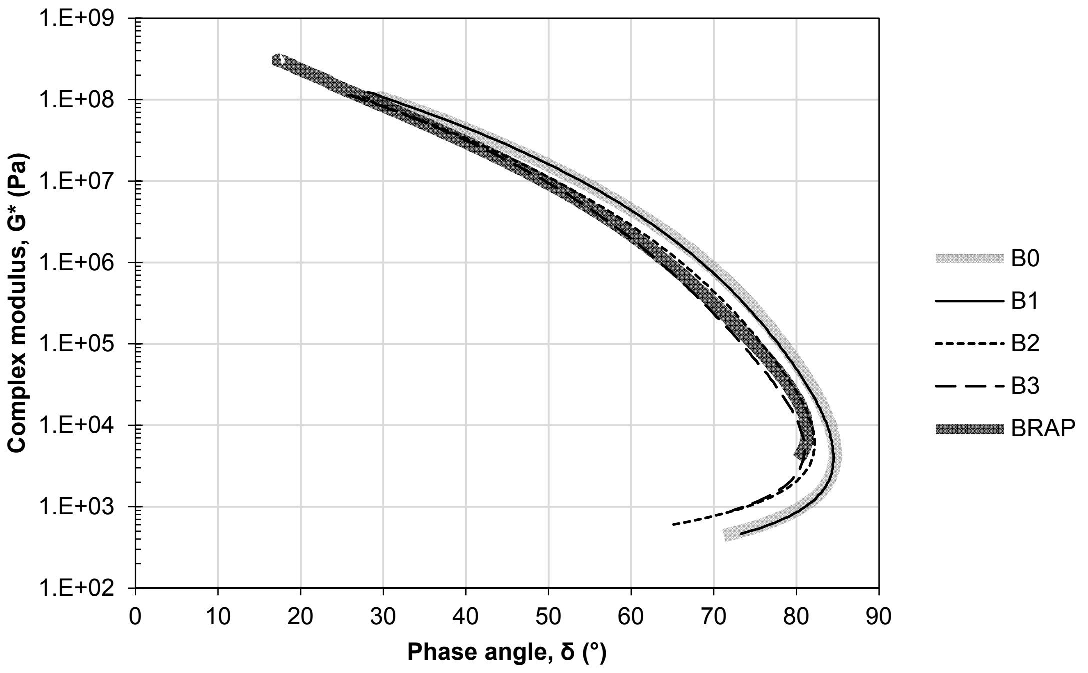
A Black diagram is a graph of the norm of the complex modulus G* versus the phase angle δ [113]. Data can be presented in one plot, without any manipulation of raw data with Time-Temperature Superposition Principle (TTSP).

Results in Fig. 6.29 show a shift of the curves towards lower values of the phase angle with the increase of the RAP binder content into the bituminous blends. This is caused by the dual effects of the increase in complex modulus and the decrease in phase angle. The first indicates a hardening of the bitumen, the second indicates an increase in the elastic response of the mixtures as RAP binder content increases.

The aged bitumen has little influence on the rheology of the final blend when the RAP content in the asphalt mixture is equal or below 10%, as can be noted from B_0 and B_1curves. The effect of RAP binder becomes relevant when the RAP content is equal or above 20%, as results from the shift of B_2 and B_3 curves on the Black space.

As previously observed referring to the plot of tgδ, the turning inward of the Black diagram curves reveals an increased elastic response of the bitumens at high temperatures. This is specific to B_0, B_1, B_2 and B_3. Therefore, this behaviour can be attributed to the rheology of the neat binder.

Fig. 6.29: Black diagram for investigated bitumens

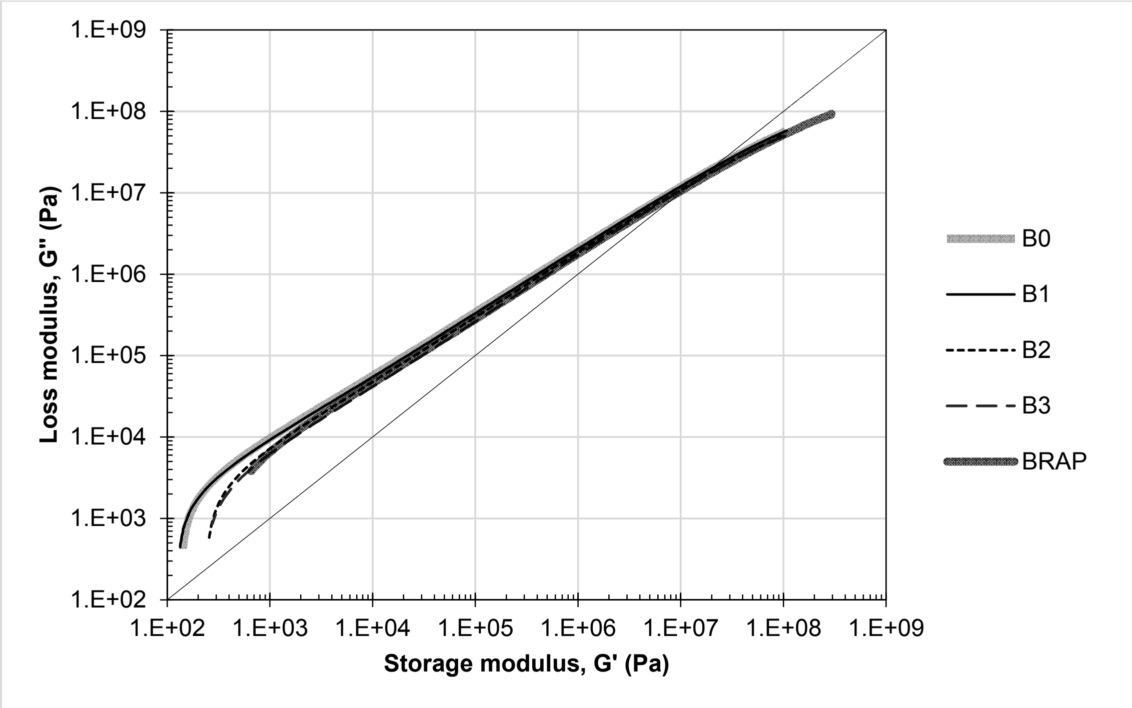


6.2.2.5 Cole-Cole diagram

The Cole-Cole diagram is a plot of the loss modulus G'' as a function of the storage modulus G'. The graph provides a means of representing the viscoelastic balance of the bitumen [87]. As for the Black diagram, the Cole-Cole diagram does not require any information about test temperature or frequency.

When the RAP binder content increases, the curves reported on the Cole-Cole diagram (Fig. 6.30) shift towards the lower right-hand corner of the plot, with a larger increase in storage modulus than loss modulus at a given temperature. Increasing the RAP binder content leads to a more elastic, less viscous and stiffer rheological behaviour of the mixes.

Fig. 6.30: Cole-Cole diagram for investigated bitumens



6.2.3 Summary of results

Findings from laboratory tests indicate that aged binder influences physical and rheological properties of the final blend due to an interaction with the virgin binder.

Results of standard characterization tests show an increase in softening point and viscosity at 160°C and a decrease in penetration grade as the RAP content in the mixtures increases. This indicates a hardening of the composite blend caused by the presence of aged binder.

The dynamic viscosity of bitumens was investigated as a function of RAP content and temperature. Results show that viscosity increases in proportion to the RAP binder content. The RAP binder changes the chemistry of the bituminous blends, resulting in a more viscous and consequently harder composite bitumen. The presence of 10% (by mass of the mix) or less of RAP binder into the mixture does not affect the viscosity of the blend, whereas a RAP binder content of 20% or higher causes a significant increase in viscosity. The trend of viscosity with temperature shows an increased temperature susceptibility as the RAP content increases.

The rheology of bituminous samples was investigated with the DSR analysing the complex modulus and phase angle isochrones, the Black space and the Cole-Cole diagram. Results show a hardening and an increased elastic behaviour of bitumens caused by the presence of RAP binder into the bituminous blends. The effects on the final blend are in proportion to the aged binder content. The aged bitumen has little influence on the rheology final blend when the RAP content in the asphalt mixture is equal or below 10%. The effects of RAP binder are significant when the RAP content is equal or above 20%. In addition, the increase in the cross-over temperature with RAP content indicates a widening of the temperature range where the mechanical response of the bitumen is predominantly elastic.

6.3 Annexes

6.3.1 Fatigue curves (Wöhler) of asphalt mixes

Fig. 6.31: Mix0 fatigue line

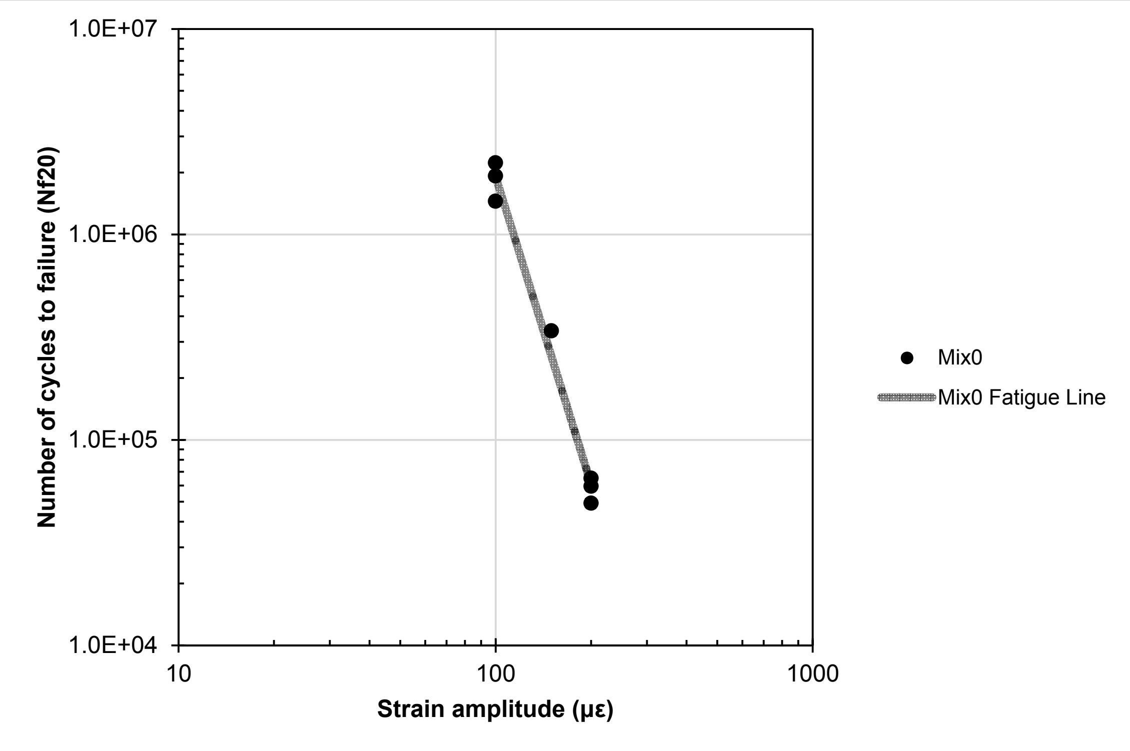


Fig. 6.32: Mix1 10%RAP fatigue line

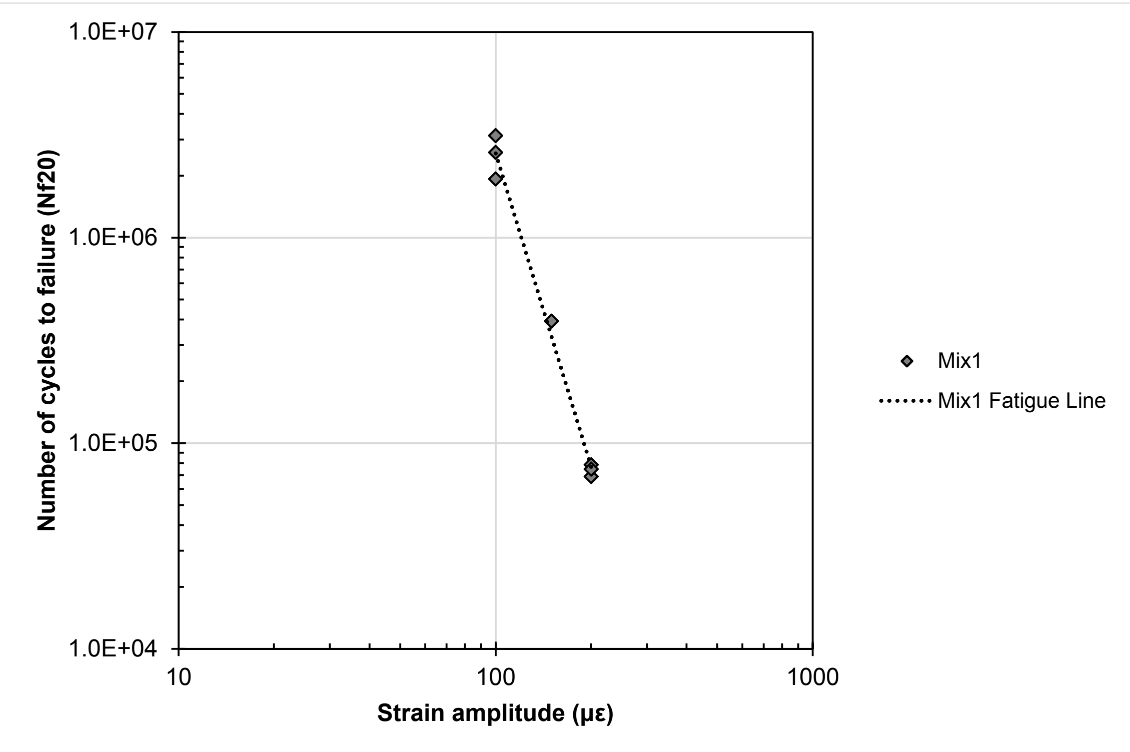


Fig. 6.33: Mix2 20%RAP fatigue line

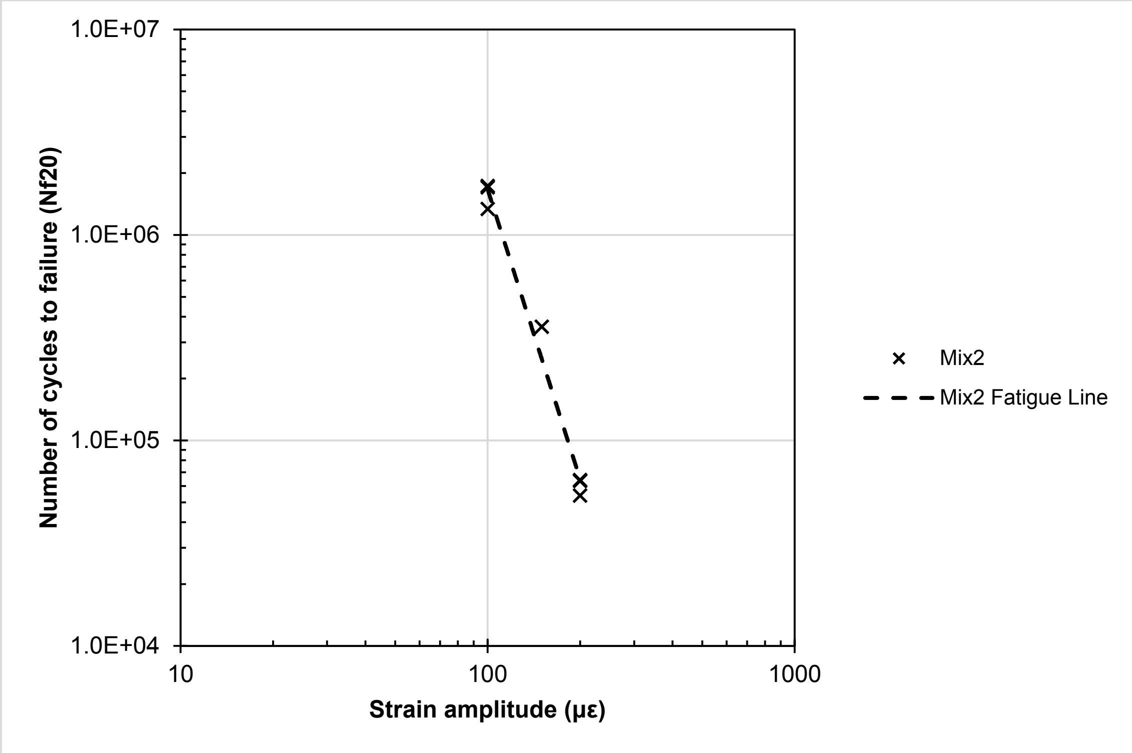
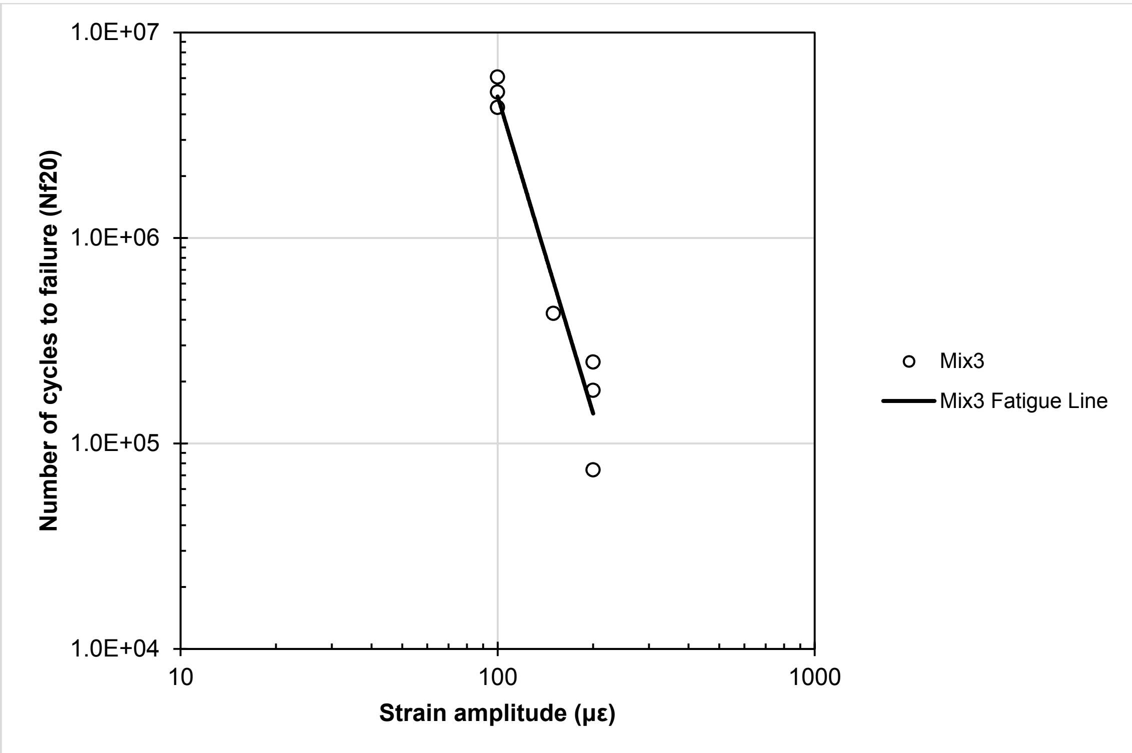


Fig. 6.34: Mix3 30%RAP fatigue line



6.3.2 Energy fatigue curves of asphalt mixes

Fig. 6.35: Mix0 energy fatigue curve

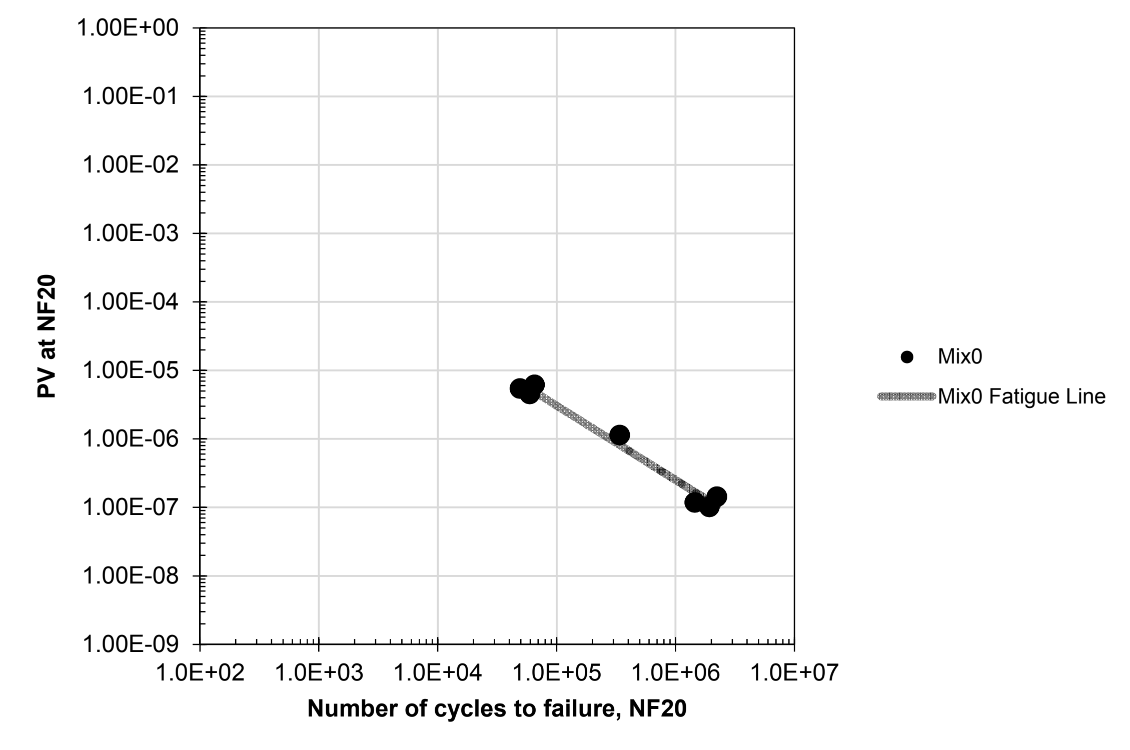


Fig. 6.36: Mix1 10% RAP energy fatigue curve

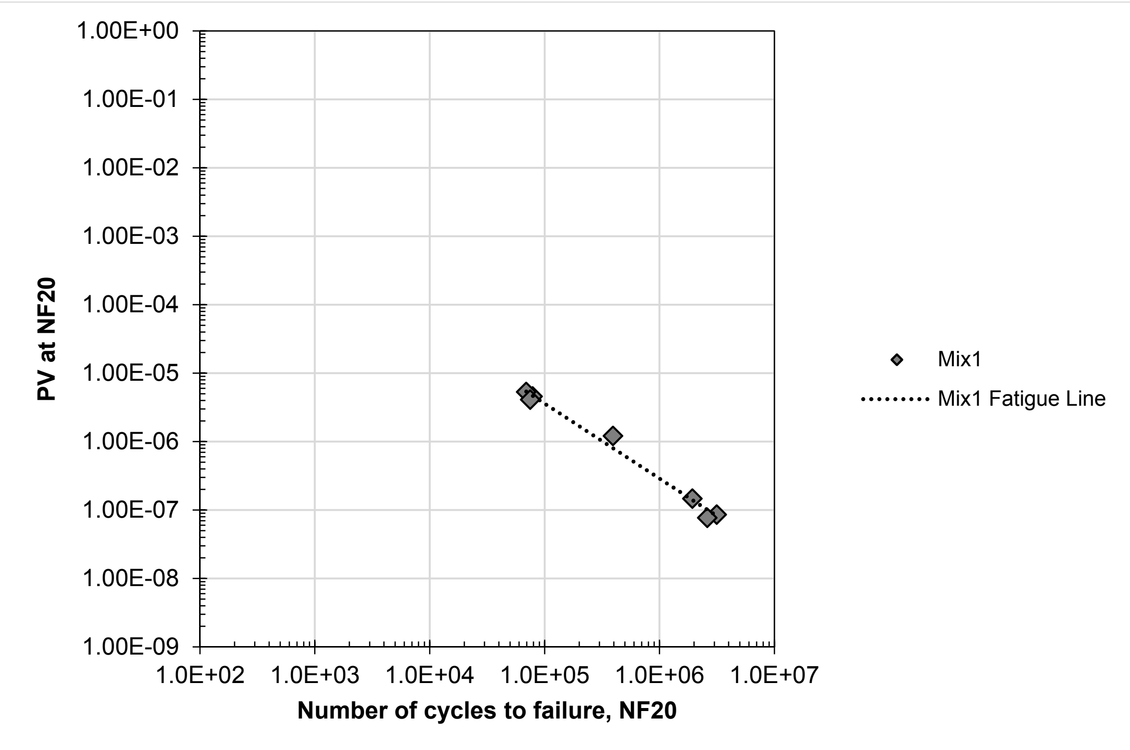


Fig. 6.37: Mix2 20% RAP energy fatigue curve

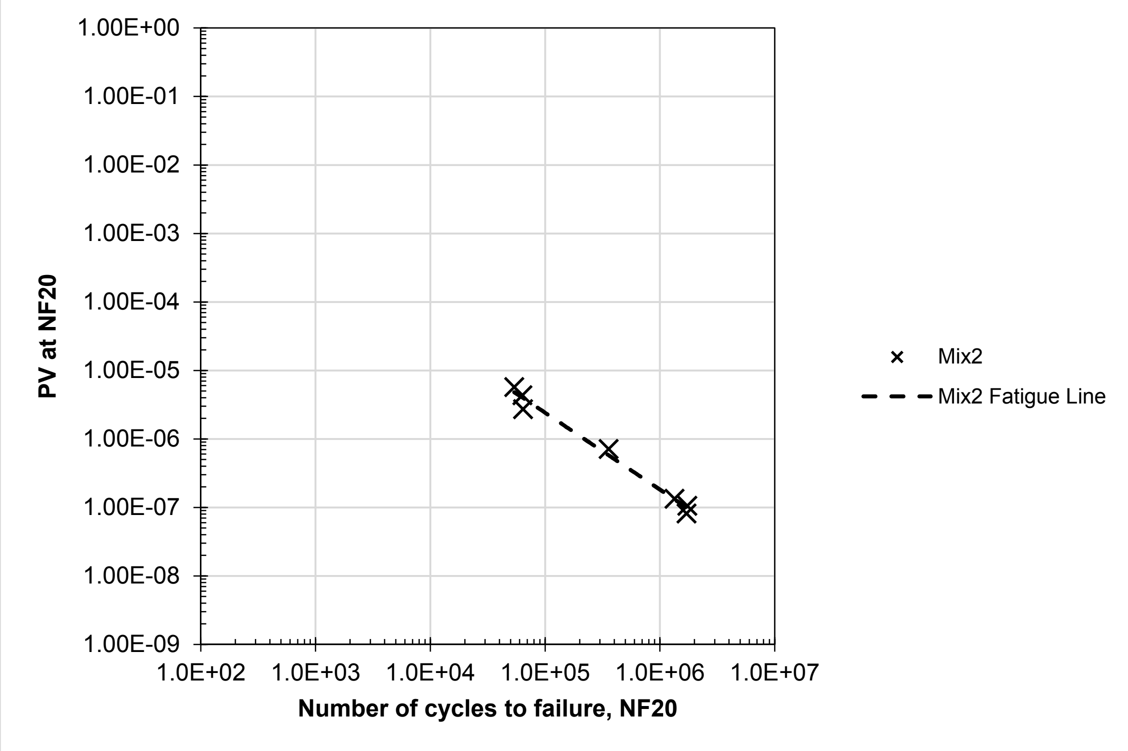
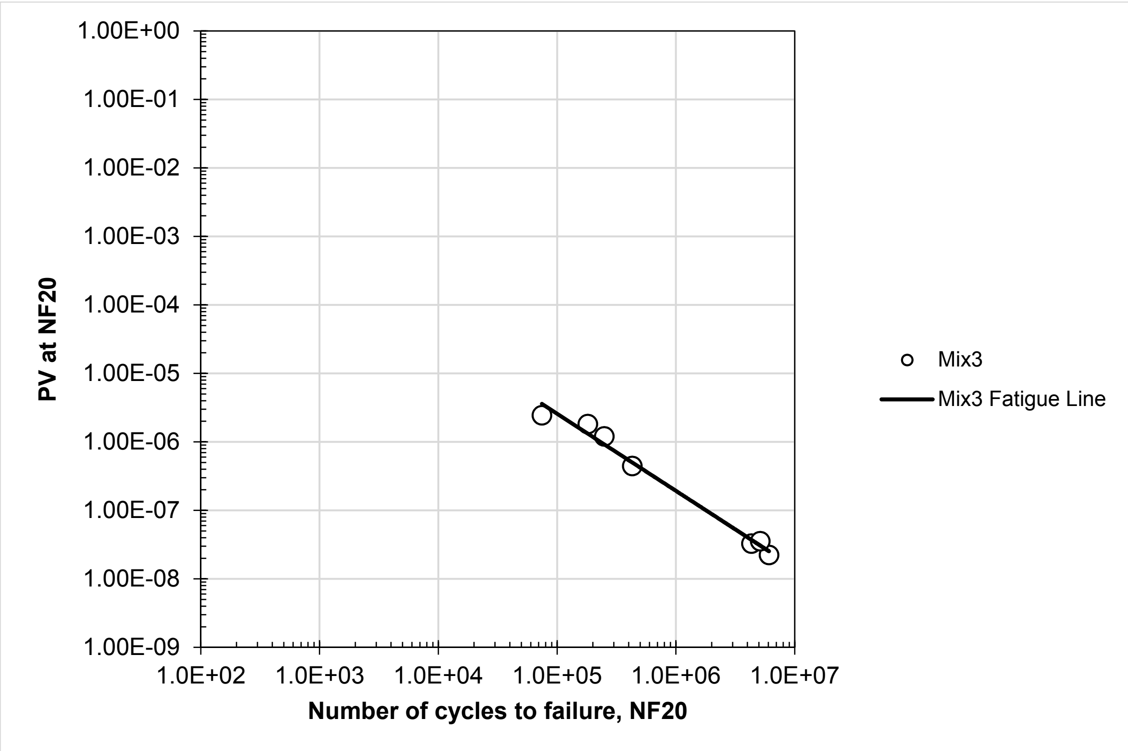


Fig. 6.38: Mix3 30% RAP energy fatigue curve



6.3.3 Calculation of dissipated energy

The energy dissipated by a prismatic asphalt sample subjected to a sinusoidal load with frequency ω_0 and amplitude ε_0 at the generic cycle *i* can be calculated from the integral of the hysteresis loop plotted on a ε-σ chart, as follows:

$$w_i = \int_0^T \sigma(t) \cdot \frac{d\varepsilon}{dt} dt$$

Where T is the duration of the load cycle *i*. Since for strain-controlled fatigue test the initial strain level ε_0 is constant, we can write:

$$\sigma_i(t) = \sigma_i \sin \omega t \ \ [kPa]$$
$$\varepsilon(t) = \varepsilon_0 \sin(\omega t - \varphi_i) \ \ [\mu m/m]$$

Where φ is the phase angle measured at i-cycle. Then, the equation of dissipated energy becomes:

$$w_i = \varepsilon_0 \int_o^T (\sigma_i \sin(\omega t) \cdot \omega \cos(\omega t - \varphi_i))dt$$

And the final expression is:

$$w_i = \pi \cdot \varepsilon_0 \cdot \sigma_i \cdot \sin \varphi_i \ \ [kJ/m^3]$$

The equation above represents the energy dissipated at cycle i with reference to a sinusoidal load applied with constant amplitude ε_0 and frequency ω. Parameters σ_i and φ_i change at each cycle and are constantly measured by the testing equipment to calculate the dissipated energy.

6.3.4 Interpretation of the Ratio of Dissipated Energy Change (RDEC)

RDEC is defined as the ratio between the change in dissipated energy between two cycles “a” and “b” and the energy dissipated at initial cycle “a”, as follows:

$$RDEC_a = \frac{|w_a - w_b|}{w_a \cdot (b - a)}$$

The three assumption presented in 6.1.4.2 are summarized below:

$$w_i = w_i^\eta + w_i^\zeta$$
$$w_i^\eta \approx w_{i+1}^\eta = w^\eta$$
$$w_i^\eta \gg w_i^\zeta$$

Where w^{η} and w^{ζ} are the dissipated energy by viscoelastic damping and fatigue damage, respectively. The expression of RDEC can be written as:

$$RDEC = \frac{w_a^{\eta} + w_a^{\zeta} - w_b^{\eta} - w_b^{\zeta}}{\left(w_a^{\eta} + w_a^{\zeta}\right) \cdot (b - a)}$$

Since the viscoelastic damping is much larger than the fatigue damage, we can write:

$$RDEC = \frac{w_a^{\eta} + w_a^{\zeta} - w_b^{\eta} - w_b^{\zeta}}{w_a^{\eta} \cdot (b - a)}$$

The viscoelastic damping is assumed to be constant for the entire test duration, therefore we have:

$$RDEC = \frac{\Delta w^{\zeta}}{w_a^{\eta} \cdot (b - a)}$$

The RDEC is then a function of the change in dissipated energy caused by the fatigue damage within the material. In the region II of the RDEC-N chart, the RDEC has approximately a constant value and the region is often referred as “plateau stage”. In this region, the entire amount of load energy is turned into a fatigue damage and it has a constant value throughout the test. The value of the RDEC calculated under condition of energy balance is called Plateau Value.

7 Conclusions

7.1 Research summary

This thesis focuses on the effects of RAP on performances and durability of new asphalt mixes produced incorporating RAP with a standard hot-in-plant process. The effects of RAP on asphalt mixes and on bitumen recovered from the same mixes have been investigated through an extensive laboratory testing campaign.

The research is divided in two phases: in the first phase, the effects of RAP on asphalt mixtures are investigated. A specific typology of asphalt mixture is produced with different RAP percentages of 10%, 20% and 30% by mass of aggregates. Fatigue resistance, stiffness and volumetric properties of each mix have been investigated and compared. A great effort was put to produce mixtures with different RAP content but similar grading curve, bitumen content, air voids and density.

The second phase of the research aims to investigate the effects of RAP on asphalt binder as a composite blend of neat and RAP bitumen. Bituminous samples are recovered from asphalt samples tested in phase 1 and then are studied and compared with standard and dynamic tests.

7.2 Use or RAP in pavement industry

7.2.1 RAP in figures

a. In Europe, 70% of the available RAP is recycled in hot, warm and cold asphalt mixture production [8]

b. RAP recycling in new asphalt mixture production is the most valuable use of available RAP

c. Hot in plant recycling is the most common practice for recycling RAP in Europe, with 61% of available RAP being used [8]

d. In Italy, only 50% of RAP is recycled in valuable applications i.e. hot, warm or cold recycling. Only 20% of available RAP is used in hot in plant recycling

7.2.2 Economic and environmental benefits of RAP usage

a. The most significant economic advantage of RAP recycling is the reduction in the use of bitumen, which is the most impacting factor on road costs [1] [9] [10]. While bitumen only constitutes approximately 5-6% by mass of materials used in asphalt, it can contribute to over half of the total cost and this price tends to rise further according to increasing world crude oil prices

b. Recycling of RAP allows to reduce the amount of natural and non-renewable resources needed to produce a new asphalt mixture, since RAP can partially replace virgin aggregates and neat bitumen

c. RAP recycling contributes reducing greenhouse gas (GHG) emissions and global warming, atmospheric pollution (ground level ozone creation and acidification) related to transportation of raw materials and combustion of fossil fuels during manufacturing of asphalt mixtures

d. The need for soil used for landfilling RAP is reduced since RAP is incorporated into new mixes instead of being damped

e. Compared with virgin mixes, RAP recycling leads to increased costs that may lower competitiveness of RAP mixtures on a short-term period

    i. RAP mixes shall perform as good as similar mixes made with only virgin materials. In order to achieve this goal, traditional asphalt plants must be retrofitted with specific equipment i.e. parallel drum or double dryer for heating RAP

    ii. RAP requires specific handling procedures from the moment it’s stored at the plant to the moment is incorporated into a new asphalt mixture. In case of high percentages of RAP incorporated into mixtures (>20%), performances and durability of RAP mixtures should be investigated in laboratory to avoid uncertainties related to RAP homogeneity

f. However, investing money in retrofitting and handling procedures allows to increase the amount of RAP being incorporated into new asphalt mixes. Short-term “savings” on retrofitting and handling may result in poor quality RAP mixes, increased emissions, increased fuel consumptions and reduced usage of a valuable resource as RAP. For this reasons, in many European countries, governments have been encouraging recycling of RAP with an intervention strategy aimed to make RAP mixes at least competitive as virgin mixtures

7.2.3 Methods for incorporating RAP into new hot asphalt mixes

a. 100% of reclaimed asphalt can be recycled [4] with different methods: hot recycling in asphalt plant, hot in-place recycling, cold in-place recycling and full depth reclamation are the most commonly applied techniques [2]

b. The maximum amount of reclaimed asphalt that can be incorporated in a new asphalt mixture is mainly limited by the production technology [34]. Considering hot in plant recycling, most conventional drum plants can accommodate 50% RAP, whereas the percentage of reusable RAP in batch plant ranges from 10% to 30%

c. Limiting factors for increasing RAP content used in batch plants are [38] [5] [2]: RAP moisture content, production rate and use of old equipment. Regarding continuous plants, the limiting factors are: RAP moisture content, blue smoke and production rate. In both cases, the production rate decreases as the amount of RAP being incorporated increases

d. In spite of the typology of asphalt plant, heating/drying of reclaimed asphalt in relation to its moisture content is the most impacting factor on RAP usage in hot-in-plant recycling

e. There are several technologies that enable re-use of recycled asphalt for the production of new mix including [8]: parallel drum dryers are able to increase the use of RAP into new hot mixes up to 100% (counter-flow dryers) or up to 60% (parallel-flow dryers). In addition to parallel drums, which are mainly used in batch plants, the double dryers are becoming a widespread technology, both in batch and continuous plants, able to recycle up to 90% of RAP.

7.3 Laboratory procedures

7.3.1 Sampling, laboratory characterization and handling of raw materials and RAP

a. Accurate sampling of RAP and virgin aggregates from plant’s stockpiles is essential (but not sufficient) to achieve consistent laboratory test results. Sampling procedures shall be carried out according to EN 932-1 and 12697-27 respectively for aggregates and asphalt mixes

b. Standard characterization tests shall be conducted on aggregate and RAP to determine their properties: particle size distribution (EN 933-1 and 12697-2), density (EN 1097-6) and RAP binder content (EN 12697-1 hot extraction with solvent)

c. RAP characterization contributes to limiting reclaimed asphalt variability. Properties of reclaimed asphalt should be representative of the stockpiled material. A sufficient number of RAP samples should be tested in order to obtain consistent results. Indications on the minimum amount of RAP being tested can be found in the EN 13108-8

d. When a high quantity of RAP is incorporated into a new asphalt mix (high quantity means >20% RAP by mass of aggregate), RAP bitumen should be characterized in terms of penetration at 25°C (EN 1426), softening point (EN 1427) and dynamic viscosity with rotational viscometer (EN 13702). Bitumen is recovered from RAP samples using a centrifuge extractor according to EN 12697-1 annex B.1.5 for cold extraction methods using dichloromethane DCM (CH_2Cl_2) organic solvent. The solution of DCM and bitumen is then distilled with rotary evaporator (EN 12697-3). Studies show that the process of extraction and recovery influences the properties of bitumen [10]

e. Materials are heated or dried at a specific temperature before being mixed together. Aggregates are heated at 180°C for 2 hours, RAP is dried at 110°C for 90 minutes and bitumen is heated for 2 hours at 165°C. RAP drying contributes to blending of old and neat bitumen [9] [10] [14]

f. The bulk density and bitumen content of prismatic beams measured from lab tests indicate homogeneity of the samples produced in laboratory. Therefore, results of fatigue testing are expected to be consistent and comparable, since they're obtained from homogeneous samples. Measured variations comply with repeatability specifications reported in the EN 12697-6 and EN 12697-1

7.3.2 Mix design of RAP mixes

a. Four mixtures have been investigated in this research, each with 5% air voids (by volume of the mix), 5% bitumen content (by mass of aggregates) and similar grading curve. Mixtures mainly differ in RAP percentages of 0% (control), 10%, 20% and 30% by mass of aggregates

b. Standard mix design procedures shall be adjusted for RAP mixes in order to take RAP variability and the presence of aged bitumen into account

c. RAP is used as "substitute" of virgin materials. Mixtures are produced with same grading, volumetric properties and bitumen content using less virgin material

d. The amount of bitumen being added in new mixes shall be calculated considering the contribute of the old bitumen which is present in the RAP aggregate. Under the assumption of total blending between virgin and old bitumen and for a specific binder content, the higher the RAP content, the lower the virgin bitumen added to the mix

e. Because of the assumption of total blending in the mix formulation, the mass of RAP that is added into the mixture shall not include the mass of the aged bitumen

f. Aggregate proportioning for hot asphalt mixes incorporating RAP is made using the grading curve of the extracted RAP, since the RAP bitumen is expected to (partially) melt and blend with the virgin bitumen

g. Attention to operative details is necessary to ensure good quality of produced mixtures, which means concurrence of target and measured mixtures properties. Laboratory operations require care, precision and accuracy to obtain desired mixtures. Recommendations are included in the research mix design

7.4 Results of laboratory testing on RAP mixes

7.4.1 Durability

7.4.1.1 Flexural fatigue testing in controlled-strain mode with four-point bending device: Wöhler approach

a. Mix0 (reference mix with no RAP), Mix1 (10% RAP) and Mix2 (20% RAP) exhibit similar fatigue performances both at 100 and 200 με

b. The mix made with 30% RAP (Mix3) shows the longest fatigue life (highest number of cycles to failure at each strain level) relative to other mixes

c. Adding up to 20% of RAP seems to have no negative effects on asphalt mixtures durability. Therefore, 20% of RAP can be considered as a threshold value for the investigated mixes

d. The RAP content appears to affect fatigue life more than other material properties as e.g. air void content and density, possibly because of the interaction between neat and RAP bitumen. The effects of bitumen content on the measured number of cycles to failure cannot be determined since all mixes have very similar binder content, accordingly to the mix design

7.4.1.2 Flexural fatigue testing in controlled-strain mode with four-point bending device: energy approach

e. The Plateau Value (PV) is an indicator of fatigue damage occurring on asphalt mixtures subjected to cyclic loading. The higher the PV, the higher the fatigue damage, therefore the shorter the fatigue life of the material

f. Mix0, Mix1 and Mix2 exhibit a similar fatigue resistance. Mix 3 shows lower values of PV and hence a better response to fatigue testing

g. Mix3, the mixture with 30% RAP, shows longest fatigue life, among the four mixes. In fact, Mix3's PV is significantly lower than the PV of other mixes, indicating a lower fatigue damage occurring in Mix3 during cyclic loading

h. The value of 20% of RAP incorporated in asphalt mixes appears to be a threshold below which the effects of RAP content on fatigue life of mixes are negligible or of low extent

7.4.2 Interaction neat-RAP bitumen

a. Both the Wöhler and energy approaches suggest that the interaction between virgin and RAP bitumen plays a significant role in the fatigue performance of asphalt mixes

b. The RAP bitumen causes a hardening of the composite blend made of old and neat bitumen that is in proportion to the RAP content. Standard characterization tests i.e. penetration at 25°C, softening point and viscosity show that the higher the RAP content, the greater the hardening

c. Viscosity test results indicate that viscosity of bituminous blends increases in proportion to the RAP content. The presence of 10% (by mass of the mix) or less of RAP binder into the mixture does not affect the viscosity of the blend, whereas a RAP content of 20% or higher causes a significant increase in viscosity of bitumen

d. The rheological analysis of bituminous blends with DSR shows a hardening and an increased elastic response of the investigated bitumen caused by the presence of

RAP bitumen into the blends. The effects on the final blend are in proportion to the aged binder content

e. The complex modulus and phase angle isochrones, the Black space and the Cole-Cole diagram reveal hardening and increased elastic behaviour of the investigated bituminous blends. The effects are caused by and in proportion to the presence of RAP binder. Results does not indicate any possible brittleness issue at low temperatures with regards to the blend with 30% RAP content

f. The aged bitumen has little influence on the final blend rheology when the RAP content in the asphalt mixture is up to 10%. The effects of RAP binder are significant when the RAP content is equal or above 20%

7.5 Practical implications

a. High quality RAP mixes start with accurate RAP handling at the production plant. In order to limit RAP variability, asphalt production plants should be able to ensure a correct RAP storage and stockpiles sorting on the basis of the source of the milled material. Cooperation with testing laboratories for routinely investigating RAP properties i.e. bitumen type and content, density and particle size distribution should be encouraged as a key tool for limiting RAP variability

b. The investigated hot asphalt mix can be produced in plant adding up to 10% RAP in the new mixes with no negative effects on durability of mixtures and with no relevant modifications to bitumen rheology. If the RAP percentage is increased up to 20%, the blend made of old and neat binder should be characterized in terms of penetration at 25°C, softening point and dynamic viscosity in order to evaluate the effects of the interaction between virgin and RAP bitumen. Consistency of RAP properties shall always be ensured with a scheduled characterization of RAP stockpiles

c. The use of RAP into asphalt mixtures causes an increase in the viscosity of asphalt binder in proportion to the percentage of RAP incorporated into the mixture. This may lead to the need of lowering the viscosity of the bitumen to ensure performances and workability of RAP mixes. An increase in production and working temperatures can reach the goal causing a drop in viscosity to required values, but might also cause the composite blend to harden, with an undesirable stiffening effect on the bitumen and negative consequences on performances of asphalt mixtures (brittleness). Therefore, the use of a softer and hence less viscous neat binder is recommended, if needed, when the RAP percentage incorporated in the mix is equal or above 20% (by mass of the mix). Viscosity blending charts can be used to determine the penetration grade of the neat bitumen to reach the desired viscosity

d. The effects of RAP binder on the bituminous blend depend on the percentage of RAP incorporated in the mixture. Results from laboratory tests show that RAP can be incorporated into the investigated mixture at percentages up to 10% with no significant effects on properties of bitumen. In this case, RAP can be added without the need to perform any additional laboratory test on recovered binder. As the RAP content is equal or above 20%, it’s highly recommended to perform laboratory investigations on the recovered binder following the procedure presented in this study

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