Thesis 417

Reclaimed Asphalt Pavement

- *A Mechanical Performance Study Across Temperatures*

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Abstract:
Reclaimed asphalt pavement (RAP) is one of the most widely recycled materials in the world and its use in asphalt mixtures has increased significantly due to environmental and economic benefits. However, incorporation of RAP may alter mechanical performance of pavement, particularly its resistance to cracking and rutting. This study aims to investigate the influence of RAP content on mechanical behavior of asphalt mixtures across a range of temperatures. Laboratory testing was conducted on mix designs with varying RAP content in order to evaluate susceptibility to cracking and rutting, respectively. Cracking resistance was assessed using Cracking Tolerance Index (CTI) at 5 and 35°C, whilst rutting resistance was evaluated using Rutting Tolerance Index (RTI) at 35 and 65°C. Results indicate that an increase of RAP content generally increases the stiffness of the mix. While higher stiffness improves resistance to permanent deformation at elevated temperatures, susceptibility to cracking may be compromised, particularly at lower temperatures. These findings highlight the importance of balancing RAP content and mechanical performance when designing asphalt mixtures for different climactic conditions. This study provides insight into optimization of mix design to ensure both durability and sustainability in pavement construction.

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Preface

I would like to thank my supervisor, Pajtim Sulejmani, for guiding me through this thesis, offering his expertise along the way.

I would also like to express my gratitude to Craig Long at Pavement Engineering Inc. for allowing me this opportunity, as well as for the continuous support and guidance provided throughout all stages of this project. Further, I would like to thank Nathan Trease, Anthony Long and Joe Sturges for their hard work and friendly attitude in the laboratory.

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Summary

Reclaimed asphalt pavement (RAP) is one of the world's most widely recycled materials and has become increasingly popular due to its potential to reduce environmental impact as well as cost. Incorporation of RAP allows for substitution of both virgin aggregate and binder, consequently resulting in more sustainable pavement construction. However, due to factors such as aged binder and fine aggregate gradation, the mechanical properties of the final asphalt mixture may be altered. The aim of this study is to investigate how varying amounts of RAP included in a mix design affects the mechanical performance of the final pavement, with regards to susceptibility to cracking and rutting. This is done at a range of temperatures in order to simulate the climactic conditions of northern California as well as those of southern Sweden. Laboratory tests were conducted on asphalt mix designs with 0, 25, 40 and 60 % RAP. Cracking Tolerance Index (CTI) was used as an indicator of resistance to cracking, whilst Rutting Tolerance Index (RTI) was used to assess susceptibility to rutting. CTI tests were performed at 5 and 35°C, while RTI was performed at 35 and 65°C.

The results indicate that increasing RAP content generally increases the stiffness of the mix. While a stiffer mix may exhibit greater resistance to rutting and permanent deformation at elevated temperatures, resistance to cracking may be compromised, particularly at low temperatures. This is believed to be caused by the fact that inclusion of RAP affects the binder grade, binder content and aggregate gradation of the final mix. Further, elevated temperatures are believed to reduce the viscosity of the binder, resulting in a softer mix, exhibiting increased risk of permanent deformation. The ideal amount of RAP to be included in a mix design used in northern California is estimated to be somewhere between 40 and 60 %. The equivalent for the climate in southern Sweden cannot be estimated due to a lack of data gathered in the laboratory phase.

Sammanfattning

Returasfalt (RAP) är ett av världens mest återvunna material vars popularitet ökat på grund av dess förmåga att reducera miljöpåverkan såväl som omkostnader associerade med anläggning. Användningen av RAP möjliggör ersättning av både jungfruligt aggregat och bitumen, vilket i sin tur resulterar i större hållbarhet inom anläggningssektorn. Dock kan vissa mekaniska egenskaper hos asfalt innehållande återvunna produkter påverkas till följd av faktorer så som åldrat bindemedel samt förfinad kornstorleksfördelning. Målet med studien är att undersöka hur varierande mängder RAP i en asfaltsmix påverkar slutproduktens benägenhet till sprick- och spårbildning. Detta utförs vid varierande temperaturer i syfte att simulera rådande klimatförhållanden i norra Kalifornien respektive södra Sverige. Laboratorietester har genomförts på mixer innehållande 0, 25, 40 samt 60 % RAP. Tester utförs med avseende på två amerikanska standarder. Dessa är Cracking Tolerance Index (CTI) vilken används som en indikator av benägenhet till sprickbildning och Rutting Tolerance Index (RTI) som används för att bedöma spårbildningsbenägenhet. CTI har genomförts på samtliga mixer vid 5 respektive 35°C, medan RTI genomförts vid 35 respektive 65°C.

Resultaten visar att ökat RAP innehåll generellt sätt resulterar i en styvare asfalt. En ökad styvhet kan innebära minskad deformations- samt spårbildningsbenägenhet, dock till priset av ökad sprickbenägenhet. Detta är särskilt framträdande vid låga temperaturer. Detta fenomen tros bero på att egenskaper så som bindemedelskvalitet, bindelmedelshalt samt korstorleksfördelning påverkas av inkludering av RAP i mixen. Vidare tros förhöjda temperaturer minska viskositeten hos bindemedlet vilket resulterar i en mjukare mix och i sin tur i högre benägenhet för deformation. Den ideala mängden RAP som bör inkluderas i en mix i norra Kalifornien uppskattas vara mellan 40 och 60 %. Det motsvarande värdet för södra Sverige kan inte uppskattas på grund av brist på tillräcklig datainsamling i laboratoriefasen.

1 Introduction

1.1 Background

The construction industry is one of the main contributors to climate change, waste generation and natural resource depletion (Behnood & Afshin, 2025). Within this subgroup, transportation infrastructure is described as a major contributor to both natural resource consumption and carbon emissions due to its high demand for large material volumes as well as its usage of processes that require high energy consumption. As cities expand and society advances, the demand for extensive paved road networks increases, resulting in both large-scale road construction as well as large volumes of road maintenance and repair. Consequently, the environmental footprint associated with transportation infrastructure is amplified. This kind of societal development has increased the popularity of concepts such as circular economy. The model focuses on reducing waste and prolonging the lifespan of materials through reusing, repairing, refurbishing and recycling. This way of handling materials at the end of their lifespan can reduce dependency of virgin materials which can be beneficial both environmentally and economically (European Parliament, 2023).

Asphalt is a commonly used construction material that, in its simplest form, consists of aggregate and bitumen. Asphalt pavements serve a vital role in the transportation infrastructure across the globe and is used to cover more than 94 % of roads in the United States (Shacat, et al., 2024). Due to the wide use of this paving material, efforts have been made to make the industry more sustainable. One way of achieving this is by recycling and reusing asphalt at the end of its lifetime to pave new roads, so called reclaimed asphalt pavement (RAP). The most common method for reusing asphalt pavement is conventional recycled hot mix and involves using RAP as a partial substitute to both virgin aggregate as well as binder (Federal Highway Administration, 1997). Hot in-place recycling is described as an alternative method that is used for correction of damaged pavement surfaces. This is performed by softening the surface through exposure to heat, removing distressed pavement and mixing it with a rejuvenator, and possibly virgin materials, and finally repaving the surface with the rejuvenated mix.

According to the Federal Highway Administration (FHWA), records of pavement recycling in the United States can be found as early as 1915 (Federal Highway Administration, 1997). However, it is stated that more serious efforts of recycling pavement were conducted later, starting 1974. With financial help from FHWA, more than 40 states performed projects involving RAP between 1976 and 1982. As of today, using RAP as substitute for virgin aggregate and binder is routinely accepted in almost every state in the country. The efforts to recycle asphalt pavement have led to a reduction of both greenhouse gas (GHG) emissions as well as landfill volumes. Between 2009 and 2019, the net reduction of GHG emissions in the United States was estimated to 21,2 million tonnes CO_2 equivalents, which equates to removing approximately 460,000 passenger vehicles from the road (National Asphalt Pavement Association, 2021). In 2019 alone, the wide usage of RAP could save landfill volumes of over 45 million cubic meters. This is due to as much as 95 % of reclaimed asphalt pavement in the United States being reused in other projects, as of 2021 (National Asphalt Pavement Association, n.d.). It is stated that the majority of this material was incorporated into new asphalt mixtures, whereas smaller quantities were used in base and subbase layers, or other construction applications.

In Sweden, recycling of aged asphalt pavement has been prominent for several decades, with approximately 90 % of reclaimed asphalt being reused (Norrman, 2004). As of 2025, the average percentage of RAP used in reclaimed pavement projects in Sweden is 30 %, and the corresponding value in the United States amounts to 22,2 %, as of 2022 (NCC, 2025; Williams, et al., 2024). While the percentage of paving materials avoiding landfill is large, there is still room for improvement regarding the amount of RAP that is incorporated into new asphalt mixtures. Increasing the average RAP content in recycled asphalt mixtures would likely allow for more efficient reaping of environmental and economic benefits. It is already possible to manufacture asphalt mixes containing 100 % RAP, and this thesis aims to investigate how much RAP can be incorporated without substantially compromising mechanical behavior of an asphalt mix in different parts of the world (NCC, 2025).

1.2 Thesis scope and objective

The following master's thesis aims to investigate how incorporating RAP as a partial substitute to virgin aggregate and binder may affect an asphalt mix's mechanical performance when exposed to varying temperatures, simulating different climates. Tests will include evaluation of Cracking Tolerance Index (CTI) as well as Rutting Tolerance Index (RTI). Gathered data will be analyzed with the aim to determine an optimal RAP range, balancing cracking susceptibility and resistance to rutting for asphalt use in different climate conditions.

1.3 Research questions

The master's thesis aims to answer the following research questions:

1. How is Cracking Tolerance Index (CTI) of an asphalt mix affected by variation of RAP content at low and high temperatures?
2. How is Rutting Tolerance Index (RTI) of an asphalt mix affected by variation of RAP content at low and high temperatures?
3. Is there an optimal range of RAP content, balancing CTI and RTI, that can be used in colder and warmer climates respectively?

1.4 Methodology

The following master's thesis can be divided into four sections. The first of which includes a literature review with the intention of gathering necessary background information within the subjects of asphalt and reclaimed asphalt pavement. This allows for deeper understanding of the material, recycling methods and properties connected to pavement which creates a foundation for further analysis.

The second section explains the standards that are used for testing the specimens, explaining for instance how CTI and RTI are calculated.

The third section includes the laboratory phase, describing the methods used to make and evaluate the properties of a number of specimens with regards to CTI as well as RTI. Specimens were created using four different mix designs to allow for a possible pattern to be recognized between the amount of RAP present in an asphalt mixture and its mechanical properties. The methodology of the laboratory phase is further explained in chapter 0.

The fourth and final section of the report includes an analysis of results. Using the data gathered, the analysis aims to determine an ideal amount of RAP to be included in an asphalt mixture in warm and cold climates respectively. Conclusions will be drawn with regard to the balance of CTI and RTI. Results of testing will also be compared to findings from the literature review to investigate to which degree they align.

1.5 Delimitations

The following master's thesis aims to evaluate cracking and rutting resistance in HMA with regards to Cracking Tolerance Index (ASTM D8225) and Rutting Tolerance Index (ASTM D8360) (ASTM International, 2019; ASTM International, 2022). No other methods of evaluation will be used to quantify cracking and rutting susceptibility in this report. Further, no other properties than cracking and rutting resistance are investigated. Temperatures are chosen to simulate climactic conditions representative of southern Sweden, Skåne, as well as Redding, located in northern California, United States. Temperatures include 5 and 35°C for CTI testing and 35 as well as 65°C for RTI. The study is limited to RAP and does not include any other recycled materials such as recycled asphalt shingles (RAS). Mix designs include RAP contents of 0, 25, 40 and 60 % and are all made with PG64-10 binder. Long-term field performance of mixtures is not included in the report.

2 Literature review

2.1 Main components of asphalt

The main components of asphalt are binder and aggregate. The binder is typically derived from crude oil and is called bitumen. Aggregate refers to granulated materials of varying particle size, such as crushed stone, gravel and sand. In addition to this, filler, additives and rejuvenators can also be included to ensure that desired requirements are met.

2.1.1 Bitumen

Bitumen is a petroleum-based binder predominantly used as the binding agent in asphalt. Although it occurs naturally in geological deposits, it's primarily produced through the refining of crude oil (Pipintakos, et al., 2020). In this process, crude oil is piped through hot furnaces, and the resulting vapors and liquids are discharged into distillation units (U.S. Energy Information Administration, n.d.). These products are then separated into petroleum components, so called fractions, based on boiling points where its described that lighter fractions rise to the top and heavier ones end up at the bottom of the distillation tower. Further, the heaviest crude oil components do not vaporize during the refining process, resulting in bitumen, a dense, viscous residue.

Bitumen generally consists of approximately 85 % carbon, 10 % hydrogen as well as smaller fractions of heteroatoms, oxygen and sulfur (Pipintakos, et al., 2020). It is explained that traces of metals may also be found in the substance. However, the structural and chemical composition of bitumen is complex and therefore efforts have been made to simplify the properties of the substance. SARA fractionation divides bitumen into Saturates, Aromatics, Resins and Asphaltenes and aims to provide a simplified model of the structure of bitumen (Pipintakos, et al., 2020). Saturates, aromatics and resins are collectively referred to as maltenes, and it is the combination of the four components that determines the properties of the bitumen. Saturates are considered to contribute to the workability of the bitumen and aromatics provides flexibility and ductility to the substance. Further, asphaltene is the component that primarily correlates to the substance's viscous behavior. Resins are believed to act as a stabilizing agent for asphaltenes, keeping them dispersed throughout the bitumen. They also enhance the binding strength of the bitumen by providing adhesive and cohesive properties, according to Pipintakos et al.

The material's viscous behavior enables it to effectively coat and bond aggregates, providing flexibility, impermeability and strength to the asphalt mixture (Agardh & Parhamifar, 2014). A flexible asphalt mixture allows the pavement to withstand traffic loads without cracking, although it may increase the risk of plastic deformation. Less flexibility reduces the risk of deformation, however the risk of cracking may increase. The substance's impermeable nature protects underlying road layers from water-related damage. The consistency of bitumen is highly temperature dependent and the material becomes a viscous liquid at higher temperatures (Agardh & Parhamifar, 2014). This phenomenon is utilized during the asphalt paving process.

2.1.1.1 Binder grade

There are different methods and systems to characterize bituminous binder according to its quality. The Performance Graded (PG) system is widely used in the United States and is a method for measuring the performance of asphalt binders (Pavement Interactive, n.d.). It is explained that the PG system is based on the theory that the properties of a binder should be chosen with respect to the conditions under which it will be used. These conditions include climate conditions as well as how the bitumen may age. Its precursors, penetration and viscosity grading for instance, are methods that also gave insight into material properties but with the limitation of them being performed at set temperatures, according to Pavement Interactive. In order to classify binders, the PG system uses a set of tests similar to, but not exclusively, penetration and viscosity grading. However, the newer method ensures that the binder passes these tests at specific temperatures that match the conditions that it will be used in. The grading of a binder is then done by the usage of two numbers. The first of which is the average seven-day maximum temperature of the pavement and the second is the minimum design temperature of the pavement that is likely to occur. Both measurements are written in degrees Celsius (Pavement Interactive, n.d.). As for Sweden, a common way to define the quality of bitumen is through penetration grading (EN 1426) (European Committee for Standardization, 2024). This is done by letting a weighted needle penetrate a material sample for a set amount of time, before noting the total penetration in tenths of millimeters. The quality of the bitumen is then given by a set interval in which the total penetration is included, for instance 70/100 mm/10. There are other methods of evaluating the properties of bitumen that complement penetration grading, to further quantify the behavior of the binder (Agardh & Parhamifar, 2014). These include, but are not exclusive to, testing of viscosity and softening point.

The grade of the bitumen directly correlates to the flexibility of an asphalt mixture. A low PG insinuates a less viscous binder, resulting in reduced stiffness and therefore greater resistance to cracking (Husain, 2021). This is particularly effective at lower temperatures where low temperature cracking might be problematic. A stiffer binder on the other hand can be beneficial at higher temperatures since this counteracts deformation of the pavement (Agardh & Parhamifar, 2014).

Since the binder in RAP is hardened by processes such as oxidation, the addition of aged binder may affect the binder grade of the mixture depending on the amount of RAP added. Current guidelines assume that significant blending occurs between reclaimed and virgin materials and suggests the following (McDaniel, 2009). If the amount of added RAP falls below 15 %, it can be assumed that the effective binder grade of the asphalt does not change. Between 16-25 %, it is assumed that a decrease in virgin binder grade may occur. If RAP constitutes more than 25 % of the mixture, it is recommended to test the reclaimed binder to then determine the appropriate grade of the virgin binder. However, it is not guaranteed that the assumption of significant blending is always correct. If it is assumed that blending occurs but it doesn't, McDaniel suggests that the grade of the binder may be softer than desired. This results in increased risk of rutting and reduced risk of cracking. On the other hand, if blending is assumed to not take place but it does, it is stated that the opposite may occur instead.

2.1.1.2 Aging of bitumen

The aging process of bitumen can be divided into two stages, short-term aging and long-term aging (Tauste, et al., 2018). Short-term aging takes place during the process of mixing, storing, spreading and compacting the bitumen and occurs at high temperatures. Long-term aging on the other hand refers to the aging the bitumen undergoes during its service life and is caused by oxidation and effects of UV radiation. According to Tauste et al., the aging of bitumen consists of three main phenomena: loss of volatile components, oxidation of the material and steric hardening.

Loss of volatile components occur when saturates and aromatics evaporate. This process is highly temperature dependent and the phenomenon is of greatest importance when the temperature of the bitumen is high, for example during mixing (Tauste, et al., 2018). It is stated that an increase in temperature by 10°C can result in double the amount of evaporated volatile components. Steric hardening affects the binder's rheological properties through reorganization of molecules within the bitumen (Santagata, et al., 2012). This process is explained to be heat reversible and does not alter the chemical composition of the material. Steric hardening is most efficient at lower temperatures and can be reversed by heating the binder. The most important aging mechanism is oxidation, which is an irreversible, diffusion-driven mechanism (Tauste, et al., 2018). It can be considered diffusion-driven due to the fact that the amount of bitumen that can be aged directly correlates to how much bitumen is exposed to oxygen. This accessibility is explained to be affected by factors like void content, cracks as well as bitumen content in the mixture. The process itself entails reactions between atmospheric oxygen and bitumen components which alter the chemical composition of the binder. Tauste et al. further states that this process can be catalyzed through photo-oxidation when the surface is exposed to UV-radiation, resulting in an increase in reactivity of the molecules within the bitumen. During the oxidation process, the increase in oxygen removes hydrogen atoms from carbon atoms resulting in a shift of fractions from aromatics to resins and asphaltenes (Pipintakos, et al., 2020). Consumption of aromatics and resins thus increase the concentration of asphaltenes. The relation between these fractions is of importance due to a deficit of resins resulting in greater asphaltene condensation and in turn accelerated aging (Tauste, et al., 2018).

The outcome of these aging processes is an increase in stiffness and viscosity of the bitumen, resulting in a reduction of agglomerating power, i.e. its internal cohesive strength (Tauste, et al., 2018). This may in turn increase risk of thermal cracking and fracture due to fatigue. Furthermore, the reduction of maltene content in aged binder may cause the asphaltene component to flocculate, which in turn may lead to increased risk of cracking (Xie, et al., 2019).

2.1.1.3 Rheological properties

Rheology can be defined as the study of flow and deformation behavior of materials, focusing on relationships between time, temperature, stress and strain. It is essential to understand the rheological properties of bitumen in order to be able to predict pavement performance (Yusoff, et al., 2011). Bitumen is a viscoelastic material, meaning that it has both viscous and elastic components of response depending on temperature and loading frequency. When subjected to load, it is the viscous part of the material that is responsible for permanent deformation, whilst the elastic part contributes to the reversible deformation. There are three main rheological properties of bitumen that have influence on flow and deformation of the material: complex viscosity (η^*), complex shear modulus (G^*) and phase angle (δ) (Remisova, et al., 2016). Complex viscosity can be defined as a material's total resistance to flow as a function of angular frequency (ω) (Malvern Instruments Limited, 2016). Complex shear modulus is used to measure the total resistance to deformation of a material when undergoing repeated oscillatory loading. Finally, the phase angle describes the delay between applied shear stress and the resulting shear strain. The phase angle also describes the relationship between viscosity and elasticity of a material. A smaller phase angle insinuates more elastic behavior and vice versa. Complex shear modulus and phase angle are directly measured using an oscillatory testing method called dynamic shear rheometer (DSR). Complex viscosity can be calculated by dividing complex shear modulus by angular frequency. The complex shear modulus and phase angle can be combined in order to quantify the resistance to rutting as well as fatigue cracking of the pavement (Pavement Interactive, n.d.). The elastic portion of the complex shear modulus can be defined as $G^*/\sin\delta$. The value of G^*describes the stiffness of the binder as well as its resistance to deformation, whilst δ influences the binder's ability to recover its original shape after deformation. Pavement Interactive states that a higher value of the elastic portion results in higher stiffness of the pavement and consequently more resistance to rutting. When evaluating fatigue cracking prevention on the other hand, the viscous portion of the complex shear modulus, $G^* \cdot \sin\delta$, is used instead. In this case, it is beneficial for the binder to be elastic, meaning that the viscous portion should be kept to a minimum (Pavement Interactive, n.d.).

Adding RAP to an asphalt mixture affects it in a predictable manner. Since ageing of binder results in increased stiffness, the value of G^* is also increased (Pavement Interactive, n.d.). Furthermore, a reduction in elasticity also compromises the mixture's ability to recover after deforming, resulting in a lower value of the δ parameter. Consequently, the elastic portion of the complex shear modulus, $G^*/\sin\delta$, sees an increase along with the addition of RAP. Similar results can be expected regarding the viscous portion of the complex shear modulus, $G^* \cdot \sin\delta$, which is also increased along with the addition of RAP. An increase of this parameter on the other hand is less beneficial since a stiffer pavement is more susceptible to fatigue cracking (Pavement Interactive, n.d.).

2.1.1.4 Binder content

Binder content can be defined as the amount of bituminous binder present in an asphalt mixture. Depending on what properties are desired, binder content can vary but generally accounts for 4-8 % by weight in the United States (Pavement Interactive, n.d.). Similar bitumen content is used in Sweden, 5-6 %, since the fundamentals of asphalt paving are similar globally (VTI, 2020). Although not constituting much of the total weight of an asphalt mixture, binder content has tremendous effect on properties such as stiffness, strength, durability, fatigue life, raveling, rutting and moisture damage (Pavement Interactive, n.d.). Low binder content results in less dense pavement, consequently increasing its permeability. Water infiltration may in turn lead to damage from freeze-thaw as well as other effects caused by the presence of water (Giompalo, 2010). High binder content on the other hand insinuates an increase in aggregate-binder ratio, which translates into greater absorption of binder on the aggregate surface (Husain, 2021). This strengthens the bond between aggregates, thus resulting in improved resistance to cracking. However, increasing the binder content reduces cohesion and consequently also the resistance to rutting (Douries, 2004). Furthermore, high binder content increases the density of the pavement, decreasing the risk of water infiltration. Beyond rutting, high density may also lead to increased risk of phenomena such as flushing and shoving (Giompalo, 2010).

When RAP is introduced, the binder content of an asphalt mixture is further complicated. This is due to the aged binder not behaving homogenously throughout the reclaimed material (Chiara Cavalli, et al., 2017). Although the addition of RAP may increase the total binder content of a mixture, it is stated that reclaimed binder is considerably stiffer and consequently may not be fully mobilized during mixing. This results in an increase in the difference between active binder and total binder content. As a result, incorporation of RAP may increase the stiffness of the pavement, exhibiting more brittle behavior. This may in turn lead to properties such as susceptibility to low temperature cracking and fatigue cracking being compromised. However, moisture as well as rutting resistance may increase due to increased stiffness (Rizk, et al., 2023).

2.1.2 Aggregates

Aggregate accounts for approximately 92-96 % of the volume in Hot Mix Asphalt and is a collective term for mineral materials with varying particle size, such as crushed stone, gravel and sand (Pavement Interactive, n.d.). These particle sizes include coarse aggregate, retained on the 2,36 mm sieve, fine aggregates, retained on the 0,075 mm sieve, and filler that passes through the 0,075 mm sieve (National Cooperative Highway Research Program, 2011). It is stated that coarse aggregate and fine aggregate are commonly referred to as gravel and sand respectively. Due to aggregate constituting such a large proportion of the volume, it is decisive for the properties of an asphalt mixture. The term can be divided into natural and manufactured aggregate. Natural aggregate includes for example naturally occurring unconsolidated sand and gravel (Langer, 2014). Crushed stone or bedrock can be considered manufactures aggregates. It can be derived from industrial processes and can include byproducts of other processes, such as slag as a byproduct of steel production (Sims, et al., 2019). For asphalt production in the United States, crushed stone is the most common alternative, although small percentages of natural sand may be included in the mixture (Williams, et al., 2024). Natural gravel sees little to no usage in Sweden due to it being a finite resource, along with the fact that access and availability of crushed stone is good (Agardh & Parhamifar, 2014). Consequently, crushed stone dominates as the most popular aggregate alternative in the country.

In order to achieve desired properties of an asphalt mixture it is important that the aggregate meets certain requirements. Aggregate interlock is a term that describes the mechanical interaction between aggregates in stone-based materials and affects load transfer, density, stiffness, shear resistance and compaction of asphalt (Liu, et al., 2013). It is of importance that aggregates are crushed sufficiently to avoid round and smooth particles. These would otherwise result in poor interlock between particles which in turn may lead to reduced strength and rut resistance of the finished product (National Cooperative Highway Research Program, 2011). Flakiness Index determines the proportion of particles that are considered flaky or flat. A high Flakiness Index implies a high amount of flaky or elongated particles whilst the opposite implies that most particles are roughly cubic. The former may lead to a large number of particles being oriented in the same direction, causing anisotropy within the material (Agardh & Parhamifar, 2014). As the pavement ages, shape and surface properties of aggregate particles are affected. Virgin aggregate tends to possess high angularity as well as rough surface texture which promotes aggregate interlock (Tutumluer, et al., 2018). Ageing of pavement results in degradation of aggregate particles which may lead to abrasion. In turn, Tutumluer states that this may result in particles becoming more spherical, losing their angularity and rough surface in the process.

The gradation of aggregate, or particle size distribution, in an asphalt mixture is of great importance when it comes to predetermining the performance of pavement. Properties such as stiffness, stability, durability, permeability, workability, fatigue resistance, frictional resistance and susceptibility to moisture damage can all be controlled by modification of the gradation of aggregate in HMA (Pavement Interactive, n.d.). Laboratory experiments have shown that large particles, retained on the 2,36 and 4,75 mm sieves, provide more than 50 % of the contribution to resist loads and rutting, while particles retained on smaller sieves, between 1,18 and 0,3 mm, contribute to more than 50 % of the strength and structure of the pavement (Yue, et al., 2022). An imbalanced distribution of particle sizes can have a negative effect on the performance of the final product. Excessive usage of large maximum particles harms the workability and may lead to segregation of the asphalt (Pavement Interactive, n.d.). On the other hand, instability may be caused by excessively including small maximum particles in the mixture. It is important to note that mentioned relationships between particle size and expected behavior of pavement focus on the maximum size of particles. The source further states that including a large percentage of small particles results in increased density of the asphalt since void space is reduced. This in turn results in reduced permeability and increased stability (Pavement Interactive, n.d.). However, experimental evaluations have been carried out where results have shown that a high dust-binder ratio can have a negative impact on flexibility (Husain, 2021). Consequently, a high amount of small, filler like particles may increase susceptibility to cracking even at higher performance grades. Dust in this case is defined as particles passing through the 0,075 mm sieve.

The process of removing pavement and extracting RAP may result in a shift in aggregate gradation compared to virgin materials (Federal Highway Administration, 1997). This is due to processes such as milling and crushing mechanically degrading aggregates, resulting in a larger concentration of fine particles and filler. Consequently, aggregate gradation of the mix becomes finer as RAP is added. Increased concentration of fine particles affects the properties of the asphalt mixture and in turn the finished pavement. Fine particles allow for more efficient filling of air void volumes in the mix, resulting in reduced volume of Voids in the Mineral Aggregate (VMA) (Copeland, 2011). Reduced VMA means there is less space available for binder to coat aggregate particles within the mixture, resulting in a stiffer mix. Furthermore, dust-binder ratio is increased along with the addition of finer particles. This results in a drier mix and also reduces the thickness of the binder coating aggregate particles (Coleri, et al., 2018). Consequently, flexibility is reduced and phenomena such as fatigue cracking become more relevant. That being said, Coleri et al. states that reduced flexibility promotes greater resistance to rutting.

2.1.3 Filler and asphalt mastic

Mineral particles that pass through the 0,075 mm sieve are referred to as filler (Wu, et al., 2021). Wu et al. states that limestone powder is commonly used for this purpose in countries where availability is good, such as the United States. The addition of particles of this size increases the cohesiveness of the mixture, consequently increasing its resistance to deformation (Carvajal-Munoz, et al., 2024). The combination of filler and bitumen is called asphalt mastic, and its primary role is to help transfer loads, coat and bond coarse and fine aggregates as well as fill voids in the material. The asphalt mastic can be identified as the weakest phase in an asphalt mixture (Wu, et al., 2021). This results in properties such as workability, rutting resistance and crack resistance of the asphalt mixture being heavily influenced by the stiffness, strength and cohesiveness of the mastic.

2.1.4 Additives and alternative methods for production

In addition to bitumen and aggregate, different additives can be added to an asphalt mixture in order to enhance or modify certain properties. These properties vary from workability of the mixture to mechanical properties of the finished pavement or even modification of the natural ageing of the bitumen. Using alternative methods to Hot Mix Asphalt, such as Warm Mix Asphalt is also an option in order to modify certain properties of the mixture.

2.1.4.1 Warm Mix Asphalt

Hot Mix Asphalt (HMA) is a technology that requires high temperatures that exceed 150°C (Federal Highway Administration, 2025). The elevated temperatures are required in order for the binder to become viscous enough to allow for a complete coating of the aggregate. Furthermore, FHWA states that it contributes to good workability during laying and compaction as well as durability during the service life of the pavement. However, reaching temperatures of this magnitude requires a high amount of energy and fuel. Warm Mix Asphalt (WMA) is a term that includes a plethora of technologies that allows for asphalt pavement to be produced and placed at lower temperatures compared to HMA (Federal Highway Administration, 2016). This reduction in temperature ranges between approximately 20-40°C. This can be achieved through the usage of additives, resulting in reduced viscosity of the binder and therefore allowing for a decrease in temperature of the mixture (Federal Highway Administration, 2016). This can in turn reduce energy consumption by approximately 15 % (Bhatt & Wu, 2025). Besides reducing energy and fuel demand in asphalt production, WMA also allows for better compaction of asphalt (Federal Highway Administration, 2016). When warm asphalt contacts the cold surface of the ground, the temperature of the mixture immediately decreases. According to FWHA, this increases the viscosity of the binder, affecting the workability of the asphalt. The severity of the consequences is dependent on the temperature difference between the mixture and the underlying layer, thus making WMA an option for more compact pavement requiring less energy and fuel to produce.

2.1.4.2 Polymer additives

The consistency of the binder in an asphalt mixture is of great importance for its properties. If the bitumen is too stiff, cracking may occur, mainly at low temperatures. At elevated temperatures however, a less viscous binder may lead to risk of rutting. An ideal temperature range can be defined where viscous and elastic properties of the material are balanced (Illinois Department of Transportation, 2005). To increase the size of the span where the bitumen preforms the best, polymers can be added to modify the viscoelastic behavior of the binder, and in turn change the ideal temperature range. According to Illinois Department of Transportation, the two most common polymers used for this purpose are elastomers and plastomers. Elastomers increase elasticity at low temperatures as well as strength high temperatures, whilst plastomers exclusively provide the latter. Besides increasing resistance to rutting and thermal cracking, addition of polymers to bitumen can contribute to reduced fatigue damage, greater elastic recovery as well as an increased softening point (Yildirim, 2005). The use of polymers as an additive also generally increases the viscosity of the binder, resulting in increased adhesion between aggregate particles as well as the time it takes for the binder to become brittle as a result of oxidation (Illinois Department of Transportation, 2005).

2.1.4.3 Hydrated lime

In the United States, one of the most commonly used additives in HMA is hydrated lime, which has been shown to extend the life of pavement up to 38 % (Little, et al., 2006). Hydrated lime is, unlike most mineral fillers, chemically active (European Lime Association, n.d.). When added to an asphalt mixture, reactions occur between the hydrated lime and the binder as well as the aggregate respectively. European Lime Association states that as the hydrated lime reacts with the highly polar molecules of the bitumen, the formation of hydrophobic calcium-based salts is promoted. Simultaneously, this reduces the formation of water-soluble soaps that otherwise would contribute to weaker bond-strength and may result in moisture damage. Moreover, reaction with bitumen increases the stiffness of the binder at high temperatures, resulting in higher resistance to rutting and fatigue cracking. As temperature decreases, hydrated lime becomes less chemically active and acts more like an inert filler (European Lime Association, 2011). As a result, the asphalt mixture becomes less brittle at low temperatures when hydrated lime is added, thereby preserving the asphalt's resistance to low-temperature cracking. It has also been shown that the addition of hydrated lime can reduce age hardening, such as oxidation, due to lime reactions slowing the rate of change in natural bitumen chemistry (Little, et al., 2006). As for chemical reactions between hydrated lime and aggregates, the result is a modification of the surface properties of aggregate particles (European Lime Association, 2011). The reactions are stated to allow for increased roughness which in turn results in greater adhesion between aggregate and bitumen.

2.1.5 Void content

The volumetric properties of an asphalt mixture can be described using the following: Air Voids (VA), Voids in the Mineral Aggregate (VMA), Voids Filled with Asphalt (VFA) and Volume of Effective Binder (VBE) (Speight, 2015). VA refers to the air spaces in the pavement, trapped between bitumen-coated aggregate. VMA indicates the space available for bitumen to sufficiently coat each aggregate particle (Dhir, et al., 2017). VFA gives the percentage of voids that are filled with binder, while VBE describes the volume of binder that is actually effective (Varma, et al., 2021). VFA is of importance since it directly correlates to the density of the finished mixture (Speight, 2015). Speight states that the result of VFA being too low is an insufficient amount of binder, compromising durability of the pavement as well as not allowing for controlled compaction during traffic load. Although all of the mentioned parameters are of importance, it is most common to refer to VA when discussing the void content of an asphalt mixture. In asphalt design, the percentage of air void is essential for pavement performance. One common threshold for this is between 3-7 % (Pavement Interactive, n.d.). The source states that there is a general rule of thumb that states that for every percentage point the air void content exceeds 7 %, the service life of the pavement is decreased by approximately 10 %. Moreover, high air void content can result in decreased stiffness and strength, reduced fatigue life as well as increased risk of raveling. Furthermore, a high percentage of air present in the pavement may promote accelerated aging of the binder, since oxygen availability is increased. Pavement Interactive states that moisture damage may also pose considerable challenges due to increased permeability and thereby infiltration of water. If the air void content falls below the threshold however, the risk of phenomena such as flushing, distortion and rutting become imminent. Today, air void content between 3-8 % is considered normal and produces a good combination of the mentioned pavement properties (Pavement Interactive, n.d.).

When incorporating RAP into the mixture, its air void content may be affected in different ways depending on the conditions during mixing. A study by Al-Mosawe et al. shows that increasing the amount of RAP directly increases air void content, resulting in lower bulk density, VMA and VFA (AL-Mosawe & Al-Mosawe, 2023). This may be caused by two phenomena, the first of which is particle clustering (Abed, et al., 2018). Particle clustering is explained to take place when RAP particles agglomerate during mixing, causing a decrease in fine particles in the mix. This may in turn lead to reduced compaction of the pavement, thus increasing VA. The second factor is affected by the size of the RAP particles. Abed et al. states that at relatively low mixing temperatures, the ratio between active to non-active binder is low. This causes an effective size increase of RAP particles since they behave according to the black rock theory, further explained in chapter 2.2.1.2. However, if mixing conditions meet certain time and temperature requirements, Abed et al. suggests that VA can be unchanged or even lowered after the addition of RAP. Increasing the temperature during mixing activates a larger portion of the aged binder, effectively reducing the size of RAP particles, consequently also reducing VA of the mix (Abed, et al., 2018). However, in doing so, aging of the binder may be significantly accelerated (da Silva Lopes, et al., 2023).

2.2 Reclaimed asphalt pavement (RAP)

Reclaimed asphalt pavement (RAP) is a term describing removed and reprocessed pavement materials containing aggregates and binder (Federal Highway Administration, 1997). According to FHWA, these materials originate from the removal of asphalt pavement during reconstruction, resurfacing or when access to underground utilities are needed. Since asphalt mainly consists of non-renewable resources, recycling of the material is frequently encountered in the paving industry and RAP remains one of the United States most recycled materials (National Asphalt Pavement Association, 2023).

2.2.1 Extraction and manufacturing of RAP

In order for asphalt pavement to be removed, one of two methods is generally used. Milling is the most common method and entails removing only the upper portion of the existing pavement, typically around 50 mm (Federal Highway Administration, 1997). This leaves the underlying pavement structure intact, and the method is stated to be commonly used for resurfacing purposes or to handle surface level damages. The alternative is full-depth removal which is a process where the entire asphalt surface as well as a portion of the road base is excavated (City of Elk Grove, n.d.). This method is explained to be used when the pavement is severely damaged or distressed and needs to be removed and replaced.

When the process of removing existing pavement is complete, the RAP material is transported to a recycling facility where it’s crushed to ensure desired particle size. The size of particles varies from coarse to fine, and screening of the material helps sort particles by size. This is done by letting particles pass through vibrating screens and this process also removes unwanted non-asphalt materials and other contaminants (Aurilio, 2025). This is followed by quality control of RAP fractions to ensure consistency throughout the stockpile. If crushing and screening is done properly, the result should be RAP consisting of high quality, well-graded aggregates coated by bitumen (Federal Highway Administration, 1997). Samples are then analyzed to determine characteristics such as bitumen grade and bitumen content of the RAP. Based on the results of the analysis, decisions are made regarding potential incorporation virgin aggregate, virgin bitumen as well as rejuvenators in the mixture (Aurilio, 2025). Rejuvenators are commonly used due to RAP containing aged bitumen, the properties of which are drastically altered as it ages, see chapter 2.1.1.2. The addition of rejuvenators restores the functional properties of aged bitumen and enables it to be reused. Finally, the recipe of the asphalt mixture can be determined, deciding the amount of RAP, virgin materials and rejuvenator with regards to predetermined quality requirements (Aurilio, 2025).

2.2.1.1 Rejuvenators

Rejuvenators are additives added to asphalt mixtures containing RAP to reduce the viscosity of the aged bitumen in order to improve workability, performance as well as environmental impact of an asphalt mixture (Saltan & Hussain Khaliqi, 2024). This agent can be divided into softening and restorative rejuvenators. Restorative rejuvenators aim to mitigate aggregation of components in aged bitumen through restoring chemical and physical properties of oxidized binder (Pahlavana, et al., 2017). This is done by rebalancing the colloidal structure resulting in restoration of the original viscosity of the bitumen. As bitumen ages, oxidation causes a reduction of maltene content. This may lead to flocculation of the asphaltene component, increasing cracking susceptibility. Restorative rejuvenators commonly contain high levels of maltene components and can be added to reclaimed bitumen in order to help restore the maltene content of the binder (Xie, et al., 2019). Xie et al. states that this process enables asphaltenes to move freely, resulting in reduced viscosity and improved flow properties of the bitumen. Softening rejuvenators, or fluxing agents, on the other hand, do not alter the chemical composition of bitumen and is limited to lowering its viscosity (Pahlavana, et al., 2017). This is not done by increasing maltene content, but rather through the addition of small molecules to the aged and brittle asphalt, according to Pahlavana et al.

Further, rejuvenators can be divided into five general categories. These include paraffinic oils, aromatic extracts, naphthenic oils, triglycerides & fatty acids as well as tall oils (Xie, et al., 2019). While these categories showcase a variety of available rejuvenators, it can be difficult to classify new rejuvenators with deviant or combined chemical composition into these set groups (Schwettmann, et al., 2022). This has led to further classification, which divides rejuvenating agents based on origin. These categories include petroleum-based and bio-based rejuvenators (Xie, et al., 2019). Petroleum-based rejuvenators are derived from non-renewable resources, and the usage of these materials introduces a plethora of challenges. When exposed to elevated temperatures, the light fractions in this type of rejuvenator tend to volatilize, reducing the efficiency of rejuvenation (Quan, et al., 2025). Furthermore, ageing resistance and long-term durability of the asphalt may be compromised due to petroleum-based rejuvenators containing high levels of unsaturated bonds as well as aromatic compounds, both of which are prone to oxidation at elevated temperatures. In addition, Quan et al. states that the presence of unsaturated aromatic structures results in environmental pollution. As a result, the popularity of bio-based rejuvenators has increased, presented as a promising alternative to petroleum-based products.

Vegetable oil-based rejuvenators are becoming more common as an alternative to petroleum-based rejuvenators (Quan, et al., 2025). Because these products are derived from materials that are both renewable and sustainable, Quan et al. states that bio-based rejuvenators may result in a significant decrease in climactic impact compared to their petroleum-based counterparts, depending on how carbon sinks are accounted for. A plethora of studies have been conducted regarding the efficiency of bio-based rejuvenation techniques, and bio-oils are often regarded as the most effective rejuvenators for enhancing fatigue performance of aged asphalt binder (Sun, et al., 2025). Rejuvenators derived from soy-bean oil as well as rapeseed oil have proven to be able to effectively restore low-temperature flexibility and increase resistance to fatigue cracking in aged asphalt (Quan, et al., 2025). Furthermore, rapeseed oil has been identified as a particularly promising alternative due to its advantageous chemical profile, reliable supply and thermal as well as oxidative stability. According to Quan et al., its oxidative stability is caused by the oil containing a high proportion of monosaturated fatty acids, enhancing resistance to oxidative degradation at high temperatures. A study by Arafat et al. shows that a bio-oil rejuvenator can exhibit greater efficiency in lowering the binder grade of aged bitumen compared to a petroleum-based rejuvenator (Arafat, et al., 2023). However, it is also shown that the bio-oil rejuvenator in this case is more susceptible to long-term ageing and therefore performed worse when evaluating cracking susceptibility of long-term aged asphalt. Furthermore, the use of bio-oil resulted in more effective softening of the binder, despite both rejuvenators possessing the same high temperature performance grade. Consequently, resistance to rutting was inferior compared to the petroleum-based rejuvenator.

Rejuvenators can be added to an asphalt mixture through three main methods (Bhatt & Wu, 2025). The first of which is through direct mixing with reclaimed bitumen-coated aggregate. This method involves heating RAP to a mixing temperature before adding a rejuvenating agent which is then followed by consistent mixing. The second method involves incapsulating rejuvenators in capsules that are added along with aggregates before paving occurs. Bhatt and Wu states that as the pavement ages and microcracks form, these eventually reach and burst the capsules, releasing rejuvenator from the inside of the pavement. Although more research is required, it is believed that this method of rejuvenation promotes self-healing of the asphalt pavement, extending its service life. The third method entails spraying rejuvenator on the pavement surface in order to soften the aged asphalt, seal micro-cracks and restore protection from moisture and oxygen (Bhatt & Wu, 2025).

2.2.1.2 Heterogeneity of RAP

It is important to note that RAP may be composed of a mix of different materials exhibiting diverse levels of ageing, gradation and binder characteristics. It can be difficult to estimate the presence of potential additives or other unknown components in the original pavement, some of which may affect the properties of the material (Ferreira, et al., 2021). It should also be noted that RAP samples can be very consistent if milled from a single, traceable source (Tarsi, et al., 2020). The properties of a RAP sample may vary depending on what aggregate-binder ratio the specific sample contains. This ratio is dependent on how well the RAP has been crushed and fractioned due to fine aggregate having the ability to bond a larger amount of binder in relation to particle size compared to coarse aggregate (Nanda & Kumar Siddagangaiah, 2024). The gradation of RAP is therefore one of the factors affecting the amount of aged bitumen present in the mixture. As bitumen ages, mechanisms such as oxidation and volatilization change both chemical and rheological properties of the binder (Ferreira, et al., 2021). Furthermore, aging is not consistent throughout all bitumen in the mix and is partially due to the external parts of the binder naturally being exposed to more oxygen. This results in a higher degree of oxidation of the outermost layer of bitumen and may in turn create aging differences within the reclaimed binder. This difference in ageing impacts the blending between reclaimed binder and virgin binder during mix production, thus also affecting the binder's ability to homogeneously coat aggregate particles (Ferreira, et al., 2021). Ferreira et al states that it is therefore reasonable to assume that this will in turn translate into higher heterogeneity of the finished recycled asphalt mixture. Furthermore, the aged bitumen present in RAP can by some simplification be divided into active and inactive binder with a transition zone existing between them. Active binder is commonly less aged and can be described as the binder that is part of the mixture design. According to Ferreira et al., there is uncertainty amongst researchers whether inactive binder can be considered to affect mixture performance, although it is very likely.

Black rock theory is the term used to describe the case where no RAP-binder is assumed to be active in the mix, leaving all RAP to be treated as black rocks. It is most common to assume that partial blending occurs between reclaimed and virgin components (Chiara Cavalli, et al., 2017). In the case where the portion of inactive binder is large, additional virgin bitumen may be added to ensure desired viscosity (National Center for Asphalt Technology, 2024). It is however possible that inactive binder acts as a coating layer on RAP particles affecting gradation as well as the shape of particles, and therefore mechanisms such as aggregate interlock (Ferreira, et al., 2021).

2.2.2 Benefits associated with the usage of RAP

Although the mechanical behavior of an asphalt mixture may be affected, there are a number of benefits that come with increased pavement recycling. Asphalt pavement is considered to be 100 % recyclable with the ability to be reused over and over again in new projects (National Asphalt Pavement Association, 2021). This characteristic of the material allows for efficient recycling which is directly beneficial with regards to environmental impact, including both conservation of natural resources but also reduction in carbon emissions (Tennessee Transportation Assistance Program, 2025). In terms of Life Cycle Assessment (LCA), the main reduction in GHG can be found in the cradle-to-gate section (A1-A3) since its incorporation enables reduction of both virgin binder as well as aggregate (Shacat, et al., 2024). In 2018 the use of RAP alone could save 4,1 million tonnes of virgin binder and 78 million tonnes of virgin aggregate in the United States (Tarsi, et al., 2020). Furthermore, studies have shown that by using rejuvenating agents in RAP mixes, RAP contents of over 90 % may become viable (Liu, et al., 2024). Liu et al states that using this amount of RAP in a mix can result in a 64 % reduction of the carbon footprint associated with the pavement compared to conventional asphalt blends. This aligns with a case study by Ma et al. that suggests that approximately 43 % of GHG that is released during the whole lifecycle of pavement, excluding recycling, comes from raw materials and production phase (Ma, et al., 2016). Additional environmental benefits include avoiding landfill space being occupied by reusable materials (Federal Highway Administration, 2025). The RAP accepted and reused by asphalt plants in the United States in 2018 resulted in over 46 million cubic meters of landfill space being avoided (Tarsi, et al., 2020). Although environmental benefits are one of the most prominent reasons to RAP usage, there are other benefits of recycling pavement, such as economic. A study by FHWA suggests that using 50 % RAP in an HMA design may result in savings of approximately 34 %, depending on the mix design and certain assumptions (Kandhal & Mallick, 1997). Using 20 % RAP, which is closer to the average RAP content in pavement projects across the United States, may lead to savings of approximately 14 %. It should be noted that these numerical values are not updated as of recently but aim to give an approximative understanding of possible savings. In 2018, a survey by the National Asphalt Pavement Association (NAPA) suggests that 2,8 billion U.S. dollars could be saved in the United States through substituting virgin materials with RAP (Tarsi, et al., 2020).

2.2.3 Challenges and limitations of RAP

Although there are benefits of including RAP in asphalt mix designs, several challenges limit its use within the paving industry. As mentioned previously, chapter 2.2.1.2, RAP may be considered a heterogenous material. It can therefore be difficult to predict how it will perform within a mix if the homogeneity and quality of the stockpile is not assessed carefully. Variations in properties such as aggregate gradation and binder content within the recycled material therefore increases the difficulty to achieve consistent results during production (Negrell & Larsson, 2021). It can also be difficult to estimate the percentage of binder that is active versus inactive, resulting in the possibility of consistency throughout different mixes being compromised (Ferreira, et al., 2021). Furthermore, the process of milling as well as crushing to reduce particle size promotes a high concentration of fine particles within RAP samples. This may result in potential set requirements for aggregate gradation becoming the limiting factor to how much RAP may be incorporated into the mix design. An increased concentration of fine particles may also result in increased retention of moisture within the sample, consequently requiring more energy to dry the material during mix preparation (Tarsi, et al., 2020). Increased stiffness of the final pavement as well as decreased workability of the mix are also factors that limit the current use of RAP (Farooq & Mir, 2017).

In conventional Hot Mix Asphalt plants, the maximum RAP content is considered to be approximately 50 % and is limited by two main factors: the heat capacity of the plant as well as gaseous hydrocarbon emissions (Federal Highway Administration, 1997). The heat capacity of the plant is important since the temperature of the virgin aggregate has to compensate for the fact that the RAP is usually only partially heated or cold before mixing. Further, FHWA states that if temperatures don't reach adequate levels, mixing between virgin and recycled binder may be compromised, resulting in inconsistency within the final pavement. As for emissions, avoiding RAP to exceed temperatures of 427˚C is of great importance since doing so may lead to excessive hydrocarbon emissions (blue smoke) (Federal Highway Administration, 1997). Hydrocarbons are often found in petroleum products, such as bitumen, and consist of hydrogen and carbon atoms. Releasing hydrocarbons may have severe impact on the environment, affecting the quality of soil, water and air (Alao, et al., 2025). Furthermore, health risks may also occur for individuals exposed to hydrocarbon-polluted air, groundwater and plants. It should be noted that the use of drum mix plants allows for as much as 60 to 70 % RAP in a mix and implementation of special microwave-based plants have been developed to enable 100 % RAP usage whilst limiting gaseous emissions (Federal Highway Administration, 1997). However, the cost of heating in this type of plant is significantly higher compared to conventional plants and this type of technology has only seen limited use in the United States. The result of these challenges is that RAP usage for road construction worldwide is usually limited to a range of 5 to 50 % (Jaawani, et al., 2021).

2.3 Mechanical degradation of pavement

2.3.1 Cracking

Cracking of asphalt can be caused by a plethora of factors such as traffic load, temperature and moisture within the pavement. Cracking types include, for example, edge cracking, reflection cracking, transverse cracking and fatigue cracking (Texas A&M Transportation Institute, n.d.). Fatigue cracking, or alligator cracking, occurs as a result of repeated traffic load and is caused by fatigue failure of the pavement surface (Pavement Interactive, n.d.). Bottom-up fatigue cracking occurs in thin pavements and refers to cracking initiated where tensile stress is the highest, i.e. at the bottom of the HMA layer. Longitudinal cracks are formed which migrate to the surface of the pavement, creating a pattern of cracks similar to the skin of an alligator. According to Pavement Interactive, top-down cracking is commonly found in thicker pavements and is due to high tensile stress caused by the aging of binders as well as interaction between tires and pavement. Fatigue cracks roughen the surface of the pavement and may result in potholes and larger infiltration of moisture. This type of cracking is considered a major structural distress (Pavement Interactive, n.d.).

2.3.2 Rutting

Rutting in asphalt can be described as the wheel path deformation of pavement and can be divided into three categories. Densification, subgrade rutting and mix rutting (Pavement Interactive, n.d.). Densification may occur as a result of insufficient compacting during the construction stage. This may cause the HMA to deform as it continues to compact under traffic loading. Subgrade rutting is a phenomenon that occurs when the subgrade deforms as a result of loading. This in turn may lead to the pavement settling into these deformations, resulting in visible wheel path depression on the HMA surface. The last category of rutting is mix rutting, which is what the evaluation of RTI focuses on (ASTM D8360) (ASTM International, 2022). During this process, the subgrade retains its original shape, yet the pavement surface deforms during traffic loading as a result of potential mix design or compaction problems (Pavement Interactive, n.d.). The source states that rutting in asphalt pavements become problematic as they fill up with water, resulting potential in vehicle hydroplaning. This phenomenon inhibits steering and may pull the vehicle towards the rut path, compromising the safety of the road.

2.3.3 Temperature dependency of pavement properties

When exposed to different climates, the temperature of pavement as well as its properties may be affected. With bitumen as a central building stone in an asphalt mixture, the result is a large dependency of temperature due to the binder becoming a viscous liquid at high temperatures (Agardh & Parhamifar, 2014). Although temperatures may not reach these heights in the field, the correlation between increased temperature and softer bitumen can still be found. This relates to the softening point of the binder, which if exceeded, results in an increase in the fluidity of the binder. This in turn increases the shear stress within the pavement, consequently resulting in a higher risk of rutting damages to occur (Yang, et al., 2025). Yang et al explains that this phenomenon is especially relevant when pavement is exposed to excessive loading, enhancing the risk of deformation related damage. Temperature dependent performance can be enhanced by using bitumen with a binder grade that matches the climate in which the pavement will be used. When pavement is exposed to temperatures at the other side of the spectrum, i.e. low temperatures, cracking rather than rutting becomes the prominent problem (Guo, et al., 2025). This is due to the pavement seeing an increase in both stiffness and brittleness as temperatures drop. It can be concluded that phenomena such as rutting and cracking see the most relevance at different ends of the temperature spectrum.

2.3.4 Climatic differences between the United States and Sweden

The different climates that the laboratory phase will focus on are northern California and southern Sweden. In the south of Sweden, more specifically Malmö, Skåne, the lowest measured monthly mean temperature during the time period of 2020-2025 was 0 ˚C, measured in February of 2021 (SMHI, 2026). It is important to note that pavement and air temperature differ from one another. Whilst the air temperature might be 0˚C, absorbed radiant heat as well as direct solar radiation causes the temperature of the pavement to rise (García-Melgar, et al., 2025). Furthermore, a datasheet containing air- and surface temperatures in Malmö is analyzed, provided by the city of Malmö[1]. It can be found that, in several cases, pavement temperatures exceed air temperatures by approximately 0-3 °C during colder conditions, depending on material depth and air temperature. It is therefore reasonable to assume 5°C as a representative service condition for winter climates in Skåne when measuring cracking resistance. As for summer climates, the same time period was studied, resulting in a maximum monthly mean temperature of 20,2°C measured in July of 2021 (SMHI, 2026). Analyzing the earlier mentioned datasheet, it is not uncommon to find surface temperatures exceeding air temperatures by 15-20°C in the summer. It is therefore fair to assume that 35°C is a reasonable service condition for warm climates in Skåne. This temperature is used to evaluate rutting resistance.

When analyzing temperatures in northern California, Redding specifically, during the time period of 2020-2025, the maximum monthly mean temperature was measured in July 2024 to be 36,6 °C (National Centers for Environmental Information, 2026). Using the same reasoning as above, it is assumed that pavement surfaces reach temperatures of 65°C in the middle of summer. This is the range where rutting resistance will be tested. Although these elevated temperatures can be measured in the peak of summer, the average monthly temperature between 2020-2025 is measured to be 21 °C (National Centers for Environmental Information, 2026). This means that pavement experiences intermediate temperatures during a substantial portion of its service life. Assuming that there are similarities between how air- and pavement temperatures relate to one another in Malmö and Redding, 35 °C is selected as a representative intermediate pavement temperature for evaluation of cracking resistance.

[1] Pajtim Sulejmani. Yt- och lufttemperatur Malmo, Malmö Stad. Data from the City of Malmö's Streets and Roads Department on pavement surface and air temperatures measured in Malmö, Sweden, VViS station Malmö Ö. pajtim.sulejmani@malmo.se. 23 04 2026.

3 Standards for testing

3.1 Cracking Tolerance Index – ASTM D8225

CTI testing is performed according to ASTM D8225 which is comparable to EN 12697-23 within European standards (ASTM International, 2019; European Committee for Standardization, 2017). Determination of CTI aims to evaluate fracture resistance related to fatigue cracking through analyzing the load-displacement curve, failure energy and deformation tolerance. Modern machinery is able to calculate CTI, but it may be calculated manually through the following steps.

The work of failure (W_f) is calculated as the area under the load vs. load-line displacement (LLD) curve through the quadrangle rule:

$$W_f = \sum_{i=1}^{n-1} \left((l_{i+1} - l_i) \cdot P_i + \frac{1}{2} \cdot (l_{i+1} - l_i) \cdot (P_{i+1} - P_i) \right)$$

where:

W_f = work of failure, [J],

P_i = applied load at the i load step application, [kN],

P_{i+1} = applied load at the $i + 1$ load step application, [kN],

l_i = LLD at the i step, [mm], and

l_{i+1} = LLD at the $i + 1$ step, [mm].

Failure energy (G_f) is calculated by dividing the work of failure by the cross-sectional area of the specimen.

$$G_f = \frac{W_f}{D \cdot t} \cdot 10^6$$

where:

G_f = failure energy, [J/m^2],

W_f = work of failure, [J],

D = specimen diameter, [mm], and

t = specimen thickness, [mm].

The post-peak slope ($|m_{75}|$) is the slope of tangential zone around the 75 % peak load point after the peak. Cracking Tolerance Index may be calculated as:

$$CT_{Index} = \frac{t}{62} \cdot \frac{l_{75}}{D} \cdot \frac{G_f}{|m_{75}|} \cdot 10^6$$

where:

CT_{Index} = Cracking Tolerance Index, [-],

G_f = failure energy, [J/m^2],

$|m_{75}|$ = absolute value of the post-peak slope m_{75}, [N/m],

l_{75} = displacement at 75 % of the peak load after the peak, [mm],

D = specimen diameter, [mm],

t = specimen thickness, [mm].

3.2 Rutting Tolerance index – ASTM D8360

RTI is performed according to ASTM D8360, which may be compared to EN 12697-22 within European standards (ASTM International, 2022; European Committee for Standardization, 2020). RTI is calculated from the peak load during testing and is used as an indicator of rutting performance of asphalt mixtures. RTI can be determined through the following steps, starting with calculation of shear strength as:

$$\tau_f = 0{,}356 \cdot \frac{P_{max}}{t \cdot w}$$

where:

τ_f = shear strength, [Pa],

P_{max} = maximum load, [N],

t = specimen thickness, [mm], and

w = width of upper loading strip, (= 0,0191 m).

Rutting Tolerance Index is calculated from the shear strength as:

$$RT_{Index} = 6{,}618 \cdot 10^{-5} \cdot \frac{\tau_f}{1\,Pa}$$

where:

RT_{Index} = Rutting Tolerance Index, [-], and

τ_f = shear strength, [Pa].

3.3 Guidelines for Cracking Tolerance Index and Rutting Tolerance Index

Although there is no universal minimum requirement for either CTI or RTI, several researchers have proposed thresholds for how a mix should perform with regards to these properties. For CTI, there is a large variation of suggested CTI values that indicate adequate resistance to cracking and can vary from a minimum CTI of 90, to as low as 26,4 (Kassem, et al., 2023). As for RTI, some sources suggest that the threshold for adequate rutting resistance depends on the binder grade used in the mix. Lower binder grade allows for lower RTI whilst increasing the binder grade increases the suggested minimum RTI. These suggestions vary but multiple sources state that a binder grade of PG 64-10, used in this project, should possess an RTI higher than 60 (Liu, et al., 2024; FHWA Mobile Asphalt Technology Center, 2022).

3.4 Control of outliers in data

To ensure that gathered data points can be considered representative of the whole population, calculations according to ASTM E178 are performed (ASTM International, 2021). The critical value for the test criterion (T_n) depends on the number of observations and is given by |1,425| in the case of four observations and |1,602| for five observations. If calculated values of T_n exceed these numbers, it indicates that the calculated value is an outlier with 90 % certainty. To calculate the test criterion, T_n, the following equation is used:

$$T_n = \frac{x_n - \bar{x}}{s}$$

where:

T_n = test criterion,

x_n = the doubtful value, i.e., the largest observation in the ordered dataset,

$\bar{x}$ = arithmetic average of all n values, and

s = estimate of the population standard deviation based on the sample data, calculated as:

$$s = \sqrt{\frac{\sum_{i=1}^{n}(x_i - \bar{x})^2}{n - 1}}$$

where:

x_i = individual observation,

$\bar{x}$ = arithmetic average of all n values,

n = number of observations in the sample

4 Laboratory phase

For the laboratory phase, assistance as well as lab equipment was provided by Pavement Engineering Inc. in Redding, California, United States. Aggregate and RAP was delivered by Tullis Inc., Redding, California, United States. The PG 64-10 binder that was used was delivered by Valero Refining in Benicia, California, United States. The laboratory phase is divided into three sections: determining mix designs, preparation of specimens and testing phase. All three are described in the following chapters.

4.1 Mix designs

Before any preparation of specimens could take place, the mix designs for the project had to be decided. Apart from the variation of RAP in the mixtures, other parameters were kept static. This was done in order to allow results to be directly connected to the variation of RAP. It should be noted that no rejuvenators were used in any mix design. Static parameters are presented in Table 1.

Table 1. Static parameters.

Parameter	Value
Desired total asphalt content of mix, [%]	5.20
Binder grade, [-]	PG64-10
Ratio between coarse and fine RAP (if applicable)	50/50
Air void content, [%]	7 ± 0,5

Four different mix designs were created with RAP contents of approximately 0, 25, 40 and 60 % and the properties of each design are presented in Table 2. The average percentage of RAP used in reclaimed pavement projects in the United States is 22,2 % as of 2022 (Williams, et al., 2024). The mix design including 25 % RAP is used to simulate this norm. In California, efforts have been made to increase the allowable RAP content in HMA mixes up to 40 % while maintaining adequate pavement performance (California Department of Transportation, 2019). This is represented by the 40 % RAP mix. 60 % RAP is a notable increase from 40 % and is chosen to give a representation of the performance of mix designs with a very high RAP content, while the 0 % RAP mix acts as a reference point. The sample size for each design is 8000 g, and they were all intended to fulfill the requirements of a ½ ", HMA, Superpave mix (Cominsky, et al., 1994). The mix is functionally comparable to ABT 11 in Swedish standards (Trafikverket, 2003). Percentages are specified by weight.

Table 2. Mix design properties.

Parameter	Design 1, NORAP	Design 2, 25RAP	Design 3, 40RAP	Design 4, 60RAP
Percentage of RAP, [%]	0	24,8	39,7	59,5
Weight of RAP binder, [g]	0	87,7	140,4	210,5
Virgin binder, [%]	5,2	4,1	3,45	2,57
Weight of virgin binder, [g]	416	328,3	275,6	205,5
Weight of RAP, [g]	0	1983,7	3174	4760,9
Virgin aggregate, [%]	95	71,1	57	38
Weight of virgin aggregate, [g]	7584	5688	4550,4	3033,6

It can be noted that the amount of virgin binder added to the mix decreased as RAP content was increased. This was done to ensure that the total binder content of the mix remained at 5,2 %, as increased RAP content implies an increased amount of reclaimed binder.

The aggregate fractions that were used for the designs were 5/8 ", 3/8 " and crusher dust. 5/8 " and 3/8 " equal 15,875 mm and 9,525 mm respectively. RAP was divided into two fractions, referred to as coarse and fine RAP. Detailed information regarding the RAP material was not available to the author at the time of this study. Table 3 showcases the weight of each fraction in each of the mixes. Sample size remained 8000 g.

Table 3. Aggregate fractions of each mix.

Fraction	Design 1, NORAP	Design 2, 25RAP	Design 3, 40RAP	Design 4, 60RAP
5/8 ", [g]	2123,5	1668,5	1213,4	834,2
3/8 ", [g]	2047,7	1516,8	1441	1061,8
Crusher dust, [g]	3412,8	2502,7	1896	1137,6
RAP, [g]	0	1983,7	3174	4760,9

4.2 Preparation of specimens

4.2.1 Batching

With finalized mix designs, preparation of specimens began. The process began with bulk batching, according to ASTM C702/C702M, which involved preparing the required amount of aggregate for each mix (ASTM International, 2024). This was performed for each aggregate fraction using riffle splitters of various sizes. A riffle splitter is a tool which allows an aggregate sample to be separated into two equal, representative sub-samples. Since all aggregate fractions contain a variety of particle sizes it was important that the balance of particles in a sample remained constant throughout the process of batching. This method of measuring aggregate produced consistent results whilst minimizing human bias and segregation of particle sizes. After each use of the riffle splitter, a brush was used to ensure that fine particles did not accumulate on the surface of the apparatus. This tool was also used in order to acquire a 50/50 ratio between the two RAP fractions. The riffle splitters used are shown in the figures below.

Figure 1. Large riffle splitter.

Figure 2. Small riffle splitter.

4.2.2 Mixing

When all aggregate and RAP had been batched, mixing took place. In this process, aggregate and virgin binder along with tools required for mixing were heated in an oven to 310°F, or approximately 154°C. RAP was heated to 140°F or approximately 60°C. When aggregate is heated to this temperature the materials dries, resulting in evaporation of water. This in turn may lead to some fine particles becoming airborne, resulting in minor mass loss of fine particles in the aggregate. To compensate for this loss, additional fine particles were added to ensure that desired mass requirements are met. The added aggregate met the requirement of passing through the No.30 sieve and thereby had a particle size of less than 0,6 mm in diameter.

When all components had been heated to the required temperature, mixing began. To a heated mixing bowl, aggregate, potential RAP, fine particles and binder were added according to the predetermined mix design. A rotating drum mixer mixed the sample for two minutes, with additional manual mixing taking place at the same time to ensure that the mixture didn't accumulate on the sides of the bowl. Mixing occurred two times with a 20-minute span between each session, reheating the mix between sessions. This ensured that each sample was adequately mixed. After this had been completed, the mixture was cured in room temperature, approximately 21°C, overnight to allow for a homogenous bitumen coating of aggregate. Figure 3 illustrates freshly mixed asphalt, ready to be cured.

Figure 3. Freshly mixed asphalt.

4.2.3 Ensuring void content

To ensure that the desired void content of 7 ± 0,5 % was met, two tests were performed. A Rice test (ASTM D2041) was done in order to acquire the theoretical maximum specific gravity of a mixture, G_{mm} (ASTM International, 2010). Furthermore, a Saturated Surface Dry test (SSD) (ASTM D2726) was performed to obtain the bulk specific gravity of a specimen, G_{mb} (ASTM International, 2017). These were then combined to calculate void content according to the equation below. Results are shown in Appendix A.

$$VA = \frac{G_{mm} - G_{mb}}{G_{mm}} \cdot 100$$

where:

VA = air voids, [%],

G_{mm} = maximum specific gravity of the asphalt mixture, [-], and

G_{mb} = bulk specific gravity of the specimen, [-].

4.2.3.1 Rice test

To perform a Rice test, an asphalt sample of 1500 g was heated to a workable temperature. Using heat protective gloves, large aggregate particles were then separated from small particles by hand with the aim of minimizing clusters in the sample. The sample was then set aside until it reached room temperature, approximately 21˚C. See Figure 4. After the sample had been cooled, it was added to a bowl. Water with a temperature of 25 ± 1˚C was then added until the sample was fully submerged. A lid was put on top of the bowl which was then anchored to an agitation device and air was removed until vacuum was achieved. See Figure 5. Once vacuum was achieved, the sample was agitated for 15 ± 2 min. The vacuum was then slowly released, lid was removed, and the sample along with the bowl was weighed in water. Mass was determined after 10 ± 1 min. The maximum specific gravity of the asphalt mixture was then calculated according to the following equation:

$$G_{mm} = \frac{A}{A - (C - B)}$$

where:

G_{mm} = maximum specific gravity of the asphalt mixture, [-],

A = mass of dry sample in air, [g],

B = mass of bowl under water, [g], and

C = mass of bowl and sample under water, [g].

A Rice test is performed for each of the four mix designs.

Figure 4. Sample prepared for Rice test.

Figure 5. Vacuum apparatus connected to an agitation device.

4.2.3.2 Saturated Surface Dry test

The SSD, or saturated surface dry test, was performed after asphalt briquettes had been compacted, see chapter 4.2.4. The test was performed by first measuring the mass of the dry specimen. The specimen was then submerged in 25 ± 1°C water for 4 ± 1 min. Weight in water was measured and the specimen was then removed from the water and surface dried by blotting with a damp towel. The mass of the saturated surface-dry specimen was then measured. The bulk specific gravity was calculated as follows:

$$G_{mb} = \frac{A}{(B - C)}$$

where:

G_{mb} = bulk specific gravity of the specimen, [-],

A = mass of the dry specimen in air, [g],

B = mass of the saturated surface-dry specimen in air, [g], and

C = mass of the specimen in water, [g].

4.2.4 Compacting

For performing the SSD test, two test specimens per mix design were prepared with masses of approximately 2500 g, differing 20-30 g in relation to one another. By doing this, a general understanding of the relationship between the mass of a specimen and its air void content was obtained. If the air void content of a specimen met the requirement of 7 ± 0,5 %, according to chapter 4.2.3, the same mass could be used when compacting the remainder of the specimens. If not, masses of the remaining specimens were adjusted. Decreasing the mass allows for a larger VA, and vice versa.

When the desired mass of each specimen had been determined, the cured asphalt was reheated to a temperature of 286°F, or approximately 141°C. During the compacting process, a gyratory compactor was used, see Figure 6. Compaction followed ASTM D6925 (ASTM International, 2024). The mold that was used for this process was heated along with the asphalt to avoid the mixture losing heat and as a result its workability during compacting. The specimen height was set to 62 mm, the diameter to 150 mm and the angle of gyration was set to 1,16°. The number of gyrations required to meet the height requirement of 62 mm was recorded for each specimen and is presented in Appendix A. When this requirement was met, a five-minute cooldown period began, during which the fan installed on the gyratory compactor cooled the specimens. This allowed the briquettes to stabilize before they were removed from the apparatus and marked according to the amount of RAP present in the mixture. In total, 20 briquettes per mix design were produced, resulting in a total of 80 specimens. This excludes the eight total test briquettes that were also compacted. The briquettes were cured overnight at room temperature, approximately 21°C, before testing.

Figure 6. Gyratory compactor.

Figure 7. Mold used for compacting.

Figure 8. Compacted specimens.

4.3 Testing of specimens

Prior to any mechanical testing, an SSD test was performed on all specimens, see Appendix A. The tests that were later conducted included investigations of CTI (ASTM D8225) and RTI (ASTM D8360) (ASTM International, 2019; ASTM International, 2022). Both tests were performed in the same universal testing machine, Humbolt electromechanical load frame, see Figure 9 and Figure 10. Both IDEAL-CT as well as IDEAL-RT tests were performed at two different temperatures per mix design. Temperatures for each mix design are presented in Table 4.

Table 4. Temperatures of specimens with regards to test type.

Specimen nr.	Temp, °C	Test type
1-5	5	IDEAL-CT
6-10	35	IDEAL-CT
11-15	35	IDEAL-RT
16-20	65	IDEAL-RT

In order for specimens to reach temperatures of 35°C and 65°C, water baths were used, see Figure 11. According to test standards, specimens prepared for IDEAL-RT testing should be submerged for 45 ± 5 min. IDEAL-CT testing on the other hand requires the specimen to be submerged in a water bath for 2 hours ± 10 min. To achieve specimens with a temperature of 5°C, they were placed in a cooling chamber for 3-4 hours. When all specimens had reached desired temperatures, testing began by placing one briquette at a time onto the load frame after being surface dried. The machine measured load and deformation over time and calculated either CTI or RTI, depending on what test was being run. It is important to note that the machine gathers data throughout the entirety of the testing period, including unloading of the specimen. This means that if the specimen breaks in two before unloading is completely finished, it is not possible to calculate CTI nor RTI.

Figure 9. IDEAL-CT apparatus.

Figure 10. IDEAL-RT apparatus.

Figure 11. Water bath.

5 Results

5.1 Cracking Tolerance Index

IDEAL-CT tests have been performed for every mix design at 5°C as well as 35°C. Results of CTI testing with a calculated average of five specimens are presented in Table 5 and Table 6 below. Cells that are missing a numerical value indicate specimens for which no CTI was able to be calculated due to the briquette breaking too quickly, hindering data gathering during the unloading phase of the test. This phenomenon only occurs for certain specimens at the 5°C range and the assumption that the specimen has minimal resistance to cracking is made. Figure 12 and Figure 13 show visual representations of average CTI values, including trend lines.

Table 5. Specimens tested for CTI at 5°C.

Specimen number	NORAP	25RAP	40RAP	60RAP
1	8,9	3,4	-	-
2	7,3	4,8	-	-
3	6,5	-	-	-
4	7,1	4,3	-	-
5	7,4	2,4	-	-
Average	**7,4**	**3,7**	**-**	**-**

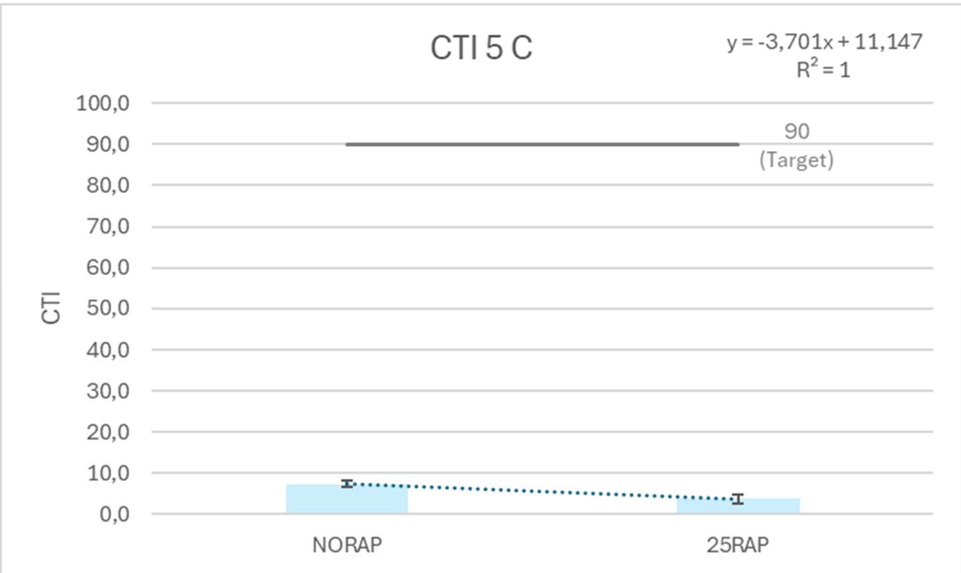

Figure 12. Visual representation of specimens tested for CTI at 5°C.

It can be seen that the CTI for specimens with a temperature of 5°C has little to no resistance to cracking. The highest resistance can be found in specimens where RAP is not a part of the mix design. At 25 % RAP, CTI is possible to be calculated for four out of five specimens. Data collection is not possible for mixes with 40 and 60 % RAP due to the phenomenon explained earlier.

Table 6. Specimens tested for CTI at 35°C.

Specimen number	NORAP	25RAP	40RAP	60RAP
6	251,8	188,7	112,7	65,6
7	226,8	237,1	117,7	40,0
8	266,3	220,2	123,8	89,5
9	185,8	230,3	115,7	56,0
10	173,9	173,3	106,0	68,2
Average	**220,9**	**209,9**	**115,2**	**63,8**

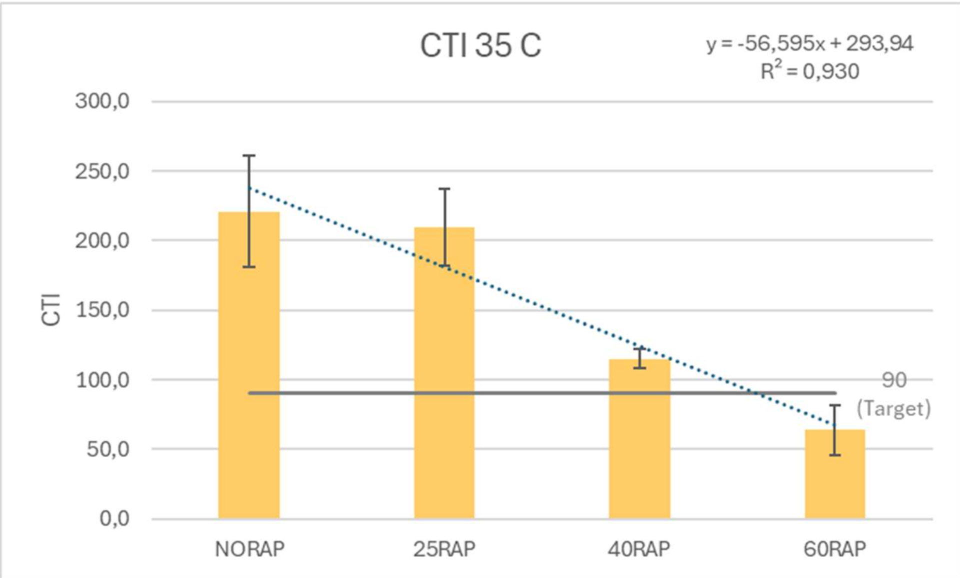

Figure 13. Visual representation of specimens tested for CTI at 35°C.

As for the case where the temperature of specimens is set to 35°C, data can be collected for all briquettes. This shows that an increase in temperature of a specimen, from 5 to 35°C, results in increased resistance to cracking. Furthermore, although the data at 5°C can be considered insufficient, a similar trend can be displayed more pronounced at 35°C. The trend shows that increasing the amount of RAP in the design directly correlates to a reduction of CTI. Table 7 shows, on average, how much of a mixtures original CTI is lost when adding RAP.

Table 7. Average CTI decrease when adding RAP to a mix compared to no RAP.

Temperature, °C	25RAP	40RAP	60RAP
5	- 50%	-	-
35	- 5%	- 48%	- 71%

5.2 Rutting Tolerance Index

To analyze how RTI is affected, IDEAL-RT tests have been performed at 35˚C and 65˚C. All specimens were able to contribute to data gathering and results are presented in Table 8 and Table 9 below. Figure 14 and Figure 15 visualize visible trends within the data.

Table 8. Specimens tested for RTI at 35˚C.

Specimen number	NORAP	25RAP	40RAP	60RAP
11	180,9	235,9	353,0	430,9
12	191,2	268,9	340,3	411,5
13	221,1	248,6	350,6	455,5
14	202,1	250,6	358,2	418,5
15	209,6	262,7	342,4	441,6
Average	**201,0**	**253,3**	**348,9**	**431,6**

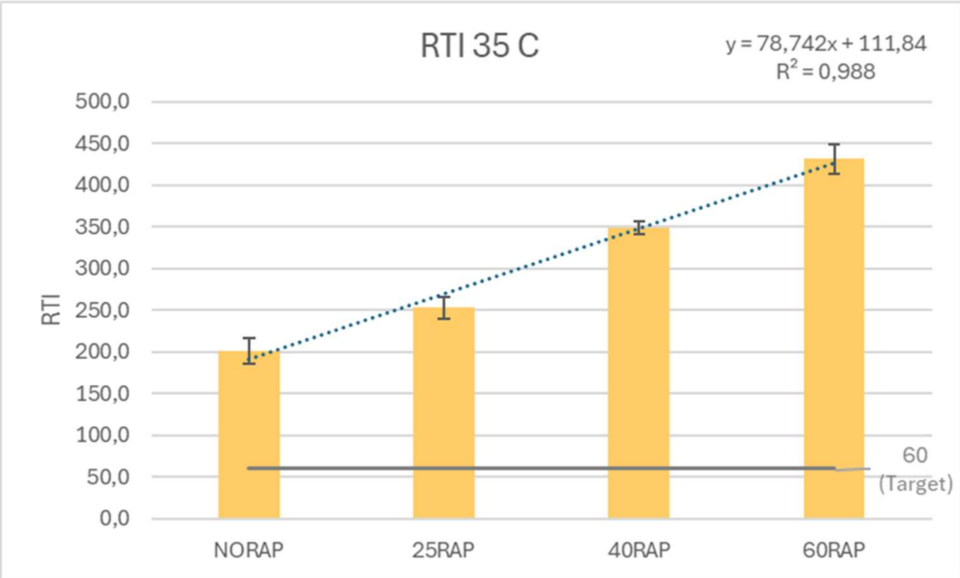

Figure 14. Visual representation of specimens tested for RTI at 35˚C.

When performing IDEAL-RT at a relatively low temperature, 35˚C, a large coefficient of determination, $R^2 = 0{,}988$, insinuates a strong relationship between increased RAP content and increased RTI.

Table 9. Specimens tested for RTI at 65°C.

Specimen number	NORAP	25RAP	40RAP	60RAP
16	46,2	49,3	64,6	79,0
17	49,2	46,4	67,1	98,9
18	48,2	50,0	65,4	91,0
19	45,1	49,4	64,2	88,9
20	46,1	47,4	68,2	98,0
Average	**46,9**	**48,50**	**65,9**	**91,2**

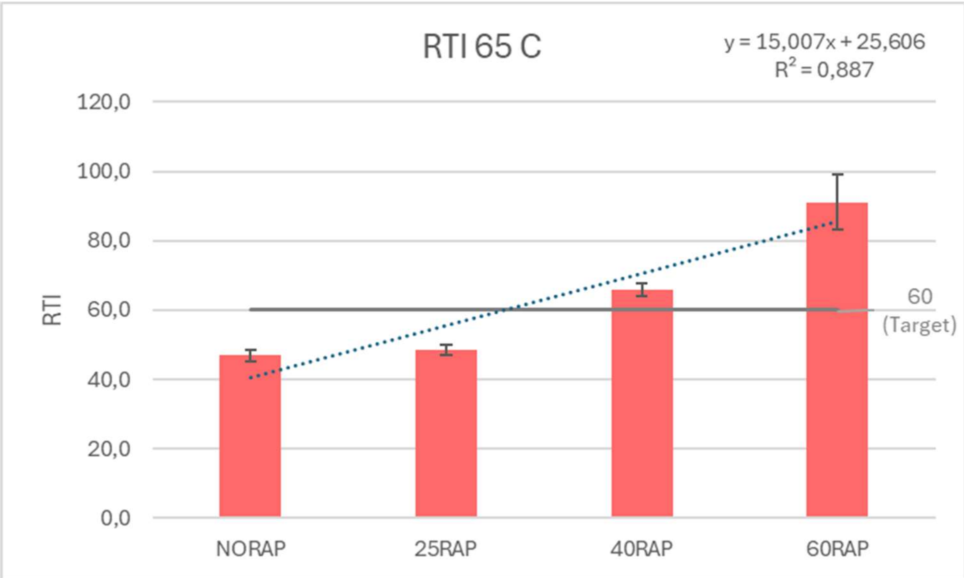

Figure 15. Visual representation of specimens tested for RTI at 65°C.

In the case where specimens are heated to a temperature of 65°C, it can be observed that RTI values decrease compared to the same mix design at 35°C. Further, although less prominent, the earlier mentioned relationship between RAP content and RTI is still valid in conditions where temperatures are elevated, 65°C in this case. Table 10 shows, on average, how much RTI increases when RAP is added to a mixture, compared to 0 % RAP.

Table 10. Average RTI increase when adding RAP to a mix compared to no RAP.

Temperature,°C	25RAP	40RAP	60RAP
35	+ 26%	+ 74%	+ 115%
65	+ 3%	+ 40%	+ 94%

5.3 Control of outliers in data

T_n is calculated for all data points according to ASTM E178, see chapter 3.4 (ASTM International, 2021). Results are presented in Table 11 where blue represents 5°C, orange 35°C and red 65°C.

Table 11. T_n for all specimens.

	NORAP	25RAP	40RAP	60RAP		NORAP	25RAP	40RAP	60RAP
Spec. \ s	**1,039**	**1,039**	**-**	**-**	**Spec. \ s**	**15,652**	**12,896**	**7,468**	**17,640**
1	1,371	-0,294	-	-	11	-1,283	-1,355	0,551	-0,038
2	-0,102	1,035	-	-	12	-0,623	1,205	-1,151	-1,138
3	-0,872	-	-	-	13	1,287	-0,366	0,227	1,353
4	-0,323	0,515	-	-	14	0,068	-0,215	1,247	-0,744
5	-0,073	-1,256	-	-	15	0,552	0,730	-0,875	0,566
Spec. \ s	**40,269**	**27,608**	**6,559**	**18,080**	**Spec. \ s**	**1,683**	**1,518**	**1,719**	**8,035**
6	0,766	-0,770	-0,380	0,098	16	-0,461	0,506	-0,763	-1,509
7	0,146	0,983	0,388	-1,318	17	1,335	-1,356	0,701	0,958
8	1,127	0,373	1,315	1,417	18	0,724	0,993	-0,272	-0,018
9	-0,872	0,739	0,080	-0,436	19	-1,117	0,602	-1,015	-0,282
10	-1,167	-1,325	-1,403	0,239	20	-0,482	-0,745	1,350	0,851

Since no observation exceeds the critical test criterion values of |1,425| for four observations and |1,602| for five observations, no data points need to be removed from the population.

6 Discussion

The following chapter discusses results and methodology, critically reviewing the credibility and reliability of the two. The results are compared with findings from the literature review to assess the extent to which they align.

6.1 Discussion of results

Based on the data obtained through laboratory testing, it is possible to discover a number of trends related to RAP content and temperature. Consistent throughout all CTI results is that the resistance to cracking is strongly influenced by the amount of RAP present in the mix design. It can be concluded that involving a larger percentage of recycled material in an asphalt mix design reduces its ability to withstand cracks when being exposed to loads. This can be seen at both 5°C as well as 35°C. It is evident that CTI is strongly associated with the temperature of which specimens are tested at, with lower temperature insinuating lower resistance to cracking. It should be noted that CTI results for 0 and 25 % RAP at 35°C overlap when considering standard deviation, suggesting no clear separation between the two mixtures. See Figure 13.

When analyzing RTI data, an opposite trend to CTI can be discovered. In this case, resistance to rutting is increased by the addition of RAP. The high coefficient of determination at 35°C, $R^2 = 0{,}988$, suggests a clear relationship between the addition of RAP and increased RTI, see Figure 14. For 65 °C, a linear relationship is less prominent. When considering standard deviation, overlap between mixtures with 0 and 25 % RAP suggests no evident separation between the two, see Figure 15. Furthermore, it can also be concluded that resistance to rutting is decreased as temperatures are elevated.

The visible trends that can be discovered are the following:

1. Increasing RAP content seems to decrease CTI and increase RTI.
2. Elevated temperatures seem to be more beneficial for CTI and less favorable for RTI.

The first trend is believed to be caused by the fact that the mix becomes stiffer along with the addition of RAP. When adding recycled material, binder grade, binder content and the gradation of aggregate are all affected. The change in binder grade affects the stiffness due to oxidized binder being added (McDaniel, 2009). This addition in turn affects binder content since it is possible that parts of the aged bitumen won't become mobilized within the mix, contributing to increased stiffness (Chiara Cavalli, et al., 2017; Rizk, et al., 2023). Furthermore, the incorporation of RAP may affect the gradation of aggregate in the mix, potentially compromising VMA and in turn further increasing the stiffness of the specimen (Copeland, 2011). Additionally, an increase in dust-binder ratio reduces the thickness of the bitumen layer coating aggregate particles, compromising flexibility (Coleri, et al., 2018). Findings in the literature review therefore support the claim that susceptibility to cracking may be compromised by the addition of RAP since the mix becomes stiffer. As opposed to susceptibility to cracking, the risk of rutting is decreased as pavement becomes stiffer and as a result less flexible. This relationship might make recycled pavement more viable than virgin materials when trying to reduce deformation in pavement.

The second trend relates to the temperature dependency of asphalt. As temperature increases, the binder is expected to see a reduction in viscosity. If the softening point of the bitumen is exceeded, greater shear stress within the pavement can be measured, resulting in increased risk of rutting (Yang, et al., 2025). Conversely, decreasing the temperature increases stiffness and brittleness of the pavement, promoting cracking at these conditions (Guo, et al., 2025). Therefore, findings of literature review seem to align with the visible trend.

In order to find an ideal amount of RAP to be added to a mix design to balance both cracking and rutting properties of pavement, the average values of CTI and RTI are analyzed. As mentioned earlier, there are no minimum requirements for neither CTI nor RTI, although there are recommended minimum values. A minimum CTI of 90 as well as a minimum RTI of 60 will be used in the further analysis. Starting with adapting RAP content to the climate of southern Sweden, it is evident that the risk of cracking in wintertime will likely be more severe than the risk of rutting in summertime. When conducting laboratory testing, the same binder was used for all mix designs, PG64-10. This binder is an alternative for mid-range temperatures but evidently sees flaws when used at lower temperatures. Since binder grade seems to have a large impact on CTI at low temperatures, it is difficult to draw any conclusions about ideal RAP content at 5°C using PG64-10. Should a more flexible binder be used, resistance to cracking would likely increase at the cost of reduced rutting resistance. In Sweden 70/100 binder is a common alternative. Since Sweden and the United States use two different methods for binder grading, it is difficult to convert one unit to the other. In laboratory environments, however, loose relationships between the two standards can be constructed through different tests. Based on such, 60/70 binder can be classified as PG64-16, whist 85/100 binder can be compared to PG58-16 (Ghuzlan & Al-Khateeb, 2012). It is therefore reasonable to assume that 70/100 binder falls within the interval of PG58-16 to PG64-16. If this is the case, and one of the mentioned binder grades would be used in the mix design for testing at 5°C, it would likely be possible to make assumptions about the ideal amount of RAP. Since this is not the case, it can only be concluded that using a binder grade of PG64-10 is unsuitable for south Swedish winter climate. This is likely the case for the entirety of the country. As for RTI, all mix designs show very positive results regarding resistance to rutting. The lowest rutting tolerance indices are obtained from specimens without RAP, as expected, with an average RTI of over 200. This is more than three times that of the recommended minimum RTI of 60. Adding more RAP to the design only improves rutting performance, leaving cracking at low temperatures as the primary limiting factor for RAP incorporation. Since RTI of all mix designs is relatively high, substituting the binder in the mix to a slightly softer one would likely not harm rutting resistance enough for it to fall below 60. This depends on the grade of the new binder.

When it comes to northern California, elevated temperatures result in the risk of rutting being the critical parameter, rather than cracking. The difference between the two is, however, not as significant as it is at lower temperatures. In this case, IDEAL-CT tests are performed under the assumption that cracking will be load-induced under warm conditions, rather than being related to cracking at low temperatures. Therefore, tests are performed at 35°C, in relation to the previous 5°C. At this temperature, specimens with 0 and 25 % RAP have cracking tolerance indices of over 200. Increasing the amount of RAP to 40 % severely reduces cracking resistance and gives a CTI of approximately 115. This is still above the recommended minimum CTI of 90. However, substituting 60 % of virgin materials with RAP gives an average CTI of approximately 64, which falls below the threshold. This means that the maximum amount of RAP that should be included in a mix design, with regards to cracking at high temperatures, is somewhere between 40 and 60 %. Evaluating rutting performance on the other hand, an increase in temperature to 65˚C heavily reduces rutting resistance. Rutting tolerance indices for 0 and 25 % RAP designs fall below 50 and consequently also below the threshold of 60. Increasing RAP content to 40 % improves RTI to approximately 66 and 60 % RAP results in an RTI of about 91. It is therefore reasonable to assume that the RAP content of the design, with regards to rutting resistance, should be minimum around 40 %. The minimum RAP content to achieve adequate RTI happens to fall within range of the maximum RAP content to achieve adequate CTI. Therefore, from this data, it can be concluded that the ideal RAP content to be used in Redding is somewhere between 40 and 60 %, using the predetermined mix designs and assuming that critical cracking is load-induced at warm temperatures.

6.2 Methodological discussion

The most prominent flaw in the execution of the laboratory phase is that not all cracking tolerance indices could be obtained through testing at 5˚C. As discussed in the previous chapter, a possible solution to this problem could be to substitute the binder in the mix design to one that provides more flexibility at lower temperatures. PG64-10 binder is used for all specimens with the intention of keeping as many parameters static as possible. By doing this, it is easier to find direct correlations between RAP content and mechanical properties of different mix designs. If a softer binder were to be used when simulating Swedish climate, more realistic and complete data would likely be obtained. However, by doing this, results of the simulated Californian climate would not be as easily comparable with results in Swedish climate due to varying parameters.

In the batching phase, the bulk method was used. This means that aggregates are batched according to chapter 4.2.1 using samples with varying particle size within every aggregate fraction. The alternative to this would be individual batching, where all aggregate of particle sizes is individually weighed and added to a pan. This ensures very consistent results but is more time consuming than the bulk method. Although there is slight variation of results within each set of five specimens, it is not considered to have a large enough impact on the final results for individual batching to be considered as an alternative. Furthermore, batching using the bulk method is arguably more accurate to how batching would be performed in the field, leading to results that are more representative of reality.

As outlined in the delimitations, the study is restricted to testing cracking and rutting resistance through CTI and RTI testing. This method of evaluating mechanical behavior gives an indication of the performance of different mixtures. However, there are a number of other mechanics that may affect the actual performance of a mixture in the field, such as moisture. If more tests were to be performed, the number of specimens needed would increase significantly and the required time consumption would likely exceed the predetermined laboratory time. The same reasoning can be used when discussing the fact that only four different mixes and temperatures were tested. Increasing the number of mixes as well as temperatures of which specimens are tested would likely result in a more precise estimation of the ideal amount of RAP that should be included in mixes in different parts of the world.

In this study, no mix design included the use of additives, such as hydrated lime, polymer additives or rejuvenators. The result of this was an understanding of how a rather simple mix design might perform with regards to cracking and rutting. Had any of these additives been used in the mix designs results would likely have varied, changing the mechanical performance of the pavement both at low and elevated temperatures. Adding polymer additives to the mix design would likely have improved both CTI as well as RTI performance by enhancing the viscoelastic behavior of the binder. Increased elasticity and decreased brittleness may have contributed to improved resistance to cracking. However, increased stiffness and elastic recovery at elevated temperatures could have reduced susceptibility to rutting. These effects may have been particularly beneficial for mixtures containing high RAP contents due to the stiff nature of RAP binder. Similarly, the addition of lime could potentially have improved RTI performance through increased binder stiffness and enhanced adhesion between binder and aggregate. Furthermore, hydrated lime may have contributed to improved CTI performance by reducing brittleness at low temperatures. Moreover, age hardening as well as moisture susceptibility might also be reduced, thereby limiting embrittlement of the asphalt mixture over time. The latter may not be visible in the lab environment but might be of importance in the field. The addition of rejuvenators would likely have improved CTI capabilities by reducing binder viscosity and restoring flexibility of the aged RAP binder. Further, rebalancing the colloidal structure and restoration of maltene content may have reduced brittleness and susceptibility to cracking, particularly at low temperatures. However, the softening effect of rejuvenators may simultaneously have reduced stiffness and deformation resistance at elevated temperatures, potentially reducing RTI performance. The actual magnitude of these effects would depend on additive dosage, binder composition, RAP content and interaction effects within the asphalt mixture. The intention of the testing was however not to optimize results straight away, but rather to gain understanding of how simple mix designs would perform. Further development regarding implementation of potential additives might increase the allowable RAP content.

6.3 Observations during mixing

During mixing, mixtures containing higher RAP content generally appeared stiffer and less workable compared to the virgin mixture. Greater effort was required to maneuver the mixture when handling it as RAP content increased. No significant segregation or incomplete coating was visually observed after mixing.

7 Conclusions and recommendations

Conclusions are drawn with regard to the research questions of the master's thesis and recommendations of future studies on the subject are made.

7.1 Conclusions

The following conclusions can be drawn with respect to the research questions of the master's thesis.

1. **How is Cracking Tolerance Index (CTI) of an asphalt mix affected by variation of RAP content at low and high temperatures?**

   The Cracking Tolerance Index (CTI) can be expected to consistently decrease with increasing RAP content at both low and high temperatures, 5 and 35°C, with overall lower CTI values expected to be observed at low temperatures, 5°C. The relationship between RAP content and CTI is less prominent at 5°C due to a lack of data.

2. **How is Rutting Tolerance Index (RTI) of an asphalt mix affected by variation of RAP content at low and high temperatures?**

   As RAP content increases, Rutting Tolerance Index (RTI) can be expected to increase accordingly at both low and high temperatures, 35 and 65°C. At elevated temperatures, 65°C, RTI can be anticipated to be compromised.

3. **Is there an optimal range of RAP content, balancing CTI and RTI, that can be used in colder and warmer climates respectively?**

   For warmer climates, such as northern California, the ideal amount of RAP to be included in a mix design can be estimated to be somewhere between 40 and 60 %. This assumes that cracking of the pavement is load-induced and occurring at warm temperatures, 35°C. For colder climates, such as southern Sweden, a similar RAP range can likely be estimated if laboratories are performed again, editing mix designs and using a softer binder grade than PG64-10. Such estimations cannot be made based on the data gathered in the laboratory phase of this master's thesis.

7.2 Recommendations

Because the usage of recycled materials is advantageous in many ways, identifying optimal RAP ranges in different climates can be considered to be an important step for the progression of the pavement industry. It is important to note that this study only focuses on resistance to cracking and rutting with regards to CTI and RTI. For further studies, it would be appropriate to broaden the range of tests to be performed, which would likely result in data that is more representative of a mix design’s actual resistance to cracking and rutting respectively. Furthermore, performing tests at a wider range of temperatures as well as RAP contents would result in a deeper understanding of how much RAP should be used in different scenarios. In the study performed, no rejuvenation techniques were used. If further research should be conducted, the incorporation of rejuvenating agents would be of interest to see how the stiffness of the mix is affected. Moreover, implementation of additives, such as hydrated lime and polymer additives, would be of interest to potentially improve the mechanical performance of the pavement at low and elevated temperatures respectively. Finally, field validation studies of laboratory findings are recommended to confirm pavement behavior under traffic and environmental conditions.

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9 Appendix A

INDIRECT TENSILE ASPHALT CRACKING TEST (IDEAL-CT)

ASTM D8225

5C	Sample Test Temperature

Compaction Parameters	
Method	D6925
Temperature	286°F
Pressure	600 kPa
Angle	1.16°

	No RAP					**25% RAP**				
	Specimen 1	**Specimen 2**	**Specimen 3**	**Specimen 4**	**Specimen 5**	**Specimen 1**	**Specimen 2**	**Specimen 3**	**Specimen 4**	**Specimen 5**
Specimen Diameter (mm)	150	150	150	150	150	150	150	150	150	150
Specimen Thickness (mm)	62	62	62	62	62	62	62	62	62	62
Gyrations	89	129	83	89	79	76	64	74	48	58
Weight of Specimen in Air	2443,8	2444,0	2443,8	2443,0	2444,7	2447,1	2446,4	2447,3	2447,7	2445,5
Weight of wet specimen in Air	2456,5	2453,1	2449,3	2449,8	2451,5	2454,5	2453,5	2452,4	2454,2	2455,1
Weight of specimen in Water	1399,8	1399,8	1396,1	1397,7	1397,8	1408,4	1405,5	1405,7	1408,2	1408,6
Maxium Specific Gravity (Gmm)	2,490	2,490	2,490	2,490	2,490	2,506	2,506	2,506	2,506	2,506
Bulk Specific Gravity (Gmb)	2,313	2,320	2,320	2,322	2,320	2,339	2,334	2,338	2,340	2,337
Air Voids (%)	**7,12%**	**6,81%**	**6,81%**	**6,75%**	**6,82%**	**6,65%**	**6,85%**	**6,70%**	**6,62%**	**6,75%**

	40% RAP					**60% RAP**				
	Specimen 1	**Specimen 2**	**Specimen 3**	**Specimen 4**	**Specimen 5**	**Specimen 1**	**Specimen 2**	**Specimen 3**	**Specimen 4**	**Specimen 5**
Specimen Diameter (mm)	150	150	150	150	150	150	150	150	150	150
Specimen Thickness (mm)	62	62	62	62	62	62	62	62	62	62
Gyrations	35	38	33	30	47	21	29	22	26	30
Weight of Specimen in Air	2447,7	2447,1	2447,8	2446,8	2447,0	2432,8	2431,4	2432,3	2432,0	2432,1
Weight of wet specimen in Air	2457,5	2456,0	2459,7	2458,5	2457,7	2450,6	2449,6	2449,6	2449,1	2448,6
Weight of specimen in Water	1407,6	1410,1	1409,9	1406,7	1410,8	1410,3	1412,5	1410,9	1412,0	1409,9
Maxium Specific Gravity (Gmm)	2,509	2,509	2,509	2,509	2,509	2,520	2,520	2,520	2,520	2,520
Bulk Specific Gravity (Gmb)	2,331	2,340	2,332	2,326	2,337	2,339	2,344	2,342	2,345	2,341
Air Voids (%)	**7,08%**	**6,75%**	**7,07%**	**7,28%**	**6,84%**	**7,20%**	**6,97%**	**7,08%**	**6,94%**	**7,08%**

INDIRECT TENSILE ASPHALT CRACKING TEST (IDEAL-CT)

ASTM D8225

35C	Sample Test Temperature

Compaction Parameters	
Method	D6925
Temperature	286°F
Pressure	600 kPa
Angle	1.16°

	No RAP					25% RAP				
	Specimen 6	**Specimen 7**	**Specimen 8**	**Specimen 9**	**Specimen 10**	**Specimen 6**	**Specimen 7**	**Specimen 8**	**Specimen 9**	**Specimen 10**
Specimen Diameter (mm)	150	150	150	150	150	150	150	150	150	150
Specimen Thickness (mm)	62	62	62	62	62	62	62	62	62	62
Gyrations	87	80	74	80	89	45	40	65	50	49
Weight of Specimen in Air	2445,7	2445,4	2444,8	2444,7	2446,0	2448,5	2448,6	2446,9	2447,3	2447,1
Weight of wet specimen in Air	2451,6	2452,0	2451,8	2452,7	2455,2	2456,1	2456,4	2455,7	2455,0	2452,8
Weight of specimen in Water	1400,0	1400,8	1397,2	1400,9	1404,0	1409,7	1409,1	1404,3	1405,8	1406,3
Maxium Specific Gravity (Gmm)	2,490	2,490	2,490	2,490	2,490	2,506	2,506	2,506	2,506	2,506
Bulk Specific Gravity (Gmb)	2,326	2,326	2,318	2,324	2,327	2,340	2,338	2,327	2,333	2,338
Air Voids (%)	**6,60%**	**6,57%**	**6,90%**	**6,65%**	**6,55%**	**6,63%**	**6,70%**	**7,13%**	**6,92%**	**6,69%**

	40% RAP					60% RAP				
	Specimen 6	**Specimen 7**	**Specimen 8**	**Specimen 9**	**Specimen 10**	**Specimen 6**	**Specimen 7**	**Specimen 8**	**Specimen 9**	**Specimen 10**
Specimen Diameter (mm)	150	150	150	150	150	150	150	150	150	150
Specimen Thickness (mm)	62	62	62	62	62	62	62	62	62	62
Gyrations	42	36	48	36	34	25	25	28	22	22
Weight of Specimen in Air	2447,8	2447,1	2448,3	2447,5	2447,4	2433,2	2432,8	2433,0	2433,9	2432,3
Weight of wet specimen in Air	2459,2	2457,0	2456,9	2459,1	2460,2	2452,6	2450,8	2449,7	2451,1	2448,0
Weight of specimen in Water	1406,7	1409,8	1411,3	1408,6	1412,9	1410,6	1413,1	1411,6	1413,0	1408,0
Maxium Specific Gravity (Gmm)	2,509	2,509	2,509	2,509	2,509	2,520	2,520	2,520	2,520	2,520
Bulk Specific Gravity (Gmb)	2,326	2,337	2,342	2,330	2,337	2,335	2,344	2,344	2,345	2,339
Air Voids (%)	**7,31%**	**6,86%**	**6,67%**	**7,14%**	**6,86%**	**7,34%**	**6,97%**	**7,00%**	**6,96%**	**7,19%**

RUTTING TOLERANCE INDEX OF ASPHALT MIXTURES (IDEAL-RT)

ASTM D8360

35C	Sample Test Temperature

Compaction Parameters	
Method	D6925
Temperature	286°F
Pressure	600 kPa
Angle	1.16°

	No RAP					25% RAP				
	Specimen 11	**Specimen 12**	**Specimen 13**	**Specimen 14**	**Specimen 15**	**Specimen 11**	**Specimen 12**	**Specimen 13**	**Specimen 14**	**Specimen 15**
Specimen Diameter (mm)	150	150	150	150	150	150	150	150	150	150
Specimen Thickness (mm)	62	62	62	62	62	62	62	62	62	62
Gyrations	82	100	77	72	77	56	53	50	49	54
Weight of Specimen in Air	2445,2	2445,5	2444,1	2446,6	2446,6	2447,1	2447,9	2447,1	2446,4	2446,6
Weight of wet specimen in Air	2452,0	2455,2	2454,8	2455,5	2456,2	2456,8	2454,9	2454,4	2456,7	2455,4
Weight of specimen in Water	1400,7	1404,1	1404,2	1404,0	1403,0	1406,2	1407,6	1409,0	1406,0	1409,0
Maxium Specific Gravity (Gmm)	2,490	2,490	2,490	2,490	2,490	2,506	2,506	2,506	2,506	2,506
Bulk Specific Gravity (Gmb)	2,326	2,327	2,326	2,327	2,323	2,329	2,337	2,341	2,328	2,338
Air Voids (%)	**6,59%**	**6,56%**	**6,57%**	**6,56%**	**6,71%**	**7,05%**	**6,73%**	**6,59%**	**7,09%**	**6,70%**

	40% RAP					60% RAP				
	Specimen 11	**Specimen 12**	**Specimen 13**	**Specimen 14**	**Specimen 15**	**Specimen 11**	**Specimen 12**	**Specimen 13**	**Specimen 14**	**Specimen 15**
Specimen Diameter (mm)	150	150	150	150	150	150	150	150	150	150
Specimen Thickness (mm)	62	62	62	62	62	62	62	62	62	62
Gyrations	51	36	34	42	33	25	29	26	25	29
Weight of Specimen in Air	2447,1	2447,3	2447,5	2447,5	2447,2	2431,9	2431,4	2432,4	2430,8	2432,3
Weight of wet specimen in Air	2457,0	2462,1	2458,7	2458,9	2457,8	2453,5	2450,1	2450,5	2451,9	2454,2
Weight of specimen in Water	1412,2	1412,3	1413,2	1413,5	1408,1	1415,0	1412,9	1411,0	1414,3	1415,2
Maxium Specific Gravity (Gmm)	2,509	2,509	2,509	2,509	2,509	2,520	2,520	2,520	2,520	2,520
Bulk Specific Gravity (Gmb)	2,342	2,331	2,341	2,341	2,331	2,342	2,344	2,340	2,343	2,341
Air Voids (%)	**6,65%**	**7,09%**	**6,70%**	**6,69%**	**7,08%**	**7,07%**	**6,98%**	**7,14%**	**7,04%**	**7,10%**

RUTTING TOLERANCE INDEX OF ASPHALT MIXTURES (IDEAL-RT)

ASTM D8360

65C	Sample Test Temperature

Compaction Parameters	
Method	D6925
Temperature	286˚F
Pressure	600 kPa
Angle	1.16°

	No RAP					25% RAP				
	Specimen 16	**Specimen 17**	**Specimen 18**	**Specimen 19**	**Specimen 20**	**Specimen 16**	**Specimen 17**	**Specimen 18**	**Specimen 19**	**Specimen 20**
Specimen Diameter (mm)	150	150	150	150	150	150	150	150	150	150
Specimen Thickness (mm)	62	62	62	62	62	62	62	62	62	62
Gyrations	77	80	93	67	92	56	49	41	48	43
Weight of Specimen in Air	2445,0	2443,6	2444,5	2446,4	2444,2	2446,7	2447,0	2446,9	2447,2	2446,6
Weight of wet specimen in Air	2452,9	2454,6	2454,5	2454,0	2456,1	2454,0	2457,0	2454,0	2454,9	2456,3
Weight of specimen in Water	1401,8	1403,5	1402,7	1401,9	1404,6	1406,0	1405,8	1406,2	1409,7	1403,8
Maxium Specific Gravity (Gmm)	2,490	2,490	2,490	2,490	2,490	2,506	2,506	2,506	2,506	2,506
Bulk Specific Gravity (Gmb)	2,326	2,325	2,324	2,325	2,324	2,335	2,328	2,335	2,341	2,325
Air Voids (%)	**6,58%**	**6,63%**	**6,66%**	**6,62%**	**6,65%**	**6,84%**	**7,11%**	**6,81%**	**6,57%**	**7,24%**

	40% RAP					60% RAP				
	Specimen 16	**Specimen 17**	**Specimen 18**	**Specimen 19**	**Specimen 20**	**Specimen 16**	**Specimen 17**	**Specimen 18**	**Specimen 19**	**Specimen 20**
Specimen Diameter (mm)	150	150	150	150	150	150	150	150	150	150
Specimen Thickness (mm)	62	62	62	62	62	62	62	62	62	62
Gyrations	36	42	36	40	40	24	34	30	28	29
Weight of Specimen in Air	2447,8	2448,2	2447,9	2447,7	2446,9	2433,6	2433,0	2432,9	2432,5	2432,2
Weight of wet specimen in Air	2464,3	2458,4	2460,9	2461,1	2459,6	2451,8	2450,8	2452,7	2452,6	2453,6
Weight of specimen in Water	1414,5	1410,8	1414,5	1415,1	1415,3	1412,7	1412,6	1412,7	1413,6	1414,5
Maxium Specific Gravity (Gmm)	2,509	2,509	2,509	2,509	2,509	2,520	2,520	2,520	2,520	2,520
Bulk Specific Gravity (Gmb)	2,332	2,337	2,339	2,340	2,343	2,342	2,343	2,339	2,341	2,341
Air Voids (%)	**7,07%**	**6,86%**	**6,76%**	**6,73%**	**6,61%**	**7,06%**	**7,00%**	**7,17%**	**7,10%**	**7,12%**