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Effect of polymer modified bitumen on deformation characteristics of low-traffic asphalt pavements

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Abstract:

Low-volume roads with an AADT less than 3000 make up 90 % of the Norwegian road network. There are many deterioration mechanisms which affect the Norwegian road network, and permanent deformation and rutting in the asphalt pavement caused by heavy traffic loads and climatic conditions is one of the most substantial types of distress on low-volume roads. Polymer modification of bitumen has been shown to improve the stability of asphalt pavements and significantly reduce rutting. This study investigates the effect polymer modified bitumen has on the deformation characteristics of low-traffic asphalt pavements. Research reviewed during the work for this thesis show improved resistance to permanent deformation and rutting of asphalt mixtures containing polymer modified bitumen. It is therefore believed that use of polymer modified bitumen in low-traffic asphalt pavements can increase pavement lifetime.

To evaluate the potential polymer modified bitumen has for use in pavements for low-volume roads, an experimental program was carried out. PG-rating of two polymer modified binders and two neat bitumens was done, and MSCRT was conducted to evaluate and compare the two polymer modified binders under heavy traffic loading. Wheel Track testing of AC11 mixtures with neat and polymer modified bitumen was used to assess the effect polymer modified binder has on rutting resistance. The results from Wheel Track and MSCR testing indicate that use of polymer modified binder can significantly decrease rutting in asphalt concrete mixture. Based on the results acquired from the laboratory testing and research reviewed during the literature study, it seems likely that an increase in pavement lifetime can be expected if using polymer modified bitumen on low-volume roads with a considerable amount of heavy traffic.

Keywords:

1. Polymer Modified Bitumen
2. Wheel Track
3. Multiple Stress Creep Recovery Test
4. Rutting

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Summary

Low-volume roads with an AADT less than 3000 make up 90 % of the Norwegian road network. Although considerable effort is being made to improve its quality, deterioration of existing low-volume roads is a significant concern. There are many deterioration mechanisms which affect Norwegian low-volume roads. The literature study conducted for this thesis found that permanent deformation and rutting in the asphalt pavement caused by heavy traffic loads and climatic conditions is one of the most substantial types of distress on low-volume roads.

Polymer modification of bitumen has been shown to improve the stability of asphalt pavements and significantly reduce rutting. The literature regarding use of polymer modified bitumen on low-volume roads is, however, limited. This study investigates the effect polymer modified bitumen has on deformation characteristics of low-traffic asphalt pavements. Low-volume roads with a high percentage of heavy vehicles are exposed to greater traffic loads than regular low-volume roads. To increase pavement lifetime, it is possible to use polymer modified binder instead of neat bitumen. Research reviewed during the work for this thesis show improved resistance to permanent deformation and rutting of asphalt mixtures containing polymer modified bitumen.

To evaluate the potential polymer modified bitumen has for use in pavements for low-volume roads with considerable amounts of heavy traffic, an experimental program was carried out. Performance Grading of two polymer modified binders and two neat bitumens was done, and Multiple Stress Creep Recovery Test was conducted to evaluate and compare the two polymer modified binders under heavy traffic loading. Wheel Track testing of AC11 mixtures with neat and polymer modified bitumen was used to assess the effect polymer modified binder has on rutting resistance. The results of the PG-classification show that the two polymer modified binders meet the criteria for permanent deformation resistance at both higher and lower temperatures for rutting and low-temperature cracking respectively than the neat bitumens. Comparison of two different polymer modified binders in the Multiple Stress Creep Recovery test showed that one of the modified binders had significantly improved rutting resistance under heavy traffic loads. The results from the Wheel Track test indicate that use of polymer modified binder can significantly decrease rutting in asphalt concrete mixture. Based on the results acquired from the laboratory testing and research reviewed during the literature study, it seems likely that an increase in pavement lifetime can be expected if using polymer modified bitumen on low-volume roads with a considerable amount of heavy traffic.

Sammendrag

Lavtrafikkerte veger omfatter omtrent 90 % av vegnettet i Norge. Utfordringene med eksisterende lavtrafikkerte veger er store, og det legges opp til en betydelig satsing på investering og nybygging i årene fremover. Valg av asfaltdekker for å sikre varige veger vil være en viktig del av denne satsingen. Det er mange nedbrytningsmekanismer som påvirker det norske vegnettet. Litteratur gjennomgått som en del av denne studien viser at en betydelig skademekanisme på lavtrafikkerte veger er permanent deformasjon og sporutvikling som følge av trafikkbelastning og klimatiske forhold. Lavtrafikkerte veger med høy tungtrafikkandel er utsatt for større belastning enn vanlige lavtrafikkerte veger, og bruk av polymer modifisert bindemiddel kan være et mulig tiltak for å øke levetiden til vegdekke. Litteraturen som ble gjennomgått viser at bruk av polymermodifisert bitumen forbedrer deformasjonsegenskapene til vegdekke og reduserer sporutvikling vesentlig sammenlignet med penetrasjonsbitumen.

For å undersøke effekten av polymermodifisert bindemiddel på deformasjonsegenskapene til asfaltdekker ment for bruk på lavtrafikkerte veger med høy tungtrafikkandel, ble det gjennomført Performance Grade-klassifisering av penetrasjon- og polymermodifisert bindemiddel, Multiple Stress Creep Recovery-test på to ulike polymermodifiserte bindemidler, og Wheel Track-test på prøver av Ab11 med penetrasjon- og polymermodifisert bindemiddel. Resultatene av PG-klassifiseringen viser at polymermodifisert bindemiddel møter kravene til permanent deformasjonsmotstand ved høyere og lavere temperaturer enn penetrasjonsbitumen, og at de polymermodifiserte bindemidlene derfor har et større temperaturbruksområde. Sammenligning av de to ulike polymermodifiserte bindemidlene i Multiple Stress Creep Recovery-test viste at det ene polymermodifiserte bindemidlet hadde signifikant bedre motstand mot deformasjon ved høye trafikklaster. Resultatene fra Wheel Track-testene viste at Ab11-prøvene med modifisert bindemiddel hadde signifikant mindre sporutvikling og deformasjon enn prøvene med 70/100 penetrasjonsbitumen. Basert på resultatene fra disse laboratorieundersøkelsene virker det trolig at man kan forvente en økning i levetiden på asfaltdekker med polymermodifisert bindemiddel sammenlignet med umodifisert bitumen.

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1 Introduction

1.1 Background

Roughly 90% of the Norwegian road network consists of roads with an annual average daily traffic (AADT) lower than 3000 and are classified as low-volume roads. In the Norwegian National Transport Plan 2010-2019 (NTP), the road network maintenance lag was calculated to be 7,3 million NOK for national roads and 20,9 million NOK for county roads. Both networks consist mostly of low-volume roads. Although considerable effort is being made to improve road quality, deterioration of existing low-volume roads is a significant concern. There are several types of distress mechanisms causing Norwegian roads to deteriorate. During the 70s and 80s, growing traffic and use of studded tires noticeably increased deterioration of the road network. Asphalt research in Norway focused for many years on problems related to wear caused by studded tires on high-volume roads, and thus, wear-resistant asphalt mixes were developed. However, on roads with lower traffic volumes, wear from studded tires was found not to be the main distress mechanism leading to deterioration. The dominating distress mechanisms on these roads are, rather, rutting and permanent deformation, aging, fatigue cracking, and other forms of cracking.

Permanent deformation in the asphalt layers is one of the most substantial types of distress on low-volume roads leading to reduced pavement lifetime. Also, examples of rapid deterioration on new, upgraded roads indicate that the potential of improvement and cost savings is significant. To combat increasing traffic loads and to reduce rutting, polymer modified bitumen has been used in high-volume traffic pavements with success in Norway and internationally. In 2015, the government program “Varige Veger” (eng: Lasting Roads) was completed. The goal of the program was to increase pavement lifetime and reduce annual costs for the complete road construction in the Norwegian road network by focusing on road pavements, pavement design and strengthening, and dissemination and implementation of knowledge. As a part of “Varige Veger”, research and documentation was initialized to increase knowledge and experience with polymer modified bitumen.

1.2 Problem statement and research objectives

Several research works have been conducted on permanent deformation of polymer modified asphalt mixtures. Most of the research found in the literature study for this thesis focused on polymer modification of asphalt mixtures suited for high-volume roads or road sections with substantial traffic loading. There is little experience and research, however, with polymer modified asphalt mixtures used on low-traffic roads. Thus, it is difficult to quantify the effect use of polymer modified bitumen has on permanent deformation o low-volume asphalt pavements. If similar results can be expected when used on low-volume roads as found in literature regarding high-volume roads, pavement lifetime could increase and annual rehabilitation costs decrease.

This graduate thesis attempts to give further insight into the effect polymer modified bitumen has on rutting in low-volume roads with a high percentage of heavy vehicle traffic. By using laboratory test that are judged to be simulative of field loading conditions, the goal of the study is to provide more knowledge on the effect PMB has on permanent deformation in asphalt concrete mixtures made for low-volume traffic conditions. Recommendations for further research work and an evaluation of the suitability of polymer modified low-traffic asphalt pavements are made.

The objectives of this graduate thesis are:

- to identify and describe the predominant deterioration mechanisms leading to permanent deformation on low-volume roads

- to review relevant research regarding the effect polymer modification has on permanent deformation of asphalt pavements

- To describe methods commonly used for evaluating pavement rutting resistance based on laboratory testing of polymer modified bitumen and asphalt mixture samples.

- to investigate the effect polymer modified binder has on rutting resistance in low-volume asphalt concrete pavements

1.3 Limitations

Due to limited time and resources, there are several limitations that were accepted in the beginning of this research work

- Roads have sufficient load bearing capacity. Thus, all deformation is assumed to occur in the asphalt pavement.

- Unpaved, gravel roads were not considered

- Due to low annual average daily traffic, wear from studded tires is assumed to not be a significant concern on low-volume roads

- AC11 was used for all asphalt mixture tests, and it is assumed that this is a typical asphalt surface for low-volume roads with a relatively high percentage traffic being heavy vehicles

- Two types of polymer modified binders were used for Multiple Stress Creep Recovery testing, while only one was used for Wheel Track testing. It is assumed that these polymer modified binders are representative of common PMBs used in Norway, however, this could not be the case.

- Some information, such as type and amount of polymer used, is not known in detail. The polymer modified binders are therefore named PMB1 and PMB2

The predominant mechanisms leading to deterioration on low-volume roads are different from road to road, and depend on traffic loading, climatic conditions, and subgrade and road material. Therefore, there are significant differences between roads within this category. This thesis focuses on roads with an AADT around 3000 and with a relatively high percentage of traffic being heavy vehicles. This is reflected in the choice of materials used for laboratory testing. Some of the materials used might not be what one would typically consider for low-volume roads, however. It is therefore debatable whether the results and conclusions represented in this thesis, apply to what one might consider a “typical” low-volume road.

2 Methodology

The methodology used to meet the objectives of this graduate thesis involves a review of literature and a laboratory investigation. The literature review was conducted to describe the predominant deterioration mechanisms leading to permanent deformation on low-volume roads, to get an overview of state of the art polymer modification of bitumen, and to assess the effect polymer modified bitumen has on rutting resistance of asphalt mixtures. Testing methods used to characterize permanent deformation properties of bitumen and asphalt mixtures were also reviewed. The laboratory investigation was conducted using the following testing procedures:

- Multiple Stress Creep Recovery Test (MSCRT)
- Rolling Thin Film Oven Test (RTFOT)
- Pressure Aging Vessel (PAV)
- Dynamic Shear Rheometer (DSR)
- Bending Beam Rheometer (BBR)
- Wheel Track Test (WTT)

To investigate the effect using polymer modified bitumen has on rutting resistance in asphalt pavements, asphalt mixtures with neat and modified bitumen were tested in WTT. DSR and BBR were used for Performance Grade classification of 70/100 and 160/220 neat bitumen, and two polymer modified bitumens, while MSCRT was performed to investigate rutting resistance of two polymer modified binders. RTFOT and PAV was used to age the binder before BBR testing.

Laboratory testing was conducted at NTNU and at Veidekke Industri Kompetansesenteret.

The laboratory tests mentioned here will be discussed further in chapter 4.

To determine the effect of polymer modified bitumen had on the test results, Student’s t-test was used. This is discussed further in chapter 6.

3 Literature study

In this section, relevant information and research found during the literature study conducted for this thesis is presented to identify and describe the predominant deterioration mechanisms leading to permanent deformation on low-volume roads. A review of relevant research regarding the effect polymer modified bitumen has on permanent deformation characteristics of asphalt pavements is also presented.

3.1 Low-volume roads

The term low-volume road is often used in literature and textbooks regarding roads with low traffic intensities. Throughout the literature study for this thesis, different definitions of what constitutes a low-volume road were found. The definitions varied from country to country, and from source to source. No universal definition was identified. Faiz (2012) defines low-volume roads as a road with an AADT less than 1000, while the Transportation Association of Canada sets the limit at AADT equal to 200. The American Association of State Highway and Transportation Officials, on the other hand, defines roads with an AADT less than 400 as very low-volume.

In Norway, the term low-volume road is often used for roads with an AADT less than 3000. This constitutes roughly 90% of the Norwegian road network. There are, however, considerable differences between roads within this category. Roads used for forestry, for example, might have a low AADT but the traffic may consist almost entirely of heavy vehicles causing considerable damage to the road. On the other hand, an intercity road might have an AADT of 3000, but have a low number of heavy vehicles. Different design methods, materials used for construction and maintenance schedules further complicate this distinction. Thus, when discussing this topic, it is important to clarify what type of low-volume road is being addressed. To simplify this discussion, three different types of low-volume roads are described in this chapter. Because of the focus of this thesis, types of low-volume roads were defined based on surface layer. However, other methods of categorizing low-volume roads exist, for example based on road geometry and road purpose.

3.1.1 Unpaved, gravel roads

In Norway, a gravel surface layer can be used on rural roads and minor access roads. The gravel surface layer consists of mechanically stabilized gravel, while the rest of the road structure resembles that of a typical paved road. The condition of gravel and unpaved roads can change much more rapidly than the condition of paved roads, so they require regular maintenance at frequent intervals. An advantage of unpaved roads, however, is that minor damage can easily be repaired simply by grading and/or compaction. Cost savings is another benefit with gravel roads. There are, on the other hand, several disadvantages one must consider. Gravel roads have a rougher surface which impedes surface drainage, ruts tend to develop quickly between grading operations, heavy rainfall can result in a muddy road and loss of bearing capacity, and dust generated on gravel roads can be a problem if not treated correctly.

3.1.2 Low-volume roads with soft asphalt pavement

For roads with traffic volumes lower than 3000 AADT, there are several soft surfacing types that can be used. These roads tend to be designed with a thin bituminous layer and soft binders. The pavement depends on a well-built granular base for structural strength and the contribution to overall bearing capacity from the soft asphalt layer is low. The most relevant surfacing types for soft asphalt pavements are asphalt concrete with gravel and soft asphalt. Recycled asphalt can also be used. Pavement thickness of 4 cm is common. Compared to thicker asphalt pavements used on roads with higher traffic volumes, soft pavements provide better flexibility and can withstand greater movement without cracking. The softer binder also gives the pavement somewhat of a self-healing ability. Improved flexibility and softer binder comes at a cost, however, as rutting and deformations in the bituminous layers can become a challenge.

3.1.3 Low-volume roads with stiff asphalt pavement

Stiff asphalt pavements can be used for all road categories and traffic classes in Norway. The asphalt pavement consists of two bituminous layers; a surface layer and a binder course. Minimum thickness of each layer is 3,0 cm, and the thickness increases with increasing traffic volume. Depending on traffic load, climatic conditions and subgrade material, asphalt concrete, asphalt gravel concrete or stone mastic asphalt is normally used. Because of a thicker bituminous layer, the asphalt pavement's contribution to the overall bearing capacity of the road is greater than that of soft asphalt pavements. This increases the road's stiffness and strength, but also reduces flexibility.

3.2 Example road: Ev 6 Brekkvasselv

Ev 6 Brekkvasselv is an E-road in Nord-Trøndelag which reflects the type of traffic conditions on low-volume roads this thesis focuses on. As figure 1 shows, the annual average daily traffic on Ev 6 Brekkvasselv is approximately 1500, while the monthly average daily traffic varies from 700 to 3500. This classifies as a typical low-volume road. Heavy vehicles, however, constitute a significant portion of the traffic. In 2016, 34% of the traffic on Ev 6 Brekkvasselv was heavy vehicles. Thus, road deterioration caused by high traffic loading is a significant concern. Low-volume roads with similar traffic conditions as Ev 6 Brekkvasselv could for example have an asphalt surface consisting of AC11 with 70/100 penetration bitumen. This is perhaps not an asphalt surface one would consider "typical" for low-volume roads, since softer binders are commonly used. Due to the amount of heavy traffic, however, one could argue that using a stiffer binder is more applicable to combat rutting and fatigue damage caused by repeated heavy loading.

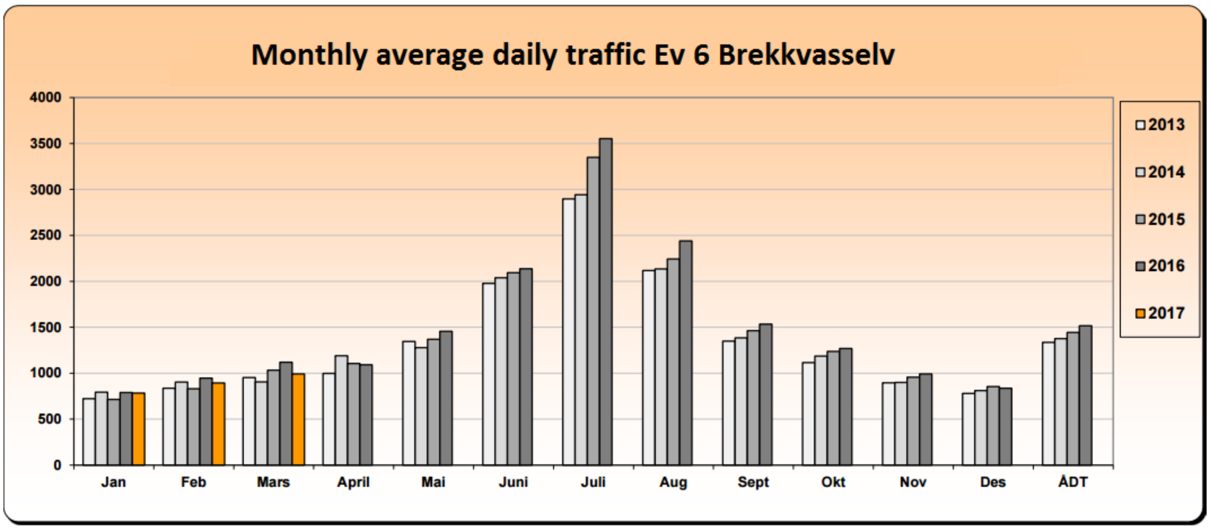

FIGURE 1 Monthly average daily traffic Ev 6 Brekkvasselv *(Statens Vegvesen)*

3.3 Types of deterioration on low-volume roads

Deterioration of asphalt pavements is a result of traffic loads, material properties and climatic conditions. When a wheel load is applied to a pavement, the primary stresses that are transmitted to the asphalt pavement are vertical compressive stress and shear stress within the asphalt layer, and horizontal tensile stress at the bottom of the asphalt layer. Therefore, the asphalt pavement must be internally strong and resistant to compressive and shear stresses to prevent permanent deformation within the pavement. In the same manner, the material must also have enough tensile strength to withstand tensile stress at the base of the asphalt layer to resist crack initiation. Otherwise, fatigue cracking may occur after repeated loading. Furthermore, rapidly decreasing temperatures and harsh weather conditions cause stress which the pavement must withstand (Asphalt Institute, 2001).

If the asphalt pavement is not able to resist these types of stress, road damage starts to accumulate. Lerfald (2000) found that the most frequent defects on existing low-volume roads in Norway were:

- Edge cracks, transverse cracks and cracks due to frost heave
- Aging
- Permanent deformation

Many roads in the Norwegian road network have insufficient structural bearing capacity and this might have affected the results Lerfald found in his study. How applicable these results are when focusing on new roads with sufficient bearing capacity is therefore questionable.

Cracks in asphalt pavements occur in a wide variety of patterns, from isolated cracks to networks covering the entire pavement surface. Cracking occurs in asphalt pavements when the applied loads overstress the asphalt materials and is usually caused by several factors occurring simultaneously. Repeated heavy loading, thin pavements and weak underlying layers lead to high deflections and cause horizontal tensile stress at the bottom of the asphalt layer. Edge cracks is a typical form of cracking on low-volume roads in Norway due to narrow roads with inadequate edge support, while transverse cracks can occur on low-volume roads due to low temperatures and bitumen hardening. Cracks due to frost heave is another frequent type of cracking. Poor drainage, poor construction and/or insufficiently designed pavements further contribute to this problem. Cracking leads to loss of load-distribution and reduced waterproofing, which in turn accelerates further pavement deterioration.

While aging is not a defect by itself, several pavement properties are influenced by the aging process. The term aging is often used to describe the change in bitumen properties during storage, mixing, laying and in service. Oxidation, evaporative hardening, structural aging and exudation have been identified as the four main mechanisms related to aging and durability of bitumen over time (Lerfald, 2000). When the binder is aged, it becomes brittle and stiff, which increases the susceptibility of crack development and raveling in the asphalt pavement. Aging of bitumen is a complex process with many factors contributing to age hardening, the most important factors being oxidation of bitumen, loss of volatiles, physical hardening and exudative hardening.

Permanent deformation is one of the major distress mechanisms in flexible pavements. It is characterized by a pavement surface cross section that is no longer in its designed state. Permanent deformation represents an accumulation of small amounts of unrecoverable deformation that occur each time a load is applied. Wheel path rutting is the most common form of permanent deformation, and because of increasing tire pressure and axle loads, rutting has become the dominant mode of failure of flexible pavements in many countries.

In the 1990s, a survey was conducted under the COST 333 program in Europe to determine the most common types of pavement deterioration. Countries were asked to rate the most common types of deterioration observed on their roads using a rising scale of increasing importance from 0 to 5, where 0 indicates not observed and 5 indicates major importance to road deterioration. Figure 2 show the results of this study.

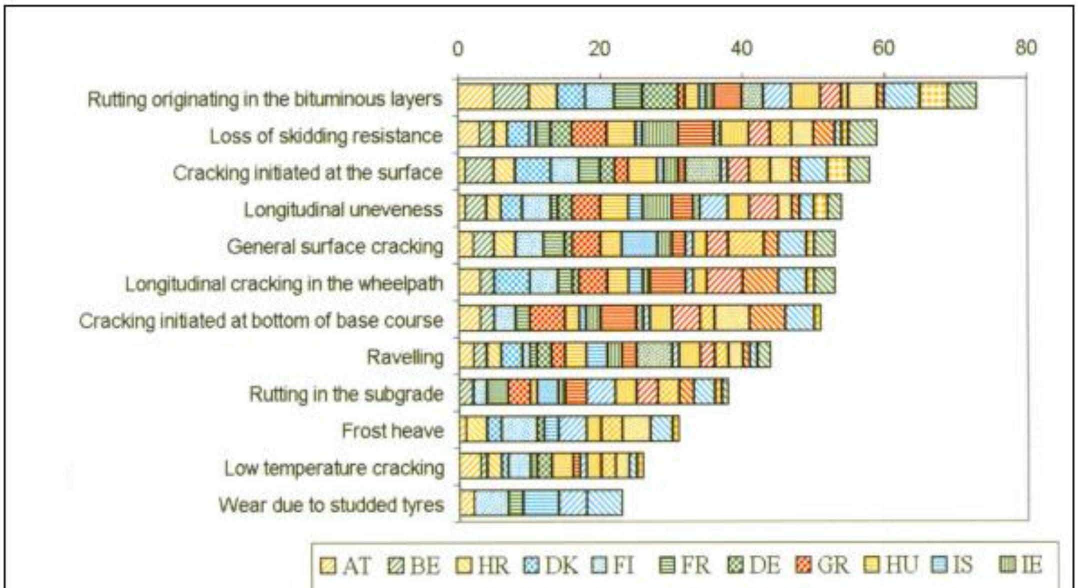

FIGURE 2 Rating of observed deterioration (European Commision, 1999)

Based on the results from this survey, rutting originating in the bituminous layers seems to be the most observed deterioration on roads in Europe. Difference between countries were considerable, however. Because of differences between road networks in European countries, it is difficult to compare these results without considering traffic loads and volumes, climatic conditions and state of the road network. The results from Norway indicated that rutting in the bituminous layers is a significant concern. Wear due to studded tires, frost heave cracking, rutting in subgrade and longitudinal unevenness are also important forms of deterioration. These results, however, reflect the whole Norwegian road network. Even though most of the road network is low-volume, it is important to note that these results might not reflect the predominant deterioration types on low-volume roads in Norway.

3.4 Rutting in asphalt pavements

The properties of asphalt concrete mixtures vary with composition, temperature, and load level and frequency. At low temperatures, low load levels, and high frequencies, asphalt concrete materials have elastic properties. At high temperatures, high loads and slow loading times, however, the behavior of the material tends to be more viscous. In response to loading, asphalt concrete materials deform and some of this deformation is non-recoverable. Over time, this permanent deformation accumulates and causes rutting in asphalt pavements. Rutting in asphalt pavements in Norway is caused by plastic deformation in the asphalt mixture, consolidation and compaction of pavement layers, and abrasion from studded tires. Combinations of these three causes further increase pavement rutting.

3.4.1 Rutting caused by wear from studded tires

Studded tires were introduced in Norway in the late 1960s, and combined with increasing traffic in the 1970s and 1980s made pavement wear from studded tires a concern. Asphalt research in Norway focused for many years on problems related to wear caused by studded tires, and wear-resistant asphalt mixes were developed. During the 1990s, studless snow tires were developed and an effort was made politically to reduce use of studded tires. Even though significant progress has been made in regards to reducing the pavement deterioration and environmental impact, wear from studded tires remains a challenge on high-volume roads and on roads with a high percentage of vehicles using studded tires. Nordal et al (1991, rev. 2016) writes that wear from studded tires is not believed to be a significant contributor to rutting in low-volume roads due to low-traffic volumes. Rutting caused by wear from studded tires will not be discussed further in this thesis.

3.4.2 Rutting caused by consolidation and compaction of pavement layers

Rutting due to consolidation and post-compaction can occur on several layers in the road structure. This is illustrated in figure 3.

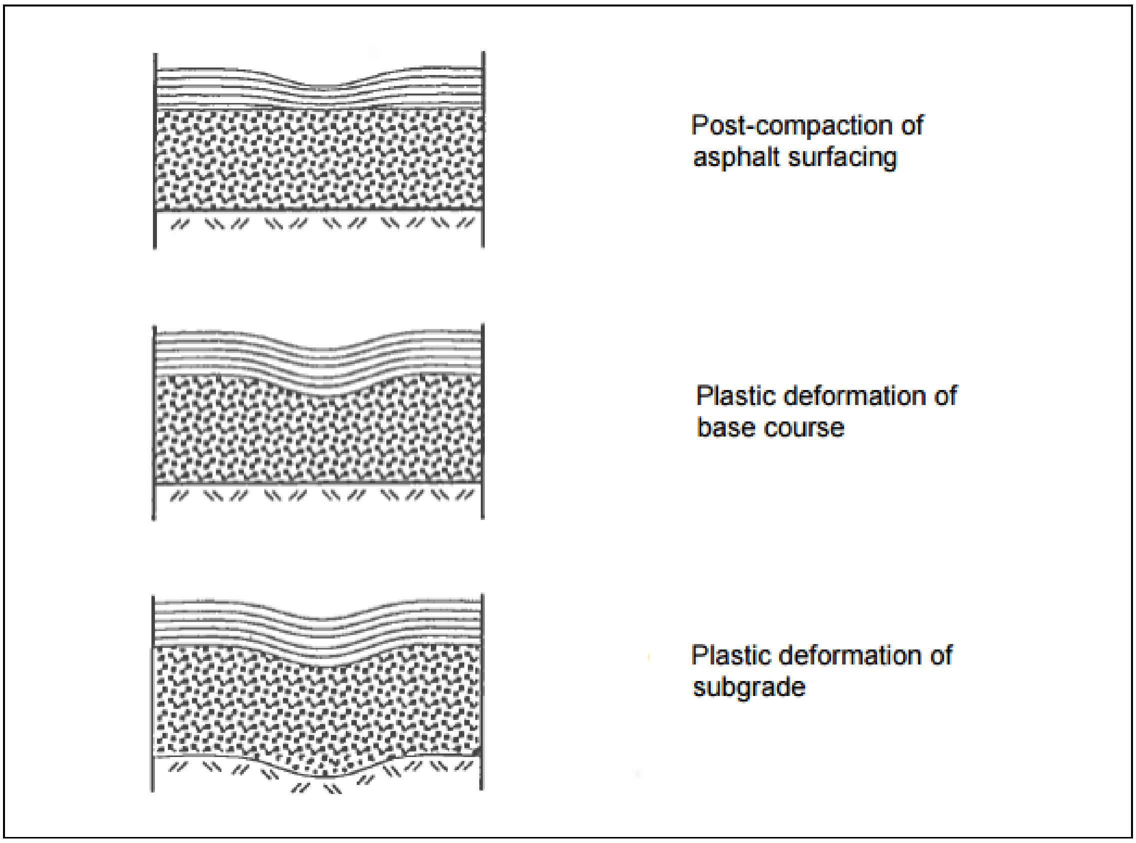

FIGURE 3 Different types of rutting caused by consolidation and compaction (Bakløkk & Mork, 2000, rev. 2016)

Firstly, if the asphalt pavement is insufficiently compacted after laying, high air void content and post-compaction of asphalt surfacing leads to development of rutting. One of the main objectives of “Varige Veger” was to address this problem and ensure proper execution of asphalt compaction. Secondly, plastic deformation of the base course leads to deformation of the road structure. Lastly, plastic deformation of subgrade material can occur which causes overlying road layers to collapse. Combinations of compaction and consolidation in multiple layers further increase rutting.

Although consolidation and post-compaction of granular layers in the road structure might be a significant factor which historically has contributed to rut development on existing roads in Norway, Hoff and Refsdal (2011) claim that deformation in the granular layers of new roads constructed following N200 (Norwegian manual for road construction) will be low.

Permanent deformation occurring in non-bituminous layers will therefore not be discussed further in this thesis.

3.4.3 Rutting caused by shear deformation in bituminous layers

In addition to post-compaction as described previously, shear deformation of the bituminous layers also contributes to development of ruts. In contrast to deformation caused by compaction, shear deformation occurs when there is no change in volume of the bituminous layers. Shear deformation in asphalt layers is caused by an asphalt mixture which does not have sufficient shear strength to resist the repeated loads it is subjected to from traffic. Challenges related to wear from studded tires for many years overshadowed the problem of rutting caused by deformation in the bituminous layers. A gradual decrease in use of studded tires since the 1990s, has, however, in combination with an increase in use of super-single tires and higher contact pressure from heavy vehicles caused deformation in the bituminous layers to become an increasingly important factor affecting pavement lifetime for high-volume roads.

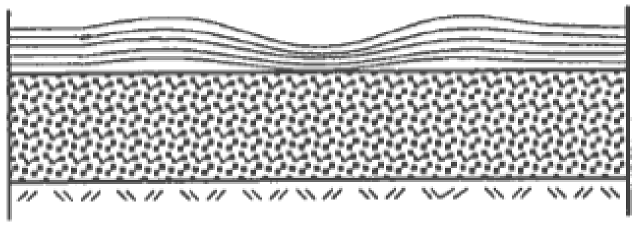

FIGURE 4 Rutting caused by shear deformation of bituminous layers (Bakløkk & Mork, 2000, rev. 2016)

3.5 Factors affecting deformation properties of an asphalt pavement

The deformation behavior of an asphalt pavement is a result of a complex system of external and internal factors. It is generally accepted that the permanent deformation observed at the pavement surface is influenced by the stress caused by traffic loading, the properties of the pavement layers and climatic conditions.

3.5.1 Stress caused by traffic loading

Traffic load, tire pressure and stress distribution are factors dependent on traffic situation affecting the stresses occurring in the asphalt pavement. Traffic imposes loads on a pavement of different nature and varying size. The pressure caused by traffic loading is applied to the pavement as relatively concentrated wheel loads and the stresses are then distributed throughout the pavement structure. Loading rate is also an important factor for the pavement's deformation resistance due to the viscous character of the binder. Furthermore, an increase in longitudinal and lateral contact stress associated with acceleration, braking and road geometry can play a role.

3.5.2 Properties of the pavement layers

Regarding the pavement itself, there are many factors affecting the deformation resistance. Composition, aggregate gradation and angularity, type of mineral filler, void content and voids in mineral aggregate influence deformation behavior, but the binder probably has the greatest importance. In addition, compaction and layer thickness also plays a significant role. Heukelom and Wijga (1973) found that the stiffness of mastic and asphalt concrete is influenced by bitumen stiffness and content of bitumen and filler. Apart from increasing the stiffness of the asphalt, an increase in bitumen stiffness particularly at high temperatures increases the pavement's resistance to rutting. In Norway, traditionally softer binders have been used to ensure flexible pavements and good low temperature qualities. This, however, leads to the asphalt pavement having a lower resistance to rutting.

3.5.3 Climatic conditions

Regarding the climatic conditions affecting the deformation properties of an asphalt pavement, the temperature is likely the most important factor. As stated earlier, the performance of asphalt pavements is influenced by the binder properties, aggregate and grading characteristics, with the binder playing a critical role. The performance of asphalts in service is influenced significantly by the rheological properties of the binder. Binders are viscoelastic materials, and their behavior varies from purely viscous to wholly elastic depending on loading time and temperature. At higher service temperatures, the binder becomes increasingly viscous and the asphalt mixture's ability to resist shear deformation decreases.

3.6 Low-temperature cracking in pavements

In cold climates or under rapidly-applied loads, asphalt pavements behave like an elastic solid; when loaded, they deform, and when unloaded, they return to their original shape. If stressed beyond material capacity or strength, elastic solids may break. The cracking of asphalt pavements in winter, is a well-known mode of road distress in Norway. Low-temperature cracks form when an asphalt pavement layer shrinks in cold temperatures. As the pavement shrinks, tensile stresses build within the layer. At some point along the pavement, the tensile stress exceeds the binder tensile strength and the asphalt layer cracks. Low-temperature cracking is characterized by intermittent transverse cracks often occurring at a consistent spacing.

The asphalt binder plays a key role in low-temperature cracking. In general, hard asphalt binders are more prone to low-temperature cracking than soft binders are. Asphalt binders that are excessively aged are more prone to low-temperature cracking. Thus, to overcome low-temperature cracking, a soft binder must be used that is not overly prone to aging, and air void content of the pavement must be controlled so that the binder does not become excessively oxidized. Various studies of low-temperature cracking have generally concluded that to reduce low-temperature cracking, the binder stiffness must not exceed some defined limit at the coldest pavement temperature (Bahia, 1991).

As figure 5 illustrates, air temperatures in some parts of Norway fall below negative 45 degrees Celsius. Flexible pavements with sufficient tensile strength to withstand low-temperature cracking is especially important in these areas. Polymer modified bitumen is starting to become a popular solution to address severe low temperatures on high-volume roads.

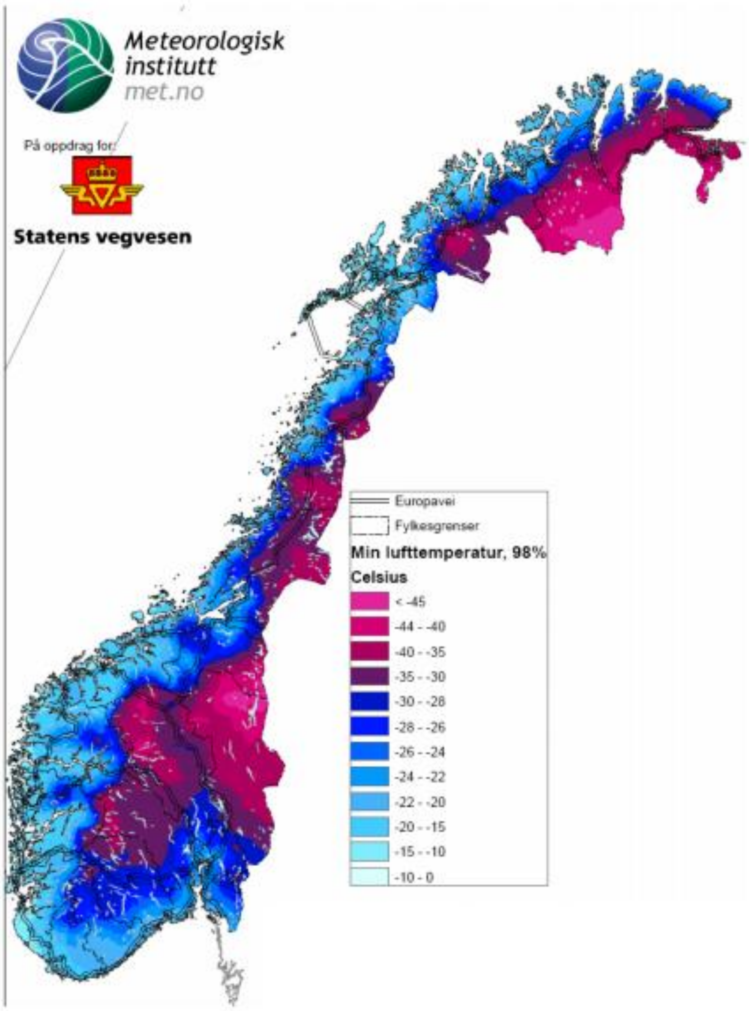

FIGURE 5 Minimum air temperature in degrees Celsius in different parts of Norway *(Statens Vegvesen, 2014)*

3.7 Polymer modified bitumen

Bitumen is a complex material. Due to its viscoelastic nature, the rheological properties of bitumen depend on both temperature and rate of loading. In hot conditions or under static loads, bitumen acts like a viscous liquid. In cold climates or under rapidly applied loads, however, bitumen is an elastic solid. The asphalt pavement's viscoelastic properties are determined by the binder, and therefore, bitumen quality plays a significant role in determining the pavement's resistance to rutting and low-temperature cracking. Neat bitumen often possesses adequate performance characteristics, but increasing the high temperature performance can sometimes lessen the bitumen's ability to resist low-temperature cracking, and vice versa. Increasing traffic volumes and loading, and a demand for longer service life, have led to the need of asphalt pavements with improved in-service qualities. By changing the characteristics of normal bitumen with the addition of polymers, one succeeds to obtain a bitumen with higher strength and resistance to fatigue, rutting and low-temperature cracking. During the last three decades, polymer modified bitumen (PMB) has become increasingly popular both internationally and in Norway as a replacement for penetration grade bitumen in the upper layers of asphalt pavements. Since 2008, use of PMB has become more common in Norway, and available data suggests that 13% of asphalt laid in 2010 in Norway contained PMB (Saba, 2013).

Polymer modified bitumen is binder whose properties have been changed using a polymer additive that alters its chemical structure and physical and/or mechanical properties. As the bituminous binder is responsible for the viscoelastic behavior of the pavement, it plays an important part in determining many aspects of road performance, particularly resistance to deformation and cracking. One of the prime roles of PMB is to increase the resistance to deformation at high pavement temperatures without adversely affecting the properties of the pavement at other temperatures. This is achieved by one of the two following methods:

1. Stiffen the bitumen so that the viscoelastic response of the asphalt is reduced.

2. Increase the elastic component of the bitumen, thereby reducing the viscous component.

Increasing the stiffness of the bitumen is also likely to increase the dynamic stiffness of the asphalt. This will improve the load spreading capability of the material, increase the structural strength and lengthen the expected service life of the pavement. Alternatively, it may be possible to achieve the same structural strength but with a thinner layer (Rodrigues & Hanumanthgaru, 2015).

In a study done by Lerfald and Aurstad (2011), laboratory tests were conducted to evaluate the long-term field performance of pavements with PMB compared to neat bitumen. Field samples were taken from asphalt concrete pavements with and without PMB, while laboratory samples of asphalt concrete and stone mastic asphalt with neat and PMB were prepared. These samples were then tested at 50 °C in a wheel tracking device. The results of the study indicated that use of polymer modified binders improves the deformation resistance of asphalt mixes, compared with asphalt mixes containing ordinary bitumen. This is shown in figure 6 and figure 7.

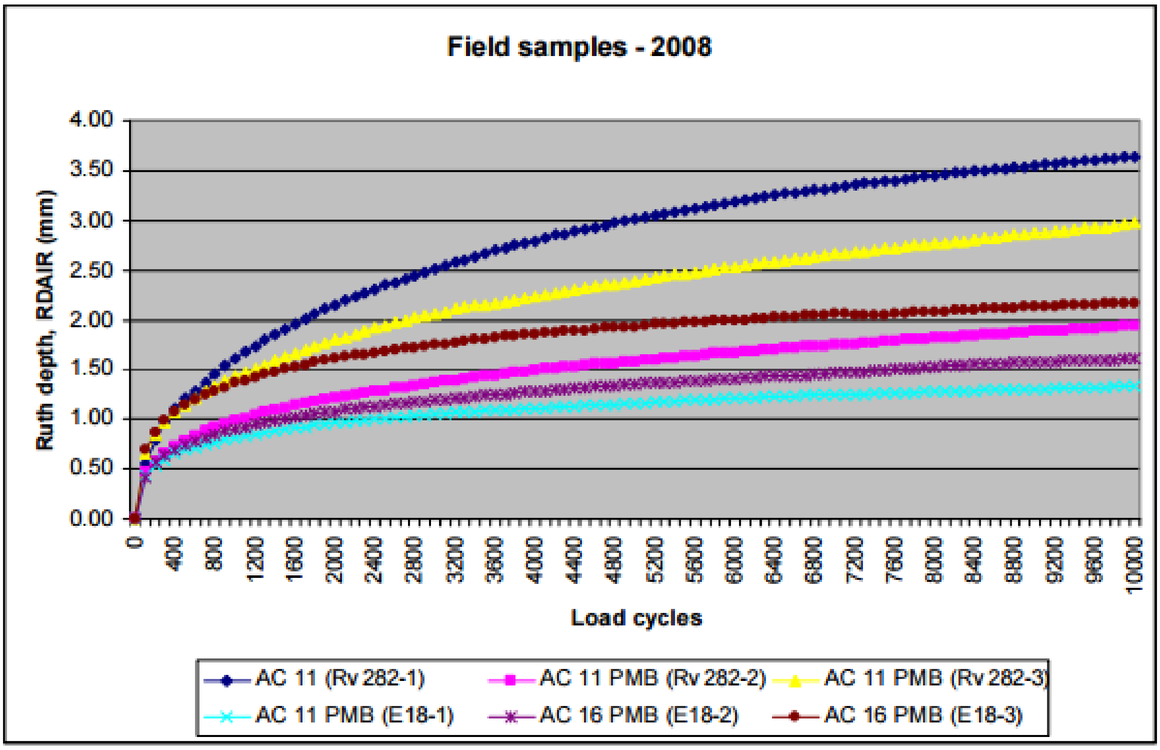

FIGURE 6 Deformation of field samples in wheel track from test sections *(Lerfald & Aurstad, Use of Polymer Modified Binders to develop more Lasting Pavements, 2011)*

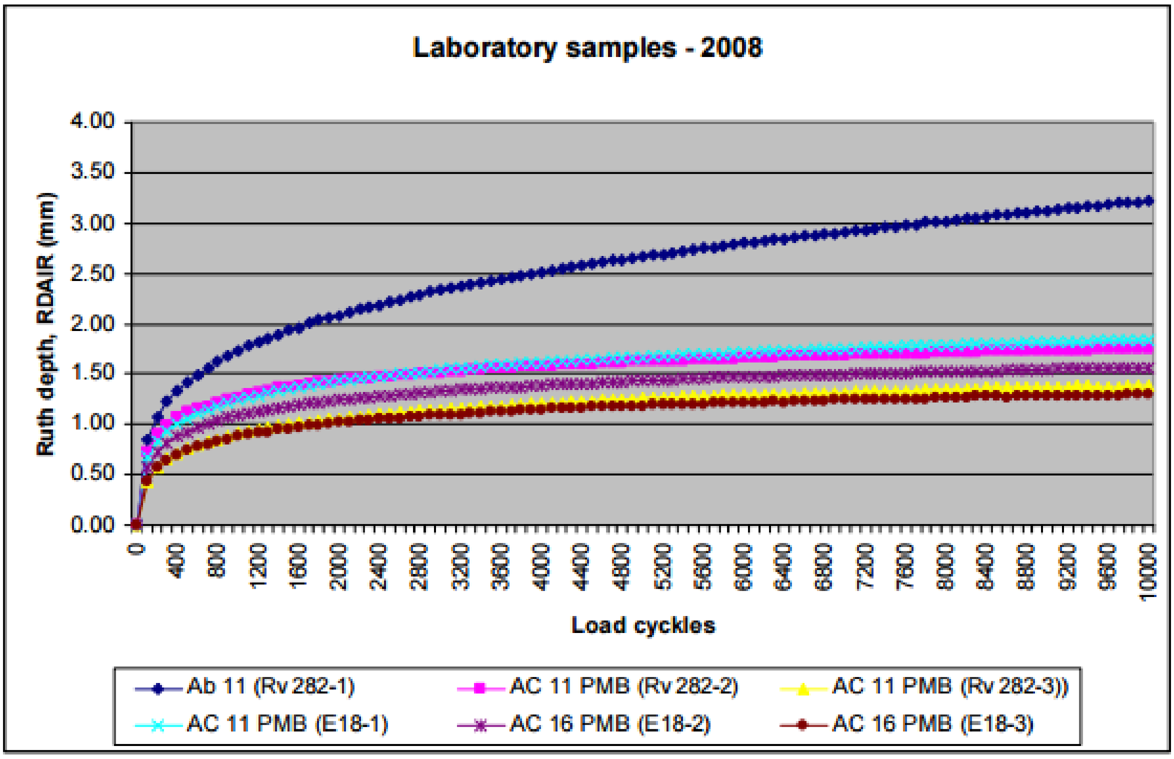

FIGURE 7 Deformation of laboratory samples from test sections *(Lerfald & Aurstad, Use of Polymer Modified Binders to develop more Lasting Pavements, 2011)*

According to Rodrigues and Hanumanthgari (2015), for the polymer modifier to be effective and for its use to be both practicable and economic, it must

- Be readily available
- Be cost effective
- Blend with bitumen
- Resist degradation at asphalt mixing temperatures
- Improve resistance to flow at high temperatures without making the bitumen too viscous at mixing and laying temperatures or too stiff or brittle at low road temperatures
- Improve binder cohesion or adhesion properties

In addition, the modifier, when blended with the bitumen, should

- Be capable of being processed using conventional equipment
- Maintain its premium properties during storage, transportation, application and in service
- Be physically and chemically stable during storage, transportation, application and in service
- Achieve a coating or spraying viscosity at normal application temperatures

Polymers used for modification of bitumen are large chains of repeating monomer molecules. These chains can be simple straight chains or variations of linked and cross-linked chains. Both the structure and the chemistry of chains can influence behavior of polymers. If the ends of a polymer are reactive with either other polymers or the surrounding solution, chemical reactions between the components can occur. Examples of typical structures are shown in figure 8. There are many different types of polymers used for bitumen modification. The polymers most commonly used are elastomers, plastomers, reclaimed tire rubbers and, to a lesser extent, viscosity modifiers and reactive polymers. Due to the scope of this graduate thesis, only elastomers and plastomers are described further in detail.

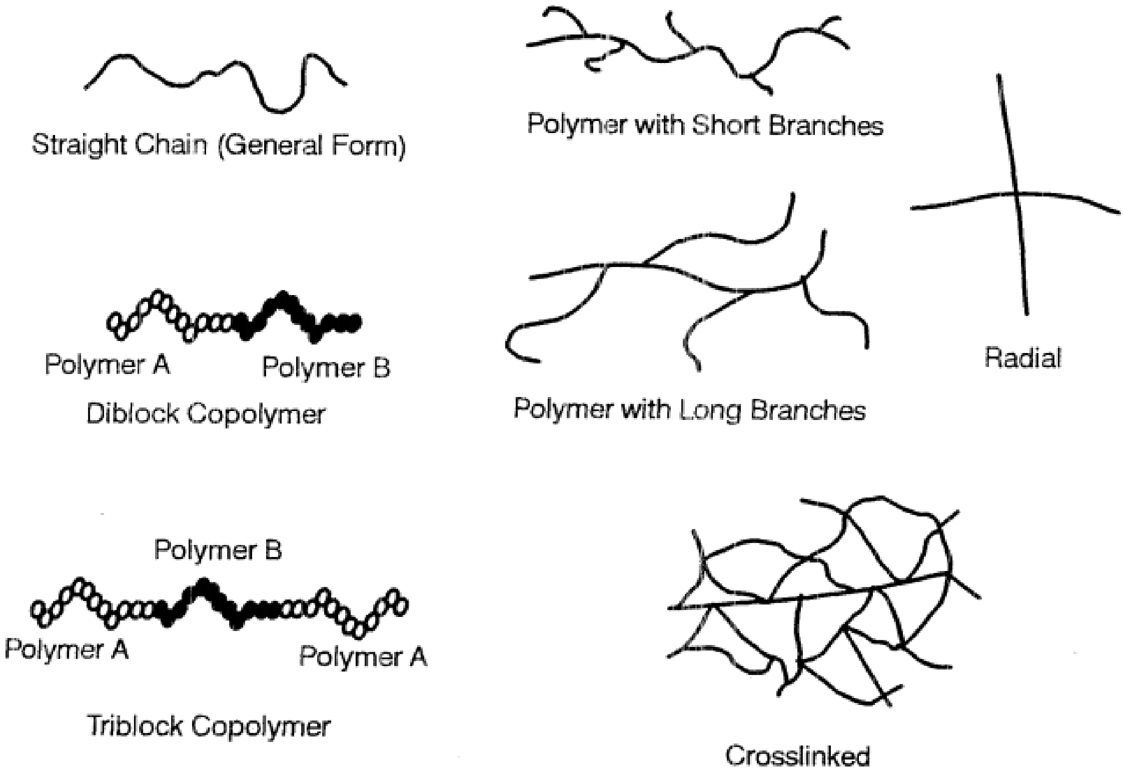

FIGURE 8 Various types of polymer structures (Stroup-Gardiner & Newcomb, 1995)

3.7.1 Elastomers

Elastomers are powdered or granulate rubbers, and include natural rubbers, synthetic rubbers (SBR), thermoplastic rubbers (SBS) and neoprene. Elastomers improve the base bitumen's low and high temperature properties by increasing viscosity and stiffness. The binder's flexibility and elasticity is improved, and its elastic properties when loaded are enhanced. Especially modification with styrene-butadiene-styrene (SBS) has shown favorable results both in laboratory testing and in-service field conditions (Rodrigues & Hanumanthgaru, 2015). The SBS-polymer consists of polystyrene end-blocks that impart strength to the polymer, and butadiene mid-blocks that gives the material its elasticity. This gives the SBS-polymer high tensile strength and flow resistance at high temperatures, while still being elastic and flexible at low temperatures (Ahmedzade, Tigdemir, & Kalyoncuoglu, 2007). When mixed with bitumen, the polymer modified binder will reflect the low and high temperature properties of the SBS-polymer, and an asphalt mixture with improved low and high temperature properties is achieved. Another advantage of SBS-polymer modification is its ability to be worked in a molten form above 150 °C, whereas at this temperature natural rubbers will begin to chemically crosslink. When the SBS is added to the bitumen, the temperature is high enough for the styrene domains to segregate, which allows for the dispersion of the individual SBS chains within the asphalt binder. As the temperature cools during laydown and compaction, the thermodynamics of the system force the styrene domains to reform, thus forming an in-place network within the binder (Stroup-Gardiner & Newcomb, 1995). This is illustrated in figure 9. In Norway, polymer modification of bitumen has mainly been done using SBS.

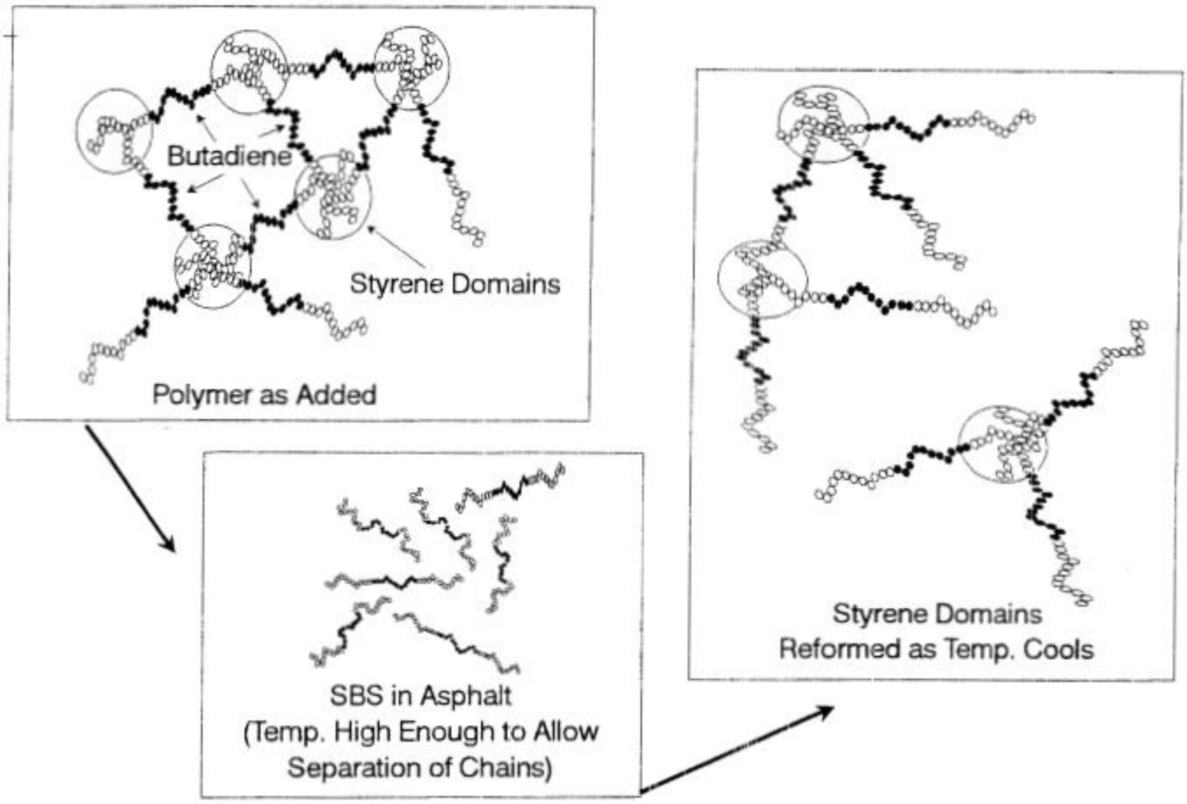

FIGURE 9 Model of SBS block polymer *(Stroup-Gardiner & Newcomb, 1995)*

3.7.2 Plastomers

Some plastomers are used in the bitumen industry as an alternative to elastomers for road paving applications. Polyethylene, polypropylene, polyvinyl-chloride, polystyrene and ethylene-vinyl acetate (EVA) copolymer are the main plastomers that have been examined in recent decades. Plastomers are characterized by softening on heating and hardening on cooling. Plastomers are different from elastomers in that it too increases the bitumen’s strength, but only during the initial loading. The function of these polymers within the asphalt is not to form a network, but to provide plastic inclusions within the matrix. At cold temperatures, these inclusions are intended to improve the binder’s resistance to thermal cracking by inhibiting the propagation of cracks. At warm temperatures, the particle inclusions should increase the viscosity of the binder and therefore the asphalt’s resistance to rutting.

3.8 Pavement lifetime

When the pavement lifetime is abnormally low in relation to what is considered acceptable for the given surface type and traffic volume, strengthening of the road is necessary. It is common to differentiate between functional pavement lifetime and normalized pavement lifetime. Functional pavement lifetime is the registered pavement life from a new-laid surface until triggering maintenance level is reached, while normalized pavement lifetime is the timespan one should expect a road has given proper design, construction and climatic- and traffic conditions. Normalized pavement lifetime in Norway is given in figure 10.

L **NORMERTE DEKKELEVETIDER FOR ULIKE DEKKETYPER**

Dekketype	ÅDT						
	≤300	301-1500	1501-3000	3001-5000	5001-10 000	10 001-20 000	>20 000
Ska				13	10	7	6
Ab			15	12	9	6	5
Agb		15	14	11			
Ma, Egt	16	13	12				
Eo	14	12					

FIGURE 10 Normalized pavement lifetime for different asphalt surfaces in Norway (Statens Vegvesen, 2014)

Since this thesis focuses on low-volume roads with a high percentage of traffic coming from heavy vehicles, an asphalt concrete surface can be considered appropriate. Figure 10 shows that the normalized pavement lifetime for AC surfaces is 15 years if the AADT is between 1501 and 3000 and 12 years if the AADT ranges from 3001 to 5000. In a report from ViaNova AS, it was found that if 10%-quantile of pavement lifetime on road sections is used as an indicator for section pavement lifetime, roughly 50% of national road sections have a lifetime lower than what is expected (Evensen & Johansen, 2016). It is written in N200 that by using polymer modified bitumen, pavement lifetime is increased by 15%. Granden (2012) concluded in her graduate thesis that by using polymer modified bitumen on low-volume roads, pavement lifetime can be increased and that the increased lifetime makes up for the cost of PMB.

In a study done by the Norwegian Road Administration (Statens Vegvesen, 2010), the pavement lifetime of regional roads in eastern Norway were analyzed. It was found that for AADT values between 1501 and 3000, asphalt concrete pavements had a higher pavement lifetime than roads built with other typical pavement types at low-volume roads. The pavement lifetime of asphalt concrete roads was found to be 17,4 years. It is, however, important to note that due to fewer roads with AC pavement with traffic levels ranging from AADT 1501 to 3000, there are some uncertainties related to the accuracy of this number.

4 Testing for permanent deformation characterization of low-volume asphalt pavements

The following sections covers different methods commonly used for evaluating pavement rutting resistance based on laboratory testing of polymer modified bitumen and asphalt mixture samples.

4.1 PG-classification of bitumen

The Strategic Highway Research Program (SHRP), initiated in the USA in 1987, was an effort to produce rational specifications for bitumen and asphalts based on performance parameters. The motivation was to produce pavements that performed well in-service. One of the results of this work is the Superpave asphalt binder specification, which categorizes grades of bitumen based on their performance characteristics in different environmental conditions. It addresses the three main causes of deterioration of asphalt pavements; rutting, fatigue cracking and low-temperature cracking. Contrary to conventional binder tests, the laboratory tests used in Superpave measure physical properties that can be related directly to field performance. The Superpave binder tests are also conducted at temperatures that are encountered by in-service pavements. Compared to other binder specification systems, a unique feature of the Superpave specification is that the specified criteria remain constant, but the temperature at which the criteria must be achieved changes for the various grades.

This is the basis for a performance grade (PG) rating, which consists of a high and a low temperature. The high temperature is the highest temperature at which the pavement has acceptable rutting resistance, while the low temperature is the lowest temperature at which the pavement is expected to resist low-temperature cracking. The fatigue cracking requirements are a function of the other two temperatures. Figure 11 shows the SHRP binder specifications for PG 46-X up to PG 64-X. The specifications are divided into three main parts; requirements for the original binder, requirements for binders after aging in a Rolling Thin Film Oven (RTFOT), and requirements for binders after aging in a Pressure Aging Vessel (PAV).

PERFORMANCE GRADE	PG 46-			PG 52-							PG 58-					PG 64-					
	34	40	46	10	16	22	28	34	40	46	16	22	28	34	40	10	16	22	28	34	40
Average 7-day Maximum Pavement Design Temperature, °C[a]	< 46			< 52							< 58					< 64					
Minimum Pavement Design Temperature, °C[a]	> -34	> -40	> -46	> -10	> -16	> -22	> -28	> -34	> -40	> -46	> -16	> -22	> -28	> -34	> -40	> -10	> -16	> -22	> -28	> -34	> -40
ORIGINAL BINDER																					
Flash Point Temp, T48: Minimum °C	230																				
Viscosity, ASTM D4402:[b] Maximum, 3 Pa·s, Test Temp, °C	135																				
Dynamic Shear, TP5:[c] $G^*/\sin\delta$, Minimum, 1.00 kPa Test Temp @ 10 rad/s, °C	46			52							58					64					
ROLLING THIN FILM OVEN (T240) OR THIN FILM OVEN RESIDUE (T179)																					
Mass Loss, Maximum, percent	1.00																				
Dynamic Shear, TP5: $G^*/\sin\delta$, Minimum, 2.20 kPa Test Temp @ 10 rad/s, °C	46			52							58					64					
PRESSURE AGING VESSEL RESIDUE (PP1)																					
PAV Aging Temperature, °C[d]	90			90							100					100					
Dynamic Shear, TP5: $G^*\sin\delta$, Maximum, 5000 kPa Test Temp @ 10 rad/s, °C	10	7	4	25	22	19	16	13	10	7	25	22	19	16	13	31	28	25	22	19	16
Physical Hardening[e]	Report																				
Creep Stiffness, TP1:[f] S, Maximum, 300 MPa, *m* - value, Minimum, 0.300 Test Temp @ 60s, °C	-24	-30	-36	0	-6	-12	-18	-24	-30	-36	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	-30
Direct Tension, TP3:[f] Failure Strain, Minimum, 1.0% Test Temp @ 1.0 mm/min, °C	-24	-30	-36	0	-6	-12	-18	-24	-30	-36	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	-30

[a] Pavement temperatures are estimated from air temperatures using an algorithm contained in the Superpave software program, may be provided by the specifying agency, or by following the procedures as outlined in PPX.

[b] This requirement may be waived at the discretion of the specifying agency if the supplier warrants that the asphalt binder can be adequately pumped and mixed at temperatures that meet all applicable safety standards.

[c] For quality control of unmodified asphalt cement production, measurement of the viscosity of the original asphalt cement may be substituted for dynamic shear measurements of $G^*/\sin\delta$ at test temperatures where the asphalt is a Newtonian fluid. Any suitable standard means of viscosity measurement may be used, including capillary or rotational viscometry (AASHTO T201 or T202).

[d] The PAV aging temperature is based on simulated climatic conditions and is one of three temperatures 90°C, 100°C or 110°C. The PAV aging temperature is 100°C for PG 58- and above, except in desert climates, where it is 110°C.

[e] Physical Hardening — TP1 is performed on a set of asphalt beams according to Section 13.1, except the conditioning time is extended to 24 hrs ± 10 minutes at 10°C above the minimum performance temperature. The 24-hour stiffness and *m*-value are reported for information purposes only.

[f] If the creep stiffness is below 300 MPa, the direct tension test is not required. If the creep stiffness is between 300 and 600 MPa the direct tension failure strain requirement can be used in lieu of the creep stiffness requirement. The *m*-value requirement must be satisfied in both cases.

FIGURE 11 SHRP Performance Graded Asphalt Binder Specifications *(Kennedy, et al., 1994)*

The equipment used in the Superpave binder specification is:

- Dynamic Shear Rheometer (DSR)
- Bending Beam Rheometer (BBR)
- Rolling Thin-Film Oven (RTFO)
- Pressure Aging Vessel (PAV)
- Rotational Viscometer (RV)
- Direct Tension Tester (DTT)
- Flash Point (FP)

DSR, BBR, RTFO and PAV will be discussed further in this chapter. RV, FP and DTT was not conducted in the work for this graduate thesis, and is therefore not discussed further.

The Superpave binder testing method has been criticized for not being able to accurately estimate deformation properties of polymer modified binders. Modified binders are more complex than conventional bitumen, and for some modified binders, the effects of polymer modification may only be apparent in the non-linear viscoelastic region. The binder test methods used in Superpave, however, only considers the binder properties in the linear region. The Superpave binder tests, therefore, do not adequately describe polymer modified binders. New specification methods for PMBs are beginning to be developed, however, and test methods such as Multiple Stress Creep Rheometer (MSCRT) is believed to describe their performance sufficiently.

4.1.1 Dynamic Shear Rheometer

The Dynamic Shear Rheometer (DSR) is used in Superpave to characterize the viscous and elastic behavior of bitumen. The test method measures the complex modulus, G^*, which is a measure of the resistance of a material to deformation when exposed to repeated pulses of shear stress, and phase angle, δ, which is a measure of the elastic response of the material. The complex modulus is a dynamic shear modulus, and is calculated as follows:

$$G^* = \frac{\tau_{max}}{\gamma_{max}}$$

where: $G^* = \text{complex modulus}$

$\tau = \text{shear stress}$

$\gamma = \text{shear strain}$

G^* consists of two components; an elastic (recoverable) component and one viscous (non-recoverable) component. δ is an indicator of the relative amounts of recoverable and non-recoverable deformation. The value of G^* and δ for bitumen are highly dependent on the temperature and frequency of loading. At high temperatures, bitumen behaves like a viscous fluid with no capacity for recovering deformation. At low temperatures, however, bitumen behaves like an elastic solid. Under normal pavement temperatures and traffic loading, bituminous binders act with the characteristics of both viscous liquids and elastic solids. By measuring the complex shear modulus and the phase angle, the DSR provides a more complete picture of the behavior of bitumen at pavement service temperatures compared to many other binder tests.

FIGURE 12 Dynamic Shear Rheometer

In the DSR test, the bitumen is poured into a silicon mold with the appropriate diameter and thickness for testing. The sample is then placed on a spindle which is fitted in the DSR. Since bitumen properties are temperature dependent, rheometers have a precise means of controlling the sample temperature. The sample is heated to a specified temperature, and the test begins. A computer control the DSR test parameters and record test results. Testing consists of setting the DSR to apply a constant oscillating stress and recording the resulting strain and time lag. The Super pave specifications require that the oscillation speed is 10 radians per second. The sample is first conditioned by loading the specimen for 10 cycles. After the 10 conditioning cycles, ten additional cycles are applied to obtain test data. The complex shear modulus, G^*, and phase angle, δ, is automatically calculated by the rheometer software and presented after testing is completed.

Research conducted as a part of the SHRP has shown that the parameter $G^*/\sin(\delta)$ correlates well with the asphalt mixture's resistance to deformation at high temperatures, while the relation $G^*\sin(\delta)$ is found to correlate well with fatigue resistance at intermediate temperatures. Validation studies have found, however, that by itself the parameters are not a sufficient indicator of an asphalt's deformation properties. Air voids and aggregate properties seem to have greater significance (Andersen, 1995, s. 14). The DSR test is not optimal for testing polymer modified binders, because the strain applied to the bitumen sample is too small for the polymer network to be activated. Due to the low strain level, the PG high temperature determined using DSR doesn't accurately represent the ability of polymer modified binders to resist rutting (The multiple stress creep recovery procedure, FHWA).

4.1.2 Bending Beam Rheometer

The bending beam rheometer (BBR) is the most widely used test device for determining the stiffness of bitumen at low temperatures. The BBR device measures how much a beam of bitumen will deflect under a constant load at temperatures corresponding to its lowest pavement service temperature when bitumen behaves like an elastic solid. The creep load applied to the bitumen sample is intended to simulate the stresses that gradually increase in a pavement as the temperature falls. Two parameters are determined in this test: the creep stiffness, S, and the creep rate, *m*. The creep stiffness is a measure of the resistance of the bitumen to constant loading, and the creep rate is a measure of how the bitumen stiffness changes as loads are applied. If the creep stiffness is too high, the asphalt will behave in a brittle manner, and low-temperature cracking is more likely. A high creep rate is desirable because, as the temperature changes and thermal stresses accumulate, the stiffness will change relatively quickly. A high value for the creep rate indicates that the bitumen will tend to disperse stresses that would otherwise accumulate to a level where low-temperature cracking could occur. Furthermore, the test is performed on binders that have been aged in both a rolling thin-film over (RFTO) and the pressure aging vessel (PAV). Therefore, the test measures the performance characteristics of binders as if they had been exposed to hot mixing in a mixing facility and some in-service aging.

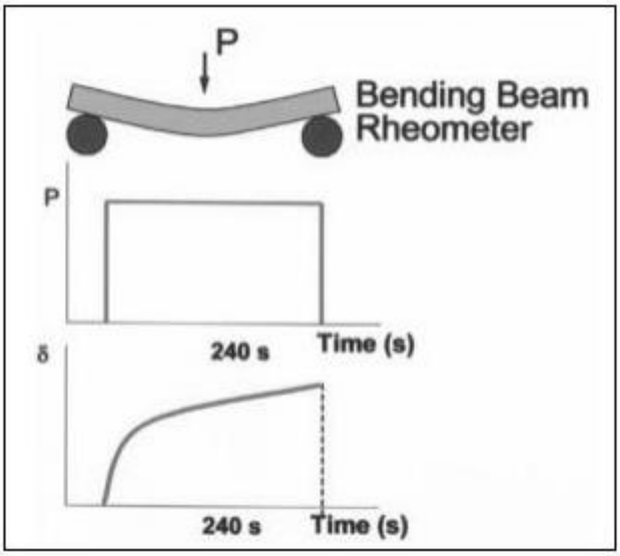

FIGURE 13 Principle of the BBR test *(Sybilski, Vanelstraete, & Partl, 2004)*

During the testing, a beam of bitumen is subjected to a constant stress by means of a loaded piston (100g) in a three-point bending machine. This is illustrated in figure 13. This beam is suspended in a cooling fluid. The displacement of the piston is measured as a function of time, and combined with other relevant parameters, this gives the relation:

$$S(t) = \frac{pL^3}{4\delta(t)bh^3}$$

where: $S(t)$ = creep stiffness of the bitumen

p = applied constant load

L = beam length

$\delta(t)$ = displacement of the piston at time t

b = beam width

h = beam height

For specification purposes, the creep stiffness, $S(t)$, and the *m* value are both determined at a load time of 60 s.

After being aged in the RTFO and the PAV, test specimens are prepared using a rectangular aluminum mold. Prior to assembling the mold, the inside surfaces of the mold are greased with a petroleum-based lubricant and strips of acetate are then placed against the greased surfaces. The mold is held together using two rubber rings. The bitumen beam sample is formed by heating the bitumen until fluid and pouring it into the mold from one end to the other in a continuous motion. A finished molded sample is shown in figure 14. After a cooling period of approximately 1 hour, excess bitumen is trimmed from the upper surface using a hot spatula. The specimen is then demolded and placed in the BBR bath for 60 minutes. After temperature conditioning, the beams are placed on the BBR supports. The beam is subjected to a series of load-conditioning steps, and then the actual test beings. A 980 mN load is applied to the beam for a total of 240 s. During the test, graphs of load and deflection versus time are continuously generated on the computer screen for observation. After 240 s, the test is completed and the rheometer software calculate creep stiffness, S, and creep rate, *m*, at 60 s.

FIGURE 14 A finished molded sample of bitumen ready for trimming

Sybilski et al (2004) write that modification of bitumen does not affect the way the BBR test is performed, and that the repeatability and reproducibility is the same as for neat binders. Although research has found a good correlation between BBR results of a bitumen and is low-temperature properties, it is less certain that how well the BBR can estimate the effect of polymer modification (Dongré, Button, Kluttz, & Anderson, 1997).

4.1.3 Rolling Thin-Film Oven Test

Oxidation and evaporation are regarded as the main causes of aging of bitumen. They are strongly accelerated by increasing temperatures and increasing surface area. The rolling thin-film oven test (RTFOT) is a bitumen aging test, and measures changes by both oxidation and evaporation. In this test, a thin film of bitumen is continuously rotated around the inner surface of a glass jar at 163 °C for 75 min with an injection of hot air into the jar every 3-4 s. The amount of bitumen hardening during the test correlates strongly with that observed during the manufacture of an asphalt (Vondenhof & Clavel, 2015). This hardening leads to a reduction in penetration value and an increase in the softening point of the bitumen. Usually a change in mass of the sample before and after the test will be observed. A loss of mass indicates volatile components in the bitumen. The mass of the sample might, however, increase due to reaction with oxygen. After testing, the RTFO-aged bitumen can be used for testing in a DSR, transferred into PAV pans for additional aging, or be stored for later use.

When tested in a RTFO, PMBs may show a decrease in the softening point, which is caused by the destruction of large polymer chains. These may then react with bitumen molecules, leading to a different structure (Vonk, Phillips, & Roele, 94).

4.1.4 Pressure Aging Vessel

Over the years there have been several attempts to simulate the long-term aging of bitumen in asphalts. This has, however, proven to be difficult because of the number of variables that affect bitumen aging, such as void content, mixture type. Aggregate type, etc. The pressure aging vessel (PAV) is used in the Superpave binder specification system to predict the changes in bitumen properties during the service life of the pavement. To do so, the PAV exposes the bitumen sample to high pressure and temperature for 20 hours. Since bitumen exposed to long-term aging has also been through the mixing and construction process, the PAV procedure uses bitumen aged in the RTFO. The artificial aging of bitumen in the PAV to simulate in-service aging has still to be fully validated, but the technique is widely accepted as a satisfactory approach (Hunter et al, p. 589).

To prepare for the PAV, RTFO-aged bitumen is heated until fluid and stirred to ensure homogeneity. Three PAV sample pans are prepared and the PAV pans are then placed in the sample rack. The aging process is conducted at different temperatures depending on the design climate. When the vessel temperature reaches set temperature, the pressure is applied and the timing for the aging period begins. After 20 hours, the samples are removed from the PAV and degassed. After this, the test is completed and the aged samples are ready for further testing or storage.

4.2 Multiple Stress Creep Recovery Test

The SHRP system was intended to use complex tests and specifications to improve the design of asphalt pavements and to reduce costs associated with road construction and maintenance. The Superpave binder test system, however, was based on studies of neat bitumen. The use of polymer modified bitumen has since SHRP's completion increased. The properties measured in Superpave are within the linear viscoelastic region and it does not appropriately capture the viscoplastic response. Multiple Stress Creep Recovery Test (MSCRT) measures parameters at higher strains and therefore engages the polymer network. This enables the MSCRT to more accurately capture the mechanical response of polymer modified bitumen compared to the high-temperature testing done using the DSR in Superpave. The MSCRT is a relatively new test developed by the Federal Highway Administration in the US and introduced by D'Angelo et al (2007).

MSCRT is a standard rheological test performed using a dynamic shear rheometer based on the repeated creep recovery test. The test method is used to determine the presence of elastic response in bitumen under shear creep and recovery at two stress levels at a specified temperature. The presence of this elastic response is determined by measuring the percent recovery, %R, and non-recoverable creep compliance, J_{nr}, of the binder:

- Percent recovery, %R, is defined as the recovered strain in a specimen during the recovery portion of a cycle, expressed in percent
- Non-recoverable creep compliance, J_{nr}, is defines as residual strain in a specimen after a creep and recovery cycle divided by the stress applied

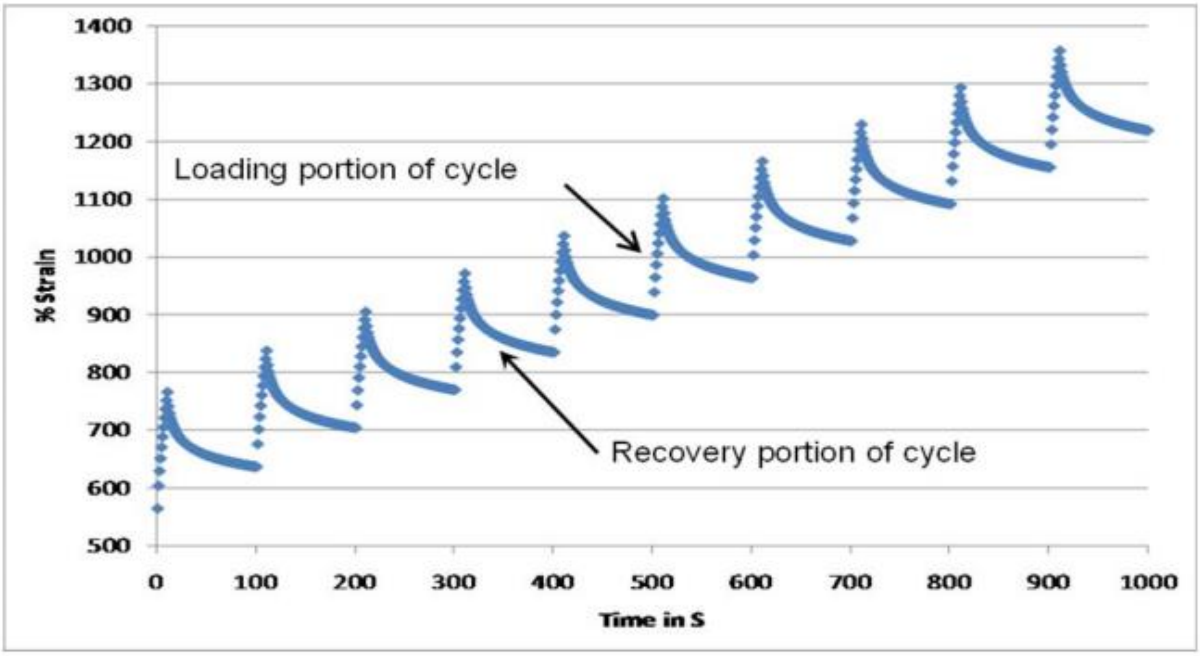

FIGURE 15 Illustration of ten creep and recovery cycles in MSCRT *(Federal Highway Administration, 2011)*

The total recoverable deformation, or percent recovery, is a measure of the elasticity of the material. The non-recoverable part relates to the viscosity. The percentage difference in non-recoverable creep compliance, $J_{nr-diff}$, between the stress levels at which the bitumen sample is tested can then be calculated. This parameter describes the bitumen's sensitivity to increased stress levels.

Ten creep and recovery cycles are run at 0,100 kPa creep stress followed by 10 more cycles at 3,200 kPa creep stress. The temperature that the test is conducted at, varies between 50 °C to 80 °C depending on the bitumen sample and the purpose of testing.

According to the Federal Highway Administration, the non-recoverable creep compliance value at a stress of 3,2 kPa has been shown to be an indicator of the resistance of bitumen to permanent deformation under repeated load (2011). In the test, a bitumen sample is subjected to a constant stress for 1 s, then allowed to recover for 9 s. A study done by D'Angelo et al. (2007) showed that by reducing non-recoverable creep compliance by half, rutting is typically reduced by half. Full scale testing of test sections constructed with multiple neat and modified binders was conducted by the Federal Highway Administration, and the results clearly showed an improved performance relationship of the MSCRT over the standard Superpave PG high-temperature binder criteria, $G^*/\sin(\delta)$ (2011). The test sections included, among others, neat and SBS-modified binders. In the report, it is written that:

> …the findings from the FHWA ALF study indicated that the existing Superpave high temperature parameter did not do a very good job of correlating with rutting. The correlation of G*/sin δ at 64°C to rutting only provided an R^2 value of only 0,13. The relationship of Jnr to rutting was significantly better, with an R^2 value of 0,82. The findings indicated that Jnr could identify the rutting performance of the modified as well as the non- modified binders used at the ALF, accurately ranking the rutting potential in all the test sections (Federal Highway Administration, 2011).

Thus, MSCRT can be used to assess the rutting resistance of both neat and polymer modified bitumen. This is perhaps the strongest incentive to add MSCRT into existing binder specification systems, as it would facilitate further development of PMB and its uses.

4.3 Wheel Track Test

So far, only bitumen tests have been discussed in this chapter. As discussed in chapter 3, however, there are several components in addition to bitumen which affects deformation properties of an asphalt mixture, such as aggregate gradation, void content, type of filler, etc. The properties of asphalts need to be known for a variety of reasons, including performance evaluation, mixture and pavement design, and production and construction specification. In situ testing of material properties in full scale trial sections or in-service pavements is impractical or uneconomical in most cases, so pavement engineers generally rely on laboratory testing to characterize material properties. As the stress conditions in a pavement loaded by a rolling wheel are complex and cannot accurately be replicated in a laboratory test on a sample of asphalt, simulative tests have been used to compare the performance of different asphalt materials. Laboratory wheel-tracking devices have been developed for this purpose. In the wheel track test, the susceptibility of a bituminous material to deform is assessed by measuring the rut depth formed by repeated passes of a loaded wheel over asphalt samples at a fixed temperature. Performance of test specimens is then used to assess, compare and rank the estimated in-service deformation properties of the asphalt mixtures.

The wheel-tracking device consists of a loaded wheel that runs on a sample held securely in place. Either the wheel moves back and forth over the sample, or alternatively the sample moves while the wheel is in place. Measurements of the deformation of the asphalt sample is registered and graphically illustrated on a monitor. Wheel-tracking devices range from large full scale pavement test facilities to smaller laboratory scale wheel tracking machines. In this thesis, laboratory scale wheel tracking machines are described. Such a device is shown in figure 16. Due to variation in testing parameters and conditions, the different devices produce different results and comparison of test results from different devices should be avoided if possible.

FIGURE 16 Asphalt specimen being tested in a wheel-tracking device

Research has shown that the test temperatures and initial air void contents have the greatest effects on the test results; rutting increases with higher test temperatures and air void contents (Cooley Jr, Kandhal, Buchanan, Fee, & Epps, 2000). Due to the temperature at which samples are tested, the WTT can test the asphalt mixture's rutting resistance at temperatures close to critical performance levels. This, combined with the simulative nature of the test, makes the WTT suitable for estimating permanent deformation properties of asphalt mixture with neat bitumen and with polymer modified bitumen.

5 Experiments and results

In this chapter, the materials used for laboratory testing, specimen preparation and testing procedure are described. Laboratory results are also presented.

5.1 Materials and specimen preparation

To test the effect polymer modified bitumen has on deformation characteristics of asphalt concrete mixtures, different materials were used. Four bitumen binders were tested in the laboratory work conducted for this thesis:

- 160/220 penetration bitumen
- 70/100 penetration bitumen
- Two polymer modified bitumen binders, named PMB1 and PMB2

Bitumen binders were provided by Veidekke Industri. Before PG-classification of 160/220 bitumen, 70/100 bitumen and PMB1 was conducted, samples of the specimens were first aged in the RTFO and then in the PAV. PG-classification of the 160/220 bitumen, 70/100 bitumen and PMB1 was done using the DSR on unaged bitumen samples, and BBR on RTFO and PAV aged samples. A verification of the PG-rating of PMB2 was conducted in a similar fashion. Unaged samples of PMB1 and PMB2 were tested in a DSR for the MSCR test.

AC11 is an asphalt mixture with nominal size of 11,2 mm commonly used in Norway. For wheel track testing, three AC11 mixtures with different gradation curves were designed using Marshall method. Binder and aggregates were heated in an oven for approximately 2 hours at 145 °C for 70/100 bitumen and 180 °C for PMB1 and mixed in a mixer. After mixing, the material was divided into appropriate portions for the wheel track mold and put into the mold. After conditioning at the temperature set for the compactor for about 2 hours, the asphalt mixtures were compacted inside the mold using a Cooper compactor. Thickness of the final samples was 50 mm. The asphalt concrete samples were then left for roughly 24 hours at room temperature before testing begun. The mixtures were named AC11-1, AC11-2 and AC11-3. The mixtures consisted of the same materials, except for the binder. Corresponding samples of each of the three AC11 mixtures were made using 70/100 bitumen and PMB1. Two samples with 70/100 and PMB1 were made for each of the three AC11 mixtures, making a total of 12 samples:

- 2 samples of AC11-1 with 70/100 bitumen
- 2 samples of AC11-2 with 70/100 bitumen
- 2 samples of AC11-3 with 70/100 bitumen
- 2 samples of AC11-1 with PMB1
- 2 samples of AC11-2 with PMB1
- 2 samples of AC11-3 with PMB1

Information regarding the polymer modified bitumen and proportioning of asphalt concrete mixtures will not be shared in detail this thesis.

5.2 Testing procedure

5.2.1 RTFO and PAV aging of bitumen

RTFO aging of bitumen samples was performed in accordance with NS-EN 12607.

PAV aging of bitumen samples was performed in accordance with NS-EN 14769.

5.2.2 Dynamic Shear Rheometer

The DSR was used to establish the PG-classification high-temperature limit for the bitumen samples. The test was performed in accordance with NS-EN 14770. The bitumen samples were heated until liquid for a sufficient amount of time so that they could be poured easily into small cups. Test samples of approximately 0,7 g were then made by pouring the hot, liquid binder onto thin plastic sheets. After being allowed to cool for 1 hour, the samples were then loaded into the DSR and trimmed using a hot spatula. The test started at an appropriate set temperature depending on the bitumen sample. After testing at this temperature, the samples either passed or failed the requirement of having a $G^*/\sin(\delta)$ ratio over 1 kPa. If the test was passed, testing at the next, higher temperature interval set by the PG-classification ensued. If the test sample failed, however, the PG-classification high-temperature limit was set to the previous temperature at which the sample passed. To ensure that the correct high-temperature limit was found, two samples of each bitumen were tested.

5.2.3 Bending Beam Rheometer

To establish the low-temperature PG-classification limit of the bitumen samples, the BBR was used. The test was performed in accordance with NS-EN 14771. After being aged in RTFO and PAV, bitumen samples were heated until liquid for a sufficient amount of time so that they could be stirred and poured easily. After stirring to ensure homogeneity and being reheated for 5 minutes, test samples were created by continuously pouring bitumen in one motion into lubricated metal molds fitted with plastic sheets. After being allowed to cool for 1 hour, excess bitumen was trimmed using a hot spatula. The test samples were then demolded and put into the BBR for temperature conditioning. The BBR temperature was set to an appropriate temperature depending on the bitumen sample. After conditioning, the test samples were put onto the BBR supports and subjected to a constant load. When the test was complete, the creep stiffness, $S(t)$, and the creep rate, m, at 60 seconds was calculated. If the creep stiffness did not exceed 300 and the creep rate was higher than 0,3, new test samples were created and testing at the next, lower temperature interval set by the PG-classification ensured. If the test sample failed, however, the PG-classification low-temperature limit was set to the previous temperature at which the sample passed. To ensure that the correct low-temperature limit was found, two samples of each bitumen were tested.

5.2.4 Multiple Stress Creep Recovery Test

In order to compare the deformation characteristics of two unaged polymer modified bitumen binders, the MSCRT was used. The test was performed in accordance with NS-EN 16659. PMB1 and PMB2 were heated in an oven maintained at 180 °C until liquid. The bitumen was then stirred, reheated and poured onto plastic sheets similarly to how samples were made for the DSR. After being allowed to cool for 1 hour, samples were loaded into the DSR and trimmed. The tests were run at 60 °C using the DSR in controlled constant creep mode. After the MSCR test was started, the bitumen samples were loaded at a constant creep stress of 0,1 kPa for 1,0 s and then followed by a recovery period for 9,0 s. Stress and strain levels were recorded every 0,1 s for the creep cycle, and every 0,45 s for the recovery cycle. This cycle was repeated 10 times. Ten cycles with a constant creep stress of 3,2 kPa and ten cycles with 6,4 kPa were then run in a similar fashion. NS-EN 16659 does not require testing at 6,4 kPa. This was done to investigate the deformation properties of the two PMBs when exposed to high stress and strain levels. Two unaged samples of each binder were tested.

5.2.5 Wheel Track

Wheel track test was used to be able to compare the rut development and permanent deformation properties of asphalt concrete mixtures with 70/100 bitumen and PMB1. After asphalt mixture test samples had been conditioned at room temperature for roughly 16 hours, they were heated up to and kept at 50 °C for an additional 12 hours. Testing at 50 °C in a Cooper wheel-tracking device was then started. Two samples were tested simultaneously.

5.3 Results

5.3.1 Results of PG-classification

Table 1 shows the highest DSR temperature and lowest BBR temperature the different binders passed when tested. The final PG-rating is also shown.

Bitumen binder	DSR [°C]	BBR [°C]	PG-rating
160/220	52	-12	PG 52-22
70/100	58	-6	PG 58-16
PMB1	70	-18	PG 70-28
PMB2	64	-18	PG 64-28

TABLE 1 Results of PG-classification of binders

5.3.2 Results of MSCRT

Table 2 shows the %Recovery and J_{nr} results of the MSCRT.

Bitumen binder	%Recover [%]			J_{nr} [kPa^{-1}]		
	0,1 kPa	3,2 kPa	6,4 kPa	0,1 kPa	3,2 kPa	6,4 kPa
PMB1 sample 1	97,7	97,2	97,2	0,005	0,006	0,007
PMB1 sample 2	97,8	97,2	97,4	0,005	0,006	0,006
PMB2 sample 1	88,5	77,6	71,9	0,114	0,258	0,354
PMB2 sample 2	82,1	78,8	75,8	0,159	0,211	0,259

TABLE 2 Results from MSCRT of unaged polymer modified binders

Figure 17 displays the mean values of the percentage recovery at 0.1, 3.2, and 6.3 kPa found in MSCR test of PMB1 and PMB2. One standard deviation is included to give an impression of the spread of the results.

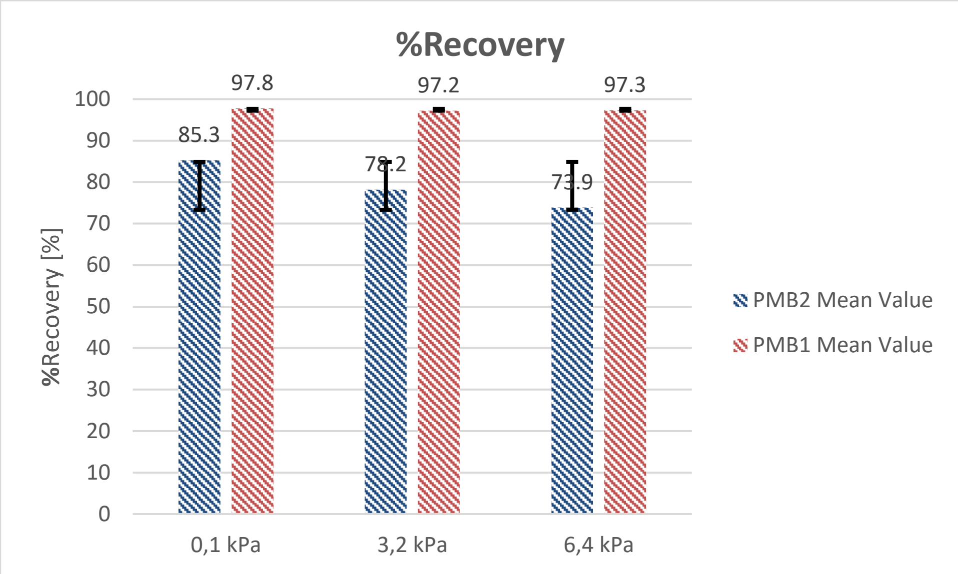

FIGURE 17 Elasticity of unaged polymer modified binders at different stress levels expressed as the mean value of %Recovery found in MSCRT

Figure 18 displays the mean values of non-recoverable creep compliance, J_{nr}, at 0.1, 3.2, and 6.3 kPa found in MSCR test of PMB1 and PMB2. One standard deviation is included to give an impression of the spread of the results.

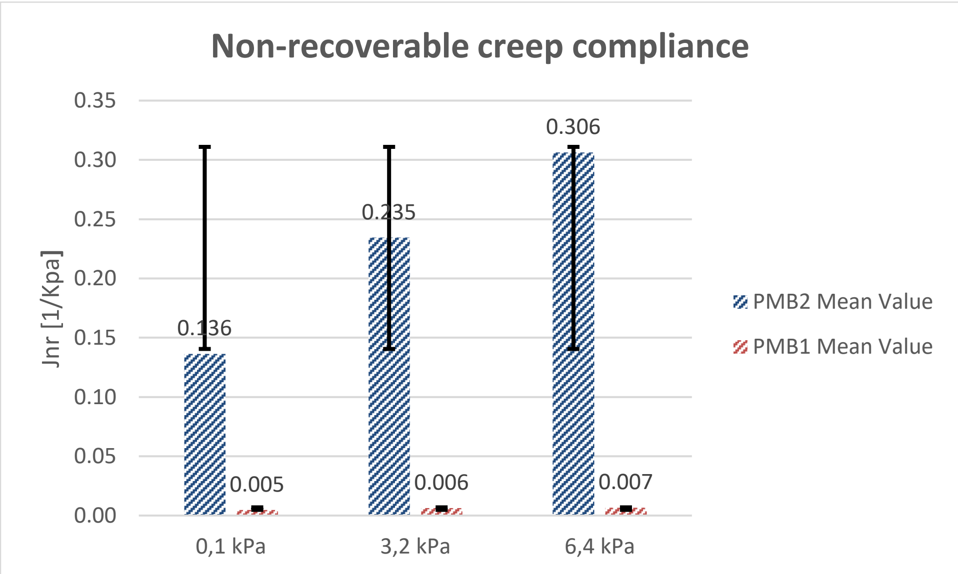

FIGURE 18 Plastic deformation of unaged polymer modified binders at different stress levels expressed as the mean value of J_{nr} found in MSCRT

5.3.3 Results of Wheel Track

Table 3 shows the results from wheel track testing of asphalt concrete mixtures made with 70/100 bitumen and PMB1.

Asphalt mixture	Mean rut depth after 1000 passages [mm]	Mean rut depth after 20 000 passages [mm]	Mean deformation [%]
AC11-1 70/100	0,234	5,3	10,6
AC11-2 70/100	0,254	5,9	11,7
AC11-3 70/100	0,181	5,5	10,9
AC11-1 PMB1	0,026	1,5	2,9
AC11-2 PMB1	0,027	1,6	3,1
AC11-3 PMB1	0,019	1,9	3,7

TABLE 3 Mean results of wheel track testing

Figures 19 to 21 displays the mean rut depth results from the wheel track testing of the different AC11 samples with 70/100 and PMB1. Figure 22 shows the total deformation that occurred during the test.

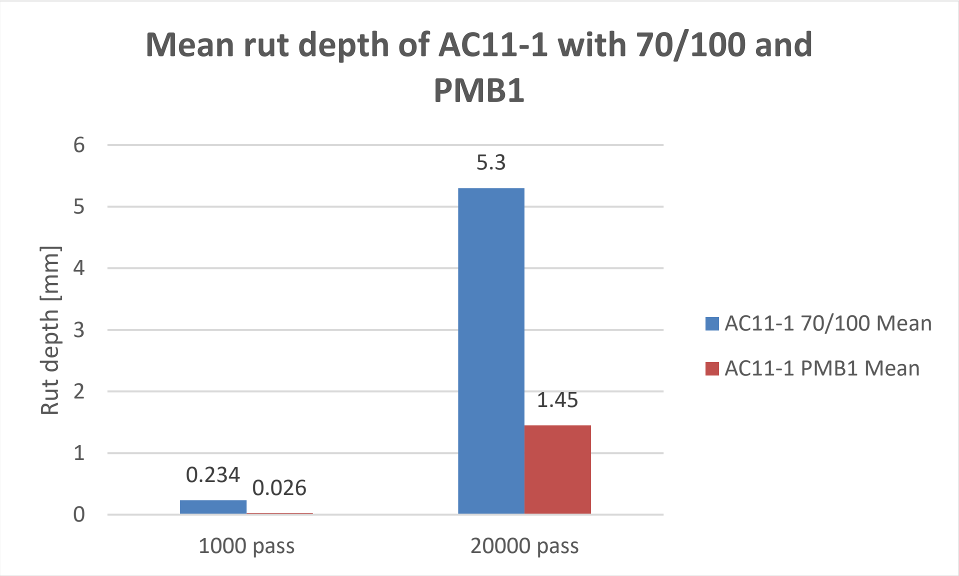

FIGURE 19 Mean rut depth of the AC11-1 samples with 70/100 and PMB1

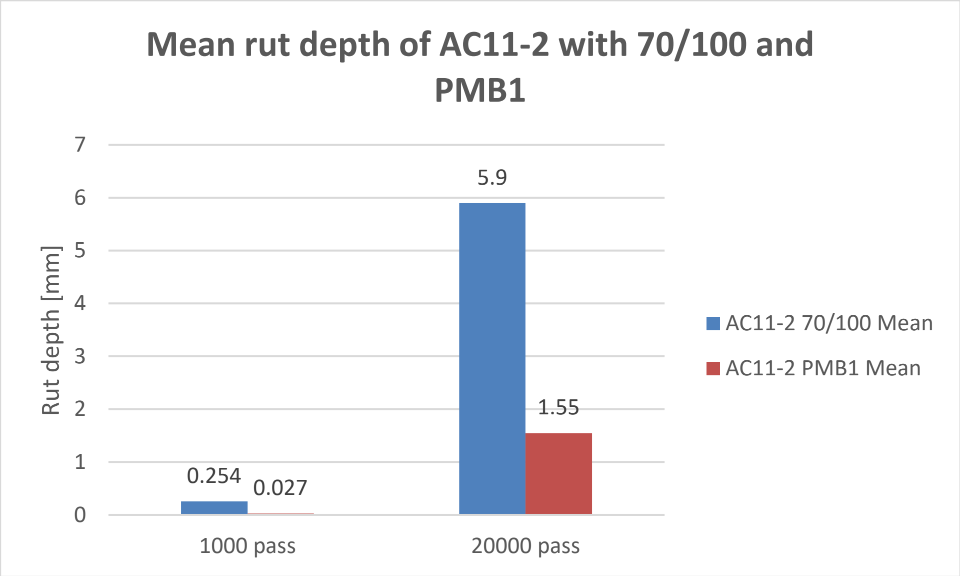

FIGURE 20 Mean rut depth of the AC11-2 samples with 70/100 and PMB1

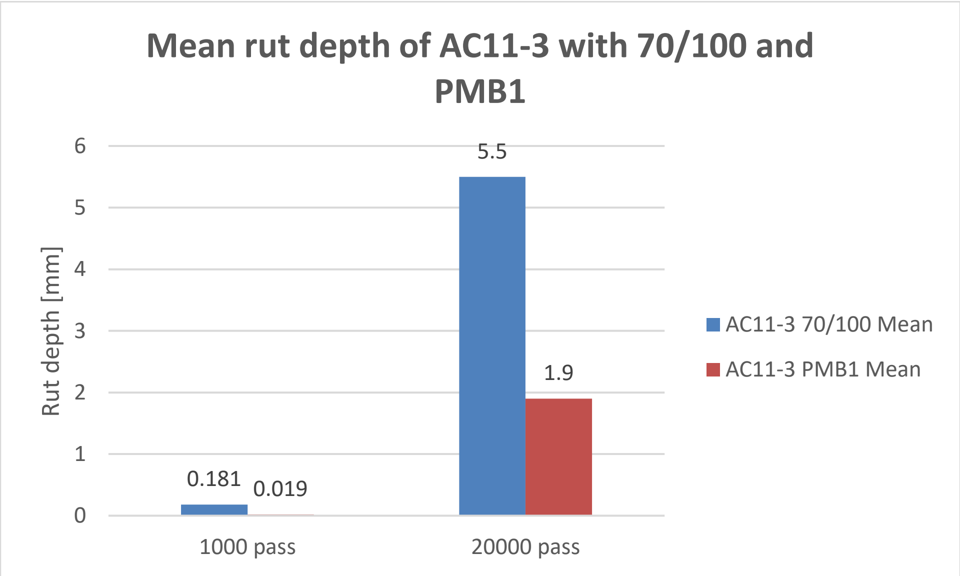

FIGURE 21 Mean rut depth of the AC11-3 samples with 70/100 and PMB1

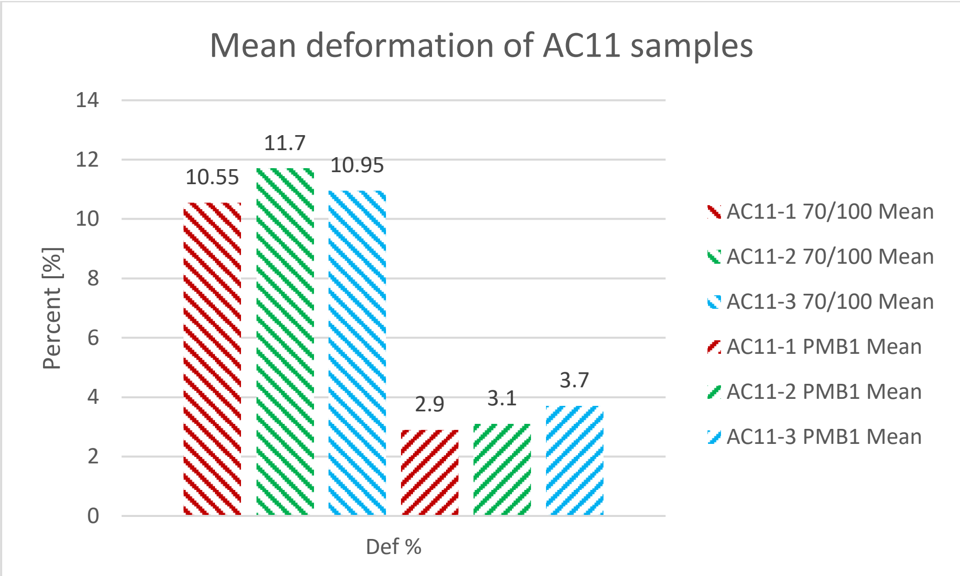

FIGURE 22 Mean deformation of the AC11 samples

6 Analysis and discussion

To analyze the test results from the MSCRT and Wheel Track, Student's t-test was used. This is a statistical test which is used to determine if two datasets are significantly different from each other. Student's t-test deals with the problems associated with interference based on small samples. Because the two datasets analyzed have a small number of samples, the calculated mean and standard deviation may by chance deviate from the "real" mean and standard deviation (College of Saint Benedict & Saint John's University, 2017). It is assumed in this thesis that the results from MSCRT and Wheel Track follow a normal distribution, and that for each test, the variance of the datasets are equal. The null hypothesis for both MSCRT and Wheel Track was that there is no significant difference between test results, and a significance level of 0,05 was chosen as the limit.

The results of the PG-classification conducted in the laboratory work for this thesis, show that the two types of polymer modified binders have a greater temperature interval at which the binders have acceptable estimated in-service properties compared to the two neat binders. The two polymer modifier binders have sufficient rut resistance at higher temperatures than the neat binders, while also having acceptable resistance to thermal cracking at lower temperatures. The temperature susceptibility of PMB1 and PMB2, therefore, seems to be lower than that of 70/100 and 160/220 bitumen. This relates well with research found during the literature review for this thesis.

PMB1, classified as PG 70-28, was found to be one high-temperature grade higher than PMB2, which was classified as PG 64-28. Based on these results, the temperature interval at which PMB1 will have acceptable permanent deformation properties is greater that of PMB2. For the neat binders, 70/100 bitumen was classified as PG 58-16 and 160/220 as PG 52-22. The Performance Grade temperature interval was consequently found to be equal, but 160/220 was offset one grade lower for both high and low-temperature limit than 70/100. This illustrates that softer neat binders have lower rutting resistance at high temperatures, but increased flexibility and thus resistance to cracking at low temperatures.

The results of the PG-classification illustrate that instead of using softer neat bitumen on low-volume roads to get a flexible pavement, polymer modified binders could be used. This would also increase the pavement's rutting resistance at high temperatures.

t-test values	**%Recover**			J_{nr}		
	0,1 kPa	**3,2 kPa**	**6,4 kPa**	**0,1 kPa**	**3,2 kPa**	**6,4 kPa**
PMB1 vs PMB2	0,0598	0,0009	0,0071	0,0280	0,0103	0,0240

TABLE 4 t-values for MSCRT test samples using a two-tailed Student's t-test

The results of the MSCRT test showed that for percent recovery, %Recovery, PMB1 was significantly different from PMB2 at 3,2 kPa (t-value: 0,0009) and 6,4 kPa (t-value: 0,0071). For 0,1 kPa, however, there was not found any significant difference (t-value: 0,0598). Based on the %Recovery of PMB1 and PMB2, it seems that PMB1 is more elastic at higher stress levels than PMB2. Because PMB1 has a significantly higher %Recovery value at 3,2 kPa and 6,4 kPa than PMB2, this means that a greater percentage of the binder returns to its previous shape after being repeatedly stretched and relaxed under heavy traffic loading and high temperatures.

For non-recoverable creep compliance, J_{nr}, PMB1 was significantly different from PMB2 at 0,1 kPa (t-value: 0,028), 3,2 kPa (t-value: 0,0103) and 6,4 kPa (t-value: 0,0240). Thus, it seems that PMB1 has improved resistance to plastic deformation at high stress levels and high temperatures compared to PMB2. It is important to note that at increasing stress levels, the non-recoverable creep compliance of PMB2 increases, while J_{nr} of PMB1 stays roughly the same. This could be due to a more successful activation of the polymer network in PMB1 at higher stress levels. Based on the findings of D'Angelo et al (2007) and the FHWA (2011), the results of the MSCR test can be interpreted such that PMB1 is likely to have a significantly higher resistance to rutting than PMB2.

t-test values	**1 000 passages**	**20 000 passages**	**Deformation**
AC11-1 70/100 vs AC11-1 PMB1	0,0456	0,0236	0,0258
AC11-2 70/100 vs AC11-2 PMB1	0,0007	0,0049	0,0050
AC11-3 70/100 vs AC11-3 PMB1	0,0230	0,0038	0,0046

TABLE 5 t-values for Wheel Track test samples using a two-tailed Student's t-test

As shown in table 5, all AC11 mixtures with PMB1 were found to be significantly different from the corresponding AC11 mixtures with 70/100 penetration bitumen. Therefore, rut depths after 1000 and 20 000 passages were found to be significantly lower in the AC11 mixtures with PMB1 than in the AC11 mixtures with 70/100. For the AC11 mixtures with PMB1, the rate of rut depth seems to stabilize after roughly 7-8000 passages. Rutting development after this stabilization is minimal. For the AC11 mixtures with 70/100, on the other hand, the rate of rutting does not seem to stabilize before the test is completed. See the appendix for Wheel Track charts.

It is, however, important to note that the air void contents of the wheel track samples were not recorded. Therefore, some of the sample deformation could be caused by compaction of the asphalt mixture rather than shear deformation. The samples should, however, have approximately the same air void content, and it is assumed that most of the deformation is due to shear deformation of the asphalt mixture.

New roads designed and constructed in accordance with N200 ensures that proper bearing capacity is achieved. The Norwegian Road Administration states that the normalized pavement lifetime of an asphalt concrete pavement with traffic volumes ranging from 1501 to 3000, is 15 years. For new roads, this number might be higher, however. If the assumption that the predominant deterioration mechanism on low-volume roads with a high percentage of heavy traffic is permanent deformation caused by shear deformation in the bituminous layers and that PMB1 is a representative polymer modified binder, the results from the wheel track test indicates that the use of polymer modified binder can significantly improve pavement rutting resistance. This in turn could lead to an improvement in pavement lifetime. Quantifying this improvement if difficult, but based on the results found in this thesis and by other researchers, 3-4 years longer pavement lifetime seems possible to achieve by using polymer modified binder. An increase in pavement lifetime is likely even if other deterioration mechanisms, such as low-temperature cracking, takes place, since polymer modified binders have improved properties at both high and low temperatures compared to neat binders.

7 Conclusions

The objective of this thesis is to give further insight into the effect polymer modification has on rutting in low-volume roads with a high percentage of heavy vehicle traffic. By using laboratory test that are judged to be simulative of field loading conditions, the goal of the study is to provide more knowledge on the effect polymer modification has on permanent deformation in asphalt concrete mixtures made for low-volume traffic conditions. Based on the results of the laboratory work conducted in this study, the following conclusions can be drawn:

- The two polymer modified binders have better rutting and low-temperature cracking resistance at a higher and lower temperatures respectively than the neat binders
- The MSCRT results indicate that PMB1 has a significantly higher resistance to permanent deformation under heavy traffic loads than PMB2.
- Asphalt concrete mixtures made with PMB1 have significantly higher resistance to rutting than asphalt concrete mixtures containing 70/100 penetration bitumen
- By using polymer modified binders, the lifetime of asphalt concrete pavements will likely increase

Further testing and research needs to be conducted to learn more about the effects using polymer modified bitumen have on the deformation characteristics of low-traffic asphalt pavements:

- more types of polymer modified bitumen should be tested
- testing of different types of low-traffic asphalt pavement mixtures should be conducted
- improvements in pavement lifetime by using polymer modified bitumen should be quantified
- low-volume road sections with polymer modified bitumen should be constructed and monitored

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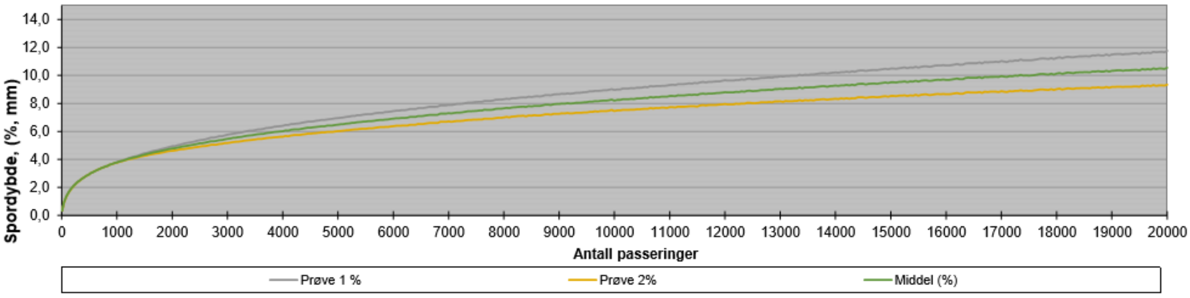
Appendix

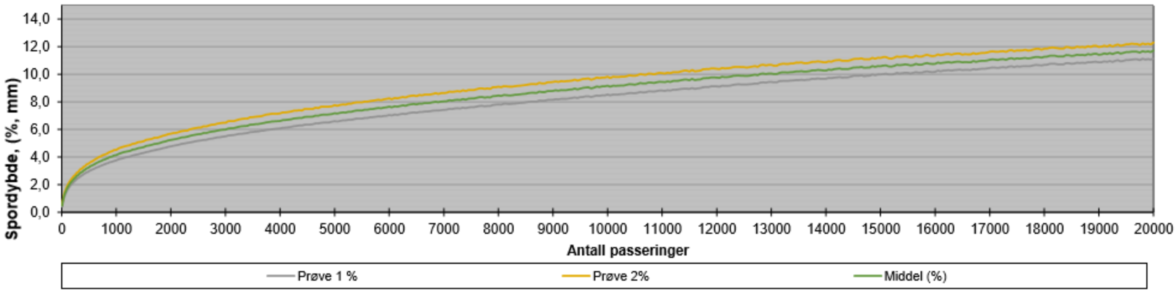
AC11-1 70/100

Test sample	Thickness	Rutting after 1 000 passages [mm]	Rutting after 20 000 passages [mm]	Deformation [%]
1	50mm	0,28	5,9	11,8
2	50mm	0,188	4,7	9,3
Mean	50mm	0,234	5,3	10,55



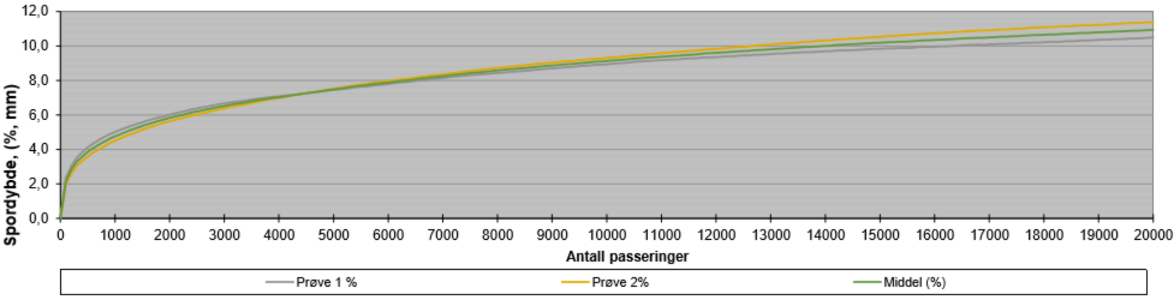
AC11-2 70/100

Test sample	Thickness	Rutting after 1 000 passages [mm]	Rutting after 20 000 passages [mm]	Deformation [%]
1	50mm	0,26	5,6	11,1
2	50mm	0,248	6,2	12,3
Mean	50mm	0,254	5,9	11,7



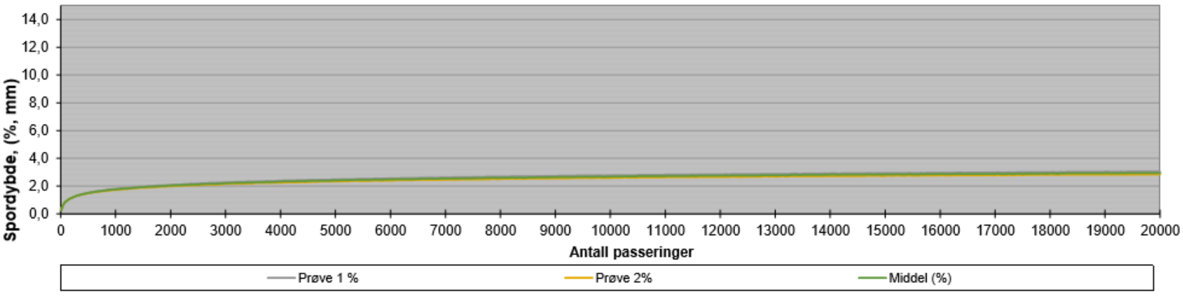
AC11-3 70/100

Test sample	Thickness	Rutting after 1 000 passages [mm]	Rutting after 20 000 passages [mm]	Deformation [%]
1	50mm	0,156	5,3	10,5
2	50mm	0,206	5,7	11,4
Mean	50mm	0,181	5,5	10,95



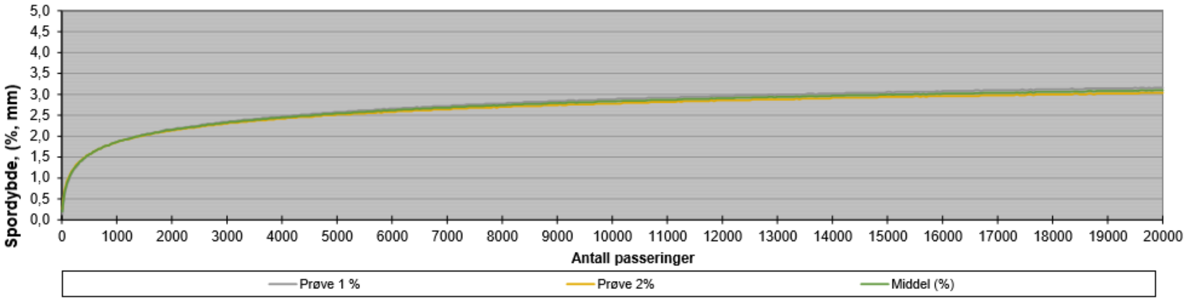
AC11-1 PMB1

Test sample	Thickness	Rutting after 1 000 passages [mm]	Rutting after 20 000 passages [mm]	Deformation [%]
1	50mm	0,026	1,5	3
2	50mm	0,026	1,4	2,8
Mean	50mm	0,026	1,45	2,9



AC11-2 PMB1

Test sample	Thickness	Rutting after 1 000 passages [mm]	Rutting after 20 000 passages [mm]	Deformation [%]
1	50mm	0,028	1,6	3,2
2	50mm	0,026	1,5	3
Mean	50mm	0,027	1,55	3,1



AC11-3 PMB1

Test sample	Thickness	Rutting after 1 000 passages [mm]	Rutting after 20 000 passages [mm]	Deformation [%]
1	50mm	0,02	2	3,9
2	50mm	0,018	1,8	3,5
Mean	50mm	0,019	1,9	3,7