Ageing and Degradation of Asphalt Pavements on Low Volume Roads

Thesis by

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SUMMARY

An important objective with this research work has been to get more knowledge about mix design and maintenance of low volume roads with asphalt pavements in Norway.

A literature review on bitumen, filler, rheology and ageing has been carried out. This has been focused on bitumen and filler because of their importance in the asphalt mixture. Bitumen as the binder in the asphalt pavement, and filler because of its relatively large surface area compared to the rest of the aggregates. Filler will therefor have a relatively large contact area to the bitumen and hence is important for the properties of the bitumen as binder in the pavement.

An investigation on deterioration of asphalt pavements on low volume roads in Norway has been carried out. The most frequent types of distress found on these roads were; edge cracks, cracks due to frost heave, rutting, transverse cracks and ageing. It is difficult to classify ageing as a type of distress, but the ageing process influences several pavement properties. The service life of an asphalt pavement on low volume roads is highly dependent on the ageing properties of the binder. An important challenge in pavement maintenance and mix-design is to find a proper binder and asphalt mixture according to local conditions, such as temperature, structure conditions and bearing capacity.

An important factor contributing to the ageing process in the field is the climate. In this work, samples from two different climatic zones have been analysed. Mechanical studies of asphalt core samples were performed. In addition recovered binder from the asphalt core samples and laboratory aged binder were studied. Both SHRP binder specification tests and conventional tests have been used in the analyses of the binder. In this investigation it was found that the age hardening of the bitumen is most severe in the upper part of the pavement and decrease with depth. Another important conclusion from this part of the study is that climatic conditions should be taken into account in a more comprehensive way than today when choosing binder for asphalt pavement.

It is well known that a relative great part of the age hardening of bitumen occurs during production, transportation and paving. An investigation on two transports with freighter was performed, to investigate the magnitude of age hardening of bitumen during long transportation (15 – 50 hours). The temperature was measured in three levels of the load in one of the transports. The temperature 0,5 m from the top and 0,5 m from the bottom of the load decreases by 1,2°C/hour and 1,34°C/hour, respectively. The temperature almost remaind constant in the core of the asphalt mixture. After a long transportation it is recommended to mix the asphalt from different levels in the load. This is due to the fact that the compaction temperature will differ significantly when taking the asphalt mixture directly from the load without any mixing. High and varying void content in the pavement could be the result when the compaction temperature is varying and sometimes is to low. Another finding from this investigation was that the hardening during transportation seems to be dependent upon type of binder. The long transportation does not seem to contribute excessive hardening of Pen180, while MB10000 (soft binder) did harden significantly during transportation.

Because the surface area of the filler material in an asphalt mixture represents a major part of the total surface area of the aggregate, a study of how different types of fillers affect the rheological and ageing properties of filler-bitumen mixes has been conducted. Four different types of fillers where used, among them hydrated lime, which is claimed to have positive influence on the ageing properties of binders. Two types of binder were used in the study. The analyses were carried out using test equipment according to the SHRP binder specification.
Based on the result from this investigation some main conclusions are:

- When producing filler-bitumen mixes of equal stiffness, the “effective filler concentration concept”, based on Rigden void content seems to correlate to the high temperature stiffness. At low temperatures (-20/-25°C) and intermediate temperatures (10 - 30°C) however, these mixtures were found to vary in stiffness.
- Based on BBR and DTT measurements of filler-bitumen mixes, the net effect of hydrated lime on low temperature cracking properties seems to be very small.
- No clear variation in ageing properties for mixes containing different types of fillers can be seen. However, there seems to be some variations between the two types of binders used in the investigation.
- The void content in the filler is an important factor, which should be evaluated in the asphalt mix-design in a more comprehensive way than today.
- The SHRP-binder equipment seems to be a step forward in evaluating the properties of filler-bitumen mixes.

PROJECT PROGRAMME

Introduction

Asphalt pavement research in Norway has for many years now mainly focused on problems related to wear caused by studded tyres on high-volume roads. As a result of this, wear resistant asphalt-mixes have been developed.

Research work on ageing on asphalt pavements on airfields in Norway has also been conducted. Long-time pavement performance is important for asphalt pavements on airfields. Ageing due to climatic exposures is among the most important factors influencing the deterioration on these pavements.

About 95% of the Norwegian road network consist of low-volume roads, that means roads with an annual average daily traffic (AADT) of less than 3000. On these roads, other distress mechanisms than wear from studded tyres dominate. The dominant distress factors here are permanent deformations, ageing, fatigue and other various types of cracking. On low-volume roads the challenges seems to be the same as for asphalt pavements on airfields, because ageing and climatic exposure are among the most important factors influencing the degradation. In Norway there is great variation in topography and climatic conditions. The quality of road building materials also varies considerably between different districts.

Research on asphalt pavements on low volume roads has only been performed to a minor degree, even though the Norwegian road network mainly consists of low volume roads. An important issue of this work is to get more knowledge about building and maintenance of low-volume roads with asphalt pavements. Better knowledge about the choice of proper binders adapted to the climatic conditions and how to limit hardening of asphalt mixtures is important. This knowledge and findings presented in this thesis, hopefully will contribute to better performance and more durable asphalt pavements.

Hence, we are facing a main challenge on low volume roads:

> ***How to evaluate material properties for proper long-time pavement performance.***

Approach

In this work the challenge of road maintenance and evaluation of material properties for long-lasting asphalt pavements on low-volume roads will be studied.

The main approach of the study has been:

How do climatic exposure and material properties affect the ageing of asphalt pavements on low volume roads?

This approach covers many factors and it is necessary to separate this into several topics:

1. *Which types of defects are predominant on asphalt pavements on low-volume roads? Which mechanisms could affects the deterioration of asphalt pavements on low-volume roads?*
2. *Is there any significant difference in pavement ageing in different climatic areas, and how is the development of hardening in soft binders with time? How do field aged samples compare to laboratory aged bitumen (using TFOT and PAV)?*
3. *The production process is important to ageing of bitumen. How do transportation of asphalt mixtures with freighter over long distances affect the binder properties?*
4. *Asphalt pavements consist of aggregates and binder. The ageing properties of the binder are important for the long-time pavement performance. What are the chemical constitutions of bitumen and what are the rheological and ageing properties?*
5. *Aggregate and filler properties could affect the ageing process of bitumen in an asphalt-mix. How does filler (aggregate less than 75 microns) affect the ageing? An investigation on hydrated lime compared to other fillers will be of particular interest in this connection.*

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LIST OF SYMBOLS AND ABBREVIATIONS

Symbol:	**Meaning:**
τ	Shear stress
γ	Shear strain
$\dot{\gamma}$	Shear strain rate
η	Dynamic viscosity
η_a	Viscosity after ageing
η_o	Steady state viscosity (Newtonian viscosity) also used for viscosity of original bitumen (before ageing)
ν	Kinematic viscosity
ρ	Density
G	Shear modulus
G^*	Complex modulus
G'	Storage modulus (elastic component)
G''	Loss modulus (viscous component)
δ	Phase angle
S(t)	Time-dependent stiffness modulus
σ_0	Uniaxial stress
$\varepsilon(t)$	Resulting uniaxial strain at time t
R	Rheological Index
ω	Frequency
ω_c	Crossover frequency
t_c	Crossover time
G^*_g	Glassy modulus
E	Elastic modulus
P_{max}	Maximum load
S_t	Tensile strength
a	Load distribution coefficient
$T_{surface}$	Temperature in pavement
T_{air}	Temperature in air
T_{cr}	Critical cracking temperature
C_e	Effective filler concentration (based on Rigden void content)
C_v	Volume concentration
C_{vmax}	Concentration compacted filler
V_{filler}	Volume of filler
V_{binder}	Volume of binder
V_o	Void in compacted filler (Rigden)
ρ_s	Specific density
ρ_d	Filler density
m_{filler}	Mass of filler

AADT	Annual Average Daily Traffic
BBR	Bending Beam Rheometer
BTDC	Bitumen Test Data Chart
CEN	European Comity for Standardisation
DSR	Dynamic Shear Rheometer
DTT	Direct Tension Test
HL	Hydrated Lime
HMA	Hot Mix Asphalt
HMWC	High Molecular Weight Constituents
HP-GPC	High Pressure Gel Permeation Chromatography
IEC	Ion Exchange Chromatography
LVE	Linear ViscoElastic
MB10000	Soft bitumen, number after MB gives viscosity at 60°C
NTNU	Norwegian University of Science and Technology
PAV	Pressure Ageing Vessel
PI	Penetration Index
PG	Performance grade
PVN	Penetration Viscosity Number
RTFOT	Rolling Thin Film Oven Test
RV	Rotational Viscometer
SEC	Size Exclusion Chromatography
SHRP	Strategic Highway Research Program
SINTEF	The Foundation of Scientific and Industrial Research at the Norwegian Institute of Technology
TFOT	Thin Film Oven Test
VTS	Viscosity-Temperature Susceptibility

CHAPTER 1 *LITERATURE REVIEW ON BITUMEN, FILLER, RHEOLOGY AND AGEING*

1.1 Introduction

The function of asphalt pavements on any given road can be defined as:

1. The roof of the road construction, preventing moisture from penetrating into the construction.
2. A riding surface for the traffic and the users of the road.
3. A structural layer in the construction.

The functional quality of an asphalt pavement will vary, depending on traffic volume. For high volume roads, rutting resistance will be most important, especially to withstand wear from studded tires. These roads have to be resurfaced after few years due to rutting. Pavements on low volume roads will serve for many years before resurfacing, and important properties for these pavements are flexibility and low temperature capacity. The overall long term pavement performance is important for low volume roads.

In spite of the fact that binder content in asphalt pavement is about 5-7 percent by weight, the binder has a great influence on pavement performance. There are arguments for and against using hard or soft binders, and modified or unmodified binders. The choice is complicated due to the fact that binder properties are altered during production and with time in the pavement. This change is called ageing.

To be able to choose the right binder and asphalt mix, it is necessary to know how binder properties change under production and over time under real condition (traffic load, climate, temperatures, etc.).

An asphalt pavement is exposed to a wide temperature range. The rheology of the binder will be of major importance for the pavement performance under different conditions.

1.2 A brief introduction to rheology

This chapter gives a brief introduction to rheology, mainly based on “An introduction to rheology”, by Barnes et al [1].
A simple description of different models for material properties is also given.

1.2.1 Definition

Barnes et al define “Rheology” as: “*the science of deformation and flow (the study of the deformation and flow of matter)*”.

1.2.2 Models for prediction of material properties

Elastic response (Hookes law)

For elastic solid materials, Hookes law is used to predict the material response. Hooke's equation for shear stress could be written (see Figure 1.1):

$$\tau = G\gamma \qquad (1\text{-}1)$$

where τ = Shear stress [N/m^2]
G = Shear modulus or rigidity modulus [N/m^2]
γ = Relative deformation in shear, shear strain.

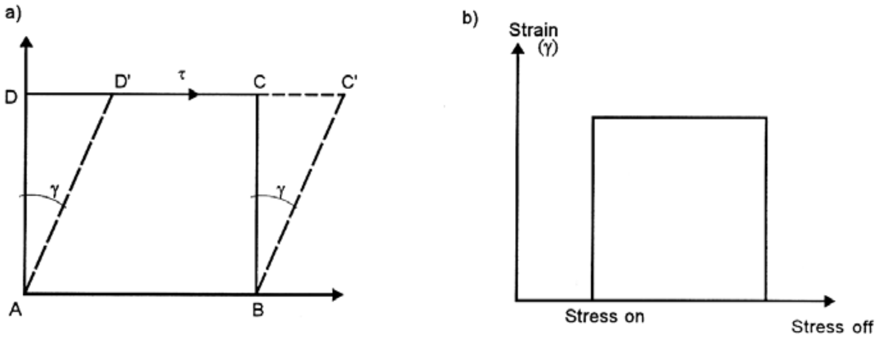


Figure 1.1 Material response for solid material, a) application of shear stress, b) strain response during application of stress.

Classical theory of elasticity establish that *if you double the tension you double the extension [1].*

Viscous response (Newtonian flow)

Barnes et al [1] refer to Isac Newton and his work where he developed the following hypothesis associated with the steady simple shearing flow shown in Figure 1.2: "*The resistance which arises from the lack of slipperiness of the parts of the liquid, other things being equal, is proportional to the velocity with which the parts of the liquid are separated from one another*".

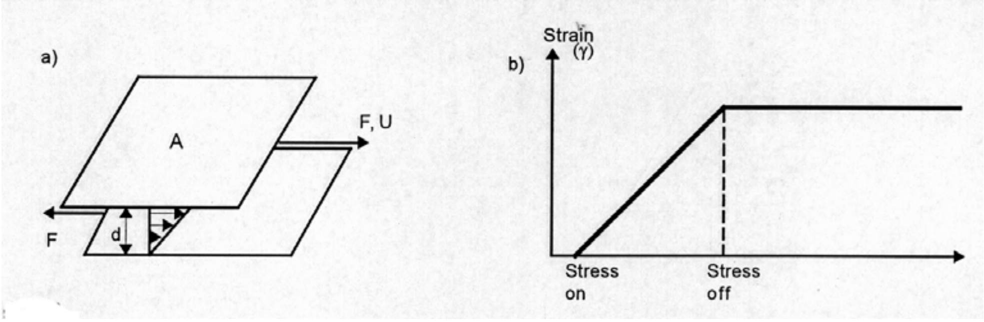


Figure 1.2 Material response for a liquid, a) application of shear stress, b) strain response during application of stress.

This lack of slipperiness is what we call “viscosity”. It is synonymous with “internal friction” and is a measure of “resistance to flow”. The force per unit area required to produce the motion is F/A and denoted by τ and is proportional to the “velocity gradient” (shear rate) U/d, i.e. *if you double the force you double the velocity gradient*. The constant of proportionality η is called the coefficient of viscosity [1].
In the Glossary of Rheological Terms [2], there are explanations of several terms of viscosity:

Viscosity: Qualitatively, the property of a material that implies a relation between stress and strain.

Dynamic viscosity, η: The ratio of shear stress to shear rate, in classical fluid mechanics.

$$\eta = \tau / \dot{\gamma} \text{ [Pa.s]} \qquad \mathit{(1\text{-}2)}$$

τ = Shear stress [Pa]
$\dot{\gamma}$ = $d\gamma/dt$, Shear strain rate [s^{-1}]

Kinematic viscosity, ν: The ratio of dynamic (shear) viscosity to density, in classical fluid mechanics.

$$\nu = \eta/\rho \qquad [m^2 s^{-1}] \qquad \mathit{(1\text{-}3)}$$

ρ = Density [kg/m^3]

Barnes et al [1] describe different factors which can affect the viscosity, such as temperature, pressure, time of shearing and shear rate.
Variation with temperature: The viscosity of Newtonian liquids decreases with increase in temperature.
Variation with pressure: The viscosity of liquids increases exponentially with isotropic pressure.
Shear dependency of viscosity: Liquids can be distinguished in Newtonian and non-Newtonian. Newtonian behaviour in experiments conducted at constant temperature and pressure has the following characteristics [1]:

(i) The only stress generated in simple shear flow is the shear stress τ.

(ii) The viscosity does not vary with shear rate.

(iii) The viscosity is constant with respect to the time of shearing, and the shear stress in the liquid falls to zero immediately after the shearing is stopped. In any subsequent shearing, however long the period of resting between measurements, the viscosity is as previously measured.

(iv) The viscosity measured in different types of deformation are always in simple proportion to one another, so, for example, the viscosity measured in uniaxial extensional flow is always three times the value measured in simple shear flow.

A liquid showing any deviation from the above behaviour is non-Newtonian.

Thixotropy is a phenomenon often mentioned in connection with viscous materials. Barnes et al [1] have also discussed and explained this phenomenon as follows:" *The measured shear stress, and hence the viscosity, can either increase or decrease with time of shearing. Such changes can be reversible or irreversible. According to the accepted definition, a gradual decrease of the viscosity under shear stress followed by a gradual recovery of structure when the stress is removed is called "**thixotropy**". Thixotropy usually occurs where the liquid viscosity levels decrease with increasing shear rate.*

Visco-elastic response

A visco-elastic material consists of both elastic and viscous properties.
Most materials can behave viscoelastic if the conditions are right. If the material is relatively slowly loaded, the material will appear to be viscous rather than elastic, whereas if the loading is relatively fast, it will appear to be elastic rather than viscous. At intermediate time-scales visco-elastic response is observed. Visco-elastic response to stress is shown in Figure 1.3.

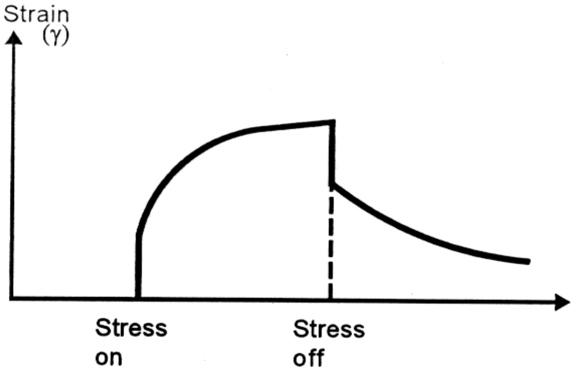


Figure 1.3 Visco-elastic response to stress.

The Maxwell model:
The Maxwell model is in Glossary of Rheological Terms [2] described as the simplest representation of the behaviour of a visco-elastic liquid. This is a two-element model comprising a dashpoint (viscosity, η) and a spring (shear modulus, G) in series, see Figure 1.4. For a constant stress, the behaviour is that of a Newtonian fluid. For a constant strain, stress tends to zero at infinite time.

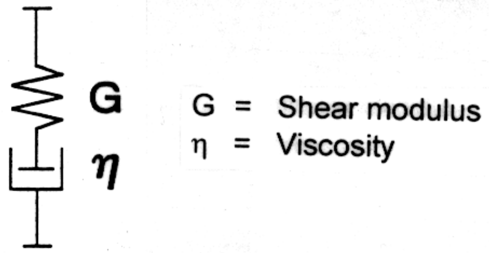


Figure 1.4 The Maxwell model, a simple linear viscoelastic model [2].

Oscillatory shear

To measure the viscoelastic properties of a material, oscillatory techniques can be used. Barnes et al [1] have also discussed this model. The complex modulus , G*, measured using this technique is given in equation 1-4.

$$G^* = G' + iG'' \qquad (1\text{-}4)$$

G' = $|G^*| \cos \delta$, describes the amount of energy stored and released elastically in each oscillation. Also called the *elastic component* of the complex modulus [2].

G'' = $|G^*| \sin \delta$, describes the average energy dissipation rate. Also called the *viscous component* of the complex modulus [2].

δ = The phase difference between stress and strain in harmonic oscillation [2]. For elastic materials, δ will approach to 0°, and for viscous materials, δ will approach to 90°.

Johansson et al [17, paper 2], have explained the tan δ as the relation between the viscous and the elastic moduli. A high tan δ indicates ability to dissipate deformation energy and relax stresses that otherwise could contribute to cracking. Johansson also mentioned that in bitumen subjected to age-hardening, tan δ values decrease over time due to build-up of products of oxidation.

1.3 Bitumen chemistry

1.3.1 General

Bitumen is produced from residues after distillation of crude oil. As described in the Shell bitumen handbook [54], the distillation process usually consist of atmospheric distillation and vacuum distillation, as shown in Figure 1.5

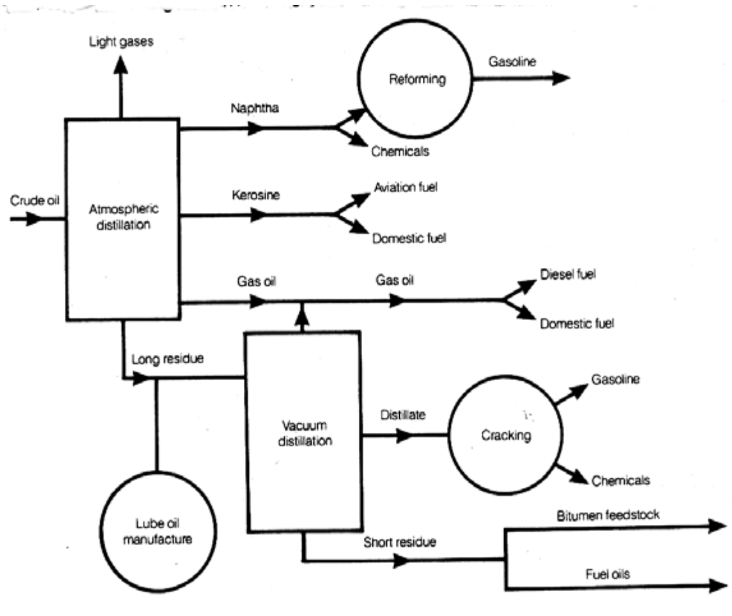


Figure 1.5 Distillation of crude oil [54].

In Altgelt et al [52] the distillation cuts of petroleum are named and categorised by their origin or refinery process. For example, there are the straight run fractions (distilled directly from crude oil) and cracked ones (produced by catalytic cracking or hydrocracking).

Algelt et al also says that with heavy petroleum fractions, knowing every chemical detail is often impossible, because of two obstacles:

1. The enormous complexity of heavy petroleum fractions and
2. The limitations of the measuring techniques.

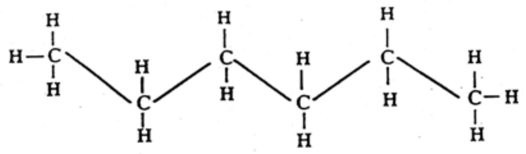
1.3.2 Molecular level

Bitumen is often referred to as a hydrocarbon, which simply means that it is composed primarily of atoms of carbon and hydrogen. In a report by Jones from the Strategic Highway Research Program [12] it is said that bitumen used for paving contains between 90 and 95 weight percent hydrogen and carbon atoms. The remaining consist of two types:

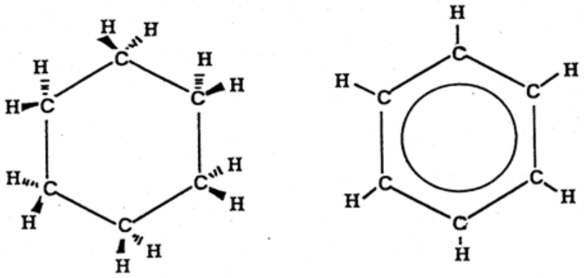
1. Heteroatoms (nitrogen, oxygen and sulphur)
2. Metals (vanadium, nickel and iron)

Further it is described that heteroatoms can replace carbon atoms in the bitumen molecular structure. Their chemistry is claimed to be very different from carbon, and heteroatoms can contribute to many of bitumen’s unique chemical and physical properties by forming associations between molecules. The presence of heteroatoms can make the molecule polar and thus more likely to react with other molecules. Both the kind and amount of heteroatoms present in a given bitumen are said to be a function of both the crude oils from which the bitumen was produced, and the state of ageing of the bitumen. Jones also mentions that sulphur plays an important role in the ageing of bitumen because it is more chemically reactive than hydrogen or carbon, and can oxidise more easily than the hydrocarbons. When bitumen oxidise, two main products are formed, sulfoxide (S=O) and carbonyl (C=O).
According to Jones, the metals play an important role in the ageing process. The metals present in the bitumen are usually indicative of the crude source from which the bitumen was refined [12]. The most common metals in bitumen are vanadium, nickel and iron, according to Robertson [5].

Jones [12] states that the molecules in the bitumen can be grouped into three groups, aliphatics, cyclics and aromatics. Examples of these three types of molecules are shown in Figure 1.6.
Jones[12] also claim that the chemical composition has a great influence on the physical properties of the molecule. This is demonstrated in Table 1.1 for 3 molecules shown in Figure 1.6.



Aliphatic
(Hexane, C_6H_{14})



Cyclic
(Cyclohexane, C_6H_{12})

Aromatic
(Benzene, C_6H_6)

Figure 1.6 Three groups of molecules in bitumen [12].

Table 1.1 Effect of chemical composition on physical properties [12].

Molecule	**Boiling Point, °C**	**Freezing Point, °C**
Hexane	69	-95
Cyclohexane	81	6
Benzene	80	5

Cyclic and aliphatic molecules are three-dimensional and form shapes that keep the molecules apart. Aromatics are flat and can closely stack on top of one another. Comparison of the boiling and freezing points of these three very similar molecules demonstrates the large property changes brought about by minor chemical changes (Table 1.1) [12].

1.3.3 Intermolecular level

When the molecules are randomised, they can move about with respect to each other more easily than when they are more organised. Robertson [5] describes the bitumen like a structured, springlike material (viscous material). Oxidation is mentioned as an important reason for the organised structure.

Johansson [17] has described one of the most generally accepted concepts of bitumen composition. In this model, bitumen is composed of four generic fractions, asphaltenes, resins, aromatics and saturates. Asphaltenes are often considered as highly polar and complex aromatic ring systems with a high concentration of polar functional groups containing heteroatoms. Boduszynski et al [7] has given the following description of the asphaltenes role in the mechanical and rheological properties of bitumens: " *Asphaltenes play a fundamental role in determining the mechanical and rheological properties of bitumens. Such important bitumen*

properties as temperature susceptibility or sol-gel transition phenomena depend on the quantity and nature of asphaltenes. Ironically, asphaltenes are the least characterised and the most controversial bitumen component, even though a great deal of effort has been devoted to investigation of their composition".
Johanson [17] refers resins as composed of hydrogen and carbon containing small amounts of oxygen, sulphur and nitrogen. Aromatics are referred to as low molecular weight naphthenic compounds with paraffinic chains, and substituted monocyclic and polycyclic aromatic hydrocarbons. Further saturates is referred to comprise straight and branch-chain aliphatic hydrocarbons, together with alkyl-naphthenes and some alkyl-aromatics. Saturates are said to not contain polar chemical functional groups.

Johanson [17] refer that the functional groups of bitumen are often characterised by heteroatoms such as sulphur, oxygen and nitrogen. The functional groups are known to be responsible for molecular interactions leading to bitumen molecular associations. The differences in chemical and rheological properties of bitumens from different sources may be mainly caused by functional groups. Important functional groups of bitumen are shown in Figure 1.7.

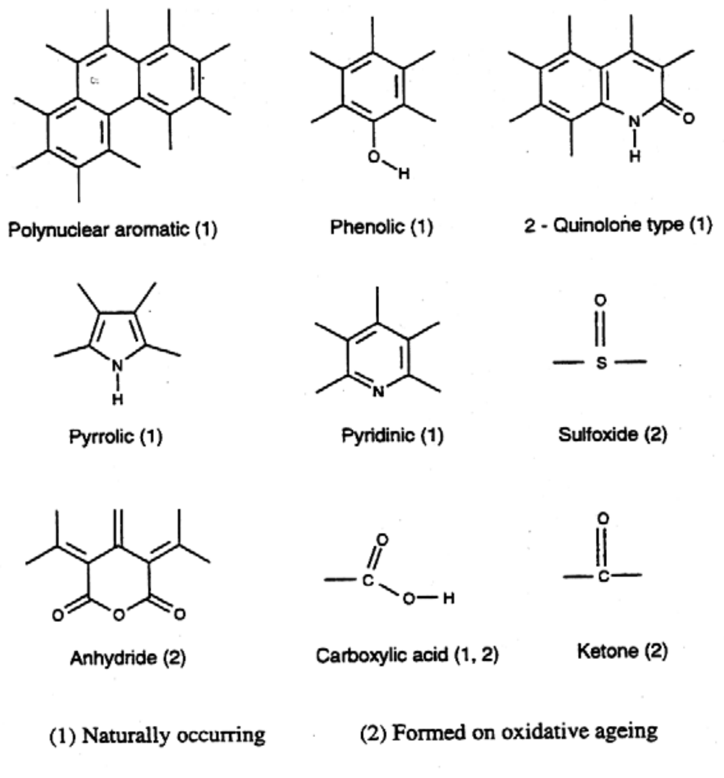


Figure 1.7 Examples of chemical functional groups of bitumens [17]).

In SHRP [12] it was claimed that molecules are formed through the formation of four types of bonding. This is said to be through the strong covalent bond, and through three weaker forms of bonding:

1. *pi-pi bonding* (The aromatics form stacks of molecules due to their flat shape, and the electrons in the aromatic rings interact with one another).

2. *Hydrogen/polar bonding* (The molecules containing heteroatoms are generally polar and play and important role in affecting bitumen's physical properties by interacting with other molecules through the formation of hydrogen/polar bonds. This occurs when a heteroatom on one molecule interacts with a hydrogen atom next to a heteroatom on a different molecule).
3. *Van der Waals forces* (In this type of very weak bonding, long chains of aliphatic hydrocarbons, such as hexane in Figure 1.6, intertwine and are weakly hold together).

In SHRP it was claimed that hydrogen/polar bonding is probably the most important of the weak molecular interactions.
Further, these types of bonds are described as weak and that they are easily broken by heat or stress. They will form again when the molecules cool or the stress is removed. This property of formation and breaking of weak bonds between bitumen molecules was in SHRP claimed to be the key to understand the physical properties of bitumen.

As a result of chemical investigations carried out by the SHRP, bitumen can be chemically characterised as follows [12]:

1. *Bitumen consist of two (and only two) functional families of molecules:*
    - *Polar*
    - *Non-polar*
2. *Polar molecules differ according to:*
    - *Strength and number of polar group(s)*
    - *Molecular weight*
    - *Degree of aromaticity*
3. *Non-polar molecules differ according to:*
    - *Molecular weight*
    - *Degree of aromaticity*
4. *The "compatibility" of the polar and non-polar fractions, or the degree to which they can dissolve in each other, is controlled by the relative aromaticity (a measure of the amount of aromatics versus aliphatic and cyclic molecules) of the two fractions.*

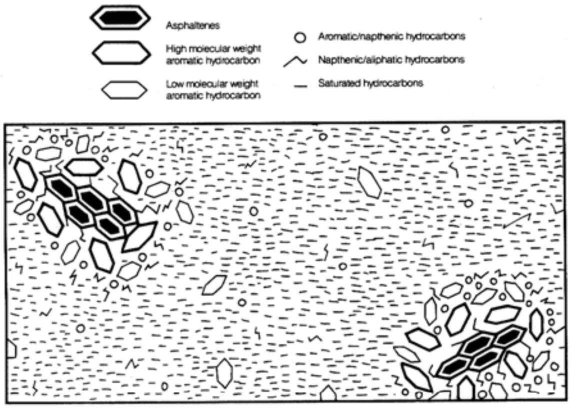
In this characterisation the molecules in the bitumen behave in one of two ways. In the following a description is given relating chemical composition to bitumen behaviour [12]: *"If they are polar at service temperature they participate in the formation of a network that gives the bitumen its elastic properties. The non-polar materials form the body of the material in which the network is formed and contribute to the viscous properties of the bitumen.*
Non-polar molecules with high molecular weight will lead to bitumen that stiffen and perform poorly at low service temperatures due to brittleness. The relative amount of aromatic character of the non-polar materials is also important. If the non-polars are waxy (a type of aliphatic molecule) in nature, they may precipitate or crystallize at low temperatures, contributing to poor performance. If the non-polars are cyclic or aromatic in nature, then they may resist the effects of low temperature better".

1.3.4 Chemical models of bitumen

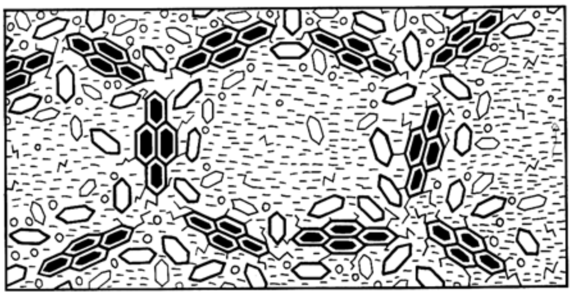
The model should be able to explain the bitumen behaviour, as well as the variations in behaviour from one type of bitumen to another.

<u>“The colloidal system”</u>

In the literature, bitumen is traditionally regarded as a colloidal system consisting of a suspension of high molecular weight asphaltene micelles dispersed in a lower molecular weight oily medium (maltenes) [6, 53, 54]. The micelles are considered to be asphaltenes together with an adsorbed covering sheath of high molecular weight aromatic resins as a stabilizing solvating layer. Further away from the centre of the micelle, there is a gradual transition to the less aromatic resins and such layers extend outwards into the less aromatic oily dispersion medium. In the presence of sufficient quantities of resins and aromatics of adequate solvating power, the asphaltenes are fully peptised and the resulting micelles have good mobility within the bitumen. These bitumens are known as solution or “sol” type bitumens. On the other hand, insufficient quantities (insufficiently solvating power) of resins and aromatics may lead to an association of micelles. These bitumens are known as gelatinous or “gel” type bitumens. Gel-type bitumen exhibit non-Newtonian flow and thixotrophy, which are typical for colloidal systems [6]. In practice most bitumens are mixed gel-sol systems [54]. Schematic representation of “sol” and “gel” type bitumen is given in Figure 1.8.



"SOL" type bitumen



"GEL" type bitumen

Figure 1.8 Schematic presentation of “sol” and “gel” type bitumen [54].

Dukattz et al [6] concluded, on the basis of the literature, that asphaltenes are not a chemically unique fraction of bitumen, but are dependent on the chemical composition of the bitumen and the type of solvent used to precipitate the asphaltenes. They also concluded that the asphaltene fraction, if it exists as such in bitumen, is a dispersed phase with varying degrees of association. Further, the degree of association depends on the polarity of the asphaltenes and the solvating power of the maltene fraction.
They further summarised that bitumen can be represented by two major components, asphaltenes and maltenes. The fysical properties and the relative quantity of each component affect the flow behaviour of the bitumen. They also claim that previous attempts to correlate composition with flow behaviour have been largely unsuccessful.

“The SHRP-bitumen model”

During the Strategic Highway Research Program a new bitumen model was developed [5]. This model is claimed to be the most consistent description of bitumen. In this model, bitumen is a collection of polar and non-polar molecules. The polar molecules tend to associate strongly to form organised structures throughout the continuos phase of the non-polar materials. As temperature is raised, the associations of polar molecules decreases and the material becomes therefor less viscous. As temperature is reduced, the opposite occurs. There have also been observations that indicate that the non-polar phase also organises, but at temperatures below 0°C. Further, bitumen is susceptible to oxidation, which is explained to increases both amount of polarity and the number of polar sites present among the molecules.
The SHRP-bitumen model is shown in Figure 1.9.

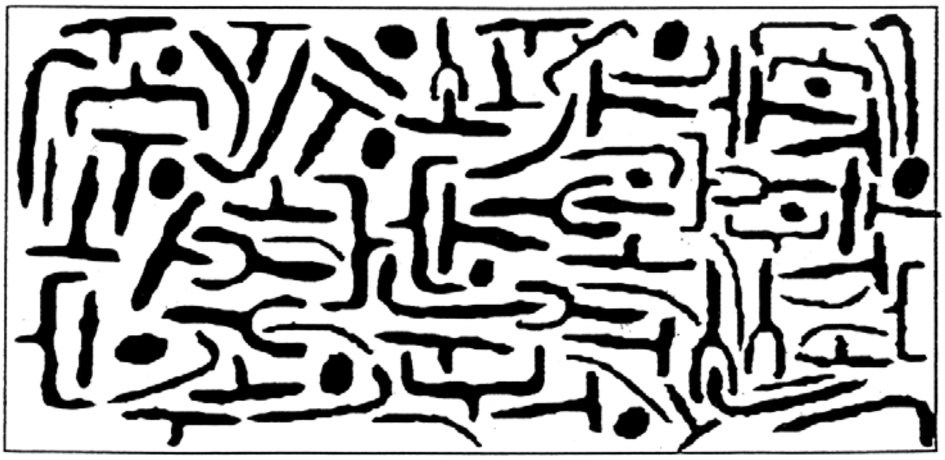


Figure 1.9 The SHRP bitumen model [12].

The different shapes in the figure represent the associations of polar molecules in the bitumen. They are surrounded by the nonpolar molecules. The polar sites on the associations of molecules interact with one another by forming weak bonds. It is believed that the majority of bitumens viscoelastic properties result from the polar-polar interactions of its associated molecules. Since the bonds between polar molecules are weak, they are constantly being broken and reformed as the bitumen is subjected to heat and/or stress.

The amphoterics

In the SHRP [12] a new chemical fraction of bitumen has been isolated using the Ion Exchange Chromatography (IEC) analytical technique. This new fraction is believed to be bitumens "key" building block. This material is believed to have unique physical properties that can explain the tendency of polar materials in bitumen to form associations. These materials are termed "amphoterics".

A definition of an amphoteric material is given as [12]: *"a material that can exhibit either acid or basic character. When applied to bitumen, the term means that a bitumen molecule has both an acid and a base group on the same molecule. Data have provided strong evidence that amphoterics play an important role in building the polar-polar bonds that give bitumen its unique properties ".*

1.3.5 Relationship of chemistry to pavement performance

In SHRP the failure modes in asphalt pavements that may be related to materials are listed as follows [5]:

1. Permanent deformation
2. Rutting
3. Fatigue cracking
4. Low temperature cracking
5. Moisture damage
6. Total loss of adhesion

The relationship of chemistry to pavement performance is also discussed in [5]. Polarity is claimed to be a major contributor to the performance characteristics of bitumen. During age hardening (oxidation) the bitumen becomes brittle due to increasing of polar materials. At very low temperature it is also mentioned that the non-polar materials tend to organise into a very rigid material, and that this rigid material shrinks at low temperature. It is speculated that this type of shrinkage is largely responsible for low temperature cracking.

Further it is mentioned that linear aliphatic materials show pronounced tendency to shrink with decreasing temperature.

Adhesion arises due to the interaction of the polar material in the bitumen with the polar surface of an aggregate. Some aggregates have positive sites, some negative and some show variation in polarity with moisture content, temperature, etc.

Water is a highly polar material and is transported into the bitumen due to attraction of polar water molecules to polar components in the bitumen. Water is said to affect the mechanical properties of bitumen by softening it [5].

1.3.6 Analysis equipment

Bitumen is a complex material. The chemical composition will depend on origin and refinery process. Johanson [17] states that composition data of bitumens are relative and meaningful comparisons should be based on the same analytical conditions. Table 1.2 gives a general survey of analytical methods used for characterisation in bitumen chemistry.

Table 1.2 Analytical methods used for characterisation in bitumen chemistry [17].

Type of method	**Reported Chemical Parameters**
Elemental Analyses	C, H, N, S, O, Trace metals (e.g. V, Ni)
Solvent Precipitation	Asphaltics, Paraffinics, Cyclics
Chemical Presipitation	Asphaltenes (n-pentane insolubles), Nitrogen bases, 1st Acidaffines, 2nd Acidaffines, Paraffines
Adsorption/Desorption Chromatography	Asphaltenes, Saturates, Naphthene-aromatics, Polar aromatics
High Pressure Liquid Chromatography	Fractions related to moving phase
Gel Permeation Chromatography	Molecular weight distributions
Ion Exchange Chromatography	Strong acid, weak acid, Strong base, Weak base, Neutral fraction
Thin-layer Chromatography	Asphaltenes, Resins, Aromatics, Saturates
Gas Chromatography	Volatile components
Supercritical Fluid Chromatography	Nonpolar fraction
Infrared Spectrometry	Functional groups
UV Spectroscopy	Characterisation of generic fractions
Nuclear Magnetic Resonance Spectroscopy	Saturate/aromatic carbon ratio, content of carboxylic acids, phenol and benzylic protons, generic compositions
X-ray Diffraction	Crystalline fractions

Due to the difficulty of performing an exact chemical analysis on individual chemical species, the molecular composition of bitumen is normally defined by fractionation [17, 52].

1.4 Bitumen rheology

1.4.1 General

Bitumen is defined as a viscoelastic material. For highway engineering purposes, the most important characteristic of viscoelastic materials is the dependency of their mechanical response on time of loading and temperature.

Mechanical properties should ideally be measured at the temperature and loading frequency ranges that correspond to pavement temperatures and loading conditions. In the Strategic Highway Research Program, this philosophy formed a basis for the development of the new SHRP binder specification [18].

1.4.2 Rheological properties of bitumen

Introduction

The mechanical response of bitumen is dependent on time and temperature. Considering only the temperature effect on the viscosity of bitumen, the principal viscosity properties are shown in Figure 1.10.

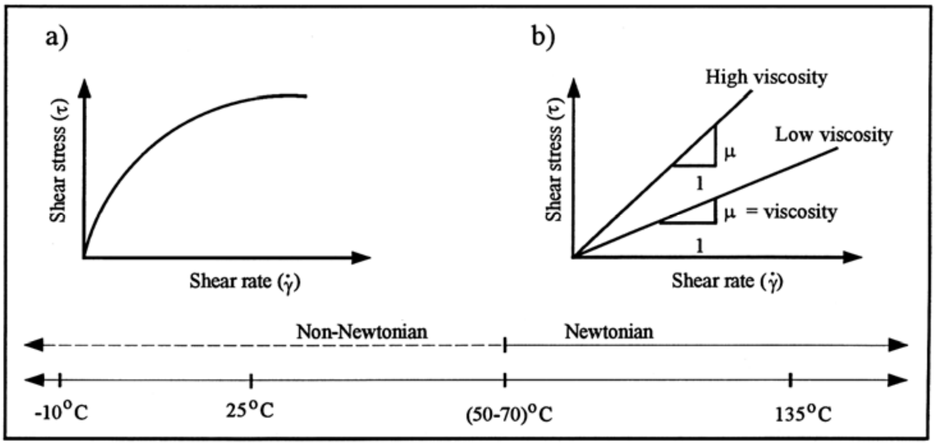


Figure 1.10 Flow properties of bitumen as a function of temperature (at a given loading time).

Viscous liquids like hot bitumen are sometimes called *plastic* because once they start flowing, they do not return to their original position. This is why in hot weather, some asphalt pavements flow under repeated wheel loads and wheel path ruts form. However, it is a common agreement that rutting in asphalt pavements during hot weather is also influenced by aggregate properties. As stated by McGennis et al [45], it is probably more correct to say that the asphalt mixture is behaving like plastic.

At low temperatures or very rapid loading times, bitumen behaves like elastic solids. If to much load is applied, elastic solids may break. Even though bitumen is an elastic solid at low temperatures, it may become too brittle and crack when excessively loaded. This is the reason why low temperature cracks sometimes develop in asphalt pavements during cold weather.

Traditionally, rheological examination of bituminous binders has often taken place in the Newtonian temperature range and one has extrapolated linearly down into the non-Newtonian range. Heukelom [54] developed the Bitumen Test Data Chart (BTDC) where this was put into a system. In the BTDC, by using penetration, softening point, Fraass breaking point and viscosity data, he was able to represent the viscosity and penetration of normal bitumens as a function of temperature as straight lines. The Bitumen Test Data Chart is shown in Figure 1.11.

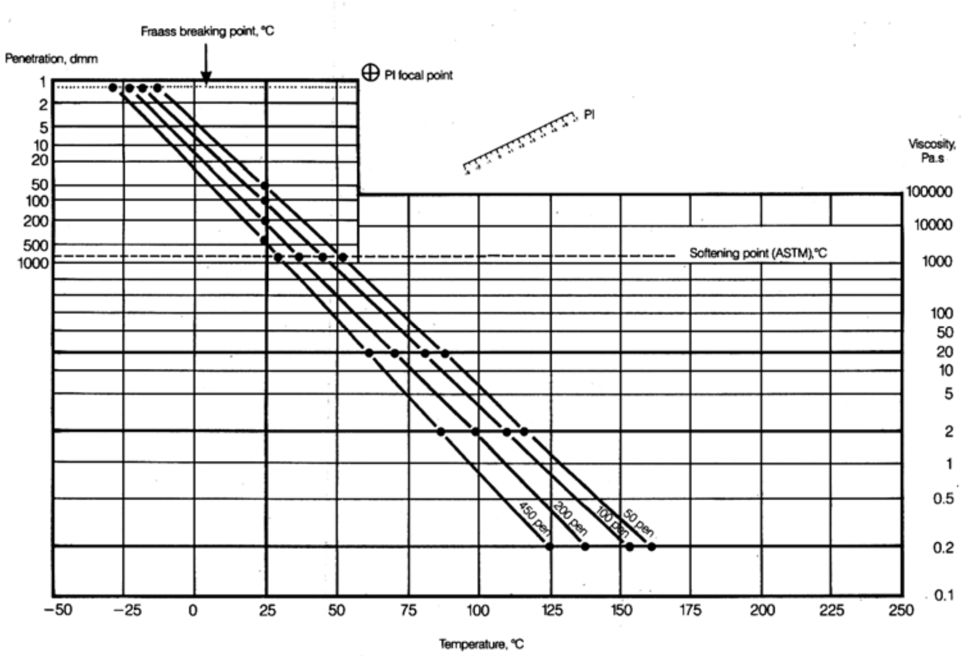


Figure 1.11 The Shell Bitumen Test Data Chart [54].

Linear viscoelastic model

Anderson et al [43] used the simple creep test to explain and characterize the stress-strain response of a material. In this test, a load of constant magnitude is applied to a material at time t_0, at time t_i, the load is removed. The inherent differences between elastic, viscous, and viscoelastic behavior is shown in Figure 1.12.

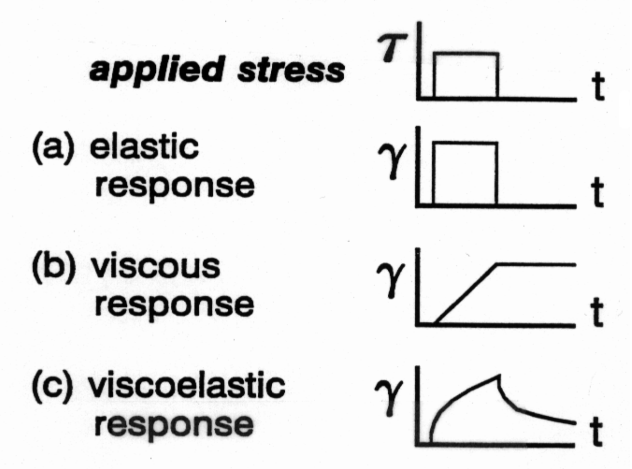


Figure 1.12 Idealized response of elastic, viscous, and viscoelastic materials under constant stress loading [43].

Bitumen is often characterised as a viscoelastic material. A viscoelastic material (Fig. 1.12 c) has both elastic and viscous components of response, followed by a gradual time-dependent deformation. This time dependent deformation may further be divided into a purely viscous component and a delayed elastic component. Upon removing the load at t_i, the viscous flow ceases, and none of this deformation is recovered. The delayed elastic deformation is, however, recovered, but not immediately as with purely elastic deformation. Instead, once the load is removed, the delayed elastic deformation is slowly recovered, at a decreasing rate, as is shown in Figure 1.12 c).

The description of elastic, viscous, and viscoelastic response given above is only valid for linear responses. Anderson et al [43] argue that linear methods of characterization and analysis are more than adequate for engineering design problems. Nonlinear response, especially for viscoelastic materials such as bitumen, is said to be extremely difficult to characterise in the laboratory and to model in practical engineering problems.

The stiffness modulus, which is commonly used when analysing creep data, is defined as:

$$S(t) = \sigma_0 / \varepsilon(t) \qquad (1\text{-}4)$$

where

$S(t)$ = time-dependent stiffness modulus, Pa
t = loading time, s
σ_0 = applied constant uniaxial stress, Pa
$\varepsilon(t)$ = resulting uniaxial strain at time t, m/m.

Dynamic mechanical analysis

There are many methods and ways of characterising viscoelastic properties. Bahia and Anderson [18] claim that dynamic (oscillatory) testing is the best technique to measure and explain the viscoelastic properties of bituminous materials. In the shear mode, the complex modulus, G*, and phase angle, δ, are measured. G* represents the total resistance to deformation under a given load. δ represents the relative contribution of an in-phase elastic component and an out-of-phase viscous component to the measured complex modulus. Bahia and Anderson explain that the elastic in-phase component can be related directly to energy stored in a sample for every loading cycle, while the viscous out-of-phase component can be related directly to energy lost per cycle in permanent flow. Bahia and Anderson explain further that the relative distribution of these components is a function of the composition of the material, loading time and temperature.

Both the complex modulus and the stiffness modulus are simply indicators of the resistance of bitumen to deformation under a given set of loading conditions. Anderson et al [43] describe how the dynamic mechanical properties are directly related to the creep properties, but in a mathematically complex way.

Anderson et al [43] further describes the typical parameters or output which is obtained from dynamic testing. The primary response in dynamic testing is the complex dynamic modulus, which is computed (in strain-controlled mode) using the following equation:

$$G^*(\omega) = | \tau(\omega) | / | \gamma(\omega) | \qquad (1\text{-}5)$$

where

$G^*(\omega)$ = complex dynamic shear modulus at frequency ω, Pa
$| \tau(\omega) |$ = absolute magnitude of the dynamic shear stress response, Pa, and
$| \gamma(\omega) |$ = absolute magnitude of the applied dynamic shear strain, m/m.

The phase angle, δ, indicates the lag in the stress response compared with the applied strain. For purely elastic materials, the phase angle will be 0°, whereas for purely viscous materials, the phase angle will be 90°. The phase angle was in SHRP [43] pointed out as an important parameter in describing the viscoelastic properties of material such as bitumen.

Anderson et al [43] describe three other parameters from dynamic testing which are often used: the storage modulus G'(ω); the loss modulus G''(ω); and the loss tangent, or tan δ. These parameters can be visualized as seen in Figure 1.13.

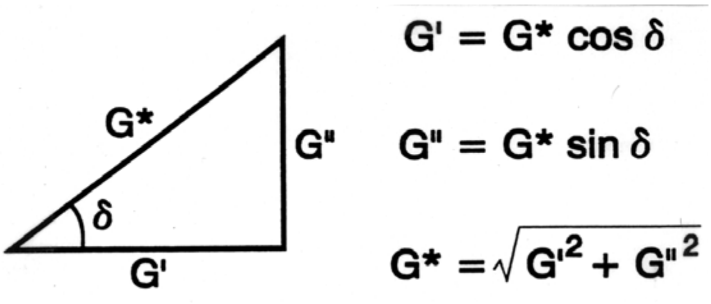


Figure 1.13 The relationship between complex modulus (G), storage modulus (G'), loss modulus (G'') and phase angle (δ) [43].*

[43] describes how the storage modulus and the loss modulus sometimes are misinterpreted as the elastic and viscous modulus; in reality, the elastic component of the response only represents part of the storage modulus, and the viscous response only part of the loss modulus. In addition to the elastic and viscous response, most real viscoelastic materials exhibit a significant amount of delayed elastic response that is time-dependent but completely recoverable. In interpreting the storage and loss moduli, Anderson et al points out how it should be kept in mind that both these parameters reflect only a portion of the delayed elastic response.

Linear viscoelastic (LVE) model for bitumen developed during SHRP A-002A research.

During SHRP, Anderson et al [43] developed a linear viscoelastic (LVE) model for bitumen based on master curves obtained from dynamic testing. As described in [43], a master curve can be seen as a rheological fingerprint of viscoelastic materials. Four parameters were found necessary and adequate to fully describe a master curve and thus characterize the LVE properties of any binder. A typical master curve is shown in Figure 1.14 illustrating the meaning of these 4 parameters.

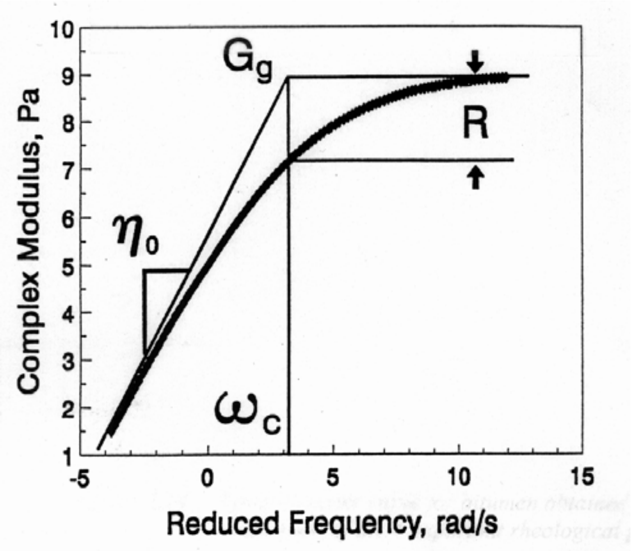


Figure 1.14 Typical master curve for bitumen obtained from dynamic analyses with illustration of the 4 important rheological parameters [43].

The four parameters needed to characterise the LVE properties of binders are described in [43] as:

- ***The glassy module, G^*_g*** – the value that the complex modulus or stiffness modulus approaches at low temperatures and high frequencies or short loading times; the glassy modulus is normally very close to 1GPa in shear loading for most bitumen.
- ***The steady-state viscosity, η_o*** – the steady state, or Newtonian viscosity. In dynamic testing, it is approximated as the limit of the dynamic viscosity, η^*, as the phase angle approaches 90°. The 45° line that the dynamic master curve approaches at low frequencies is often referred to as the viscous asymptote. It is indicative of the steady-state viscosity, and the value of η_o is binder specific.
- ***The crossover frequency, ω_c, or crossover time, t_c*** – the frequency at a given temperature where tan δ is 1. At this point, the storage and loss moduli are equal. For most binders, the crossover frequency is nearly equal to the point at which the viscous asymptote intersects the glassy modulus. The crossover frequency can be thought of as a hardness parameter that indicates the general consistency of a given bitumen at the selected temperature and is bitumen specific. The crossover frequency is the reciprocal of the crossover time,
$t_c = 1/\omega_c$.
- ***The rheological index, R*** – the difference between the glassy modulus, G^*_g, and the dynamic complex modulus at the crossover frequency, $G^*(\omega_c)$. The rheological index is directly proportional to the width of the relaxation spectrum and indicates rheological type. R is not a measure of temperature bur reflects the change in modulus with frequency or loading time and therefor is a measure of the shear rate dependency of bitumen. R is bitumen specific.

Low-temperature physical hardening

Low-temperature physical hardening is a phenomenon which was first seen and described in SHRP [43]. The low-temperature physical hardening is believed to be caused by the time-dependent volume changes in the binder when it is stored at low temperatures. It has been shown that this hardening may have great influence on the low temperature rheology of bitumen. Anderson et al [43] found that the degree of physical hardening depends on the bitumen source, isothermal ageing temperature, and time of storage. Data indicate that the low-temperature physical hardening also affects the failure properties, causing a decrease in the failure strain as physical hardening proceeds. It appears that there is a relationship between wax content and the degree of low-temperature physical hardening [43]. In SHRP [41], the following description of isothermal physical hardening is given: *"As bitumen is cooled within the temperature range for which the molecular processes are relatively rapid (e.g., above the softening-point temperature or in the Newtonian flow region), the change in volume with temperature is relatively instantaneous, and no time-dependent volume change is observed. However, as the temperature is further depressed, the volume change does not occur instantaneously but over a nonzero time. This time-dependent volume change is called* ***isothermal physical hardening*** *because the volume change causes a concomitant time-dependent increase in the stiffness of the asphalt. The temperature at which the molecular processes is slow enough that the volume change is not instantaneous but time dependent is called the* ***glass transition temperature****. The glass transition temperature is, however, not uniquely defined but is dependent on the rate of cooling and other aspects of thermal history. Physical hardening is observed at temperatures typically below* 0^oC*"*.

Thixotropy

Saal and Labout [3] concluded that bitumens must be considered as mixed gel-sol systems and that the degrees of structure exhibited can vary widely with composition. This investigation also indicates that bitumen exposed to deformation has the property of self-healing or thixotropy. It was observed that it could take 1-3 days to rebuild the broken structure.

1.4.3 Bitumen chemistry and rheology

For many years, great efforts have been made to determine relationships between bitumen chemistry and rheology. The influence of asphaltene content has been studied several times. In 1975 the influence of bitumen composition on its rheological behaviour was studied by Marvillet [4]. In this work, the influence of hydrocarbon groups (asphaltenes, resins and oils) on the final behaviour of the binder was studied. Some results from [4] are:

- The rheological behaviour of a binder depends on the quantities of each hydrocarbon group present. [4] points out as a recognised fact that a large quantity of polar and aromatic compounds will enable the asphaltenes to be peptised and then rendered soluble, thus giving the bitumen a "sol" type behaviour. Where there is a substantial concentration of non-volatile maltenes however, soft asphaltenes may to a large extent lose this solvent effect.

- Marvillet [4] claims that it should be possible to predict the behaviour of the material from the ratio of hard to soft asphaltenes, with the lowest ratio resulting in a typical Newtonian bitumen.

Analysis of data from a work carried out in 1984 by Dukatz, Anderson and Rosenberger [6] showed that the commonly accepted view that the flow properties of bitumen are a simple function of asphaltene concentration and maltene viscosity is incorrect. The following conclusions were drawn from this study:

1. In terms of explaining the flow characteristics of bitumen, composition per se is inadequate. The physical-chemical interactions among different components must be considered in order to adequately explain flow properties such as temperature and shear susceptibility.
2. The commonly accepted asphaltene/maltene model is not a valid descriptor of bitumen structure and is inadequate to explain the flow properties of bitumen.
3. Asphaltenes as such does not exist in bitumen.

[6] concludes that flow properties should be extended to a more comprehensive consideration of dynamic mechanical properties and their relation to composition.

Johansson [17] summarises in his literature study several interesting findings; "*despite the fact that certain relationships exist between chemical compositions and rheological properties, considering fraction contents alone is not adequate for an accurate understanding of bitumen rheology*". He refers to how bitumens with similar rheology may have different fraction contents and on the other hand, bitumens with an identical content of fractions may display different rheological properties. In these cases, it is pointed out how the structural characteristics of bitumen, which are dominated by both chemical constitution and molecular forces (molecular interactions), are important.

Furthermore, [17] refers to investigations where it is shown that the presence of different functional groups, their quantitative amounts and relative positions are responsible for molecular forces, and are therefore important in characterising the rheological properties of bitumens.

The importance of chemical "functional groups" to bitumen rheology was studied and emphasised in SHRP. In SHRP [42], a new bitumen structure model was developed. According to this model, the bitumen polarity, rather than global chemical parameters such as elemental composition, can best describe the physical properties of bitumen.

In SHRP [42] it was pointed out that two of the most fundamental chemical properties of any organic substance is molecular weight and chemical functionality. Many physical properties, particularly rheological properties, are in theory functions of molecular weight and molecular weight distribution. Chemical functionality determines extent of intermolecular associations; it therefore influences effective molecular weights, as distinct from true molecular weights of organic materials. Hence, chemical functionality governs those physical properties that depend on effective molecular weight. [42] claims that chemical functionality also influences ageing and stripping behaviour. Molecules composed of carbon and hydrogen tend to associate with one another less strongly than molecules composed of carbon, hydrogen, and heteroatoms such as nitrogen and oxygen. The latter kinds of molecules are considered to be polar; they form relatively strong associations of molecules, depending on such factors as temperature, shear and solvation. These associations have the effect of increasing effective molecular weights of polar molecules.

1.4.4 Bitumen rheology and its relationship to mixture performance

Four viscosity levels are of immediate importance to the pavement performance:

1. Proper viscosity for mixing.
2. Proper viscosity for compaction.
3. Viscosity at the highest temperature to which the pavement will be exposed.
4. Viscosity at the lowest temperature to which the pavement will be exposed.

Davis [15] is one of many who has shown that the viscosity of the bitumen has only a relatively small effect on the stiffness of the mix at high temperature as compared with the properties of the mineral aggregate. At low temperatures, the viscosity of the bitumen has been shown to play a very important role in the stiffness of the mixture [51]. Experience has shown that low-temperature cracking is almost directly related to the stiffness of the binder [8, 9, 14, 15, and 24].

A general phenomenon in asphalt pavements is that the bitumen gets stiffer as the pavement gets older. Problems with ageing of pavements are associated with hardening of the bitumen. This suggests that the softer the bitumen is at the time of construction, the longer the life expectancy of the pavement. Davis [15] therefore argues that the primary rheological requirement should be a maximum level of bitumen viscosity under the expected lowest temperature during the life of the pavement.

Furthermore, Davis [15] argues that the complex rheology of bitumen and asphalt pavements and the poor precision of test methods used to measure these properties, require that the conditions of test be carefully matched to road conditions in order that the results be meaningful.

Skog [8] and McLeod [16] have investigated the temperature susceptibility of bitumen. Figure 1.15 illustrates the influence of temperature susceptibility of bitumen on the hardness at temperatures below 25°C (77 F). The bitumen with a PVN (Penetration Viscosity Number) = -1,5 will be harder at temperatures below 25°C, and therefore has a greater tendency to provide pavements with low temperature transverse cracks than the bitumen with low temperature susceptibility, PVN = 0. Figure 1.15 also shows the effect of temperature susceptibility when bitumen is graded by viscosity at 60°C (140 F). For two bitumens with the same viscosity at 60°C, but with different temperature susceptibilities, the difference in hardness between these at any temperature below 25°C is substantially greater than if they had been graded by penetration at 25°C.

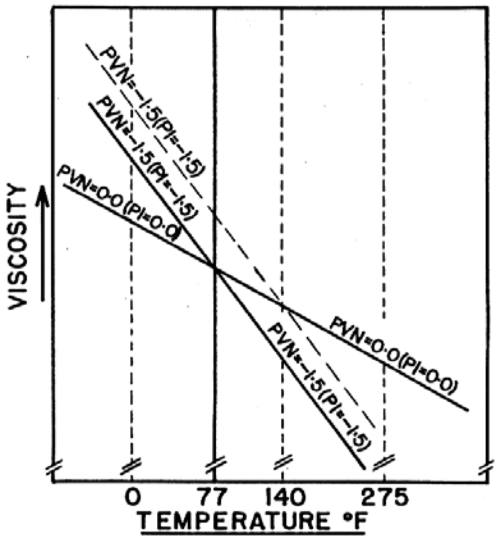


Figure 1.15 Relationship between viscosity and temperature for bitumen with different PVN-values [8].

Hopman et al [10] studied the viscoelastic properties of asphalt concrete mixtures. A conclusion from this work is that a viscoelastic characterisation of asphalt mixes is essential to understand the behaviour of asphalt mixes and asphalt pavements fully.

Bahia and Anderson [18] have argued why new rheological parameters for bitumen are needed. At temperatures above 100°C a measure of viscosity is sufficient to represent the workability of the bitumen. At temperatures between 45 to 85 °C, typical of the highest pavement in-service temperatures, the main distress mechanism is rutting, and they argue that G* and δ need to be measured. Within the intermediate temperature zone, bitumens are generally harder and more elastic than at higher temperatures. The prevailing distress mode at these temperatures is fatigue damage, which is caused by repeated cycles of loading at levels lower than the static strength of a material.
The fourth and last temperature zone is the low-temperature zone at which thermal cracking is the prevailing failure mode. Thermal cracking results from thermal stresses generated by pavement shrinkage as a result of thermal cooling. To reliably predict binder contribution to cracking, they claim that both the stiffness of a binder and its rate of relaxation need to be evaluated.
In Figure 1.16, typical rheological behaviour of fresh and field aged bitumen, in relation to the main pavement distress modes is shown.

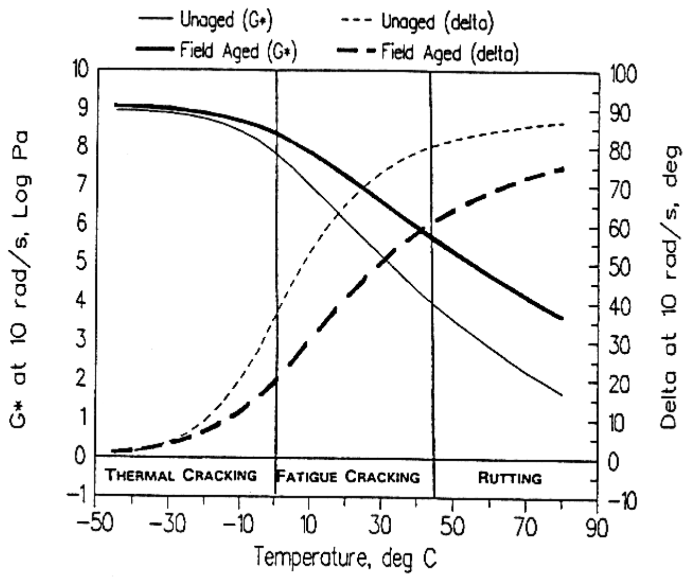


Figure 1.16 Typical rheological behavior of fresh bitumen and after aging in the field, in relation to the main pavement distress modes [18].

The relationship between rheology and pavement performance is very complex. Davis [15] has given some recommendations, which should be met to minimise the problems caused by this complexity:

- The complexity of asphalt pavement rheology requires that the extrapolation of rheological testing be reduced to a minimum.
- The primary rheological requirement for bitumens is that they not reach a critical low-temperature viscosity during the design life of the pavement.
- The primary rheological requirement for asphalt pavements is that their bearing capacity not be exceeded under the highest temperatures reached during the design life of the pavement.
- The parallel plate viscometer is a suitable instrument for measuring the rheological properties of bitumen, mastics, sand asphalt, and asphaltic concrete under realistic conditions of temperature, time of loading, and stress.
- The best approach to reduce problems with oxidation and moisture in asphalt pavements is a sufficient level of bearing capacity so that air voids can be kept below 3 %.
- An important factor in increasing the bearing capacity of pavements, is selection of a maximum nominal aggregate size that is greater than two thirds of the thickness of the pavement layer.

Puzinauskas [9] has carried out a study on physical properties of bitumen used in pavements in the USA. A general evaluation of the physical properties, with emphasis on properties at low temperatures, was the main study objective. Another objective was to compare the properties of materials at that time (1979) to properties of bitumen produced in the past. Some conclusions from this study are:

- Bitumen produced in 1979 was not found to differ substantially from those produced in the past.
- Bitumen, within a given grade, differs substantially in their properties.
- Both the source of parent oil and the method of manufacture affect the physical properties of bitumen.
- Different methods used to evaluate temperature susceptibility or temperature effects on bitumen correlate rather poorly (the Penetration Index (PI), the Penetration-Viscosity Number (PVN) and Viscosity-Temperature Susceptibility (VTS)). Furthermore, it was found that all temperature susceptibility indexes vary and depend on the temperature range selected for the calculation of such indexes.
- Measurements at high or intermediate temperatures cannot be used to predict behaviour of bitumen at subfreezing temperatures.

1.5 Bitumen and filler

1.5.1 General

Filler is an important part of the asphalt mixture and historically many early investigations on filler materials have been conducted. Tunicliff (1962) [65] states that Clifford Richardson probably was the first to recognise and describe the importance of filler as early as 1905. Richardson believed that particles smaller than 0.05 mm were the most valuable particles of the filler material. The function of filler was described by Richardson as:
"..a part of the mineral aggregate of asphalt surfaces for the purpose of rendering the surface more dense, so that it will be less acted upon by water, and less liable to interior displacement or movement".
Richardson also believed that the function of filler was to make the asphalt cement less susceptible to changes in consistency caused by heat.

In 1965 Heukelom [62] presented the following statement concerning filler in bituminous mixes; "*The workability and practical performance of bituminous mixes depend largely on the quantity and type of filler used. The influence of the quantity on mix properties is fairly well known, but relatively little information is available concerning the effect of the type of filler used. This applies in particular to the mechanical properties at high service temperatures of mixes prepared with different fillers. At low service temperatures, the mechanical properties of the mix depend mainly upon the volume concentration of the total minerals, and to a much lesser extent upon the percentage and type of filler".*
Today too, the same statement can be made. There is a great lack of knowledge about filler materials and how they act in different asphalt mixes with different binders and aggregates.

1.5.2 The function of fillers in asphalt mixtures

The filler materials influence the characteristics of the asphalt mix, this is explained by Slyngstad [79] in Figure 1.17.

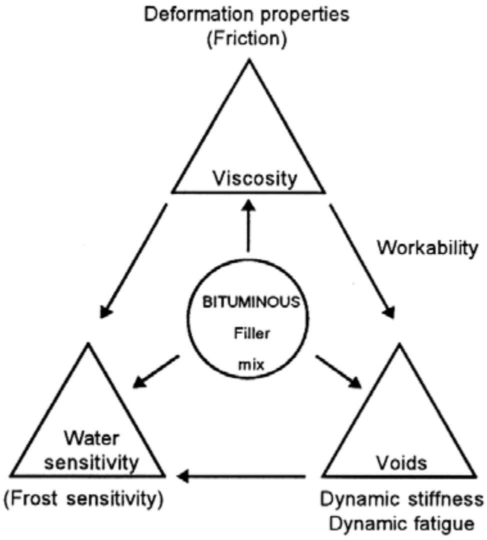


Figure 1.17 The fillers influence on asphalt mix characteristics [79].

Definition of mineral fillers

Two main functional theories of filler are mentioned in the literature [63, 64, 65,76,79]:

(1) Filler is that portion of the mineral aggregate generally passing the number 200 sieve (0.074 mm) which occupies void space between the coarser aggregate particles in order to reduce the size of these voids and increase the density and stability of the mass. The filler is a part of the aggregate-curve and works as a part of the aggregate.
(2) The filler is a part of the binder, and the filler-bitumen mix acts as a binder for the aggregate.

Ruud [63] has discussed these theories. Ruud states that filler material with particle size less than the binder film thickness, act as a suspension with the binder, thereby becomes part of the binder. Filler material with particle size larger than the binder film thickness will act as a part of the aggregate. The thickness of the binder film will vary depending on the type of mix and also within a single type of mix. Therefore it is impossible to give a general rule on how different parts of the filler functions in asphalt mixes [63]. Tunnicliff [66] claims that an important mixture design problem is to determine how to restrict the type and quantity of suspended particles so that required mixture properties are produced.

The literature examined in a study by Slyngstad [79], also explains the effect of filler materials. The theories given above are called the aggregate theory and mastic theory, respectively.

Kallas and Puzinauskas [64] states that a number of factors will affect how filler acts in an asphalt mix, among these are: gradation of the rest of the mineral aggregate, degree of compaction, thickness of binder films and relative proportions of filler.

The filler-bitumen system

Two mechanisms of attraction, absorption and adsorption, have been described in the literature [65]. Tunnicliff [65] claims that if absorption occurs, there must be a space or volume for the absorbed material to occupy after penetrating through the surface. Filler presents a large surface and hence a large capacity for adsorption, but its volume, and thus its capacity for absorption is small. Absorption may occur, but probably its effect is negligible. Tunnicliff further explains that because of the surface energy of attraction between the two materials, the adsorbed layer of bitumen assumes a different, and a stiffer consistency. It seems reasonable to suppose that the bitumen film not necessarily have a constant thickness. The bitumen is assumed to have its most rigid character immediately adjacent to the particle, and becomes increasingly less rigid as the distance from the particle increases.

The function of the two mixture components, mineral aggregate and asphalt cement, is distinctly different for load bearing purposes, as described by Tunnicliff [65]. Mineral aggregates resist deformation because of its internal stability, while asphalt cement, within a mass of aggregate, resists deformation because of its cohesion (rheological properties). Tunnicliff has given the following definitions of these properties:

1. *Internal stability* is that mechanical property of granular masses which produces resistance to displacement by mutual support of adjacent particles in the mass, involving static forces and reactions between particles too large to be noticeably affected by molecular forces.
2. *Cohesion* is that property of a material, which produces resistance to displacement by mutual attraction between particles involving forces of molecular origin, characteristics of microscopic and submicroscopic matter. This definition is intended to include both cohesion and adhesion.

1.5.3 Characteristics of filler materials

So far there has not been established an accepted description of the material, which is defined as filler. The term mineral filler has been generally applied to a fraction of the mineral aggregate, most commonly defined as material less than 0.075mm. Tunnicliff [66] refers to Miller and Traxler who have given the following filler characteristics:

1. Primary characteristics of fundamental importance: particle size, size distribution, shape.
2. Primary mineralogical characteristics of less importance: texture, hardness, strength, specific gravity, wettability.
3. Secondary characteristics dependent on one or more primary characteristics: void content, average void diameter and surface area.

Slyngstad [79] states the most important filler characteristics in asphalt mixes to be:

- Content
- Type
- Void content
- Gradation

- Water susceptibility
- Aggregate form
- Surface area.

Rigden [75] performed a study where the objective was to find a simple characteristic of the filler, which could be directly related to the behaviour of filler-binder mixtures. Some of the conclusions from the study are:

- The percentage passing a No. 200 sieve is inadequate as a means of expressing or defining the fineness of filler.
- None of the primary fineness characteristics of fillers are suitable, by itself, for evaluating the influence of a filler on the flow properties of filler-binder mixtures.
- A compaction factor, which expresses the concentration of filler in any filler-binder system in terms of the degree of compaction of filler, provides a promising means of relating the flow properties of such systems to a simple characteristic of the filler.

Heukelom [62] refers that Rigden has shown that the fractional voids are of major importance for the behaviour of fillers, and it followed from this investigations that the volumetric proportion of filler including these voids is a main parameter for the rheological properties of filler/bitumen mixtures. Heukelom further states that the apparent or bulk volume percentage of filler is the main parameter determining the consistency of filler/bitumen mixtures. Other factors affecting rheological properties according Heukelom are: differences in the chemical nature, grading and related properties of the filler and the type of bitumen used. According to Heukelom, these secondary effects may be ignored in practice.

Höbeda [76] refers to Kandahl (1980) who claims that void content of 50 % in filler is critical. Filler with higher void content must be investigated before use.

1.5.4 Effects of adding filler to bitumen

In binder specifications there are requirements to different properties for the bitumen used in the asphalt mixture. Anderson and Goetz [67] claim that to assume that this in turn specifies the properties of the filler-bitumen mix is to imply that the consistency of the bitumen in the mix is not influenced by the nature of the filler. In other words, the tacit assumption in customary usage is that there is an absence of interaction between the bitumen and the filler.

Höbeda [76] refers to the following effects of filler on asphalt concrete given by Anderson (1987):

- Makes the binder stiffer.
- Extend the bitumen.
- Improves moisture resistance.
- Reduces bitumen ageing.
- Influence the mix workability and compaction effort.

Stiffening effect

Höbeda [76] refers that strongly stiffening filler normally has fibrous, flaky particle shapes. The shapes seem to be more important than grain-distribution curves. It may be porous and in some cases chemically reactive with bitumen. Stiffening fillers may be very fine-grained, but certain fine fillers have poor stiffening properties and "extend" the bitumen instead.

Tunnicliff [65] explains that the stiffening is caused by the surface energy of attraction between the mineral surface and the bitumen. Tunniclif further indicates that the stiffened or adsorbed bitumen can be characterised by a condition of closer packing of its molecules than existed initially.

Temperature susceptibility

Tunnicliff [65] states that at low temperatures, where adsorption is usually strongest, the internal resistance to temperature stresses should be greater in the adsorbed bitumen than in the original bitumen. At high temperatures the effects of adsorption are somewhat reduced, but nevertheless present.
Tunnicliff also states that the selective adsorption of asphaltenes may have a bearing on temperature effect. Tunnicliff further refers to discussions by Miller and Traxler, and Spielman and Hughes, were it was noted that the asphaltenes are adsorbed more readily, that they are the most complex components of bitumen and that they are also the most viscous materials. The remaining bitumen, which is less susceptible to adsorption, is more fluid and of an oily nature. With respect to low temperature, if this selective adsorption occurs, the material that is the least affected by adsorption, and therefore the most available for cracking, is the most fluid and the least likely to crack.

The interaction between bitumen and aggregate surfaces before and after low temperature storage at various film thickness was investigated by Huang et al [74], using the sliding plate microviscometer with standard glass plates and machined aggregate plates. The conclusions from this study are:

- The findings indicate that there is an effect of the aggregate surface chemistry on the properties of thin bitumen films after long-term storage at room temperature.
- The results show clearly that the different chemical compositions and interactions of the bitumens in relation to the different types of aggregate affect the hardening of the specimens differently, especially after long-term storage.
- Binders that are graded alike by the current Superpave performance-grading system may behave differently as they contact aggregates, especially after long-term storage at room temperature.

Höbeda [76] refers that when using filler with stiffening effect, softer bitumen quality could be used in the asphalt mix, which in turn could affect the low-temperature properties.

Increased water resistance

Tunnicliff [65] states that the improved water resistance can probably, as for stiffening, be explained due to closer packing of molecules by the surface energy of attraction between the mineral surface and bitumen.

Bitumen ageing

Mineral filler in mastic is assumed by Gubler et al [78] to have two different effects on binder ageing. Filler can act as a catalyst and enhance the oxidation. On the other hand the filler is a hindrance to diffusion and can protect against reaction with oxygen which reduces hardening and ageing.
Gubler et al have investigated the effects of mineral filler on ageing of bituminous binders. Pure bitumen was aged in laboratory and then mixed with filler. The results from these mixes were then compared with aged mastic (filler and bitumen were aged together). The test equipment used in the investigation were dynamic shear rheometer (DSR) and bending beam rheometer (BBR). The results from the investigation showed that the ageing depends on the properties of the filler and, to a lesser extent, of the bituminous binder. Since hardening and ageing of the binder generally are of importance for low temperature thermal and fatigue cracking, these findings are of practical importance. Gubler et al claims that filler that strongly enhances oxidation may be detrimental, especially in pavements were cracking is the critical failure mechanism. Gubler et al further argues that the combination of filler and bituminous binders should be tested in such cases.

Höbeda [76] refer that polar components in bitumen are bounded to the mineral surface and the low-temperature and ageing properties of the remaining bitumen are then improved when adding filler. Further it is referred that increased bitumen content with given filler content could give improved ageing properties.

How filler acts

Beneficial effects resulting from the addition of filler to bitumen is already mentioned. In general it gives more durable pavement. Tunicliff [65] refers that it is believed that all of these effects can be explained on the basis of adsorption.

A preferred definition of filler is based on the premise that certain mineral particles are suspended in bitumen and that these particles alter the consistency of the bitumen. The suspension has been called colloidal.

Particle size

Primarily the suspended filler particle offers a large surface area compared to its volume. This in turn can explain the dominance of the adsorption effect.

Adsorbed layer of bitumen

Tunnicliff [65] has presented the following sketches to clarify the concepts of surface energy, adsorption, and stiffening. Figure 1.18 shows a filler particle of diameter D surrounded by an adsorbed layer of bitumen of thickness T1, which represents the optimum film thickness or the layer under the maximum influence of adsorption. A second layer, of thickness T2, which represents the additional bitumen influenced by adsorption, surrounds this. The double crosshatched area depicts the surface energy gradient, or the stiffening effect.

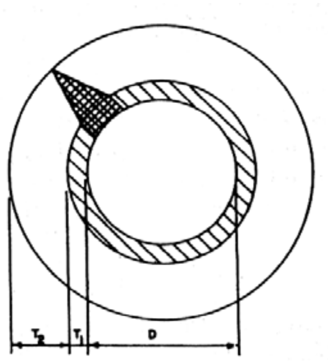


Figure 1.18 Suspended mineral particle in bitumen [65].

Figures 1.19-1.23 illustrate filler-bitumen mixes with different compaction effort.

Figure 1.19 Loose packing [65].

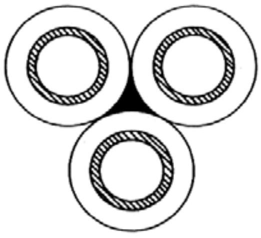


Figure 1.20 Loose packing with maximum influence on bitumen [65] .

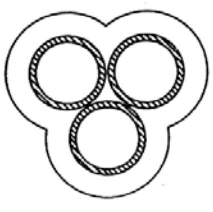


Figure 1.21
Tight packing [65]

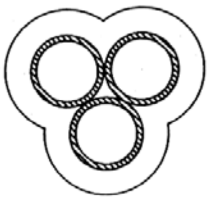


Figure 1.22
Tight packing
Maximum stiffening [65]

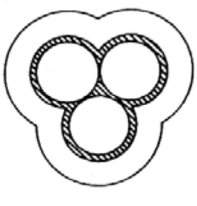


Figure 1.23 Repulsion[65]

Figure 1.22 shows tight packing and a system with maximum resistance to deformation. (This is the optimum binder content, which was presented by Rigden).

Particle size distribution
Tunnicliff [65] claims that if filler materials can be evaluated properly, any available material can be used, and an ideal grading should be viewed as a goal, not a requirement.

1.5.5 Rheological properties of bitumen and filler mixes

The rheological properties of the mastic (bitumen and filler) in an asphalt pavement certainly influence the behaviour of the pavement subjected to different types of loading. Total surface area and chemistry of the filler particles are mentioned by Kallas and Puzinauskas [64] to be important in influencing the flow behaviour of the bitumen-filler mixture.

Bitumen and filler-bitumen systems may exhibit both Newtonian and non-Newtonian behaviour depending of the loading and temperature conditions. Tunicliff [66] states that the general relationship, that viscosity of the system is a function of filler concentration, filler type, and viscosity of the bitumen, appears to be valid at all levels of filler concentration.

The bitumen molecules closest to the filler mineral particles are bounded to the surface as illustrated in Figure 1.24.

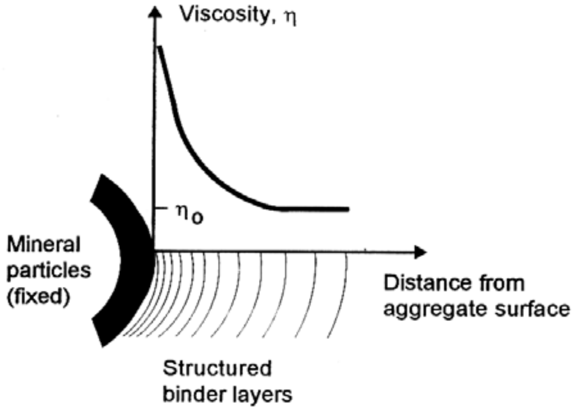


Figure 1.24 Schematic picture of mineral surface and bounded bitumen and how the viscosity is influenced [76].

Permanent deformation of hot mix asphalt (HMA) is influenced by the aggregate interlock and the viscous behaviour of the bitumen/filler mastic. In the Strategic Highway Research Program (SHRP), tests are proposed to determine the rheological properties of the pure bitumen. In a study performed by Wortelboer et al [68] the rheological properties of different binders mixed with different fillers where examined. In Figure 1.25 the result from permanent deformation of HMA in dynamic creep testing is shown.

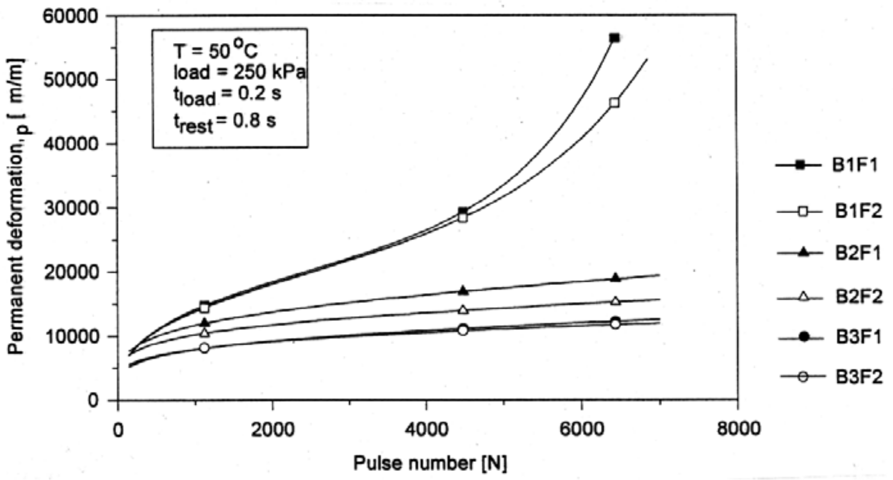


Figure 1.25 Permanent deformation in HMA in dynamic creep test with varying types of filler and binder [68].

As can be seen in Figure 1.25, the permanent deformation is dependent on type of binder as well as type of filler. B1, B2 and B3 are three different binders and F1 and F2 are two different fillers.
From the study by Wortelboer et al it was also concluded that although the bituminous binder is of principal importance to the rutting resistance, the influence of the filler must not be underestimated.

Different fillers can give different rheological properties to the bitumen-filler-mix, as can be seen in Figure 1.26.

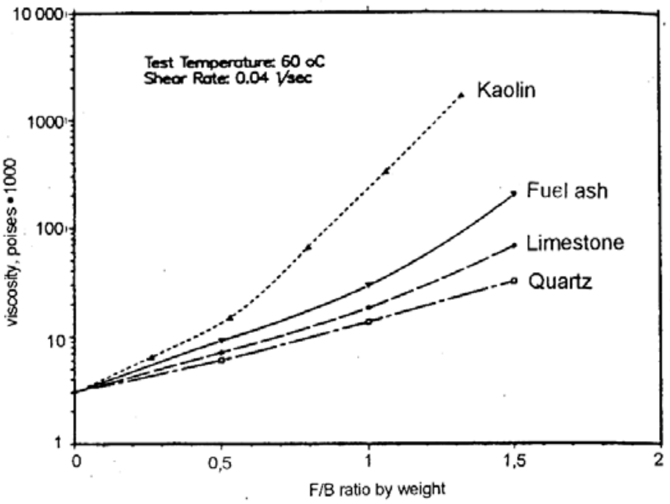


Figure 1.26 Viscosity of different filler-bitumen-mixes (Kavussi and Lees 1992) [76].

Soenen and Teugels [77] concluded, based on their study of filler-bitumen interactions using DSR, that;

- The relative stiffening effects are only dependent on the volume percentage of filler added.
- The stiffening is independent on the type of filler as well as on the origin of the bitumen.
- The oxidative ageing is not influenced by any of the different fillers used.

It is concluded by Anderson and Goetz [67] that both filled and unfilled bitumen can be satisfactorily represented as thermorheologically simple linear viscoelastic materials.

For the design of HMA mixtures, the properties of the bituminous binder are important. Wortelboer et al [68] also claim that the rheological properties of the bitumen/ filler mixtures should be included in pavement design considerations.
Also Dukatz and Anderson [29] states that bitumen properties alone are not sufficient to describe the creep and compaction behaviour of asphalt concrete mixtures made with fillers that exhibit stiffening effects.

1.5.6 Effect of adding hydrated lime to bitumen

Hydrated lime (HL) is described by Lesueur and Little [69] as an active filler in some bitumens. They claim that HL interacts with certain bitumens to develop an adsorbed (interactive) layer around the HL particle. The level of interaction between HL and bitumen is said to be dependent of the type of bitumen. The interaction between bitumen and HL is said to strongly affect high temperature rheology in certain bitumens but less in others. Lesueur and Little states that the low temperature stiffening effects of HL are less prominent. Lesueur and Little describe HL as a multi-functional additive with potential benefits to the binder and to the mixture that include: resistance to deformation, low temperature fracture toughening and increased resistance to age hardening. They claim that HL may be competitive with some polymer additives.

The objective of a laboratory study performed by Hanson et al [73] was to determine if adding the hydrated lime to the fine aggregate fraction and then adding the hydrated lime/fine aggregate mixture to the remainder of the aggregate would produce the same results as if the hydrated lime had been added to the entire aggregate stream. It appears on the basis of the data developed for this study that these two methods of hydrated lime addition are equivalent in reducing moisture damage susceptibility.

In a work performed by Little et al [82] it is concluded that the influence of the filler effect is dependent on certain mixing conditions needed to allow complete adsorption of the polar molecules of the bitumen onto the hydrated lime particle surface. Little et al characterise hydrated lime as an effective, multi-functional asphalt modifier which has the ability to improve asphalt concrete performance through a filler effect, improved bonding of the bitumen and aggregate, fracture toughening and reduction in the effects of ageing.

Höbeda [76] also refers that hydrated lime improves moisture resistance, reduces bitumen ageing and appears to have stiffening properties. Further it is referred that special fillers, such as hydrated lime, must be used in low contents in the asphalt mix.

1.5.7 Filler specifications

Norwegian filler specifications

The aggregate mix is normally produced by a blend of several aggregate fractions. If it is difficult to achieve sufficient fines content by a composition of the material fractions available, additional filler has to be added. Filler should be produced from crushed high quality stone. The filler should be sufficient dry and should not contain any clumps or any organic material. The filler material has to satisfy the requirements in Table 1.3 and 1.4.

Table 1.3 Requirements of commercial filler, gradation [80, 81].

Mesh, µm	Rest, percent of mass
500	0
75	0 – 20

Table 1.4 Requirements to filler particles less than 75 µm [80,81].

Particle size, µm	Percent of mass passing mesh
40	> 45
2	< 20

The material fraction less than 40 µm and less than 2 µm shall be calculated in percentage of the total mass of the material less than 75 µm.

The void content in the material less than 75 µm shall be in the range 25 – 50 percent by volume, and preferably in the range 30 - 50 percent by volume in dry condition. Void content shall be calculated according to the Rigden-test method.

New CEN filler specifications

In the development of European standards and specifications, a new specification for filler materials has also been developed. The proposed new filler specification from technical committee 154 (CEN TC 154) is given in Appendix 1. In this proposal several requirements are given and divided into categories. One of these categories is “no requirement”. The contractor should specify the category needed to meet the conditions for each specific case.

Factors to be evaluated in filler specifications

In the literature, several authors have recommended that filler properties should be specified in the mix design. Examples of these recommendations are given in the following.

Wortelboer et al [68] claim that for the design of HMA mixtures, the properties of the bituminous binder are important, but establishing the rheological properties of the bitumen/filler mixtures should also be included in pavement design considerations. Tunnicliff [65] refer that in determining the bitumen content of a mixture, four factors must be considered. These are absorption by the aggregate, adsorption by the aggregate in conjunction with its surface area, void space in the aggregate, and the stiffening affect of the filler [65].

Slyngstad [79] concludes that a filler-specification should contain requirements on the following parameters:

- Gradation - the function of the filler material is to fill voids in the other aggregates. The finest filler will strongly contribute to increased viscosity in the filler-bitumen-mix.
- Thermal stability - asphalt mix is produced at high temperatures (130-180^{o}C) and it is important that the filler material is not deteriorated at these temperatures.
- Bitumen content - the viscosity increasing effect from the filler material will affect the mixing and compaction energy. Void content and surface area in filler material seems to be important parameters for increasing viscosity effect.
- Water susceptibility – asphalt pavements are often exposed to moisture. It is necessary for the filler material not to be dissolvable in water.
- Contamination and moisture – Contamination and moisture could change the material quality, and problems during production could arise.

1.6 Ageing

1.6.1 General

Bitumens in asphalt pavements are exposed to ageing processes during storage, mixing, transport and paving, as well as in service life. During the different stages, ageing conditions and mechanisms may vary considerably.

1.6.2 Mechanisms of bitumen ageing

Of several mechanisms for bitumen ageing, the following four are among the most important :

- Reaction with atmospheric oxygen.
- Evaporation of volatiles.
- Phase separation in connection to porous aggregates, exudation.
- Physical hardening due to altering the molecular configuration.

Johanson [17] refers that oxidation probably is the most important one.

Short term ageing, which takes place during mixing and laying of bituminous materials, is mainly related to oxidation and evaporation. Long term ageing, which occurs during service, is mainly related to oxidation and physical hardening. When using porous aggregate, absorption of certain oily components into the aggregate leads to exudation (phase separation) and a corresponding bitumen hardening [17].

Oxidation
Robertson [5] describes oxidation as permanent hardening in bitumen, while the hardening from reduced temperature and from molecular organisation is reversible.

Oxidative ageing is an irreversible chemical reaction between components of bitumen and atmospheric oxygen, and can occur during mixing, laying and service. Johansson [17] states that oxidation is a chemical process, and that the ageing that results from oxidation is a function of the composition of the original bitumens. Further, Johanson refers that the inherent reactivity of generic fractions in bitumen with oxygen was reported as: asphaltenes > polar aromatics (resins) > aromatics > saturates.

In SHRP [42] it is claimed that the major products of bitumen oxidation resulting from oxygen incorporation are carbonyl and sulfoxide chemical functionalities.
The stiffening of bitumen that occurs upon oxidation is a result of increased structuring caused by an increased amount of polar molecules.
In SHRP it is also concluded that the temperature of the oxidation reaction of bitumen is a very important factor that determines the rate of oxidation, the amount of oxidation, and the resulting stiffness of the bitumen.
Ageing tests conducted at temperatures substantially higher than service temperatures can produce oxidation and stiffening in some bitumens that may not be representative of the ageing that occurs at field conditions.
An important conclusion from SHRP is that an ageing test used to predict long-term durability of bitumen or asphalt pavements must take into consideration the climate to which the pavement will be exposed.

Evaporation
The volatility of bitumen is most severe at high temperatures and when the bitumen is present in the form of thin films. SHRP binder specifications includes a 1 % RTFOT mass loss limit, to avoid performance problems due to evaporation.

Zupanic and Baselice [11] states that bitumen volatility arises due to three primary changes which occur when asphalt is exposed to high temperatures in the presence of air:

1. existing molecules are volatilised,
2. oxygen reacts with existing molecules to create new molecules (generally more polar and thus less volatile),
3. thermal cracking of existing molecules creates new molecules (generally smaller and thus more volatile).

They further explain that the volatility of bitumen is of specially interest at the mixing plant, primarily due to the environmental effects.

Johansson [17] refers that this type of ageing can be reduced by minimising the volatility of the binder. Johansson further refers that the penetration grade bitumens are relatively involatile and hardening due to the loss of volatiles is usually fairly small within the pavement during service.

Exudation
Johansson [17] has explained exudative ageing as a result from loss of oily components, which exude from the bitumen into the mineral aggregate. This type of ageing is explained to depend on both the exudation tendency of the bitumen and the porosity of the aggregate. Despite the fact that it is not a significant contributor to bitumen pavement hardening, irreversible exudation of bitumen components will produce compositional changes in the bitumen that may also significantly affect bitumen properties and other types of ageing.

Physical hardening
Physical hardening is hardening due to molecular organisation. Physical hardening is a reversible process, which can produce major changes in the rheological properties of bitumen without altering the chemical composition [17].
The hardening due to molecular organisation is often called “steric hardening”. Robertson [5] explains that hardening resulting from molecular organisation can be reversed periodically by recycling pavements. Robertson also describes that traffic tends to disorient molecular species, but it is not clear whether traffic loads speed up the orientation by providing additional energy for molecules to move or if orientation is slowed by keeping the system “stirred”. In SHRP [42] another reversible hardening phenomenon was observed: low temperature isothermal physical hardening. The hardening rate has been observed to decrease rapidly with time, to be highly temperature dependent, and to be bitumen specific. The lower the temperature, the faster the hardening rate.
There is a great need of techniques for measuring and evaluating this type of ageing and of methods for determining the relationship between physical hardening and bitumen chemistry [17].

Other mechanisms affecting bitumen ageing and performance
In addition to the four main mechanisms mentioned above, several other mechanisms could affect the bitumen properties in varying degree. An overview of important factors affecting bitumen properties is given in Table 1.5.

Table 1.5 Mechanisms affecting bitumen ageing and performance [55].

	Influence by					**Occurs**	
Effects	**Time**	**Heat**	**Oxygen**	**Sunlight**	**B&G Rays**	**At Surface**	**In Mass**
1. Oxidation (in dark)	X	X	X			X	
2. Photioxidation (direct light)	X	X	X	X		X	
3. Volatilization	X	X				X	X
4. Photooxidation (reflected light)	X	X	X	X		X	
5. Photo chemical (direct light)	X	X		X		X	
6. Photo chemical (reflected light)	X	X		X		X	X
7. Polymerization	X	X				X	X
8. Development of an internal structure (ageing) (thixotropy)	X					X	X
9. Exudation of oil (syneresis)	X	X				X	
10. Changes by nuclear energy	X	X			X	X	X
11. Action of water	X	X	X	X		X	
12. Absorption by solid	X	X				X	X
13. Adsorption of components at solid surface	X	X				X	
14. Chemical reactions or catalytic effects at interface	X	X				X	X
15. Microbiological deterioration	X	X	X			X	X

1.6.3 Factors affecting bitumen ageing

The circumstances under which hardening occurs varies considerably. The conditions where hardening occurs could be divided into:

1. Hardening during production and laying
2. Hardening of bitumen on the road.

In Figure 1.27 the ageing of bitumen during mixing, subsequently during storage, transportation and application and finally in service are shown.

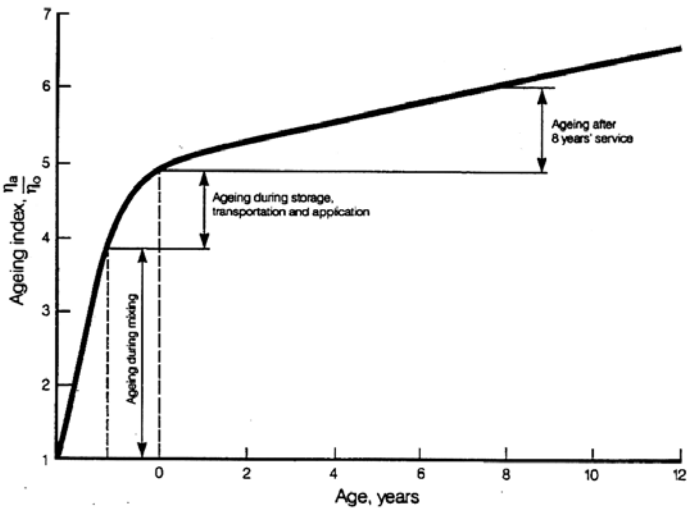


Figure 1.27 Ageing of bitumen during mixing, subsequently during storage, transportation and application and finally in service [54].

Hardening during production and laying

Hardening of bitumen in bulk storage

According to [54], when bitumen is stored in bulk at high temperature, very little hardening occurs. The reason for this is explained to be because the surface of the bitumen exposed to oxygen is very small in relation to the large volume. However, it is mentioned that if the bitumen is being circulated and is falling from the pipe entry at the top of the tank to the surface of the bitumen, significant hardening may occur.

Hardening during mixing with aggregates

During the mixing process all aggregate and filler is coated with a thin film of bitumen. As stated in the literature [17, 54, 56], when bitumen is mixed with hot aggregate and spread into thin films in an asphalt pugmill, conditions are very favourable for oxidation and loss of volatiles. Production related factors, such as temperature and mixing time is stated to be of great significance for the ageing.

Johansson [17] refers that the nature of aggregates and particle size distribution could also affect the bitumen ageing. In contradiction to this, bitumen-aggregate ageing experiments from SHRP [42] indicate that there is no predictable pattern of influence of aggregate on ageing of bitumen. However, there are some observations of bitumen ageing in the presence of aggregates that differ from ageing of neat bitumen. In these observations it is proposed that the aggregate surface induces structuring in the bitumen. It is believed that this structuring reduces the amount of viscosity increase that occurs upon oxidation compared with the neat bitumen, resulting in lower viscosity of the bitumen recovered from the aggregate surface.

Ruth and Roque [56] and Page et al [71] states that the type of mixer used also affects the amount of hardening during mixing. According to [54] it has been recognised that the amount of hardening in a drum mixer is often less than in a conventional batch mixer.

In [54] a rough rule-of-thumb concerning hardening during mixing is given: *"during mixing bitumens harden by approximately one grade, e.g. 100 pen to 70 pen, 70 pen to 50 pen, etc".*

Hardening during hot storage, transportation and laying.
Some hardening of the bitumen will take place during hot storage, be it in a silo or in a vehicle. In [54] it is claimed that the amount of hardening will depend principally upon duration of exposure to oxygen, the thickness of the bitumen film, the time of the mixing process and the temperature of the mix.

Hardening during service

In [54] it is claimed that the main factor which influences bitumen hardening on the road is the void content of the mix. Figure 1.28 shows the in-situ bitumen properties of five year old asphalt concrete with void contents ranging from 3 % to 12 %. At void content less than 3-4 % very little hardening in service has occurred.

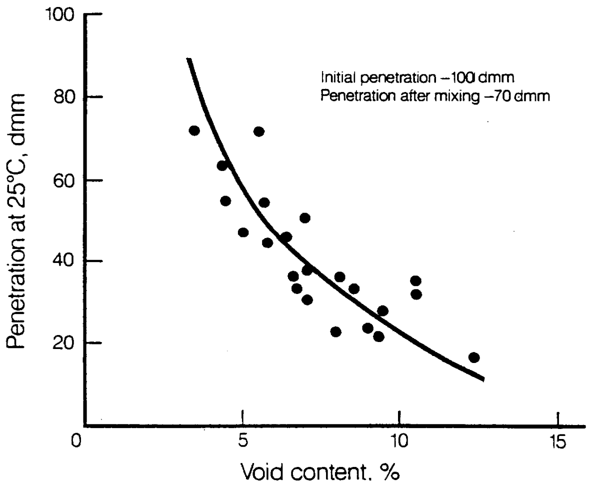


Figure 1.28 The effect of void content on the hardening of bitumen on the road [54].

1.6.4 Ageing and chemistry

It is concluded several places in the literature [13, 28] that the chemical composition of bitumen varies with crude, distillation and conditions after production. Corbett and Schweyer [28] claim that in the world today, about 260 crudes may be used or are suitable for manufacturing bitumen. An example of the differences in composition of bitumens from different crudes is shown in Figure 1.29. Yaw et al [32] claim that it is generally not enough to judge a bitumen solely on the basis of the quantity of a particular fraction.

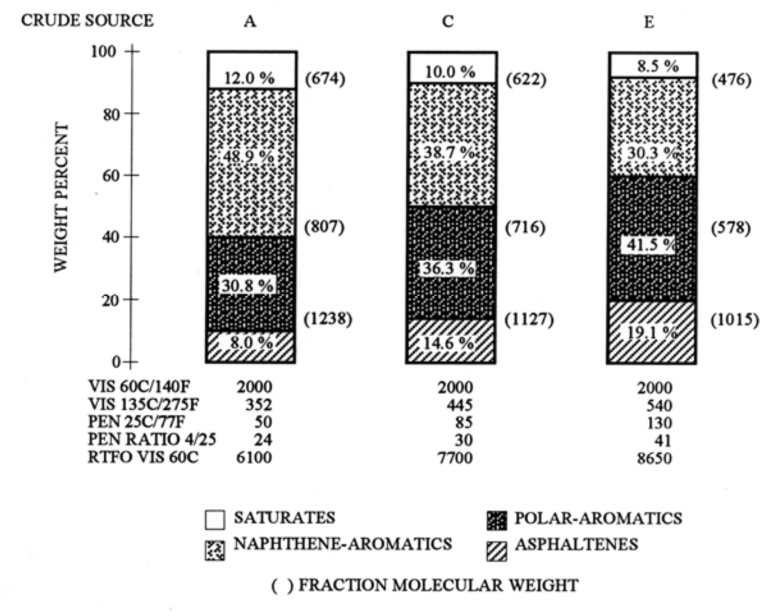


Figure 1.29 Differences in composition of bitumens from different crudes [28].

Corbett and Schweyer [28] have further studied the compositional changes during age hardening. In Figure 1.30, a significant reduction in Naphthene-aromatics with age can be seen. A conclusion from this study is that age hardening is a result of a change in the bitumen composition in which the naphthene-aromatics are converted to polar-aromatics and they in turn to asphaltenes. These changes create increase in viscosity and a lowering of penetration.

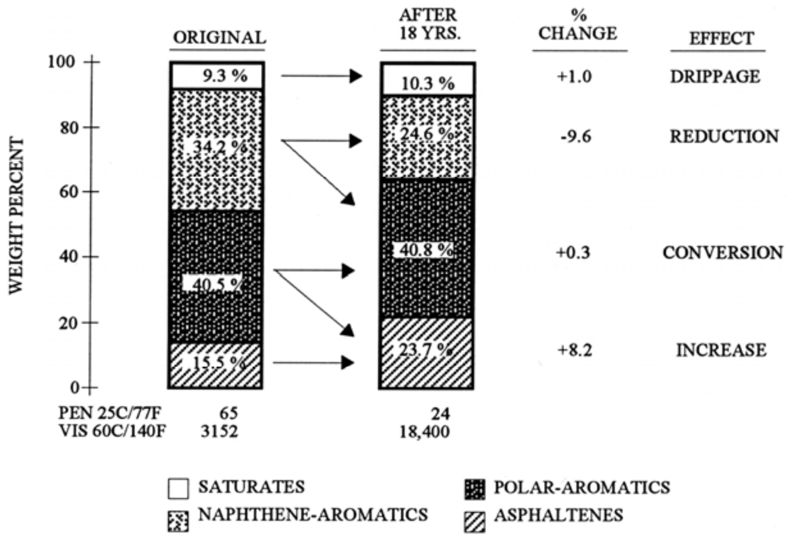


Figure 1.30 Compositional changes in a bitumen during age hardening [28].

As earlier mentioned, an important ageing mechanism is oxidation. Several places in the literature [5, 12, 13, 17, 41] oxidation is described to take place where heteroatoms (N, O and S) and chemical active hydrogen atoms are available. Polar groups such as hydroxyl, carbonyl and carboxyl are created. This in turn contributes to additional molecular associations. Johansson [17] mentions the carbonyl (C=O) formation as an important result of bitumen oxidation. It is claimed that the oxygen atom augments the polarity of the molecules containing carbonyl functionalities and thus enhances the intermolecular forces that build up the stiffness of the bitumen.

Johansson et al [36] refers that bitumen oxidation may in part be catalyzed by the naturally occurring metal complexes that also are found in most bitumens. Johansson at al further claim that the most abundant metal complexes found in bitumens are those of vanadium and nickel.

In SHRP (Jones) [12] it is argued that the production of polar materials during ageing is not in itself sufficient to cause major changes in the physical properties of the bitumen. It is claimed that the production of polar materials must generate molecules that are amphoteric (the bitumen molecule has both an acid and a base group on the same molecule) in nature to result in significant physical property changes. Bitumens with two or more active sites per molecule are illustrated in Figure 1.31. The dot indicates a site where oxidation can take place, and the asterisk indicates an oxidized atom.

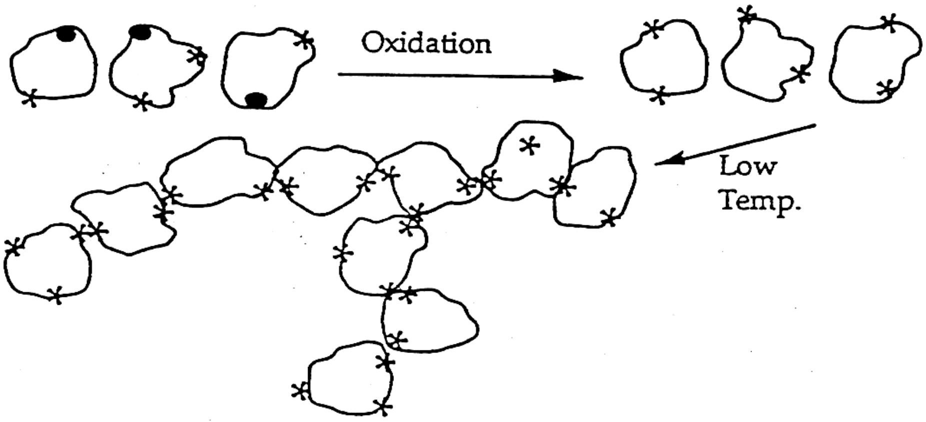


Figure 1.31 Bitumen with multiple active sites [12].

The effect of associated and solvent materials on pavement performance is shown in Figure 1.32.

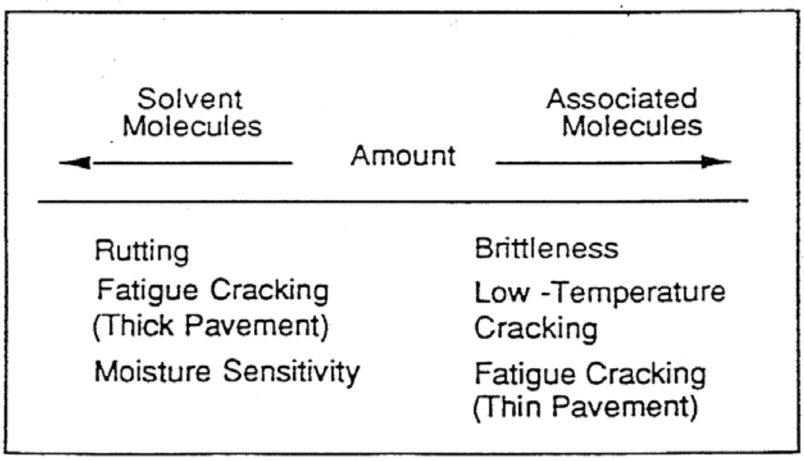


Figure 1.32 Relationship of molecule type to performance [12].

In SHRP [12] it was concluded that the nonpolar molecules play two critical roles in bitumen (12):

- The nonpolar solvent makes a critical contribution to the low-temperature properties of the pavement. The onset of low-temperature cracking in pavements is virtually independent of any other variable in the pavement system. At low temperatures the nonpolar bitumen molecules align themselves, resulting in shrinkage of the bitumen volume. This shrinkage takes place without crystallisation, and will cause thermal cracking if it is too severe. It is primarily a function of the molecular weight, but the shape of the molecules is also important, hindering collapse and retarding low-temperature cracking.
- As a solvent for the polar materials.

Figure 1.33 illustrates the relationship between bitumen chemistry and pavement performance.

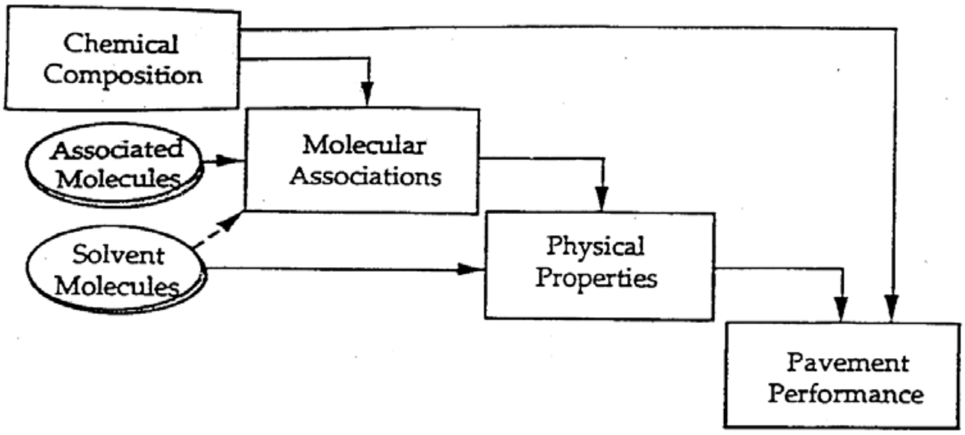


Figure 1.33 Relationship of chemistry to pavement performance [12].

1.6.5 Ageing and rheology

Bitumen has historically been graded by specifications based on consistency values at one or just a few temperatures. Penetration measurements at 25°C and viscosity measurements at higher temperatures (for example 60°C and 135°C) have often been used.

Bahia and Anderson [20] claims that to evaluate the effect of ageing, it is necessary to measure the visco-elastic properties, before and after ageing, across the temperature or loading time spectrum usually occurring in the field.
Dependency on time of loading or temperature implies that a single ageing index measurement at one temperature or loading time cannot be assumed to be applicable to estimate changes at other times or temperatures. For example, one ageing index using viscosity at 60°C cannot be used to estimate the rheological changes at low-temperatures where cracking is critical.

Johansson [17] refers that the magnitude of the viscosity or stiffness increase in ageing depends on bitumen type, severity and method of ageing, as well as evaluation conditions (e.g. temperature). The change, which appears during rheological testing, becomes less evident at lower temperatures or higher frequencies. This is referred to be a consequense of the fact that ageing has little effect on the glassy modulus, which in shear is approximately 1 GPa for most unmodified paving bitumens.
Johansson further refers that field ageing, measured in terms of viscosity, is highly dependent on climate and correlates fairly well with daily minimum temperature.

In SHRP [41] the use of empirical methods in comparison to rheological type was discussed. The empirical methods discussed were penatration index (PI) and penetration-viscosity number (PVN). The relationship between PI and rheological type as measured by more rigorous means is only a weak one, as is demonstrated in Figure 1.34, in which PI (from pen at 25^{o}C and softning-point temperature) is plotted against the rheological index found from rheological master curves (DSR-data).

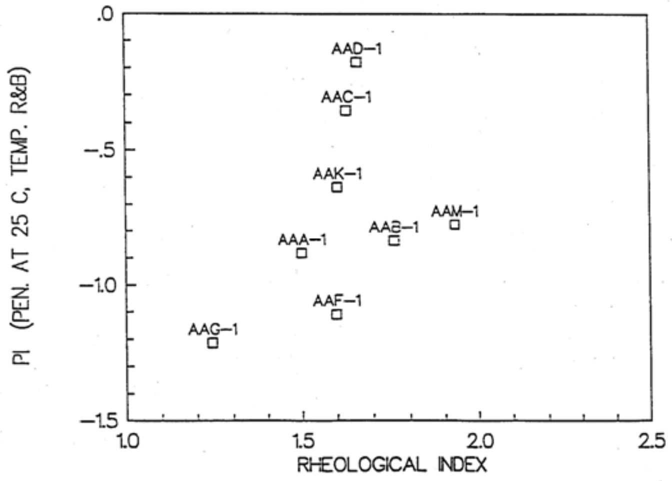


Figure 1.34 Penetration Index (based on pen at 25 ^{o}C and Ring and Ball Softening-Point Temperature) versus Rheological Type as Measured by Rheological Index, for eight bitumen types [41].

In SHRP [41] it was further stated that the rheological type and temperature dependence change with ageing, as illustrated in Figure 1.35, and it was concluded that the temperature-susceptibility parameter also changes with ageing. In SHRP it was stated that the PI values typically changes with ageing, but PVN values appear to remain unchanged with ageing, casting suspicion on the validity of PVN as a measure of temperature susceptibility.

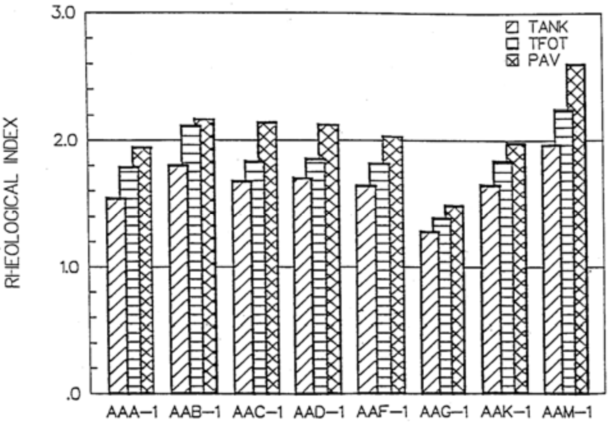


Figure 1.35 Affect of ageing on Rheological Index [41].

1.6.6 Laboratory analyses for bitumen ageing properties

Several laboratory methods have been used in the quantitative determination of bitumen ageing at various stages of the production process as well as in service. Bitumen display changes in mechanical and chemical characteristics when subjected to ageing. Either a chemical analysis or rheological test may evaluate ageing.
This chapter is divided into chemical analyses, conventional tests, SHRP binder specification and other tests used to evaluate ageing.

Chemical analyses

Analytical techniques for the chemical analysis of bitumens have been summarised in Table 1.2. Johansson [17] claim that the methods based on spectroscopy and chromatography are commonly used in characterising the ageing properties of bitumens. The main aim of these tests is to determine the changes in bitumen composition, and in turn to identify the ageing mechanisms as well as the relationship between chemical composition and physical changes. A description of two of these types of analytic methods is given in the following.

Infrared spectroscopy
Infrared spectroscopy is by Algelt et al [52] described as relatively inexpensive, simple and fast. Although it is ordinarily used for the identification of single compounds, it can measure distribution of several structural and functional groups in a sample. For bitumens the identification of functional groups, ketone and sulphoxide, before and after ageing is done using infrared spectroscopy.

Size exclusion chromatography (SEC)
SEC, or gel permeation chromatography (GPC), is the separation method according to Algelt et al [52], which comes closest to differentiating by molecule weight only, almost unaffected by chemical composition. Actually, it separates by molecular size.

Conventional tests

Thin Film Oven Test (TFOT) (ASTM D 1754 –76)
TFOT simulates the ageing occurring in bitumen during mixing, transportation and laying. During this test, bitumen is exposed to a temperature of 163°C for 5 hours.

Rolling Thin Film Oven Test (RTFOT) (ASTM D 2872)
RTFOT simulates, as TFOT, the ageing occurring in bitumen during mixing, transportation and laying. During this test bitumen is exposed to a temperature of 163°C for 75 minutes, when rotating in a bottle and injection of hot air every 3 – 4 seconds.

Penetration (ASTM 05-73)
The penetration test is maybe the most used test for specifying or measure binder consistency, both original and aged. In this test a needle of specified dimensions is allowed to penetrate into a sample of bitumen, under a given load (100g), at a fixed temperature (25°C) and for a given time (5 seconds). The distance the needle penetrates, in units of 0,1 mm, is termed the penetration value.

The softening point test (ASTM D36 – 70)
In this test a steel ball (3,5g) is placed on a sample of bitumen contained in a ring; this is suspended in a water or glycerol bath. Water is used for bitumens with a softening point of 80°C or below, and glycerol is used for softening points greater than 80°C. The bath temperature is raised at 5°C per minute, the bitumen softens and eventually deforms slowly with the ball through the ring. At the moment the bitumen and ball touch a base plate 25 mm below the ring, the temperature of the water is recorded. The test is performed in duplicate and the mean of the two measured temperatures is reported as the softening point.

Viscosity, kinematic (ASTM D2170)
This is a fundamental characteristic of bitumen as it determines how the material will flow at a given temperature. For many purposes it is common to measure the viscosity of bitumen by measuring the time required for a fixed quantity of material to flow through a standard orifice.

The Fraass breaking point test (DIN 1995/1960)
The Fraass breaking point test can be used to describe the behaviour of bitumen at very low temperatures. A steel plaque is coated with 0,5 mm of bitumen and slowly flexed and released. The temperature of the plaque is reduced at 1°C per minute until the bitumen reaches a critical stiffness and cracks. The temperature at which the sample cracks is termed the breaking point. The Fraass temperature (breaking point) can be predicted from penetration and softening point because it is equivalent to the temperature at which the bitumen has a penetration of 1,25.

Ductility test (ASTM D 113) (CEN TC 19)
In this test bitumen is immersed in a water bath and stretched at a constant speed of 50 mm per minute until fracture occurs. The distance the specimen has been stretched at failure is reported as the ductility.

SHRP binder specification

Background
In [41] the objective of the Strategic Highway Research Program (SHRP) Project A-002A, "Binder Characterization and Evaluation," is described as: *"to develop a clear understanding of the fundamental chemical and physical properties of bitumens and to correlate the effects of variations in composition on physical properties, including those that change with ageing, that affect performance of asphalt in roadways"*. The importance of this was to be able to describe, or predict, expected behavioural characteristics for any given bitumen. Hence, one should be able to select the most suitable bitumen for a given road construction project.

The following distress modes were considered as the most important in the SHRP binder specification: low-temperature, thermal fatigue, load-associated fatigue cracking and plastic deformation in the upper hot-mix asphalt layers that leads to rutting [41].

Bahia et al [40, 46] have raised the question to the applicability of the SHRP binder specification (PG-grading system) used for modified binders. Bahia et al [46] claim that it is evident that the Superpave specification is based on assumptions that apply only to binders that exhibit simple rheological behaviour, such as conventional unmodified binders. Bahia et al argue for a new classification system that differentiates between binders that can be considered as part of the PG-grading system and others that can not. Binders that satisfy the assumption made in the PG-grading system can be considered simple binders. The other binders should be classified as complex binders because they exhibit complex behaviour and do not satisfy the assumptions made in the PG-grading system.
Bahia et al further warn against ignoring modifier specific characteristics which may lead to underestimating or overestimating the contribution of modified bitumens to pavement performance.
Bahia et al also claim that modification of the current RTFOT procedure is needed to allow ageing binders with high viscosity.

In the following, a description of the tests in the SHRP Performance Graded Asphalt Binder Specifications system (PG-grading system) is given.

The Dynamic Shear Rheometer (DSR)
McGennis et al [45] have given the following description of DSR-measurements. The Dynamic Shear Rheometer (DSR) is used to characterise the viscous and elastic behaviour of bitumen. In the DSR the complex shear modulus, G*, and phase angle, δ, are measured. G* is a measure of the total resistance of a material to deforming when repeatedly sheared. McGennis et al describe G* to consist of two parts; a part that is elastic (recoverable) and a part that is viscous (non-recoverable)(This is a simplification compared to the explanation given by Anderson et al [43] in chapter 1.4.2.3). δ is an indicator of the relative amounts of recoverable and non-recoverable deformation. G* and δ are highly dependent on the temperature and frequency of

loading. By measuring G* and δ, the DSR is said to give a more complete picture of the behaviour of bitumen at pavement service temperature. Figure 1.36 explains G*, its components and δ.

DSR is performed on original bitumen to control the original stiffness.
On the RTFOT (or TFOT) treated binder, DSR-measurements are performed to control the rutting resistance of a new pavement. DSR-measurements are also performed on PAV-aged binder to control the fatigue properties.

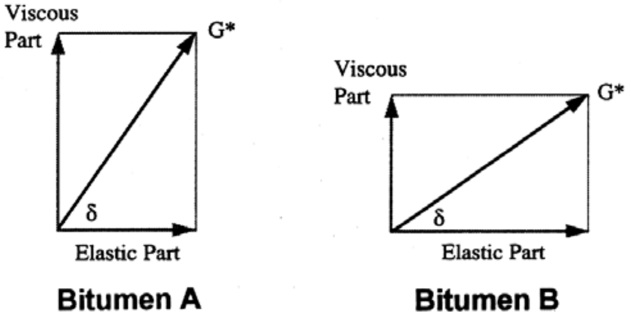


Figure 1.36 Data from DSR measurements [45].

The bending beam rheometer (BBR)
Bahia and Anderson [19] have given the following description of BBR-measurements. The bending beam rheometer is proposed by the SHRP for direct measurements of the rheological properties of bitumens at very low temperatures. Bahia and Anderson explain that two parameters are evaluated with the BBR. *Creep stiffness*, S (t), is a measure of how the bitumen resists constant loading and the *m-value* is a measure of how the bitumen stiffness changes as loads are applied. The m-value represents the rate of change of the stiffness, S(t), versus time.
BBR-measurements are performed on PAV-aged bitumen to control low-temperature thermal shrinkage cracking.
A schematic representation of the BBR-equipment is given in Figure 1.37.

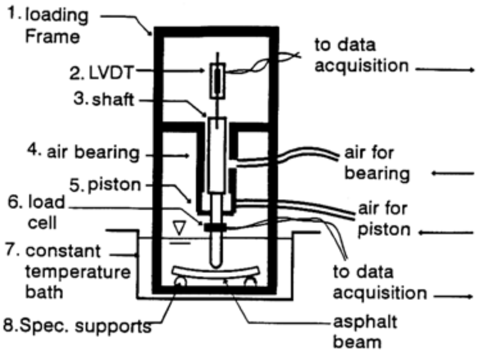


Figure 1.37 Schematic description of the Bending Beam Rheometer (BBR) [19].

The direct tension test (DTT)

The direct tension test measures the low temperature ultimate tensile strain of bitumen. In DTT the failure strain of the PAV-aged bitumen is measured to control the low-temperature thermal shrinkage cracking performance [45].

Rotational viscometer (RV)

The Rotational viscometer is used to evaluate high temperature workability of binders. High temperature binder viscosity is measured to ensure that the bitumen is sufficiently fluid when pumping and mixing [44].

The pressure ageing vessel (PAV)

The pressure ageing vessel (PAV) is an ageing procedure proposed by the SHRP to simulate long-term field oxidative ageing of bitumens [20]. It is concluded that temperature can be used effectively to accelerate ageing in the PAV and that the binder thickness and time of exposure are important factors that need to be carefully controlled.

Bahia and Anderson states that the SHRP A-002A project concluded with three main findings:

1. There exists no reliable simple procedure for characterising in-service, long-term, ageing behaviour of bitumens.
2. There are factors other than bitumen composition that may affect ageing rates of bitumens in the field, among which temperature is of most importance.
3. Ageing should be evaluated by considering not the amount of change in properties (ageing indices), but by considering important properties of bitumens measured, after ageing, at critical temperatures prevailing in the field.

Bahia and Anderson [20] have shown that the effect of ageing temperature on rate of ageing is bitumen-specific in the lab, as can be seen in Figure 1.38. This is also claimed to be the situation in the field.

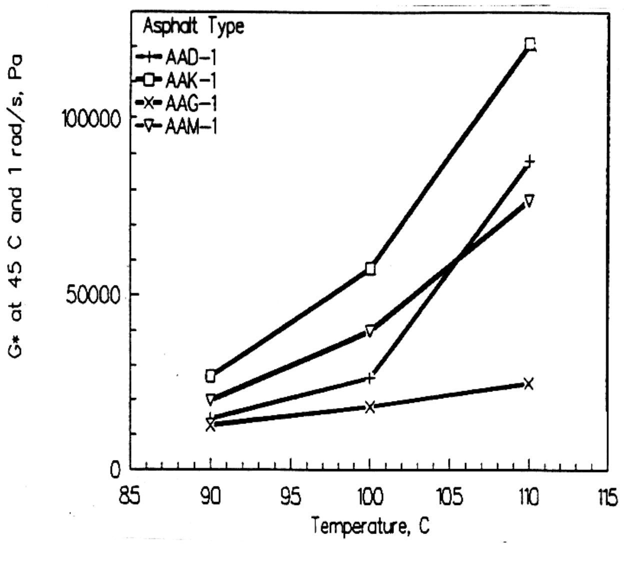


Figure 1.38 Effect of PAV temperature (90-110 °C) on measured G at 45 °C and 10 rad/s for four SHRP bitumens [20].*

Bahia and Anderson also point at the important finding that can be drawn from Figure 1.38, the general trend of the effect of temperature on the ageing level. The dependency of ageing on temperature is different among bitumens, but all bitumens age more with increasing temperature.

1.6.7 Investigations carried out on ageing

Several studies have been carried out on ageing of different binders and different types of mixes.

Artificially aged neat bitumen

Blomberg et al [50] have made an evaluation of the ageing methods TFOT and RTFOT .
Some of the conclusions from this investigation were:

- No correlation was found between change in mass and change in the physical properties examined. This indicates that oxidation is more significant than evaporation in hardening of the binder.
- The change in mass is about 50 percent greater in RTFOT than in TFOT.
- There is a high or very high level of correlation between the results in TFOT and RTFOT regarding the parameters analysed (Kinematic viscosity at 135°C, Dynamic viscosity at 60°C, penetration at 10 and 25°C, softening point and Fraass breaking point)
- The preceding point is valid provided that the correlation coefficient has been calculated over a wide range, e.g. for penetration at 25°C between 50 and 300. If the calculations are limited to a single penetration class, correlation is no longer significant. This means that penetration for a bitumen sample after heating cannot be predicted on the basis of penetration before heating.
- Hardening of the binder is independent of penetration class.

Noureldin [21] presents a study in which the changes in molecular size distribution and consistency, of a given bitumen, caused mainly by oxidation during the thin film oven test (TFOT) are investigated. The effect of artificial ageing is given in Table 1.6, where it can be seen that the amount of large molecular size components is increasing with increasing time of oven exposure during TFOT. In contrary the amount of medium and small molecular size components are decreasing with increasing time of oven exposure.

Table 1.6 Effect of artificial ageing [21].

	Time of Oven Exposure During TFOT				
Response variable	**0**	**2 hours**	**5 hours**	**10 hours**	**24 hours**
Penetration	65	43	33	25	17
Absolute Viscosity at 60°C, Poise	1890	3920	8780	25870	72190
Large Molecular Size Component, LMS, %	25.2	27.5	30.5	33.2	37.8
Medium Molecular Size Component, MMS, %	50.1	49.3	48.0	46.1	42.4
Small Molecular Size Component, SMS, %	24.7	23.2	21.5	20.7	19.8

Note: Values included represent the average of 3 replications.

There have been several attempts to characterise bitumen by chemical composition and structure. In a study by Price and Burati jr. [38], High Pressure Gel Permeation Chromatography (HP-GPC) has been used to predict the results of standard laboratory tests for bitumen. Good correlations between chemical composition and seven physical properties were found (penetration, specific gravity, cinematic viscosity, thin film oven test loss, penetration viscosity-number, viscosity-temperature susceptibility, and cinematic viscosity of thin film residue).

Johansson [17] refers that simulation of ageing at excessive temperatures may oxidise species that otherwise would not have been oxidised, and yields oxidation products that would not have been generated at a lower temperature. Johansson claims that it is important to simulate field ageing close to service temperature.

Ageing of asphalt-mixes

Agnusdei et al [57] have studied ageing of bitumen during the mixing process. They have concluded that oxidation and evaporation are the most important causes of hardening in the mixing process. Further they states that evaporation is dependent on temperature, mixing process, mixing time and type of bitumen. Results from one study indicate that the change in binder consistency was most significant in the first 30 seconds. In the USA, recommended mixing time is between 30 and 60 seconds.

Jacson and Brien [58] recommend the mixing time to be finished within 30 seconds. In Figure 1.39 the hardening of bitumen as a function of mixing time is presented.

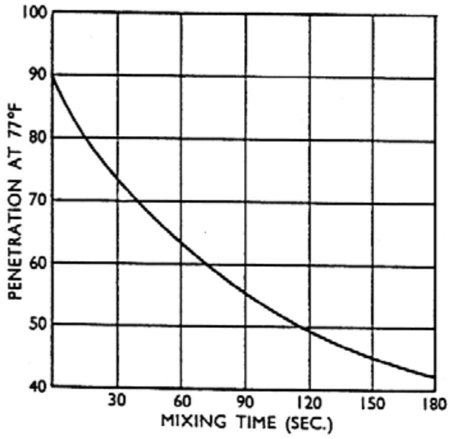


Figure 1.39 Hardening of bitumen with mixing time [58].

Höbeda and Chytla [23] refer to several investigations on ageing; From France it is reported that the reduction in penetration in a drum mixer is about 20-30 percent, in a batch mixer the reduction could be as high as 50 percent. Another investigation, by Lund and Wilson [30], also concluded that the bitumen hardening during mixing appears to be less in a drum mixer than in a batch mixer.

Höbeda and Chytla [23] refer to an investigation from Germany where it was found that hardening of bitumen is dependent on penetration class. After production, hot storage and laying the penetration value was reduced by 22-33 percent for Pen 80, and 9-14 percent for Pen 45. The softening point was for both increased by 2,5-5,5°C. Höbeda and Chytla also refer to experience from France which shows that cracking occurs when the binder in the pavement reaches the following "critical values";

Penetration (25 °C)	<20
Softning point	>70°C
Fraass breaking point	>0°C
Asphaltene content	>20 %

In Canada it is found that cracks due to temperature conditions increase significantly when the penetration value is below 35 [23].

Aurstad [59, 60, 61] has studied ageing of asphalt pavements on Norwegian airfields. Mix tests and tests on recovered binders have been performed on field samples and samples aged in a laboratory "wheatherometer" test. In the artificial wheatherometer-ageing, several parameters were varied in search of more ageing resistant asphalt mixes [59, 61]:

- Aggregate grading
- Penetration grade in binder
- Binder content
- Additives (polymer, fiber)
- Density, void content.

One conclusion from the testing on field samples is [60]:

> *Long term pavement performance is depended on binder content and void content. Critical values for long time performance seems to be minimum 5,5 percent binder content and 4,5 percent void content. The workability of the asphalt mix is also of great importance. Homogeneity and smooth surface structure also is of importance. Laying and compaction will also affect the pavement properties.*

Other conclusions from the studies by Aurstad are:

- The performance life of asphalt pavements on Norwegian airfields is dependent on climatic exposures.
- Ageing of the binder is the most critical factor for long-term pavement performance. Due to ageing, the pavements become stiffer and more brittle. This contributes to development of the most typical pavement distress in Norway; stripping and cracking.
- Several mechanisms contribute to ageing. During production and laying a significant ageing occurs. Material properties and climatic conditions will affect the ageing rate over the pavement life.
- Interesting results have been found using the “weatheromether”. It is necessary with further investigations to correlate results from weatheromether-testing to field conditions.
- To develop better long-term performance properties for asphalt pavements on airfields it is necessary to develop binder specifications and mix-design procedures and production and laying guidelines.

Santucci et al [27] refer to a study (a study by California Department of Transportation) of the hardening of three Californian bitumens at four geographically different exposure sites. In this study it was found that all the bitumens are affected by the percent of air voids in the mix, with less hardening in samples with the lower voids, especially the 4 percent void range. The study also showed that bitumen hardening progressed from the top of the cores to the bottom with the greatest effect in the top 13-mm slice, which was general twice the viscosity of bitumens recovered from the next lower 13 mm. Santucci et al further refer that aggregate type did not seem to influence bitumen hardening after two years’ exposure. Another conclusion from the study was that the most severe bitumen hardening took place at the desert site. Significantly less hardening was observed in bitumens from cores exposed at the mountain and coastal sites.

Harvei and Tsai [35] have studied how long term oven ageing affects on the fatigue properties and initial stiffness of asphalt concrete. The primary objective was to evaluate the effects of long-term oven ageing, air-void content and binder content on initial stiffness and pavement fatigue life, for typical California asphalt concrete mixes.

After mixing, the uncompacted, loose mix was subjected to four hours of short-term oven ageing at 135°C in a forced-draft oven. Long-term ageing of beams was performed using a forced-draft oven at 85 °C. Long-term ageing was performed for 0, 3 and 6 days. Harvei and Tsai concluded that increase in stiffness caused by long-term ageing, as simulated using long-term oven ageing, is not always detrimental to pavement fatigue life. They further conclude that the effect of long-term ageing on pavement fatigue life depends upon type of bitumen, aggregate type, pavement structure i.e. thick versus thin, and air-void content.

Höbeda [49] refers to experiences from Denmark on pavements with age over 20 year. These pavements can be characterised by the following factors; good quality aggregates and bitumen, strong adhesion, use of hydrated lime and amin, optimal mixing time and temperature at mixing. The void content was always less than 5 percent. Jacobsen and Wågberg [22] also mention the void content to be an important factor in the hardening of the binder.

Ageing of bitumen in cold-mixing (emulsion) versus hot-mixing has been studied by Boussad et al [48]. In the case of cold-mixes, bitumen is not heated, and is not subjected to ageing during the mixing operation. In order to investigate the difference between the ageing occurring in cold mixes versus hot mixes, Boussad et al carried out tests on cold mixes and hot mixes made from the same raw materials. Bitumen characteristics were tested before and after mixing. Characteristics for bitumen before and after RTFOT are given in Table 1.7.

Table 1.7 Bitumen characteristics before and after RTFOT [48].

35/50 pen bitumen	**Units**	**Method**	
Penetration 25°C	mm/10	NFT 66004	43
Softening point	°C	NFT 66008	52
Pen 25°C after RTFOT	mm/10	T 66-032	26
Soft. Point after RTFOT	°C	T 66-032	57
Insoluble n-Heptane	%	NFT 60115	10

Results from analysis on recovered binders are given in Table 1.8.

Table 1.8 Results from analysis on recovered binders [48].

Residual binder	**Units**	**Method**	**From hot mix**	**From cold mix**
Penetration °C	mm/10	NFT 66004	27	37
Softening point	°C	NFT 66008	56	52.5
Insoluble in n-Heptane	%	NFT 60115	14.5	10.2

Boussad et al gives the following comments based on the result in Table 2.7 and 2.8:

- Binder extracted from hot mix resembles the original 35/50 submitted to RTFOT in terms of penetration and softening point. The increase in asphaltene content is in line with this result.
- Binder extracted from cold-mix was subjected to lesser ageing. Its softening point is almost the same as the one of the original bitumen and penetration is slightly lower. However, its penetration is significantly higher than the penetration of the original bitumen submitted to RTFOT. No change in asphaltene content was observed.

Boussad et al further conclude that RTFOT is highly suitable for hot mixes but not for cold-mixes. However, they claim that bitumen is also subjected to a slight ageing during a cold-mixing operation. For cold applications, a less severe ageing test than the current RTFOT has to be used.

Bitumen ageing and hydrated lime

In an investigation by Edler et al [70] two different pen grade bitumens treated with 6 and 12 % hydrated lime were subjected to several laboratory-ageing tests. This laboratory study has shown that the addition of hydrated lime to bitumens retards the increase in oxidation products and high molecular weight constituents (HMWC). The result of this is less hardening. The addition of 12 percent of hydrated lime has a greater effect in retarding the formation of oxidative and HMWC than the addition of 6 percent of hydrated lime.
In a study by Plancher et al [31] hydrated lime decreased age hardening of the binder, using laboratory ageing at high temperatures (130^{o}C and 163 ^{o}C). They concluded that lime treatment:

1. Reduces the viscosity increases upon ageing
2. Removes reactive polar compounds
3. Reduces the formation of carbonyl-type oxidation products upon ageing
4. Reduces the ratio of asphaltenes formed to oxidation products formed upon ageing
5. Reduces asphaltene formation.

Plancher el al propose that the beneficial effect of lime treatment in reducing bitumen oxidative hardening is due to two synergistic effects: *(a) lime reduces the formation of oxidation products by the removal of oxidation catalysts or promoters and (b) lime reduces the sensitivity of the bitumen to these oxidation products by removing reactive polar molecules that would otherwise interact with the oxidation products to cause an increase in viscosity.*

In a study by Petersen et al [72] it was shown that lime treatment of bitumen reduced age hardening, increased the high-temperature stiffness of unaged bitumen, reduced the stiffness of aged bitumens at higher temperatures, and increased the bitumen tensile-elongation at low temperatures. These effects should benefit asphalt pavements by increasing asphalt durability, reducing rutting, shoving and other forms of permanent pavement deformation, improving fatigue resistance in aged pavements and improving pavement resistance to low-temperature transverse cracking. In this study the ageing was performed using a thin film accelerated test with ageing temperature of 113°C and ageing time of 3 days. Petersen at al summarises that the net result of the combined effects of lime treatment should result in longer lasting pavements with improved performance during the life of the pavement. As can be seen in Figure 1.40, limestone does not provide the beneficial effects on the ageing properties and low-temperature properties that the hydrated lime provides.

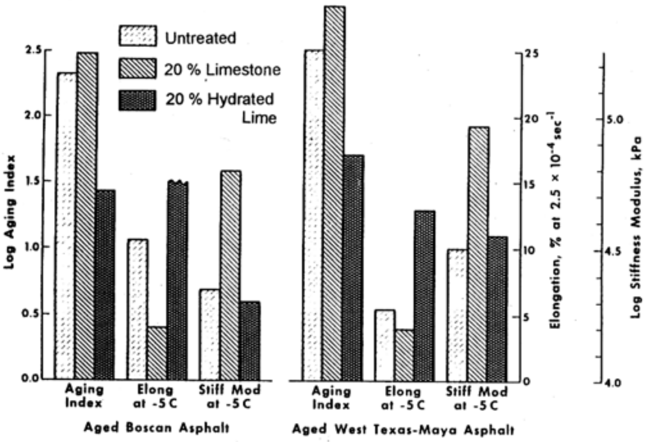


Figure 1.40 Comparison of the effects of limestone and hydrated lime on selected properties of aged bitumen [72].

A study of rheological properties of lime treated paving bitumens aged at 60°C in a pressure ageing vessel was carried out by Johansson et al [37].
In this study, eight bitumens, the SHRP core bitumens, were mixed with 20 mass % hydrated lime and oxidatively aged using the PAV procedure for 0, 1 and 2 weeks. Untreated control samples were aged along with the lime-treated bitumens. Based on carbonyl build-up, the extent of oxidation was approximately the same in the lime-treated and untreated bitumens. Based on the G* changes, the lime-treated samples harden at a slower rate, or to a lesser extent. The ageing indices of the lime-treated bitumens are smaller than the controls. Johansson et al conclude that lime treated bitumens aged at 60°C do not show the large reductions in ageing as reported from other studies where lime-asphalt mixtures were aged at higher temperatures (>100°C). Johansson et al explains the difference at different ageing conditions by fairly small molecular associations of polar molecules in bitumens at higher temperatures. At lower temperatures, polar molecules form associations and lime must compete with these. At higher temperatures, lime may inactivate oxidation promoters. This effect is not observed at lower temperatures
Johansson et al argue for the beneficial effect of lime as an additive to asphalt. The filler effect protects against early rutting, and there is claimed to be a slowing of low temperature oxidative age-hardening in bitumens treated with lime. Further they summarise that the initial differences in rheological properties between lime-treated and untreated bitumens decrease with time, but the major part of the roadway service life is said to be affected positively by addition of lime.

The influence of testing conditions on the capacity of hydrated lime to inhibit bitumen ageing has been studied by Johansson [17].
Ten bitumens with and without hydrated lime (20 % by weight) were tested before and after artificial ageing. The ageing method chosen was Thin Film Oven Test (TFOT) modified by using different ageing temperatures in the range of 60-170 °C. The following conclusions were drawn from this study:

- *In temperature modified TFOT, samples containing hydrated lime displayed more weight loss than samples without. The effect should be attributed to interaction between hydrated lime and bitumen and not hydrated lime itself*
- *The response to hydrated lime, with respect to the sulphoxides and carbonyls detected, is dependent on bitumen type.*
- *When comparing development of stiffness in terms of ageing index, it appeared that samples with hydrated lime generally show a lower ageing index than neat bitumen.*
- *The capacity of hydrated lime to reduce ageing indexes appears highly dependent on the methods and testing conditions used.*
- *When laboratory ageing is performed in very severe conditions, it is sometimes possible to demonstrate a striking effect of hydrated lime with respect to reduction of the ageing index.*

CHAPTER 2 *DISTRESS TYPES ON ASPHALT PAVEMENTS ON LOW VOLUME ROADS IN NORWAY*

2.1 Background

Deterioration of asphalt pavements takes several forms including wear due to studded tires, permanent deformation (rutting), ageing, fatigue cracking and other types of cracking. The deterioration of asphalt pavements is caused by 3 main factors: traffic, environmental factors and material properties.
Different types of cracks in the asphalt pavement could bee caused by frost heave, temperature cycling, low temperature, reflective cracks (cracks in the underlaying layers), deformation in subgrade and traffic loading.

About 95 % of the Norwegian public road network are low volume roads. On these roads, ageing and weathering due to climatic exposures are among the most important factors influencing the deterioration of asphalt pavements.

The main intention in this part of the work was to get an overview of the most important pavement defect types occurring on the low volume road network. This as a basis for further research work on asphalt mix design and construction for more durable pavements on low volume roads.

The climate in Norway varies significantly from south to north and from coastal areas to inland areas. Several investigations from different parts of the country were studied to cover some typical climate zones in Norway.

Most of the investigations were carried out as a visual inspection when driving on the road at a slow speed, 20 – 30 km/h. Identification of the defects was based on different defect catalogues [26, 33 and 34].
Many of the observed defects did not fit neatly into any particular defect type. There could be various reasons for this, such as simplification of defect types and the existence of multiple defects. An example is a combination of cracking and rutting. The classification system here is based on a simplification to main or dominating factors.

2.2 Factors influencing the deterioration of asphalt pavements on low volume roads

Deterioration of asphalt pavements on low volume roads is affected of several factors. Each of these factors have different influence on the total deterioration of the asphalt pavement. Deterioration often results in visual defects in the pavement surface, such as cracks, deformations or other surface damages. In Figure 2.1, an overview of different factors affecting the deterioration is sketched.
As can be seen in Figure 2.1, a lot of factors could affect the deterioration of asphalt pavements on low-volume roads.

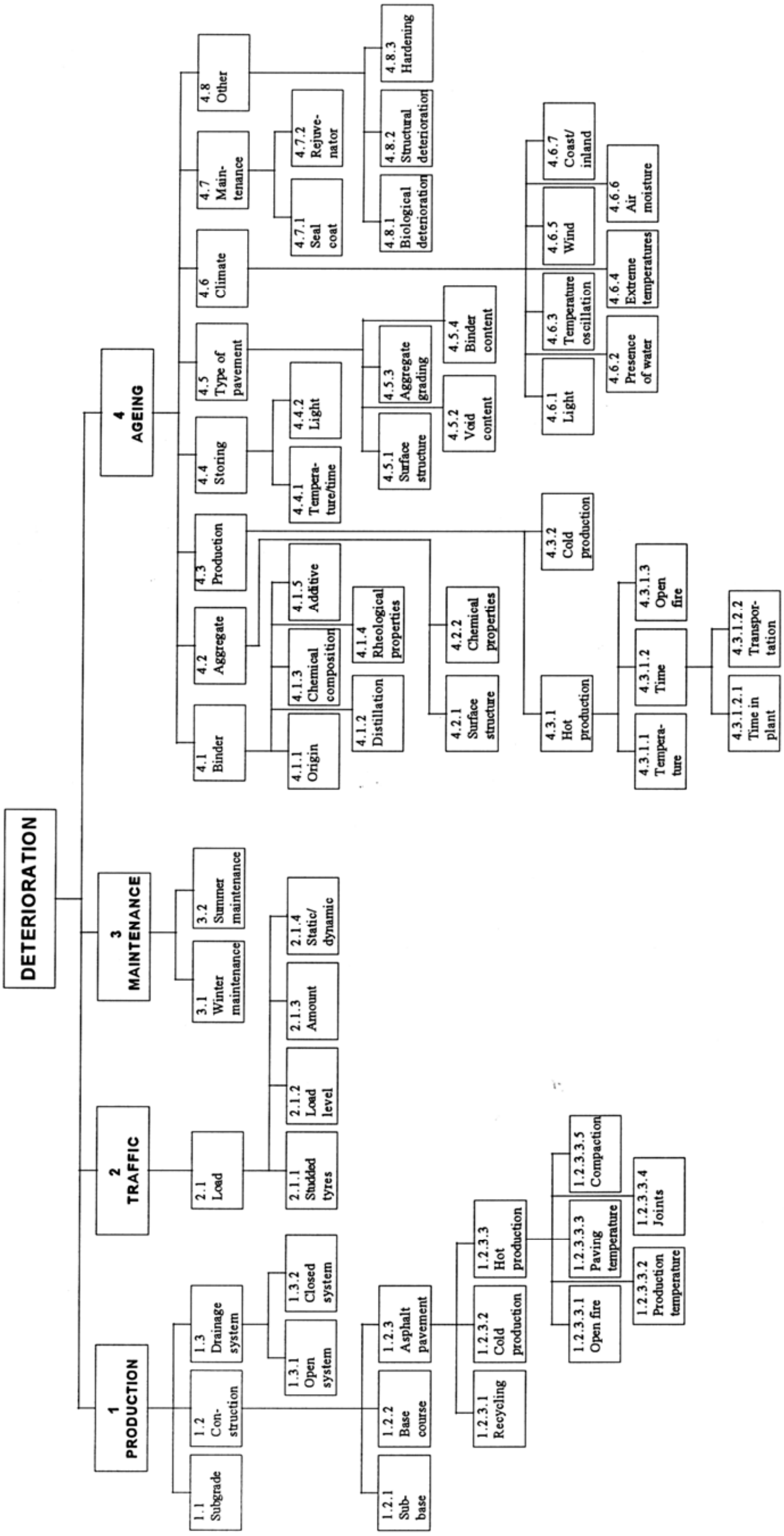


Figure 2.1 Factors influencing the deterioration of asphalt pavements on low-volume roads.

2.3 Defects in asphalt pavement surface

The different defects occurring on an asphalt pavement can be grouped into the following classes:

- Deformation
- Cracks
- Surface damages
- Potholes

Deformation
The term deformation in this connection is permanent deformation, being the difference between the theoretical profile and the real surface. Deformation may occur after construction due to traffic (load associated) or environmental (non-load associated) influences. In some cases, deformation may also occur in a new road construction due to inadequate compaction [26]. Deformation is an important element, which affects the pavement condition. Deformation may in some cases lead to cracking of the pavement. Deformation is often caused by:

- Traffic loading
- Local frost heave
- Local subsidence
- Edge defects

Cracks
Cracks in road pavement surfaces can occur in a wide variety of patterns, from isolated single cracks to network of cracks covering the entire pavement surface. The consequences of cracks in the pavement are many, including:

- Loss of waterproofing of the pavement layers.
- Pumping and loss of fines from the base course.
- Increasing pavement roughness.
- Loss of load-distributing ability.

The loss of load-distributing ability and waterproofing will usually lead to accelerated deterioration of the pavement condition. Factors, which lead to cracking, include:

- Local frost heave.
- Deformation, plastic and elastic.
- Extreme temperatures/fast temperature changes.
- Traffic.
- Ageing of the pavement – the surface is getting brittle.
- Reflection of cracks from underlying layers.
- Poor construction joints.
- Widening of existing road.
- Edge breaking.

An overview of different types of cracks is given in Figure 2.2.

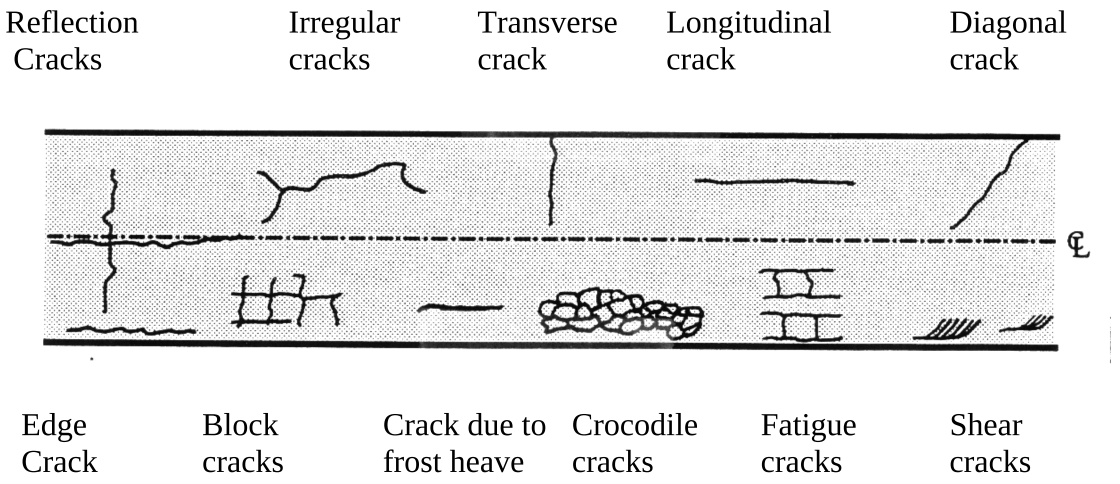


Figure 2.2 An overview of different types of cracking occurring on asphalt pavements [25].

In Figure 2.2 the terminology for different crack types are given according to the causes for the cracks or the appearance of the cracks. Where the cause for the crack is known, the cause is linked to the name, for example “crack due to frost heave”.

Surface damages
Surface damages are damages occurring on the pavement surface. A surface damages are not necessarily an indicator of bad structural condition. But surface damages could affect the driving comfort or the road safety.
Among surface damages are:

- Loss of surfacing materials.
- Flushing – the stone is immersed in the binder.
- Rough surface.
- Stripping – loss of stone from the surface.
- Damages caused by mechanical equipment.

Potholes
Potholes are bowl shaped holes in the pavement surface. Potholes have usually sharp edges, which in certain circumstances could cause damages to the vehicle and be a danger to the road safety. Growth of potholes is usually accelerated by the presence of water.

2.4 Field inspections

A field inspection was carried out in central Norway, which will be referred to as investigation number 1 [25]. Some other field inspections, as part of diploma work for some MSc (Master of Science) degrees, are also included. All inspections were visual inspections of the pavement surface condition, when driving a car at low speed. An overview of these studies is given in Table 2.1.

Table 2.1 Overview of visual field inspections on pavement surface conditions on low volume roads in Norway.

Reg. no [ref.]	**Inspection areas**	**Inspected length (km)**	**Comments**
1 [25]	Central-Norway	662,27 km	In this registration only longitudinal cracks were considered. The following types of cracks were of interest: • Cracks due to frost heave. • Edge cracks. • Cracks due to plastic deformation. • Fatigue cracks Roads were selected from both the coastal area and the inland area. The selected roads have an average annual daily traffic of less than 4000.
2 [83,84]	Central-Norway	Total length inspected was 328,97 km. (268,29 km governmental roads, 60,68km localroads)	In this work all defects on the road surface was registered. Requirements for the roads considered in the inspection were: • Average annual daily traffic less than 3000. • Existing pavement constructed later than 1985. • Roads from both the coastal area and the inland area. A plan for resurfacing was used to select roads for inspection.
3 [85]	Western-Norway	20,40 km	
4 [39]	Northern-Norway (Finnmark)	Total length inspection was 283,44 km. (261,13 km governmental roads , 22,3km localroads)	This work was carried out using a computer in the inspection. The system used is called NorDic. The road was divided into three zones, left, right and middle. Requirements for the roads considered in the inspection were: • Pavement older than 9 year (laid before 1984). • Roads from both the coastal area and the inland area. It was not possible to register more than one type of defect at the same time in a zone. Roads in the inspection were chosen from the Public Road data base (Vegdatabanken, VDB)
5 [86]	Northern-Norway (Nordland)	28,40 km	The procedure of the inspection was as following: 1) Selection of sections for detailed inspection was carried out using a car driving at low speed. This inspection was combined with other data for the road, such as bearing capacity and rutting 2) Detailed inspection was carried out by walking on the road. The Public Road Administration in Nordland wanted 6 roads to be evaluated due to rutting and low bearing capasity.

2.5 Results

The main objective of this work was to get an overview of the deteriorating factors on low volume roads in Norway. This was considered to be a prerequisite for further research on low volume roads. This investigation is based on the five different inspections mentioned in the previous chapter. There were two inspections in central Norway, two inspections from northern Norway and one from western Norway. Roads were chosen from both the coastal area and the inland area. The range of defects varies between inspections. The most frequent defects from each inspection are summarised in Table 2.2. Values in Table 2.2 are in percentage of the total inspection length. Defects occurring only at one single point are summarised in Table 2.3.

Table 2.2 Results from visual inspection of asphalt pavement surface conditions on low volume roads in Norway. The extent of defects is given in percentage of the total inspection length.

Type of defect	**Central Norway (Inv.no 1) *)**	**Central Norway (Inv.no 2)**	**Western Norway (Inv.no 3)**	**Northern Norway (Inv.no 4) **)**	**Northern Norway (Inv.no 5) ***)**
Crack due to frost heave	7.16	19.0	3.5		5.5
Edge cracks	6.10	26.2	10.3	4.0	29.0
Cracks due to plastic def.	1.13	2.4	4.3		3.9
Fatigue cracks	0.99				
Rutting		5.5	11.0		14.2
Crocodile cracks		4.6	10.4	0.4	2.0
Stripping		1.9	10.6		
Ageing		1.8	7.7	12.3	
Longitudinal cracks				1.9	
Flushing					8.4
Defect after patch		0.5	7.4	0.31	6.7
Total inspection length (km)	**662.27**	**328.97**	**20.4**	**283.42**	**28.4**

*) In this inspection only longitudinal cracks was in focus.

**) In this inspection the road was divided in three zones. The value in the table is the average value for all zones.

***) Only defects in the longitudinal direction are registered.

Table 2.3 Results from visual inspection of asphalt pavement surface conditions on low volume roads in Norway. Defects occurring at a single point, number pr km.

Type of defect	**Central Norway (Inv.no 1) *)**	**Central Norway (Inv.no 2)**	**Western Norway (Inv.no 3)**	**Northern Norway (Inv.no 4) **)**	**Northern Norway (Inv.no 5) ***)**
Transverse cracks	1)	1.8	2.4	17.3	1)
Pothole	1)	0.9	6.8	0.7	1)
Total inspection length (km)	**662.27**	**328.97**	**20.4**	**283.42**	**28.4**

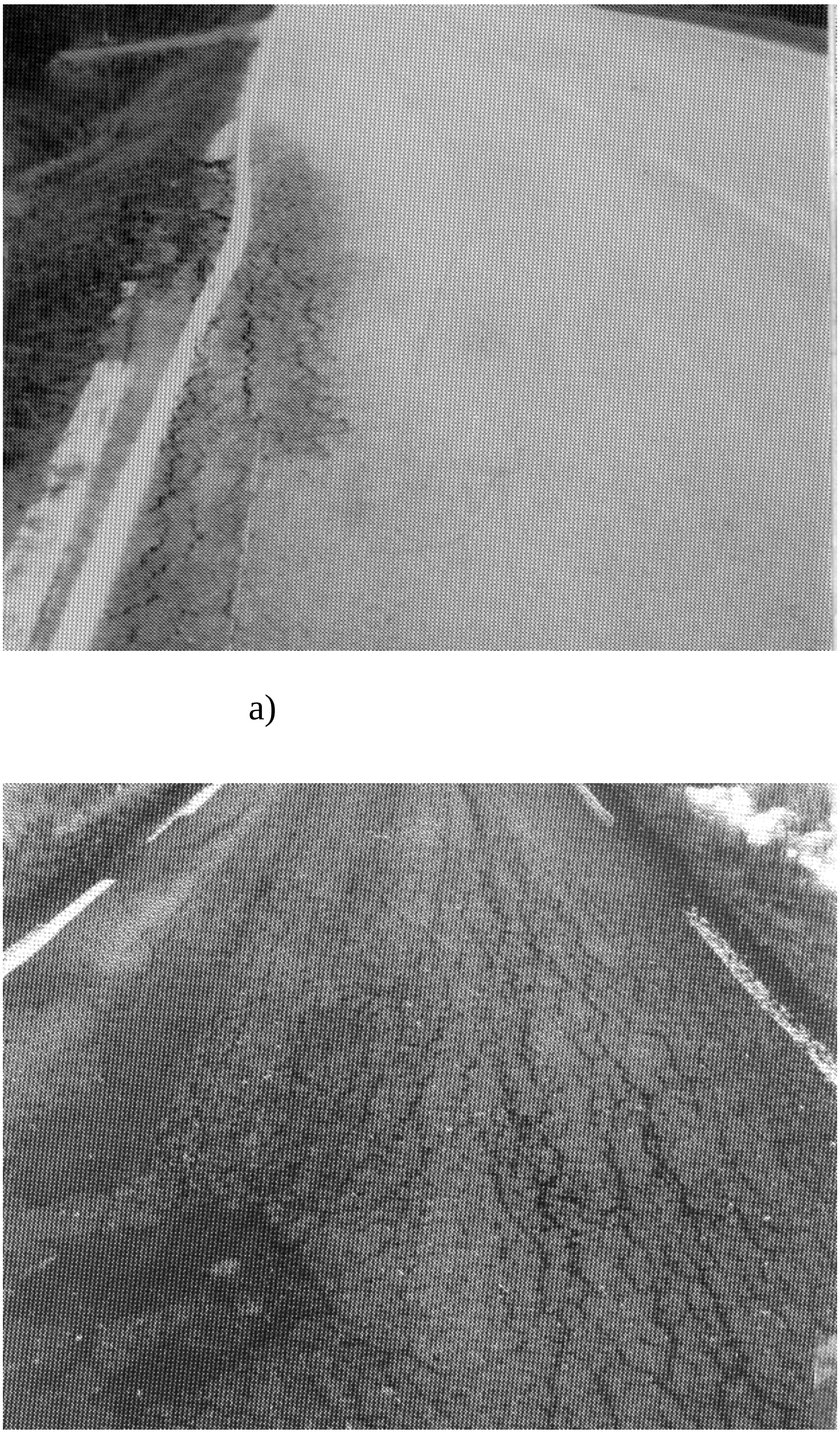
*) In this inspection only longitudinal cracks was in focus.
**) In this inspection the road was divided in three zones. The value in the table is the average value for all zones.
***) Only defects in longitudinal direction are registered.
1) Not selected as a parameter to be registered.

As Table 2.2 shows, edge cracks is the defect type occurring most frequently on low volume roads. Other dominant defects are cracks due to frost heave, rutting, ageing and crocodile cracks.
In Table 2.3 we can see that transverse cracks is a defect type occurring frequently on some roads. It is important to remark that transverse cracks was not a part of the inspection neither in the investigation no. 1 nor in the investigation no. 5.

Detailed results from each investigation are given in Appendix 2.

2.6 Discussion

It is often difficult to point at one distinct reason for a damage in a pavement. In many cases there will be interaction between several mechanisms causing deterioration or failure. Figure 2.3 a) shows a local defect, which was frequently encountered in coastal areas. Figure 2.4 shows the mechanism of this type of failure in principle.



a)

b)

Figure 2.3 Examples of defects on low- volume roads. a) “shear rupture” and b) cracks in the full road width [25].

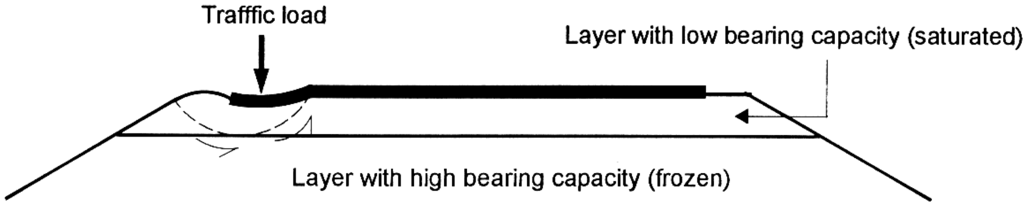


Figure 2.4 “Shear-rupture” [25].

Loading under critical conditions in the spring, when the upper layer due to thawing is saturated with water and the lower layer is frozen, could be a reason for the “shear rupture”.

In Figure 2.3 b) a pavement with cracks in the whole road width is shown. In this case the pavement is lying on a road with good drainage and almost no deformation. Ageing and brittleness combined with traffic loading could be the reason for the development of cracks in this case.

In inspection number 1, cracks due to frost heave and edge cracks are the dominating defects.
The frequency of defects in the second field investigation (inv. no. 2) is probably somewhat higher than for the low volume roads in general. This is due to the fact that the inspected roads were selected among roads which were planned to be resurfaced the same year or the next year. In this case too, cracks due to frost heave and edge cracks are the dominating defects.
In investigation number 3, rutting, stripping, crocodile cracks and edge cracks are the dominating defects.
From the results of investigation number 4, we can see that ageing and transverse cracks are dominating. There seems to be some difference in type of the defects between the coastal areas and inland areas. There is registered more ageing in the coastal regions, and roads in inland regions seem to have more edge crack and transverse cracks (Appendix 2).
In investigation number 5, edge cracking is the defect, which occurs most frequently.

From the investigations we can see that a considerable amount of low volume roads have defects. It is hard to observe if the defects in the coastal regions differ from the defects in inland regions or not. One investigation describes shear-rupture as a defect occurring more frequently in the coastal areas and ageing and brittleness occurring more frequently in the inland areas. Another investigation found ageing as a typical coastal problem, and edge cracks as a typical problem in inland areas.

The inspections were based on catalogues describing defects occurring on asphalt pavements. In these catalogues, the names of the defects are given with a short description and often with a picture. Possible causes for defects are also given.

Inspection of roads while driving a car at a speed of 20 –30 km/h makes it difficult to register all defects. One problem is to decide what type of defect is occurring in visual inspection at high speed. Another problem is to observe all the defects on the pavement. There are often several defects occurring at the same time.
A third problem is that inspections, which follow this procedure, can not be very objective.

In the following a description and possible causes for the most frequent defects occurring on the low volume roads is given:

Edge cracks

Description
Longitudinal cracks running parallel to the shoulder. The cracks are often relatively wide and deep. Sometimes whole flakes are loose [25].

Possible causes [26]
- Inadequate pavement width and edge support.
- Alignment which encourages drivers to travel on pavement edge.
- Edge drop-off.
- Weak seal coat, loss of adhesion to base.

Cracks due to frost heave

Description
These cracks depend on the road, the subground, the existence of water and the climate. These cracks are usually longitudinal cracks running in the middle of the road or at the edge, depending on the road width. The cracks are usually relatively wide and deep. On roads with width of 5-6 meter, cracks usually occur at the edge (Figure 2.5 a). On roads with width of 7-9 meter, one wide and deep crack usually occurs in the middle (Figure 2.5 b). On roads with width of 12-13 meter, cracks usually occur at some distance from the edge (Figure 2.5 c).

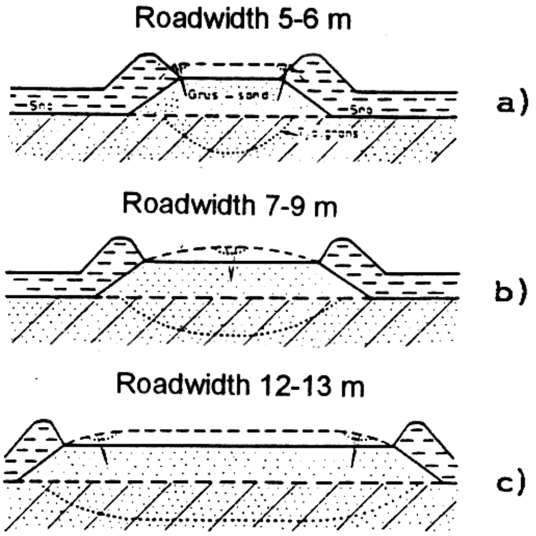


Figure 2.5 Cracks due to frost heave [34].

Possible causes
- Differential frost heave across the pavement.
- Development of cracks could be influenced by snow on the roadside.

Rutting [26]

Description

Longitudinal deformation in a wheel path. May occur in one or both wheel paths of a lane.

Possible causes

- Wear due to studded tires (problem on high volume roads).
- Inadequate pavement thickness.
- Inadequate compaction in surfacing or base – leading to consolidation (Figure 2.6)
- Inadequate strength (stability) in surfacing or base.

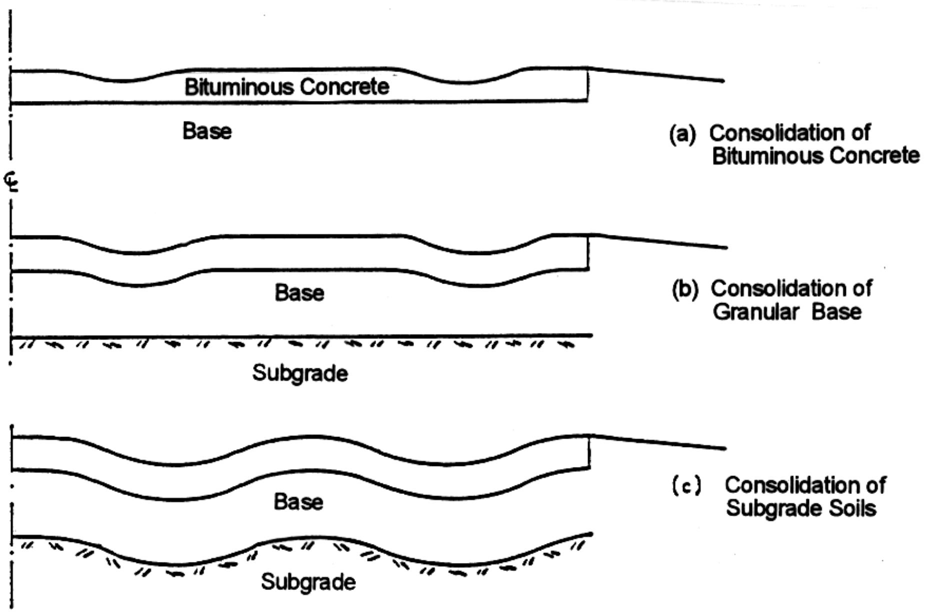


Figure 2.6 Rutting due to consolidation [87].

Ageing

Description

Ageing is not a defined defect in a pavement. Ageing of a pavement will lead to several defects, which include stripping, ravelling and cracking.
Due to ageing the binder and the asphalt-mixture becomes stiffer and more brittle.
Ageing is defined as change in the binder properties over time [88].

Possible causes

There are several factors affecting ageing. The most important factors are [88]:

- Oxidation – the polar oxygen containing groups formed, tend to associate into micelles of higher molecular weight and thereby increase the viscosity of the bitumen [54].
- Loss of volatiles – evaporation of volatile components depends mainly upon temperature and the conditions of exposure [54].

- Physical hardening – occurs when the bitumen is at ambient temperature and is usually attributed to reorientation of bitumen molecules and the slow crystallisation of waxes [54].
- Exudative hardening – results from the movement of an oily component which exudes from the bitumen into the mineral aggregates [54].

Crocodile cracks

Description
Interconnected or interlaced cracks forming a series of small polygons resembling a crocodile hide. It is usually associated with wheel paths and may have a noticeable longitudinal grain. Cell sizes are generally less than 150 mm across but may extend up to 300 mm [26].

Possible causes
- Inadequate pavement thickness.
- Low modulus base.
- Brittle base or wearing course (e.g. cemented, aged).

Transverse cracks

Description
Unconnected crack running transversely across the pavement.

Possible causes
- Reflection of a shrinkage crack or joint in an underlying base.
- Construction joint.
- Shrinkage crack, due to low temperature or bitumen hardening, in asphalt surfacing (most important on low volume roads).

2.7 Conclusions

As a result of these investigations, some defects typically occurring on low volume roads can be classified. Among the most frequent defects are:

- Edge cracks
- Cracks due to frost heave.
- Rutting
- Transverse cracks
- Ageing

The extent of some of the defects is given in Table 2.4. The results are based on the 5 investigations summarised in this study. The values are in percentage of the total inspected road length. For example, edge cracks occurs on 11 % of the low volume roads network.

Table 2.4 Extent of defects on the low volume roads.

Type of defect	**Percent of the low volume road network with the defect**
Edge crakcs	11
Cracks due to frost heave	11
Rutting	6
Ageing	7

The main reason for ***edge cracks*** is in many cases inadequate edge support, because of paving very close to the shoulder edge. This is due to the fact that low volume roads are narrow. When resurfacing, the pavement is often laid to the edge of the shoulder to utilise the total road width.

On low volume roads, ***cracks due to frost heave*** occurs more frequently than on the high volume roads because of poor material quality and poor drainage systems.
To minimise the construction costs, the materials used in low-volume roads are often taken from gravel deposits close to the road, even if the material quality is poor.

Rutting could occur on low volume roads due to poor bearing capacity, poor material quality and unsufficient compaction during construction (see Figure 2.6). Another reason could be poor drainage system, or inadequate edge support. Wear due to studded tyres is probably not a problem on low volume roads because the effects mentioned before will be much more dominant.

Transverse cracks can occur on low volume roads due to low temperature and/or bitumen hardening.

Ageing could be a problem on asphalt surfaces mainly where mechanical wear from studded tires is not a problem. During ageing the binder and asphalt-mixture become stiffer and brittle. There is no defect directly linked to ageing, but several pavement properties are influenced by the ageing process. When the binder is aged, it becomes more susceptible to development of cracks. It will be more sensitive to cracks due to rutting, low temperature, deformation and frost heave. On the aged material, some other deterioration mechanisms will also occur more frequently, such as stripping and ravelling.

The service life of an asphalt pavement on low volume roads is highly dependent on the ageing properties of the binder and the asphalt-mixture. It is important to keep asphalt pavements ductile to withstand cracking due to low temperatures. On the other hand certain stiffness is required to avoid permanent deformation under traffic loads. The challenge is to find a proper binder and the right asphalt-mixture according to local conditions, such as temperature and bearing capacity.

CHAPTER 3 *A FIELD INVESTIGATION OF MATERIAL PROPERTIES RELATED TO AGEING*

3.1 Background

Ageing due to climatic exposure is among the most important factors influencing the degradation of asphalt surfaces on low volume roads. The ageing properties of asphalt mixtures are as shown earlier, related to material factors such as binder stiffness, binder content, aggregate mineralogy and void content and also to production related factors and climate. In asphalt pavements on low volume roads, generally softer binders are used than on high volume roads. These pavements are also produced at a lower temperature. This means that the ageing during mixing and production should be less than for high traffic mixtures. On the other hand, paving on low volume roads often requires long transportation due to often long distances to the asphalt plant. The asphalt mixture is typically transported several hours, and even days with freighter (boat). This way, ageing during production on low volume roads frequently reaches a high level.

In this field investigation, asphalt cores are sampled at 24 different places. 9 of the samples are from a typical coast area and 15 from a typical inland area. The investigation is concentrated on low volume roads in central Norway.

Binders aged in situ and corresponding laboratory aged binders are tested by traditionally tests and test methods recommended from the SHRP-project.
There are limited experiences on the use of SHRP-binder test equipment on soft binders for Norwegian conditions.

3.2 Objective

The objective of this investigation has been to examine to what extent ageing is occurring on asphalt pavements on low-volume roads. The primary goals of this study are to:

- Evaluate the mechanical properties of asphalt mixes exposed to field conditions.
- Examine if there are significant differences in the ageing development and hence mechanical properties of pavements at different climatic exposure, inland versus coast.
- Characterise the rheology of binders recovered from field samples.
- Examine the development of ageing with time in pavements with soft binders.
- Characterise the rheology of original and artificially aged bitumen using SHRP-binder test equipment and conventional tests.
- Examine the rheology of recovered bitumen compared to the original and laboratory aged. Specially examine whether the SHRP binder test equipment is useful for soft binders under Norwegian conditions or if the testing procedures have to be modified.

3.3 Procedure

3.3.1 Selection of samples

Asphalt pavements from a typical inland climate area and a typical coastal area are selected in this study. Inland climate typically has extreme low temperatures at winter and could also have high temperatures at summer. The coastal climate does not have the extreme temperatures that occur at typical inland climate. A typical coastal area has more precipitation, mainly as rainfall. The map in Figure 3.1 shows the locations of the areas where the samples are taken.

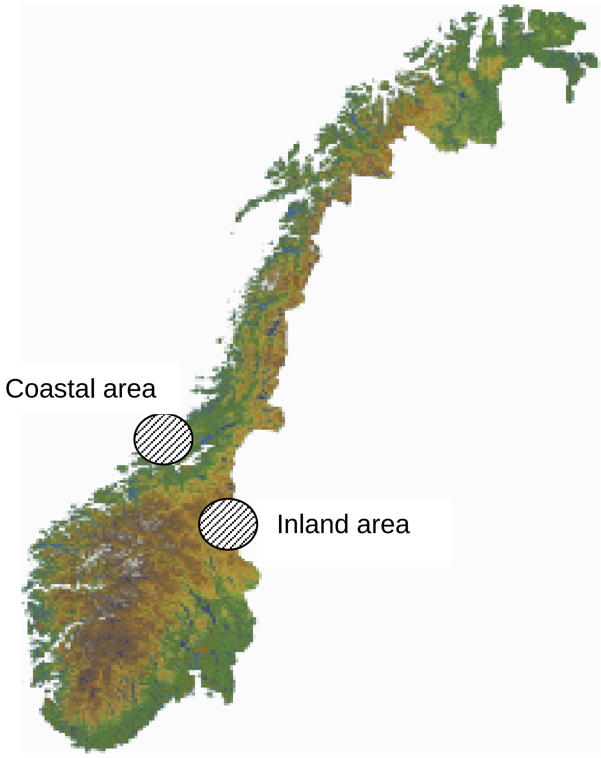


Figure 3.1 Areas for field investigation of asphalt-mixtures.

Maps over the highest average 7-days summer temperatures and minimum winter temperatures for Norway are given in Appendix 3. In this particular inland area, air temperatures below – 46^{o}C can occur in the winter and temperatures above 32^{o}C may occur in the summer. For the coastal area the corresponding temperatures are -25^{o}C and 26^{o}C respectively.

The same type of asphalt mixtures but with different age have been selected to get an overview of the ageing process in situ and also to compare coastal and inland climate effects. Pavements with age 1-2, 3-5, and 7-14 years have been selected. The Road Pavement Data Bank has been used to make a selection of road sections. In addition, data from the mixture formula and the quality control is achieved from the Public Roads Administration. The available design data is: aggregate grading, origin of the aggregates, aggregate density, binder content, penetration value (for Pen180) and viscosity at 60°C (for soft binders). An overview of all field samples is given in Table 3.1.

Table 3.1 Overview of field samples based on mixture formulation.

Specimen nr.	**Coast/ Inland**	**Asphalt mix**	**Binder**	**Age (year)**	**Planned binder content (%)**	**Pen. 25 C °**	**Viscosity mm^2/s (60°C)**	**Planned void content (%)**
1	Coast	Ma16	MB10.000	4	4,8		10774	5,5
3	Inland	Eg11	BE70S (MB1500)	1	2,7			
7	Inland	Og16	VO550	11	3,4		748	
8	Coast	Og16	VO550	11	4,4 (3,5)		530	
9	Coast	Og16	VO550	12	3,5		546	
11	Inland	Ma16	MB10.000	3	4,3		10006	
12	Inland	Ma14	MB10.000	1	4,5		11067	5,5
13	Inland	Ma16	MB8.000	7	4,7		8517	<8
14	Inland	Ma16	MB10.000	3	4,3		10871	
15	Inland	Agb14	Pen180	1	5,5	180		4,5
16	Coast	Ma16	MB10.000	4	4,8		10774	5,5
17	Coast	Ma16	MB 8.000	8	4,5		8779	<8
18	Coast	Ma11	MB10.000	1	4,8		9833	5,5
19	Coast	Ma16	MB6.000	9	4,5		5974	
20	Coast	Ma11	MB10.000	1	4,8		9833	5,5
21	Coast	Ma16	MB6.000	9	4,5		5974	
22	Inland	Agb16	Pen180	11	5,5	177		4,0
26	Inland	Agb16	Pen180	3	5,3	183		4,5
27	Inland	Agb16	Pen180	3	5,3	183		4,5
28	Inland	Agb14	Pen180	1	5,5	180		4,5
29	Inland	Ag14	Pen180	1	4,5	186		6,0
30	Inland	Ma16	MB6.000	14			6940	
31	Inland	Ma16	MB6.000	14			6940	
32	Inland	Ma16	MB8.000	7	4,7		8517	<8

Core specimens from all the selected pavements have been obtained. 15 cores have been sampled at each location. It was necessary to take 15 cores at each location to be able to carry out all the tests planned for the investigation (these pavements are rather thin).

A visual registration of the pavement surface condition was carried out driving at a speed of 40 km/h.

3.3.2 Original and laboratory aged bitumen

The bitumen used for the investigation was delivered from Nynäs, Sweden. The bitumen is produced from Laguna crude oil. Binders used in this investigation are Pen180 (penetration grade 180), MB10.000 (viscosity at 60°C), MB8000 and MB6000, which are commonly used binders on low volume roads in Norway. The selected binders were of the same grades as the binders used in the field samples.

3.4 Experimental work

3.4.1 Asphalt-mix analysis

Mechanical properties have been measured using the indirect tensile test. Maximum load and deformation has been measured during the test. Tensile strength, E-modulus and load distribution coefficients (a) have been calculated using the following formulas:

Tensile strength: $S_t = (61{,}47 * P_{max})/t$ *(3-1)*

P_{max} = Maximum load in kPa.
t = specimen height in mm

E-modulus: $E = 6{,}1 * S_t + 100$ (MPa) *(3-2)*

S_t = tensile strength in kPa

Load distribution coeff.: $a = 0{,}38 * (E)^{1/3}$ *(3-3)*

A load distribution coefficient, a, is used in the Norwegian Road Design System as a material parameter for structural strength. In addition to tensile strength, the following parameters have been measured in these analyses:

- *Specimen height*
- *Aggregate density*
- *Binder content*
- *Bulk density of asphalt mix samples*
- *Maximum density of the asphalt mix (voidfree density)*
- *Void content*
- *Aggregate grading*

3.4.2 Analysis of recovered bitumen

Before laboratory testing, the specimens were prepared by cutting near the underlying layer. Ten cores were also divided in slices of 10 mm, as shown in Figure 3.2.

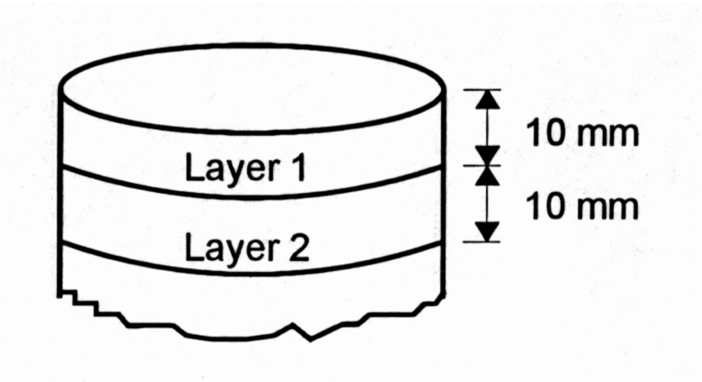


Figure 3.2 Preparation of samples for binder analysis.

The specimens have been cut as shown in Figure 3.2, to allow analysis on recovered binder at different levels in the pavement. The objective of this is to study differences in the binder properties at different levels. It was not possible to get 2 slices from all samples.

The binder from the core samples was extracted using metylenechloride as solvent and recovered using a rotavapor unit for evaporation of the solvent. The binder properties of the recovered binders were then examined. The rheological properties of the binder are important to evaluate the binder durability. Test methods developed in the Strategic Highway Research Program (SHRP) in USA were used for the binder examination. These test methods are specially developed to examine rheological properties. In addition, conventional tests were used.
The tests and the test conditions are given in Table 3.2. A detailed description of the tests is given in chapter 1.6.6.

Table 3.2 Tests and test conditions for recovered binder.

Conventional tests	**SHRP binder specification tests**
Penetration: Test temperature was 25°C for Pen180 and 10°C for soft binders (MB-graded)	*DSR:* DSR have been carried out at temperatures 10, 16, 22, 28 and 34°C. The frequency at all measurements was 10 rad/s.
The softening point test	*BBR:* The BBR-test was carried out at -20°C and -30°C for Pen 180 and -30°C and -35°C/-36°C for softer bitumens (MB-graded). The temperature where S=300 MPa and m=0,300 is determined (values required in the SHRP performance Graded Asphalt Binder Specifications). The BBR test was performed on layer 1.
Viscosity using a Brookfield rotational viscometer: The testing temperatures were 60°C and 90°C.	

The test program for recovered binders is shown in Table 3.3.

Table 3.3 Program for testing of recovered binder from field samples.

Sample nr	**Binder**	**Age, years**	**Coast/ inland**	**Pen 10°C/ 25°C**	**Ring and ball**	**Viscosity 60°/90°**	**Dynamic Shear Rheometer 10-34°C**	**Bending Beam Rheometer –20,-30,-35**
22	Pen180	11	Inland	25	X	*)90°	X	-20°C/-30°C
26	Pen180	3	Inland	25	X	*)90°	X	-20°C/-30°C
27	Pen180	3	Inland	25	X	*)90°	X	-20°C/-30°C
15	Pen180	1	Inland	25	X	*)90°	X	-20°C/-30°C
28	Pen180	1	Inland	25	X	*)90°	X	-20°C/-30°C
29	Pen180	1	Inland	25	X	*)90°	X	-20°C/-30°C
11	MB10000	3	Inland	10	X	*)60°/90°	X	-30°C/-35°C
14	MB10000	3	Inland	10	X	*)60°/90°	X	-30°C/-35°C
12	MB10000	1	Inland	10	X	*)60°/90°	X	-30°C/-35°C
1	MB10000	4	Coast	10	X	*)60°/90°	X	-30°C/-35°C
16	MB10000	4	Coast	10	X	*)60°/90°	X	-30°C/-35°C
18	MB10000	1	Coast	10	X	*)60°/90°	X	-30°C/-35°C
20	MB10000	1	Coast	10	X	*)60°/90°	X	-30°C/-35°C
13	MB8000	7	Inland	10	X	*)60°/90°	X	-30°C/-35°C
32	MB8000	7	Inland	10	X	*)60°/90°	X	-30°C/-35°C
17	MB8000	8	Coast	10	X	*)60°/90°	X	-30°C/-35°C
30	MB6000	14	Inland	10	X	*)60°/90°	X	-30°C/-35°C
31	MB6000	14	Inland	10	X	*)60°/90°	X	-30°C/-35°C
19	MB6000	9	Coast	10	X	*)60°/90°	X	-30°C/-35°C
21	MB6000	9	Coast	10	X	*)60°/90°	X	-30°C/-35°C
**)3	MB1500	1	Inland	10	X	*)60°	X	-30°C/-35°C
**)7	VO550	11	Inland	10	X	*)60°/90°	X	-30°C/-35°C
**)9	VO550	12	Coast	10	X	*)60°/90°	X	-30°C/-35°C
**)8	VO550	11	Coast	10	X	*)60°/90°	X	-30°C/-35°C

*) Measured with Brookfield thermosel apparatus.
**) Tests are carried out on recovered, but not destillated binder.

3.4.3 Analysis of original and laboratory aged bitumen

The test program for original and laboratory aged bitumen is identical to the tests on recovered bitumen. In addition, artificially ageing is obtained in the laboratory by the Thin Film Oven Test (TFOT) and the Pressure Ageing Vessel (PAV) device.

Thin Film Oven Test
This is a binder ageing test method simulating the ageing taking place during hot mixing and production of the asphalt mixture. The test samples are held at 163°C for 5 hours for Pen180 and at 120°C for soft bitumen (MB).

Pressure Ageing Vessel
The PAV simulates the ageing occurring in service. The PAV procedure is conducted by placing the bitumen in a heated vessel pressurised with air, for 20 hours. The temperature of the test is varied, depending on the climate in which the bitumen will be used. In this case 90°C (MB6000 and MB8000) and 100°C (MB10000 and Pen 180) were chosen.

The test program for original and laboratory aged bitumen is given in Table 3.4.

Table 3.4 Test program for original and laboratory aged bitumen.

Sample no	Binder	TFOT	PAV °C	Pen.	Ring and ball.	Dynamic Shear Rheomet.	Bending Beam Rheomet.	Rot. Viscomet.
A1	Pen180			25°C	X	10-34°C		60,90°C
A2	Pen180	163°C		25°C	X	10-34°C		60,90°C
A3	Pen180	163°C	100	25°C	X	10-34°C	-20,-30°C	60,90°C
B1	MB10000			10°C	X	10-34°C		60,90°C
B2	MB10000	120°C		10°C	X	10-34°C		60,90°C
B3	MB10000	120°C	100	10°C	X	10-34°C	-20,-30, -36°C	60,90°C
C1	MB8000			10°C	X	10-34°C		60,90°C
C2	MB8000	120°C		10°C	X	10-34°C		60,90°C
C3	MB8000	120°C	90	10°C	X	10-34°C	-20,-30, -36°C	60,90°C
F1	MB6000			10°C	X	10-34°C		60,90°C
F2	MB6000	120°C		10°C	X	10-34°C		60,90°C
F3	MB6000	120°C	90	10°C	X	10-34°C	-20,-30, -36°C	60,90°C

3.5 Results

In this chapter the main findings are presented. Documentation of more details is given in Appendices 4-6.

3.5.1 Field registration

A visual registration of the pavement surface condition was carried out from a vehicle driving at a speed of 40 km/h. This was done at all locations where core samples were taken. The results of the visual registration are given in Table 3.5.

Table 3.5 Results from visual registration of the pavement surface condition.

Specimen no.	Coast/ Inland	Asphalt mixture	Binder	Age, Year	Visual registration of pavement surface condition
1	Coast	Ma16	MB10.000	4	The surface is grey. Deformation in wheel tracks. Some longitudinal corrugation. Edge cracks occurring frequently.
3	Inland	Eg11	BE70S (MB1500)	1	Rough surface with stripping and potholes.
7	Inland	Og16	VO550	11	Rough surface with stripping. Edge cracks occurring frequently.
8	Coast	Og16	VO550	11	Smooth surface, some places stripping is occurring. Innuendo of deformation in wheel tracks.
9	Coast	Og16	VO550	12	Grey surface. Some places cracks are occurring in wheel tracks. Almost no deformation.
11	Inland	Ma16	MB10.000	3	Smooth surface. Some edge cracks and transverse cracks are occurring.
12	Inland	Ma14	MB10.000	1	Smooth surface.
13	Inland	Ma16	MB8.000	7	Smooth surface. Frequently cracks in outer wheel track and edge cracks. Almost no deformation in wheel track.
14	Inland	Ma16	MB10.000	3	Smooth surface. A few edge cracks.
15	Inland	Agb14	Pen180	1	Smooth surface. No deformation in wheel tracks.
16	Coast	Ma16	MB10.000	4	Smooth surface. Some places deformation is occurring in wheel tracks.
17	Coast	Ma16	MB 8.000	8	Smooth grey surface. Almost no deformation.
18	Coast	Ma11	MB10.000	1	Smooth surface.
19	Coast	Ma16	MB6.000	9	Smooth surface. Some places edge cracking is occurring.
20	Coast	Ma11	MB10.000	1	Smooth surface.
21	Coast	Ma16	MB6.000	9	Edge cracking and cracks due to frost heave are occurring.
22	Inland	Agb16	Pen180	11	Grey surface with little binder in the top surface. Innuendo of deformation in wheel tracks. Some transverse cracks are occurring.
26	Inland	Agb16	Pen180	3	Smooth surface. Almost no deformation in wheel tracks. Transverse cracks occurring frequently.
27	Inland	Agb16	B180	3	Smooth surface. Almost no deformation in wheel tracks. Transverse cracks occurring frequently.
28	Inland	Agb14	Pen180	1	Smooth surface
29	Inland	Ag14	Pen180	1	Smooth surface. Some edge cracks are occurring.
30	Inland	Ma16	MB6.000	14	Grey surface with frequently transverse cracks.
31	Inland	Ma16	MB6.000	14	Grey surface. Almost no deformation in wheel tracks, but some deformations are occurring. Transverse cracks are occurring frequently.
32	Inland	Ma16	MB8.000	7	Smooth surface. Deformation in wheel tracks. Cracks due to frost heave. Some transverse cracks are occurring.

3.5.2 Asphalt-mixture analysis

Detailed results from the analysis of asphalt mixtures are given in Appendix 4. In Table 3.6, the average values for the samples are given for a selection of parameters.

Table 3.6 Average value for asphalt mixture parameters.

Sample No.	**Binder**	**Binder content (%)**	**Void content (%)**	**Voids Filled With Binder (%)**	**Indirect Tensile Strength Measurements**					**Standard value (a)**
					Max. Load (kN)	**Defor-mation (mm)**	**Tensile strength (kPa)**	**E-modulus (Mpa)**	**Load distribution coeff. (a)**	
1	MB 10000	5,24	10,3	53,8	0,99	2,33	221	1446	2,30	1,5
3	BE70S	3,18	12,7	35,6	0,12	1,60	24	248	1,10	1,25
7	VO550	3,43	9,8	43,8	0,91	1,62	150	1017	2,02	1,25
8	VO550	3,87	11,5	42,7	1,17	1,52	206	1355	2,24	1,25
9	VO550	3,06	7,2	49,8	0,43	1,52	72	539	1,58	1,25
11	MB 10000	5,01	6,1	67,0	0,55	1,96	199	1316	2,22	1,5
12	MB 10000	5,58	4,5	75,6	1,29	2,77	334	2137	2,64	1,5
13	MB 8000	4,01	8,8	50,9	1,09	1,60	224	1464	2,31	1,5
14	MB 10000	5,22	5,6	70,0	1,50	2,95	378	2408	2,75	1,5
15	Pen 180	4,45	9,5	53,3	0,53	1,91	139	946	1,97	3
16	MB 10000	4,94	9,8	53,5	1,05	2,33	264	1712	2,44	1,5
17	MB 8000	5,28	3,0	80,8	0,72	1,95	171	1141	2,11	1,5
18	MB 10000	4,92	9,3	54,6	1,03	1,99	238	1554	2,36	1,5
19	MB 6000	4,86	5,7	66,5	1,02	2,15	159	1068	2,06	1,5
20	MB 10000	4,77	8,3	56,8	1,02	1,82	185	1231	2,17	1,5
21	MB 6000	4,52	9,3	52,4	0,56	1,72	106	747	1,80	1,5
22	Pen180	5,46	3,8	77,3	2,36	3,07	323	2070	2,61	3
26	Pen180	5,81	4,2	77,3	2,00	3,58	300	1932	2,54	3
27	Pen180	5,67	4,8	74,9	1,82	3,52	305	1958	2,56	3
28	Pen180	5,71	1,8	88,9	1,74	3,29	312	2001	2,58	3
29	Pen180	4,26	6,9	60,1	2,68	2,82	581	3644	3,17	3
30	MB 6000	4,25	5,0	66,2	0,85	2,07	153	1031	2,03	1,5
31	MB 6000	4,58	7,4	58,6	1,18	2,39	258	1676	2,42	1,5
32	MB 8000	5,16	6,8	63,3	0,77	2,01	160	1074	2,06	1,5

Important factors for the long-term pavement performance are the binder content and the void content. Some investigations carried out on ageing have shown that void content above 4-5 % could dramatically increase the rate of hardening of the bitumen. The hardening of bitumen is also dependent on the film thickness. The actual binder content in the samples is compared to the binder content in the mixture-formulations in Figure 3.3.

The measured binder content in the samples is higher than in the mixture-formulations for 14 samples, and lower for 7 samples, see Figure 3.3. For 3 samples there was no specified binder content in the mixture-formulation.

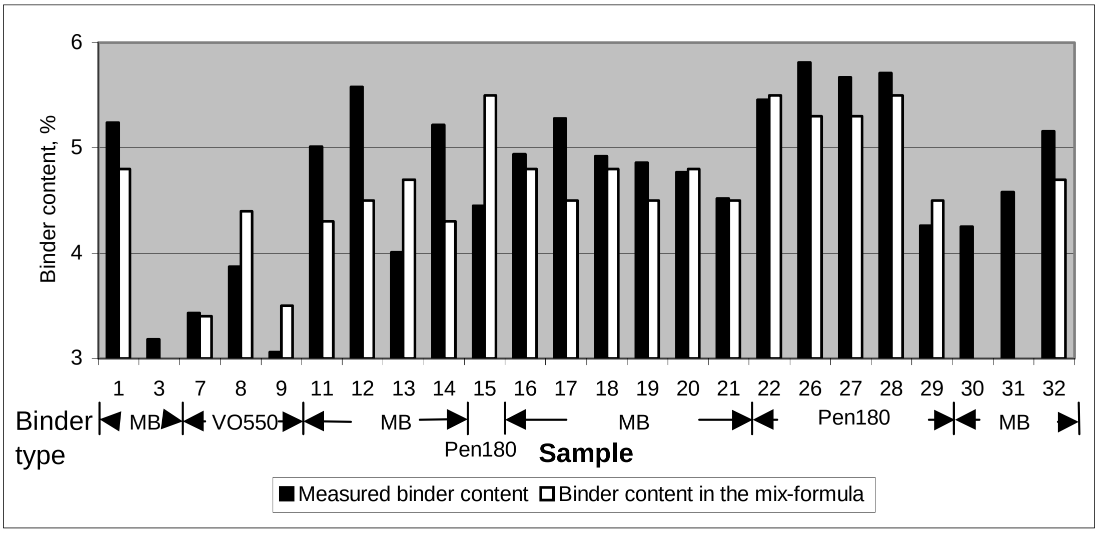


Figure 3.3 *Measured binder content in samples compared to the binder content in the mix-formula.*

The void contents of the analysed samples are compared to specified void contents in Figure 3.4.
From Figure 3.4 we observe that the measured void contents are higher than in the mix-formula for 8 samples, and lower for 6 samples. For 9 samples no specified void content in the mix-formula was available.

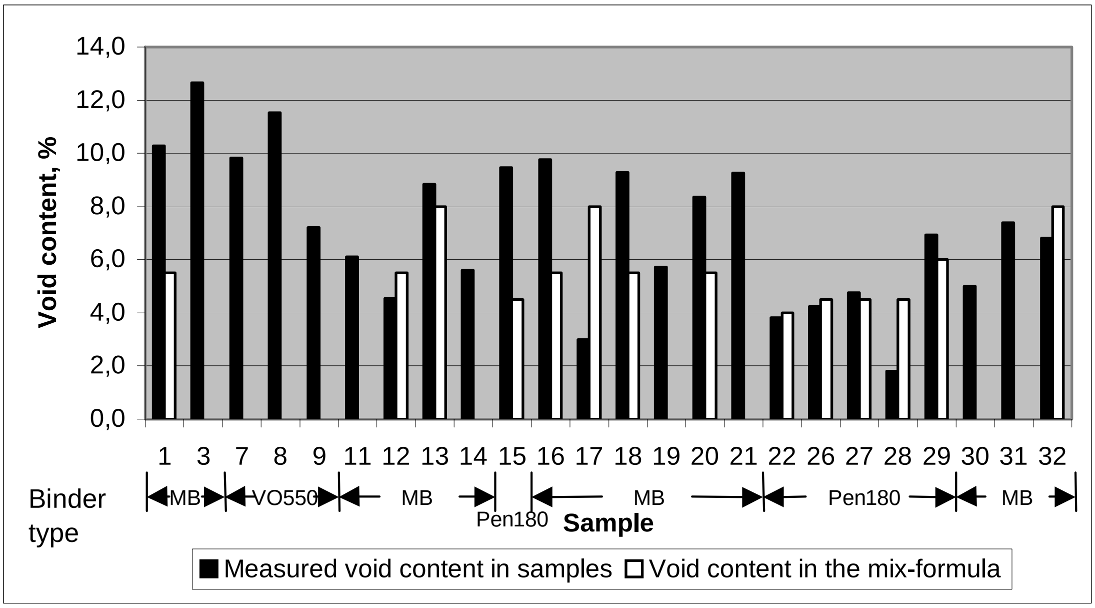


Figure 3.4 *Measured void content in samples compared to the void content in the mix-formula.*

In Figure 3.5 the calculated load distribution factors, a-factors, for samples and corresponding standard a-values used in Norwegian Road Design System are shown.

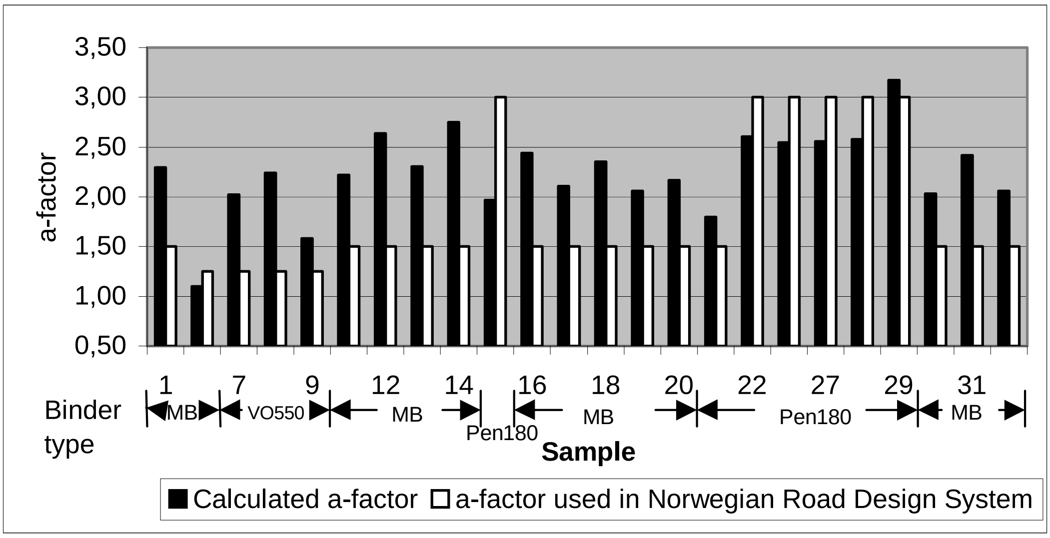


Figure 3.5 A comparison between calculated a-factor (load distribution factor) for samples and a-factor used in the Norwegian Road Design System.

Figure 3.5 shows that six samples have lower a-factor than the standard and eighteen have higher a-factors. Five of the six samples with a-factors lower than the a-factor in the Norwegian Road Design System, are asphalt-mixes with Pen180 binders. Higher a-factors than standard indicates better bearing capacity than predicted by the road design system. Lower a-factors indicate less bearing capacity.

In Figures 3.6 – 3.13, the coherence between age and location (inland/coast) is shown for E-modulus and load distribution coefficient for different asphalt-mixes. Detailed results are given in Appendix 4. For the Ma-mixtures (asphalt mixes containing MB-graded binders) there is a tendency to larger E-modulus in the inland area compared to the coastal area. The load distribution factor, a-factor, is higher than the a-factor used in the Norwegian road design system for mixtures containing soft binders (MB-graded and VO). For Agb-mixtures (asphalt mixtures containing Pen 180) the calculated a-factors are lower than the standard a-factor.

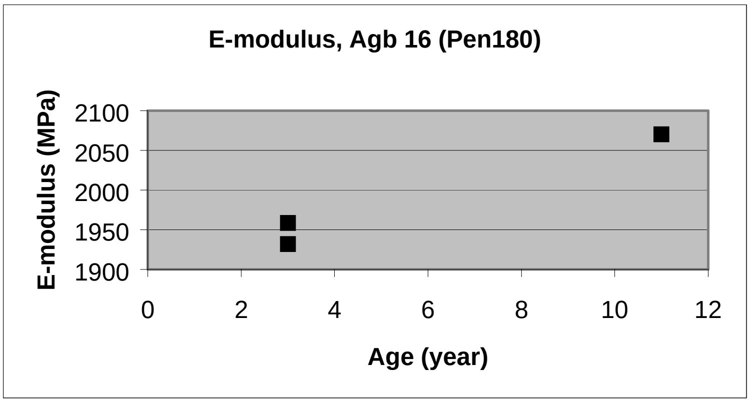


Figure 3.6 E-modulus, Agb 16 from the inland area with Pen 180 as binder.

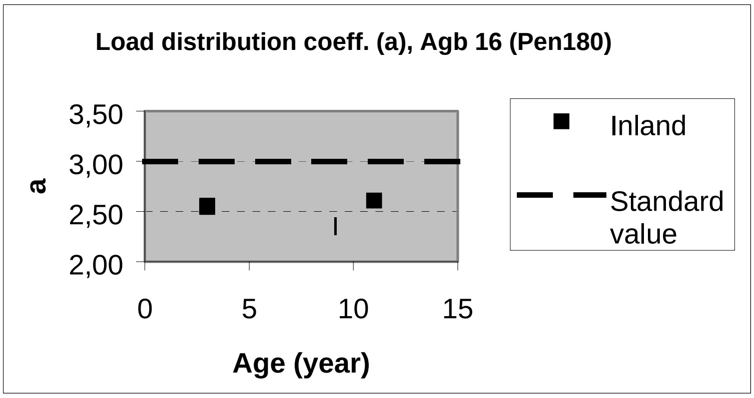


Figure 3.7 Load distribution coefficient, Agb 16 with Pen 180 as binder.

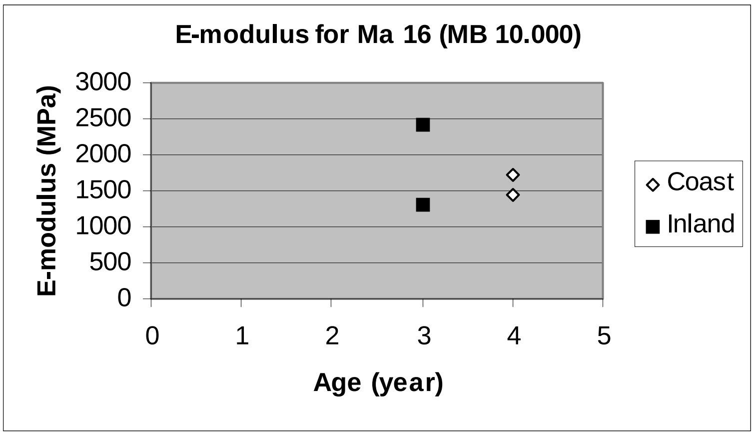


Figure 3.8 E-modulus, Ma16 with MB10.000 as binder.

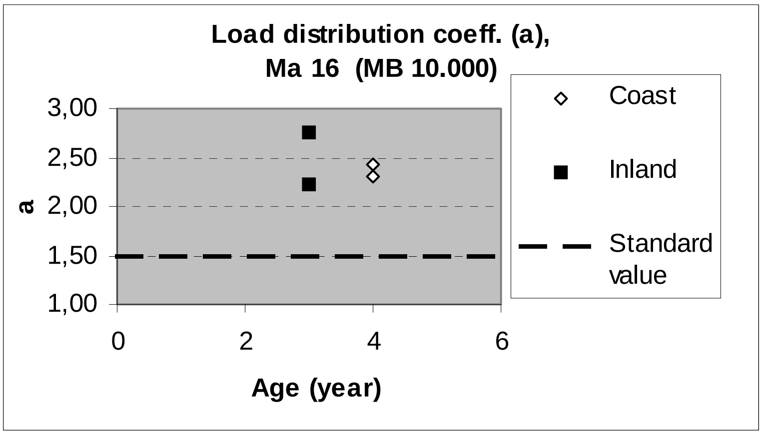


Figure 3.9 Load distribution coefficient, Ma16 with MB10000 as binder.

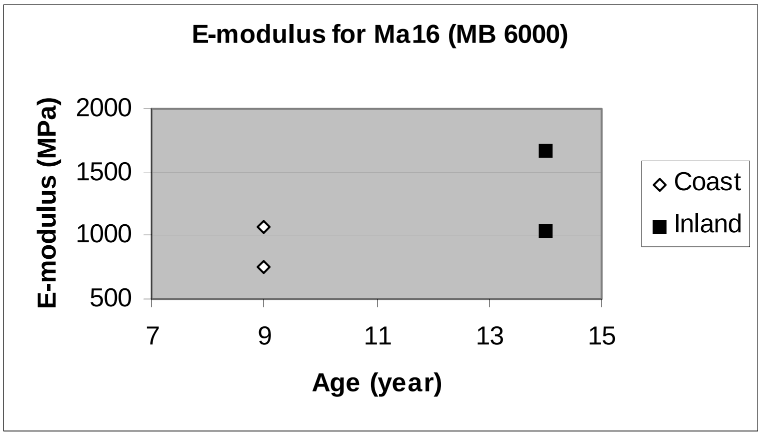


Figure 3.10 E-modulus, Ma16 with MB6000 as binder.

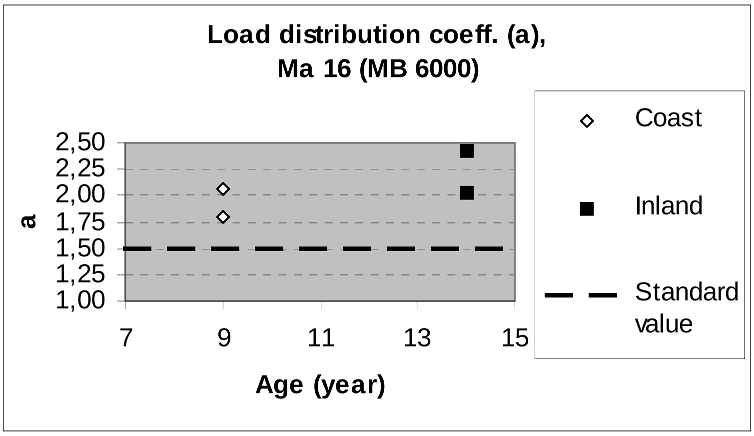


Figure 3.11 Load distribution coefficient, with MB6000 as binder.

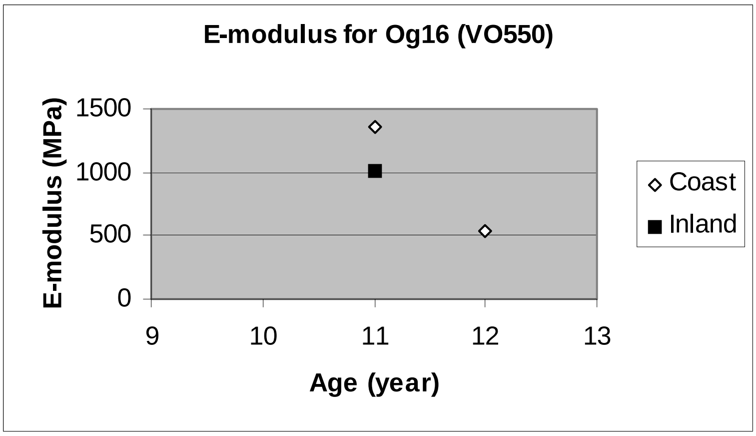


Figure 3.12 E-modulus, Og16 with VO550 as binder (550 indicates viscosity in mm^2/s at 60℃).

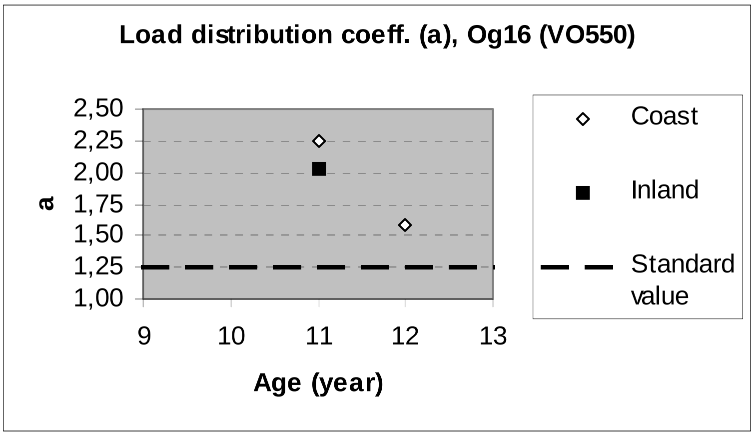


Figure 3.13 Load distribution coefficient, Og16 with VO550 as binder (550 indicate viscosity in mm²/s at 60 °C).

3.5.3 Analysis of recovered bitumen

Detailed results from the analysis of recovered bitumen are given in Appendix 5. In the following the results from the SHRP binder tests and the conventional tests are shown separately.

Conventional tests

Penetration test

Results from penetration tests carried out on Pen180 at 25°C are shown in Figure 3.14. As can be seen in Figure 3.14, layer 1 has lower penetration value than layer 2, which indicates that layer 1 has hardened most.

Sample 29, with one year of age, has hardened more than sample 27 and sample 22 with respectively 3 and 11 years of age. There are definitely different factors affecting the rate of hardening for these samples. This could be the void content, which was 6,9 % for sample 29 and 3,8 and 4,8 % respectively for sample 22 and 27. Other factors, which may influences the hardening, are transportation during production, aggregates, production, etc.

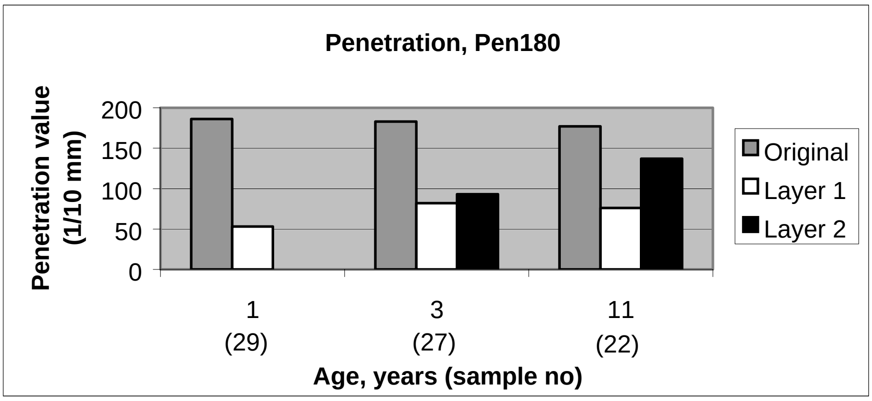


Figure 3.14 Penetration at 25°C for Pen 180 with different age.

In Figure 3.15, penetration values for MB-binders are shown.

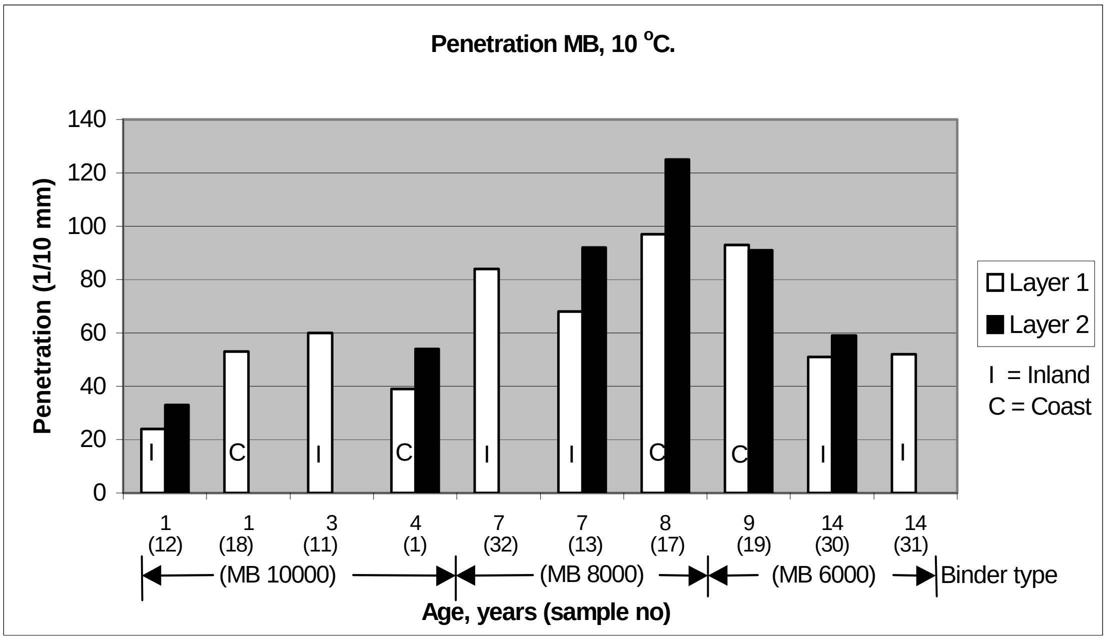


Figure 3.15 Penetration values at 10°C for MB-binders with different viscosity and age.

Figure 3.15 shows that the hardening of layer 1 is somewhat higher than for layer 2, except for sample 19 where the penetration value for layer 1 and layer 2 are about the same. MB10000 samples from the inland area shows less hardening after 3 years than after 1 year. This must be explained by other effects than pure climate exposure. For samples from the coastal area the hardening is increasing with time as expected.

For MB8000, samples from the inland area have different hardening after same time in service. Sample 13 is stiffer than sample 32. This could be explained by higher void content and lower binder content in sample 13 than in sample 32. The sample from the coastal area has higher penetration value than the two samples from the inland area, even though the sample from the coastal area has several years in service. This could be explained by low void content and high binder content in the sample from the coastal area, or it may be caused by higher temperature exposure in the inland.

For MB6000, the samples from the inland area have penetration values at about the same level. Here the sample from the coastal area has higher penetration value than samples from the inland area. This is also expected, as the time in service is less than for the samples from the inland area.

Viscosity (Brookfiels rotational viscosity test)
Results from viscosity tests are shown in Figure 3.16 and 3.17. In Figure 3.16 viscosity measurements at 60°C is shown for soft binders. In Figure 3.17 the results from measurements at 90°C are shown for all recovered binders.

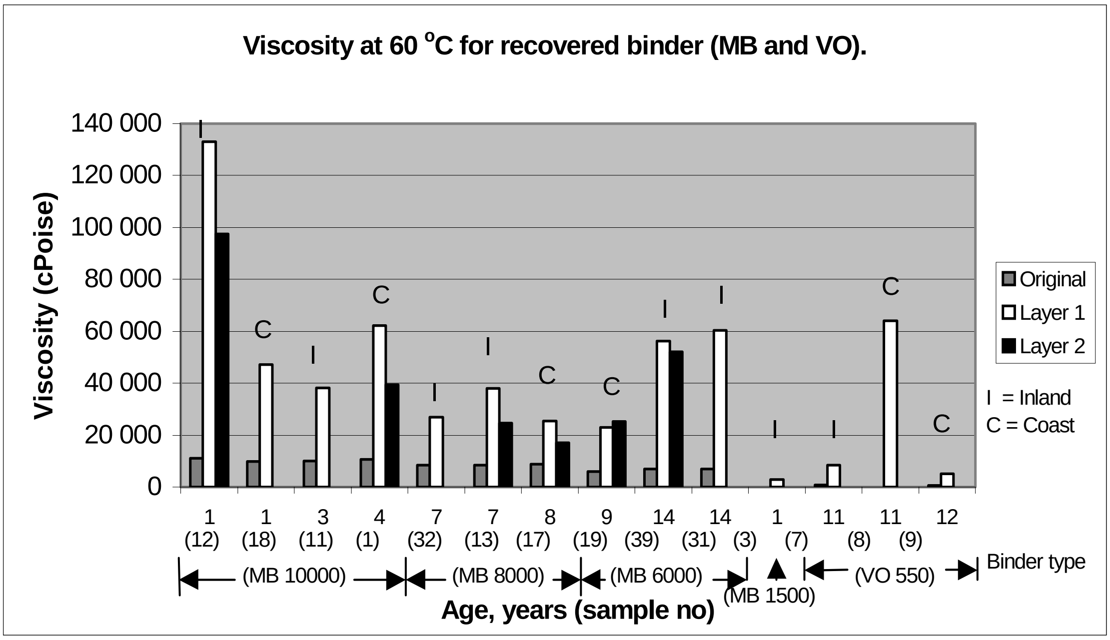


Figure 3.16 Viscosity at 60°C for soft binders (MB and VO).

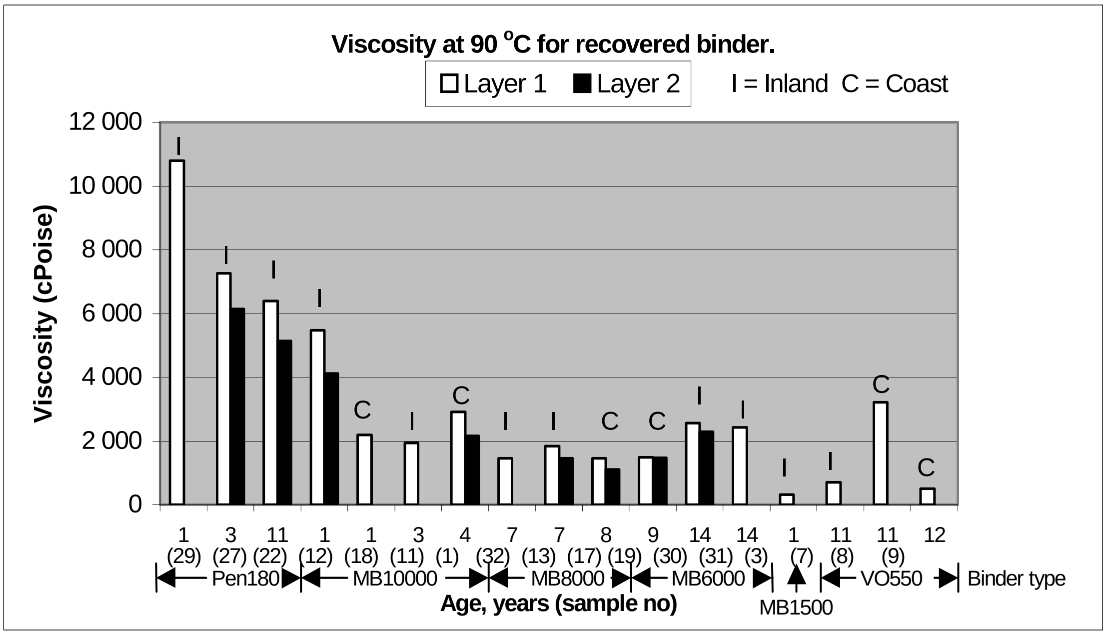


Figure 3.17 Viscosity at 90°C for all recovered binders.

From Figure 3.16 it can be seen that all samples have hardened significantly compared to the original viscosity. Layer 1 has hardened more than layer 2 for all samples, except sample no 19. The results after the viscosity measurements give the same trend as the penetration test.

Softening point test
Results from the softening point test are shown in Figure 3.18.

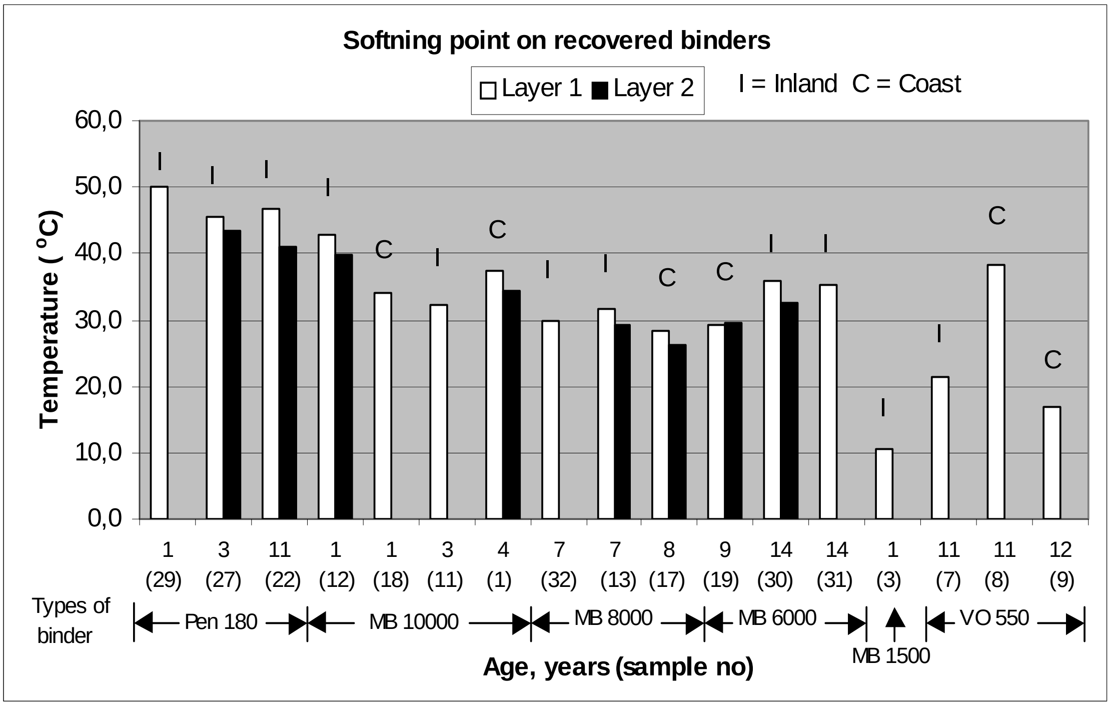


Figure 3.18 Softening point recovered binders.

The results from the softening point test give the same trend as the penetration and viscosity tests.

Ageing Index

An ageing index is calculated based on viscosity at 60°C for original and recovered bitumen. The ageing index is calculated for layer 1 and layer 2 for soft binders (MB and VO). Results are shown for layer 1 and 2 in Figure 3.19 and 3.20 respectively.

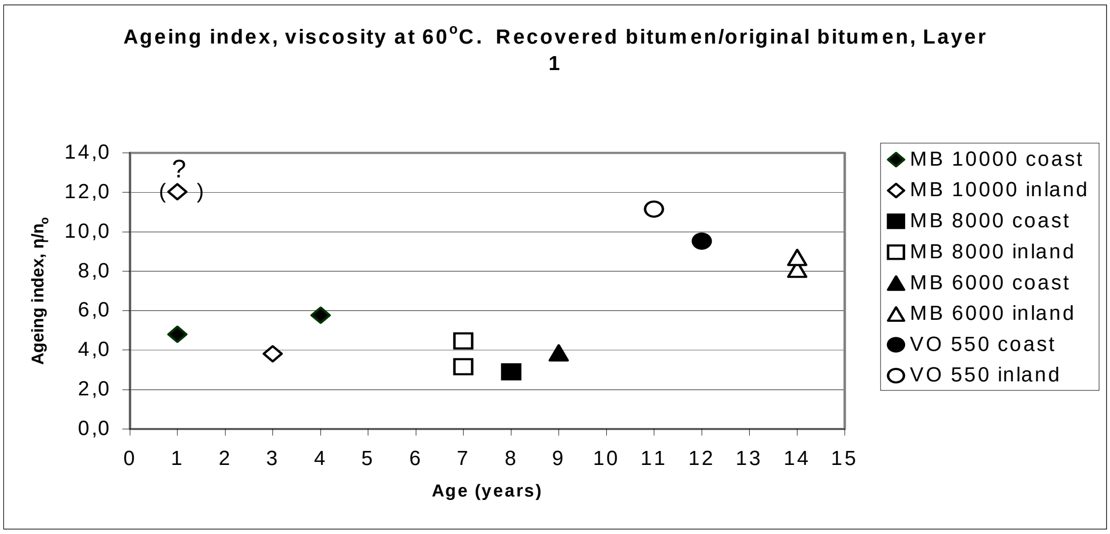


Figure 3.19 Ageing index layer 1 (upper surface). Ageing index are calculated as the ratio of viscosity of recovered to original binders at 60°C.

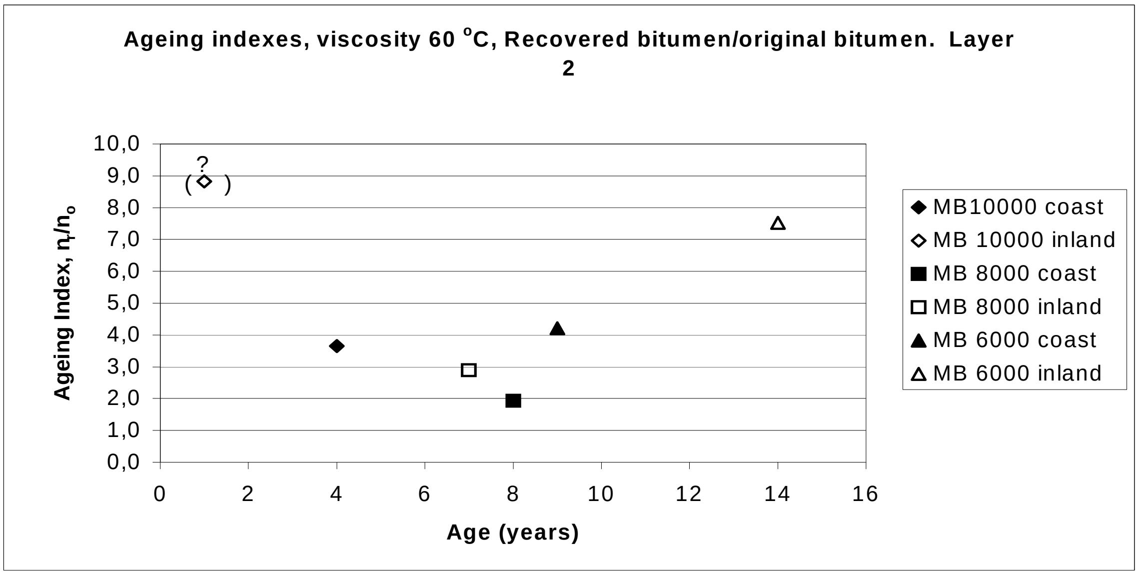


Figure 3.20 Ageing index layer 2. Ageing index are calculated as the ratio of viscosity of recovered to original binders at 60°C.

Figures 3.19 and 3.20 show a trend of higher ageing indices for samples from the inland area than for the coastal area when comparing each binder type separately. Only one sample differ from this trend, and that is sample no 11 (MB10000, 3year old, layer 1). Sample no 12 (MB10000, 1 year old from inland area) has an abnormal ageing index value.

SHRP binder specification tests

Dynamic Shear Rheometer (DSR)

The DSR was used to measure fatigue characteristics of recovered and PAV-aged bitumen. In this case, the temperature where $G^{*}\sin \delta = 5000$ kPa was determined, see Figure 3.21.

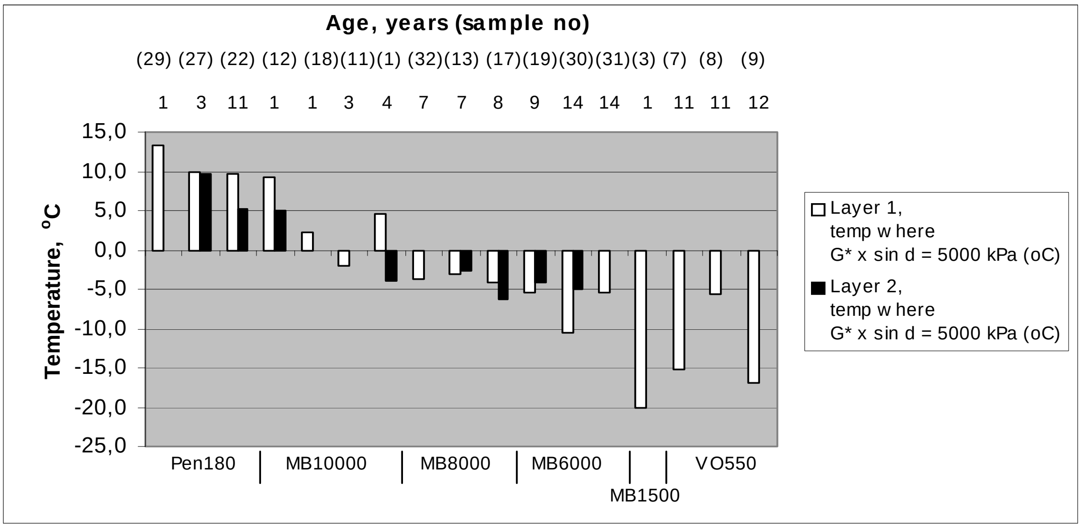


Figure 3.21 Results from DSR-measurements on recovered binder. Temperatures where $G^{*}\sin \delta = 5000$ kPa. Extrapolated values based on measurements at 10, 20 and 30°C.

Bending Beam Rheometer
The corresponding temperatures where S=300 MPa and *m*=0,300 have been measured and are shown in Figure 3.22.

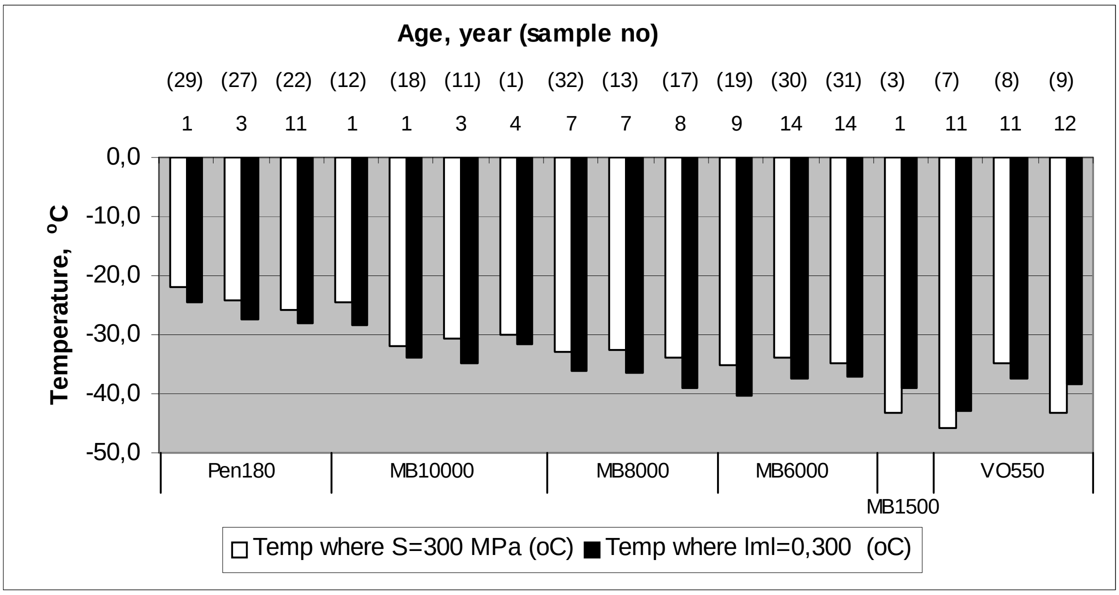


Figure 3.22 Results from Bending Beam Rheometer-measurements on recovered binder. Temperatures where S=300 MPa and m=0,300.

As can be seen in Figure 3.22, the critical low pavement temperature based on the BBR-analysis are in the range of -55°C to -30°C.

3.5.4 Analysis of original and laboratory aged bitumen

Detailed results from analysis of original and laboratory aged bitumen are given in Appendix 6. The results for conventional tests and SHRP-tests are presented separately in the following.

Conventional tests

Penetration test.
The results from penetration test on Pen180 at 25^{o}C and on soft binders (MB) at 10^{o}C are given in Figures 3.23 and 3.24.

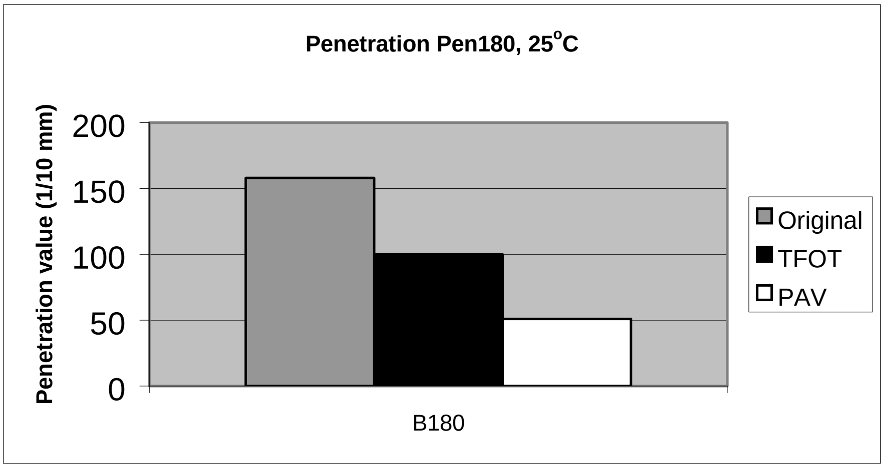


Figure 3.23 Results from penetration test on binder Pen180. The test is performed at 25 °C on original and laboratory aged (TFOT and PAV) bitumen.

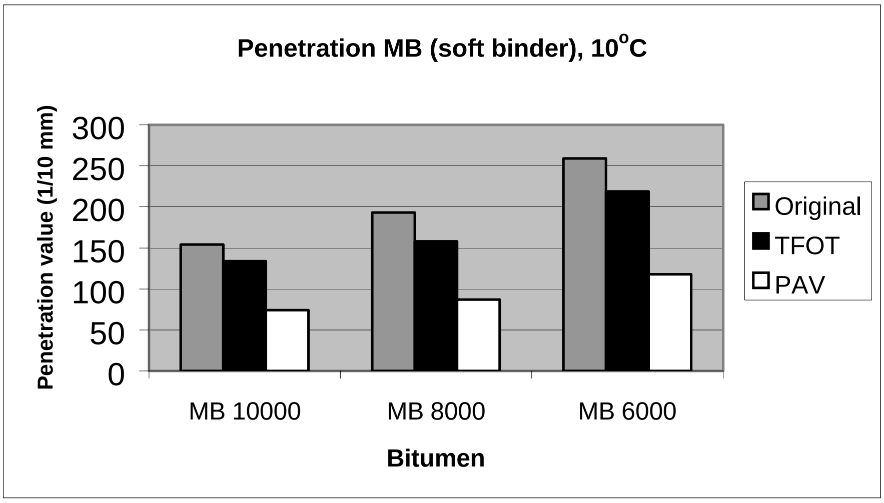


Figure 3.24 Results from penetration test on soft bitumen (MB). The test is performed at 10 °C on original and laboratory aged (TFOT and PAV) bitumen.

Figure 3.23 and 3.24 show that the relative change in the penetration value (ageing effect) after TFOT is less for MB-graded binders (soft binders) compared to Pen180 It has to be reminding of that TFOT is performed at 163°C for Pen 180 and at 120°C for MB-graded binders.

Viscosity (Brookfiel rotational viscosity test)

Viscosity at 60°C and 90°C for Pen180 is shown in Figure 3.25. In Figure 3.26, the viscosity at 60°C for soft binders are shown.

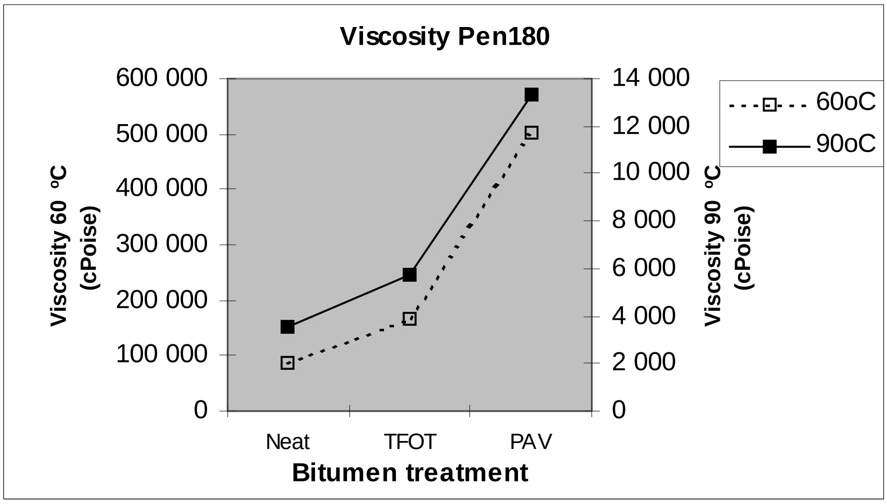


Figure 3.25 Viscosity measured on Pen 80 with a Brookfield rotational viscosimeter at 60 and 90°C.

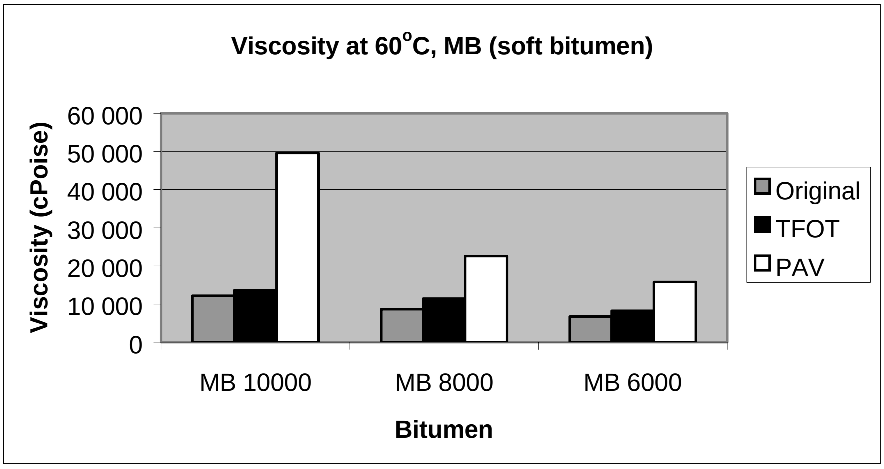


Figure 3.26 Viscosity measured on soft bitumen (MB) with Brookfield rotational viscosimeter at 60°C.

Figure 3.25 and 3.26 shows that the increase in stiffness after PAV is more severe for stiff binders. MB10000 has hardened relatively more than MB6000.
It has to be reminding of that PAV is performed at 100°C for Pen 180 and MB10000 and at 90°C for MB8000 and MB6000. This could explain the more severe hardening for Pen 180 and MB10000.

Softening point test.
Results from the softening point test are given in Figure 3.27.

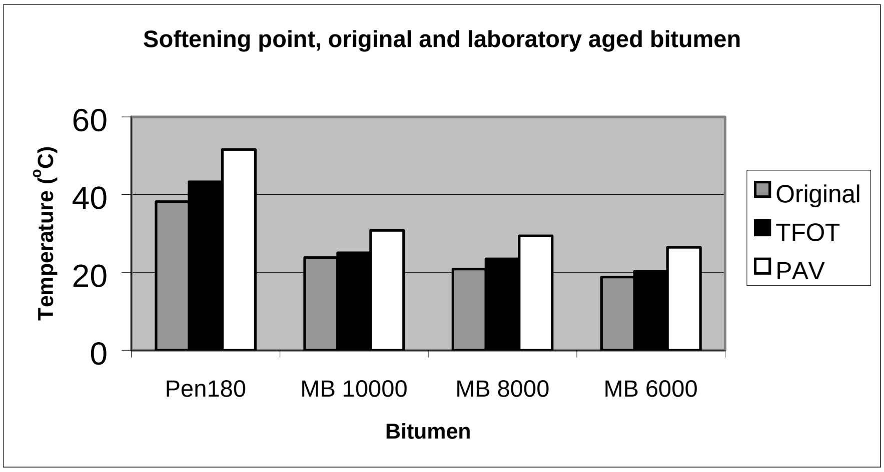


Figure 3.27 Softening point, original and laboratory aged bitumen.

Ageing Index

The ageing index is calculated based on viscosity at 60°C. The ageing index is calculated for both TFOT- and PAV-treated binders. The ageing index is calculated using the following formula:

$$\eta_a/\eta_o \qquad (3\text{-}4)$$

η_a = viscosity after ageing (TFOT or PAV)
η_o = viscosity of original bitumen

The ageing indices are shown in Figure 3.28.

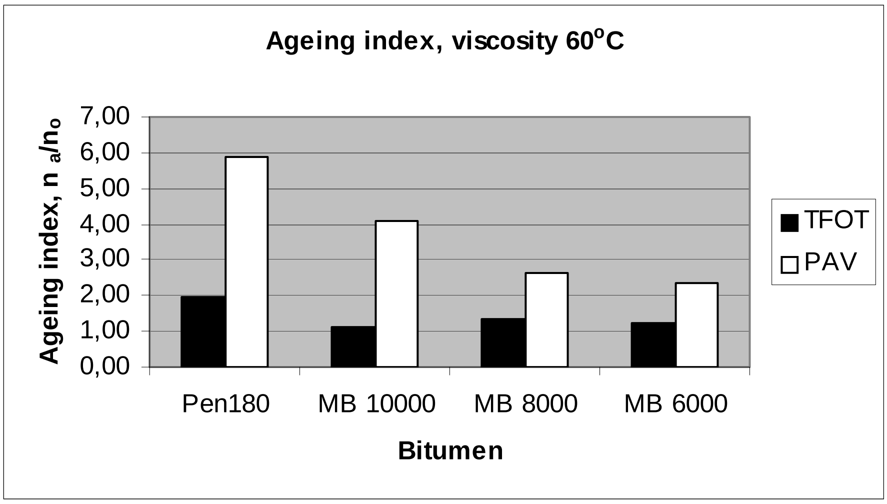


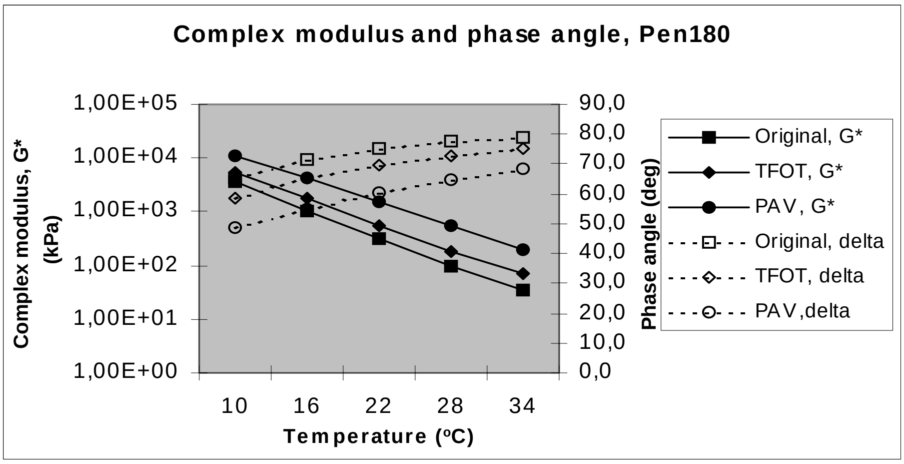
Figure 3.28 Ageing index, viscosity measurements at 60°C on original and laboratory aged bitumen.

Figure 3.28 shows that the ageing index after PAV is decreasing with decreasing stiffness in the original binder. After the TFOT, the ageing indicis for MB-graded binders are in about the same level. The ageing index for Pen180 is higher after TFOT compared to the MB-graded. This could however be explained by a higher

TFOT ageing temperature for the Pen 180 binder (163°C) than for the MB binders (120°C). The ageing indices after PAV are higher for the stiffest binder.

SHRP binder specification tests

Dynamic Shear Rheometer (DSR)
DSR measurements were carried out on original, TFOT- and PAV-aged binders. The results are shown in Figure 3.29.



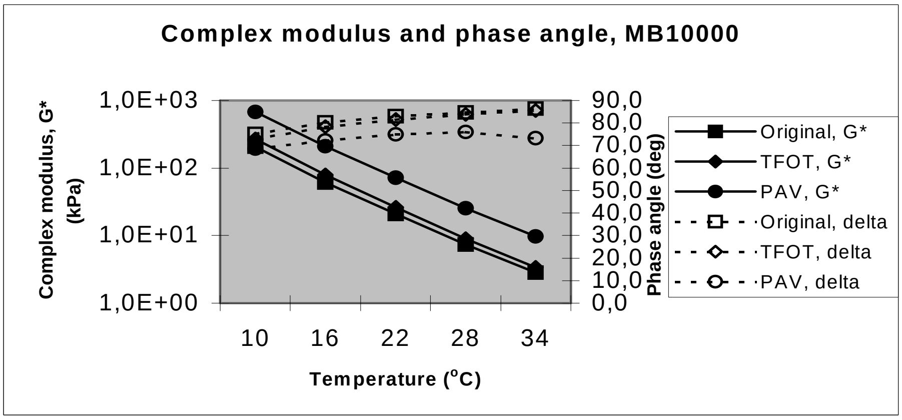
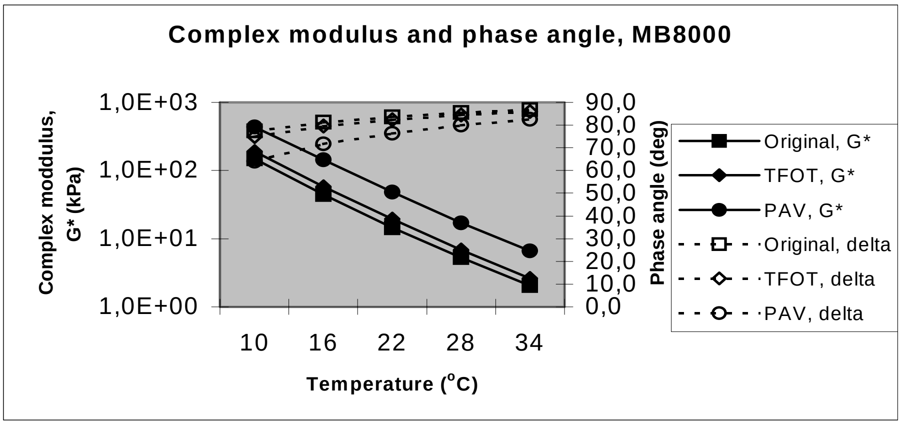


Figure 3.29 Complex modulus and phase angle, original and laboratory aged binder. (Figure 3.29 to be continued on page 96)



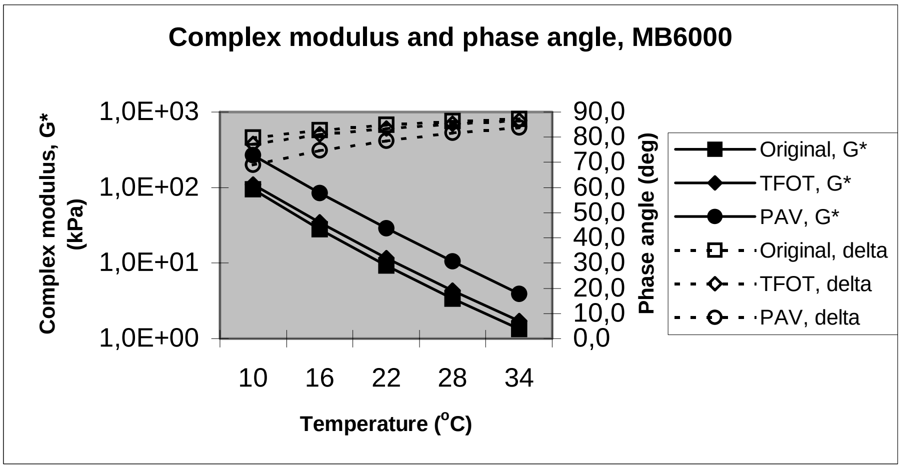


Figure 3.29 (continued) *Complex modulus and phase angle, original and laboratory aged binder.*

It can be seen, when studying Figure 3.29 (and the measured values in Appendix 6), the relative changes in complex modulus and phase angel after the TFOT is higher for Pen180 compared to the MB-grades[1].

Bending Beam Rheometer

Temperatures where S=300 MPa and *m*=0,300 have been measured and are shown in Figure 3.30.

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[1] The result for PAV aged MB10000 shows an odd result at 34°C, where the phase angle is somewhat low.

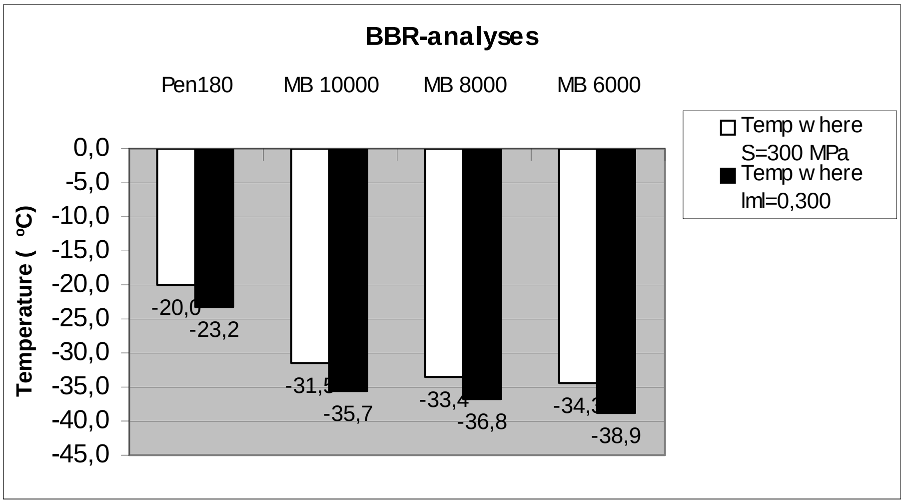


Figure 3.30 BBR measurements on laboratory aged bitumen (PAV-test).

Figure 3.30 shows that the limiting low temperature is much higher for Pen180 compared to the MB-graded binders. There are small differences in the limiting low temperature between the MB-graded binders.

3.6 Discussion

Asphalt-mixture

As a result of the tests carried out on asphalt mixtures the following could be highlighted:

- The binder contents in the analysed samples are on average higher than the binder content given in the mixture-formulation.
- The void contents in the analysed samples are slightly higher than in the mixture-formulation. The void content varies between 1,8 – 12,7 %. Only 3 samples have a void content lower than 4 %. High void contents could contribute to an accelerated hardening of the binder in the asphalt mixture.
- Calculated load distribution factors, a-factors, are generally higher than the standard a-factors used in the Norwegian Road Design System for Ma-mixtures with soft binders (MB 6.000 – MB10.000). This is also the case for cold produced mix (Og-mixture) with VO550 as binder. For Agb-mixtures, the calculated a-factors are lower than standard. For cold produced mix with MB1500 as binder (emulsion), there is small difference between the standard and calculated factor. The values calculated from the indirect tensile test could be questioned due to the fact that some of the specimens were rather thin.
- The E-modulus has a tendency to increase with age for the Agb-mixtures. For the Ma16-mixtures there are a tendency of higher E-modulus in inland areas compared to coastal areas. A trend for the Og-mixture is decreasing E-modulus with age. It is important to be aware of the fact that the type of aggregates as well as the binder influence the mechanical properties of the asphalt-mix. In Figure 3.31, E-modulus for asphalt-mixes with MB10000 and MB6000 are shown. This figure shows a tendency of higher E-modulus in inland areas compared to coastal areas. There is also a tendency of increasing E-modulus with age.

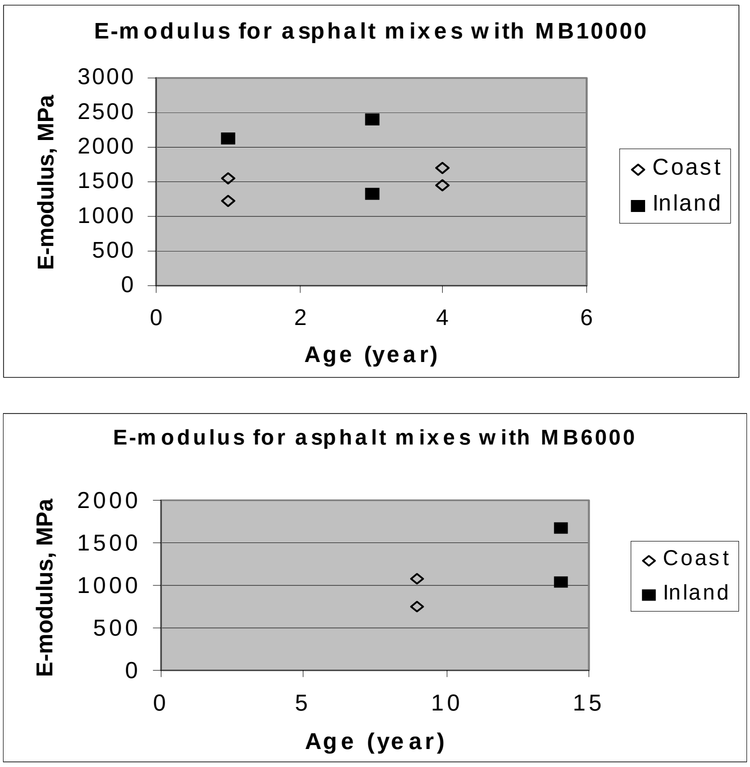


Figure 3.31 E-modulus in asphalt-mixes with MB10000 and MB6000 as binders. The E-modulus is calculated after the indirect tension test.

Recovered binder

As a result of the tests carried out on recovered binders, the following could be pointed out:

- It is obvious that pavements show more severe ageing in the top layer (10 mm) than in the underlying layer.
- In this limited investigation we found no obvious coherence between increased hardening of the binders and increasing void content as seen in Figure 3.32. It is necessary to have in mind that the binders are recovered from different mixtures, from different locations, and that the production processes probably are different.

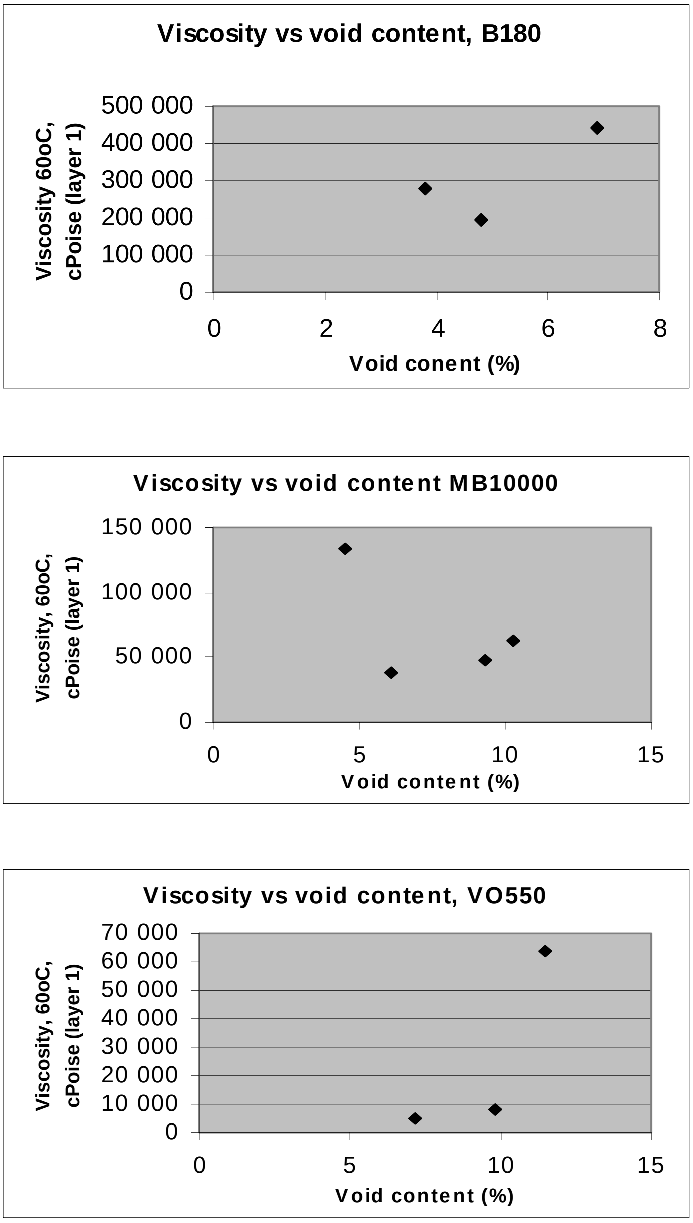


Figure 3.32 Viscosity at 60°C vs void content for three binders. The binders are recovered from different mixes and from different locations.

- From Figure 3.19 and 3.20, a slight tendency of higher ageing indices for the inland areas compared to the coastal area is indicated when comparing each binder type separately. Even though there are several factors influencing the hardening of the bitumen, this is an indication of increased binder hardening in the inland areas compared to the coastal areas.
- The Bending Beam Rheometer test on recovered binders is of special interest when evaluating the low temperature capacity of asphalt mixes. As can be seen in Appendix 3, the air temperature in the inland area can go below -46°C. In the coastal area the corresponding temperature is -25 °C. When classifying binders according to the SHRP-specification, BBR is used on PAV-aged binders, which simulates several years in service. Figure 3.22 and Table 3.7 shows critical minimum temperatures according to the SHRP specifications on PAV aged binders.

Table 3.7 BBR-analysis on recovered binders.

Binder	**Limiting temperature measured using BBR (^oC)**	**Lowest field temperature which satisfies the SHRP binder specifications (^oC)**
Pen180	-22	-32
MB10000	-24	-34
MB 8000	-33	-43
MB 6000	-34	-44
MB 1500	-39	-49
VO 550	-35	-45

The critical low-temperature in a pavement can be calculated using the following formula:

$$T_{surface} = 0,859*T_{air} + 1,7 \quad [89] \qquad (3\text{-}5)$$

According to this, the critical pavement temperature in the inland area is –38^{o}C and in the coastal area -20^{o}C.

As a result of the BBR-analysis, MB8000 is the stiffest binder satisfying the SHRP binder specification for the inland area. Comparing this with the result from the visual registration of pavement surface given in Table 3.5, transverse cracks are occurring frequently in pavements with Pen180 after three years in service. For MB10000 transverse cracks are also occurring after three years. For MB 8000 some transverse cracks are occurring after seven years. For MB 6000 frequent transverse cracks are observed after fourteen years. No transverse cracks are observed in pavements with VO550 as binder. MB8000 and MB6000 satisfy the low temperature criterion and can be evaluated as suitable binders for pavements in this area.
From this we can conclude that binders no harder than MB8000 should be used in the actual inland area with regards to low temperature cracking. From this we can also conclude that the SHRP binder specification can be used for low temperature specifications on soft binders for Norwegian conditions. It has to be remarked that more field investigations should be carried out to make a final conclusion.

In the coastal area, all the evaluated binders satisfy the low temperature criterion in the SHRP binder specifications. The visual registration of the pavement condition shows no transverse cracks in the coastal area. This also supports the use of the BBR-criterion for soft binders.

Original and laboratory aged bitumen

It is important to compare results from field samples with laboratory aged bitumen to judge if adjustments have to be done to the SHRP binder specification for Norwegian conditions. This has to be done in future studies.

Comparing laboratory aged bitumen to recovered bitumen from field samples (layer 1)

Studies of laboratory aged binders as well as recovered binders from mixtures in service are needed to evaluate models for prediction of service life. An issue of this work has therefore been to examine the rheology of recovered bitumen compared with the original and laboratory aged binders. It is of special interest to examine whether the SHRP-binder test procedures and specification limits can be used directly for soft binders under Norwegian conditions or if they have to be modified.
Only the type of binder is evaluated when comparing recovered and laboratory aged bitumen. In this study we have not paid any attention to the aggregate or type of mixture.

Pen180

Results from conventional tests are given in Table 3.8.

Table 3.8 Results from conventional testing, recovered and laboratory aged Pen180.

Sample	Age (year)	Pen	Viscosity, 60^{o}C, cPoise	Viscosity, 90^{o}C, cPoise	Softening point (^{o}C)
Original		158	85400	3520	38,2
TFOT		100	166000	5760	43,3
PAV		51	504000	13300	51,6
29	1	53	442000	10800	49,9
27	3	82	196000	7260	45,4
22	11	76	281000	6400	46,6

The testing of Pen180 gave some odd results. Sample 29 after 1 year in service is stiffer than sample 22, which has had 11 years in service. This is in contradiction of what is expected and conclusions are hard to be drawn. More field samples have to be analysed to get significant results.

The BBR-analysis shows that the laboratory aged bitumen had a low limiting temperature of –20^{o}C (Figure 3.30), while the recovered binders had limiting temperatures varying from –21,8 to –25,8°C (Figure 3.22).

MB10000

Results from conventional tests are given in Table 3.9.

Table 3.9 Results from conventional testing, recovered and laboratory aged MB10000.

Sample	Age (year)	Pen (10^{o}C)	Viscosity, 60^{o}C, cPoise	Viscosity, 90^{o}C, cPoise	Softening point (^{o}C)
Original		154	12200	885	23,8
TFOT		134	13600	995	25,1
PAV		74	49600	1480	30,8
12	1	24	133000	5480	39,8
18	1	53	47200	2200	34,0
11	3	60	38200	1940	32,3
1	4	39	62200	2920	37,5

From Table 3.9 it can be seen that MB10000 in the field is stiffer than the laboratory aged specimens.

In the BBR-analysis, the laboratory aged bitumen showed a limiting temperature of –31,5^{o}C (Figure 3.30), while the recovered binders showed limiting temperatures of –24,5 ^{o}C (sample 12), -31,9 ^{o}C (sample 18) –30,6 ^{o}C (sample 11) and –30,0°C (sample 1) (Figure 3.22).
The DSR-analysis on recovered and PAV-aged MB10000 is shown in Figure 3.33. As can be seen, sample 18 and 11 are at about the same level as the PAV-aged. The age of the field samples varies from 1 to 4 years. In this case, the PAV and TFOT do not seem to produce as severe an ageing as occurs in the field.

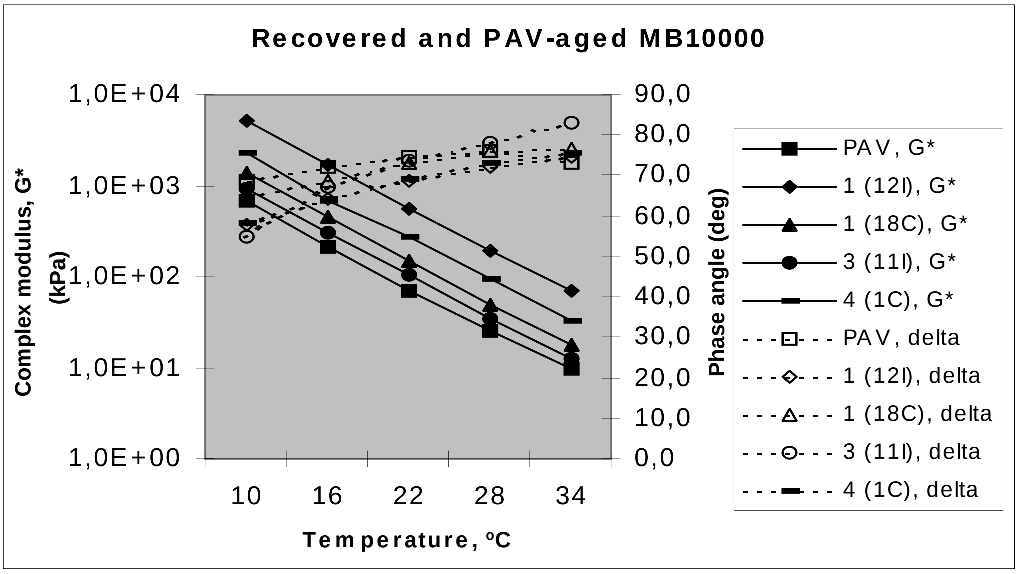


Figure 3.33 Complex modulus and phase angle on recovered binder and laboratory aged MB10000. In the legend key the first number denotes age in years. In parenthesis, the sample number and location (I=Inland and C=Coast) are given.

MB 8000

Results from conventional tests are given in Table 3.10.

Table 3.10 Results from conventional testing, recovered and laboratory aged MB8000.

Sample	Age (year)	Pen (10°C)	Viscosity, 60°C, cPoise	Viscosity, 90°C, cPoise	Softening point (°C)
Original		193	8640	680	20,9
TFOT		158	11400	820	23,5
PAV		87	22600	1390	29,4
32	7	84	26800	1460	29,8
13	7	68	38000	1840	31,8
17	8	97	25400	1450	28,4

From Table 3.10 it can be seen that sample 32 and 17 have about the same stiffness as PAV-aged MB8000. Sample 13 is somewhat stiffer than the PAV-aged.

In the BBR-analysis, the laboratory aged bitumen has a limiting temperature of –33,4°C (Figure3.30), while the recovered binders have limiting temperatures of –32,8 °C (sample 32), -32,5°C (sample 13) and –33,9 °C (sample 17) (Figure3.22). This gives a fairly good correlation between field samples and laboratory aged samples.

The DSR-analysis on recovered and PAV-aged MB8000 is shown in Figure 3.34. The recovered and PAV-results correlate well, only sample 13 has somewhat higher complex modulus and somewhat lower phase angle.

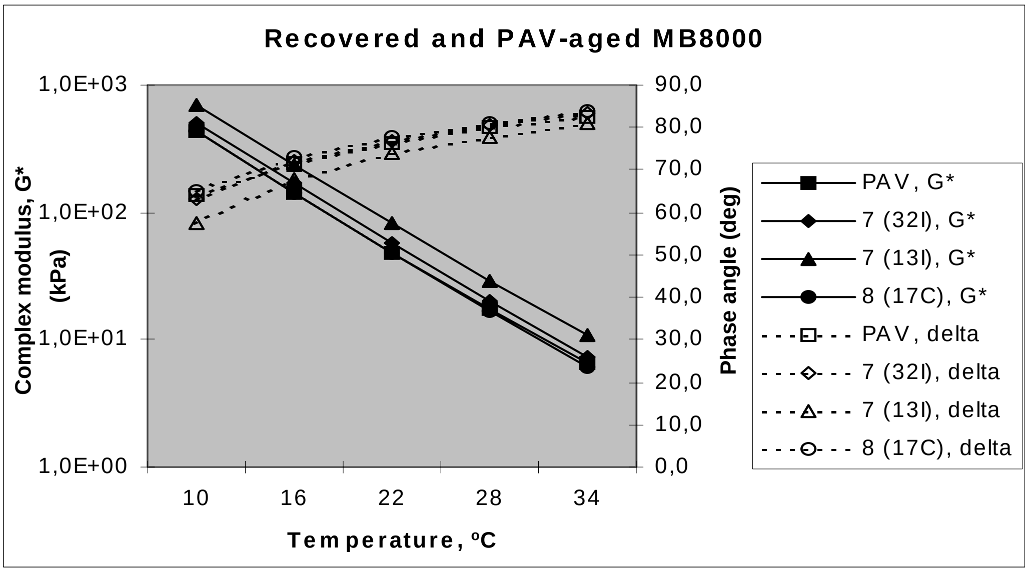


Figure 3.34 Complex modulus and phase angle for recovered and laboratory aged MB8000. In the legend key the first number denotes age in years. In parenthesis sample number and location (I=Inland and C=Coast) are given.

The PAV treatment of MB8000 has been performed at 90 ^oC. This seems to be the “right” temperature for this binder under the actual climatic conditions. It is also necessary to notice the age of the recovered binder. PAV simulates ageing during several years in service. Low volume roads ought to have a service life longer than high volume roads. On this basic, question could be raised if PAV is “rough” enough for low volume roads.

MB 6000

Results from conventional tests are given in Table 3.11.

Table 3.11 Results from conventional testing, recovered and laboratory aged MB6000.

Sample	Age (year)	Pen (10^oC)	Viscosity, 60^oC, cPoise	Viscosity, 90^oC, cPoise	Softening point (^oC)
Original		259	6740	588	18,8
TFOT		219	8200	620	20,3
PAV		118	15800	1030	26,5
30	14	51	56200	2560	36,0
31	14	52	60400	2420	35,4
19	9	93	23000	1490	29,2

From Table 3.11 one can see that the recovered binder is harder than PAV-aged MB6000.

In the BBR-analysis, the laboratory aged bitumen gave a limiting temperature of –34,3^oC (Figure 3.30), while the recovered binders had limiting temperatures of –34,0 ^oC (sample 30), -34,7 ^oC (sample 31) and –35,3 ^oC (sample 19) (Figure 3.22). This gives a fairly good correlation between field samples and laboratory aged samples.

The DSR-analysis on recovered and PAV-aged MB6000 is shown in Figure 3.35. We here observe that the recovered binders are harder than the PAV-aged. The ages of the field samples are in this case 9, 14 and 14 years. The pavement life on the low volume roads in Norway should be at this level of age. Due to this fact, the hardening after TFOT and PAV should have been somewhat greater to simulate the situation in service very well.

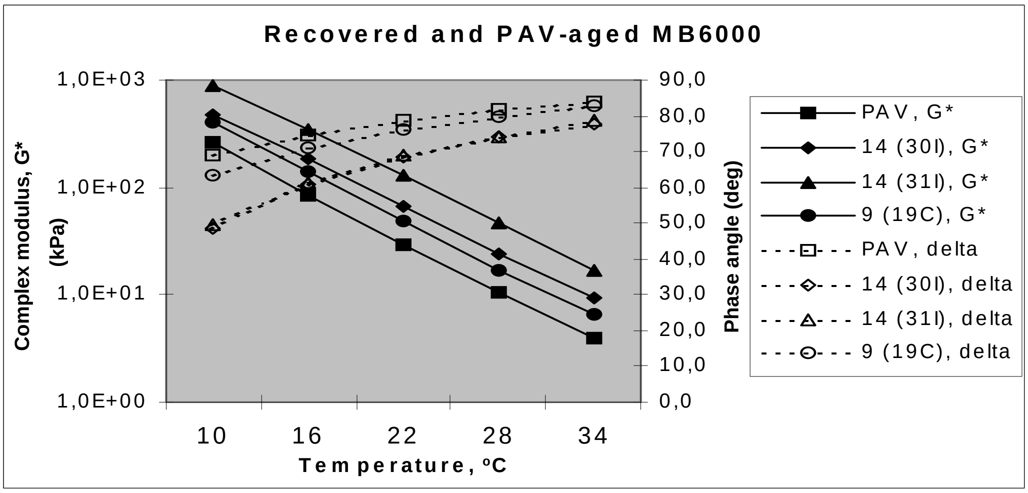


Figure 3.35 Complex modulus and phase angle on recovered and laboratory aged MB6000. In the legend key the first number denotes age in years. In parenthesis sample number and location (I=Inland and C=Coast) are given.

3.7 Conclusions

Ageing of bitumen is a very complex process and many factors contribute to the age hardening. In this investigation, field samples from different climatic conditions have been studied. Recovered binders have been compared to original and laboratory aged bitumen. The SHRP binder specification tests and conventional binder tests have been used in the analysis of the binders.

To verify the conclusions of this investigation, it will be necessary to do additional field investigations and to make regression analysis with more laboratory data.

Asphalt mixtures

Calculated load distribution factors (a-factor) are higher than the standard load distribution factors used in The Norwegian Road Design Manual for Ma-mixtures with soft binders, MB6000 – MB10000. For Agb-mixtures however, the calculated a-factors are lower than the standard a-factors. Values calculated from the indirect tensile test are uncertain in this case due to the fact that some of the samples were rather thin.

From the field data we observe that the E-modulus has a tendency to increase with age for Agb-mixtures. For Ma-mixtures (mixtures with soft binder) there is a tendency of larger E-modulus in inland compared to coastal areas. Probably the E-modulus will increase in the mixtures with increasing age until micro-cracks occur in the pavement giving reduced E-modulus in the last part of the pavement life.

The void contents in the investigated field samples are slightly higher than in the mixture formulation.

Binder

It is found that the bitumen ages most severely in the top of the layer and decrease with depth.

There is a tendency of increased binder hardening in inland areas compared to coastal areas.

Pen180 is not suitable for inland climatic conditions when evaluating low-temperature properties. This conclusion is drawn from the visual registration of pavement conditions and from laboratory results using the SHRP binder specification tests (BBR).

Based on this investigation, it is however difficult to make a good and unambiguous description of the development of ageing in pavements with soft binders. Some younger pavements have age hardened more than older. Reasons for this could be different aggregates, different climatic exposure, production processes, transportation length, etc. To give an accurate characterisation of the development of age hardening in pavements, it will generally be necessary to take samples from the same pavement over several years. (An alternative is to make estimates based on relevant data from the quality control under production together with climatic data during service life, as done here).

When comparing laboratory aged soft bitumen to recovered bitumen from field samples, it is obvious that field samples age harden more than bitumen aged by TFOT and PAV. Laboratory ageing of soft binders, using these procedures, has to be evaluated for Norwegian conditions (low volume roads).

It is necessary to be aware of that only the top layers of field samples have been used in comparing recovered and laboratory aged bitumen.

CHAPTER 4 *TRANSPORTATION OF ASPHALT MIXES*

4.1 Background

Asphalt mixtures for pavements on low volume roads are often transported over long distances, on land by trucks or on see by freighter or a combination. The transportation time may vary from some few minutes to several hours or even days.

It is well known that a significant amount of bitumen hardening occurs during mixing, transportation and paveing process. The amount of hardening depends on a number of factors such as temperature, bitumen content, bitumen film thickness, etc.
As illustrated in principle in Figure 1.27 (chapter 1), the ageing effect during transportation and laying can be in the same order of magnitude as ageing during 8 years in service, and about 30 % of the ageing during mixing [54].

The two main factors influencing the ageing process during transportation and laying are:

- Reaction with atmospheric oxygen.
- Evaporation of volatiles.

4.2 Objective

In this study, field and laboratory investigations of two different asphalt mixtures were conducted. The asphalt mixtures were an Agb 16 (asphalt concrete, d_{max}=16mm) with B180 (pen 180) as binder and Ma 16 (asphalt concrete, d_{max}=16mm) with MB 10000 (soft binder) as binder. The primary goal of the study was to:

- Examine the change of temperature with time in an asphalt-mixture during transportation with freighter.
- Examine the ageing of the recovered binder. An important objective is to study how much the binders harden due to the long exposure time at high temperatures.
- Compare the ageing of recovered binder with laboratory aged binder. The binder aged in the laboratory was taken from the binder tank at the mixing plant.

4.3 Experimental set up

4.3.1 Transportation condition

In this study, two asphalt-mixtures were studied during transportation from the mixing plant with trucks from the plant to the harbour, then by freighter and finally with trucks from the freighter to the laying site. Both asphalt-mixtures were produced at the same plant. The asphalt was loaded into the freighter as shown in Figure 4.1.

Figure 4.1 Loading the freighter with asphalt.

The transportation time on the trucks was about 20 minutes and the time used for loading the freighter was about 5 hours. For Agb 16 mixture the transportation time in the freighter before unloading started was about 32 hours. For the Ma 16 mixture the transportation time was about 10 hours in this study. During transportation on the freighter the asphalt was covered with thin felt and a tarpaulin in addition to the cover of the stowage. From the freighter the asphalt was loaded on trucks with an excavator. The transportation time with trucks to the laying site was only a few minutes for both mixes.

A sketch of the stowage on the freighter is shown in Figure 4.2. As could be seen in Figure 4.2 there is no insulation in the hull. There is 6 cm concrete in the bottom of the stowage.

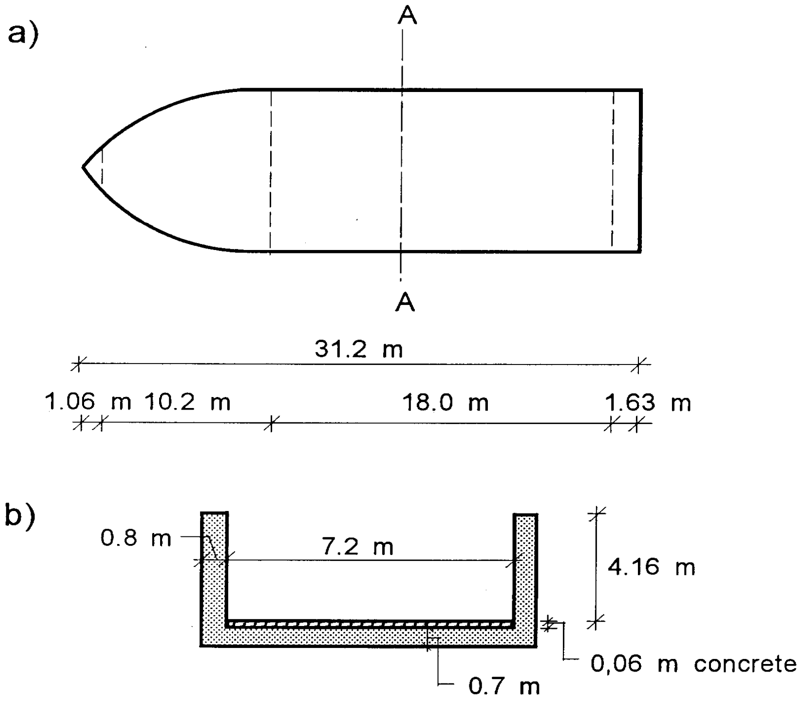
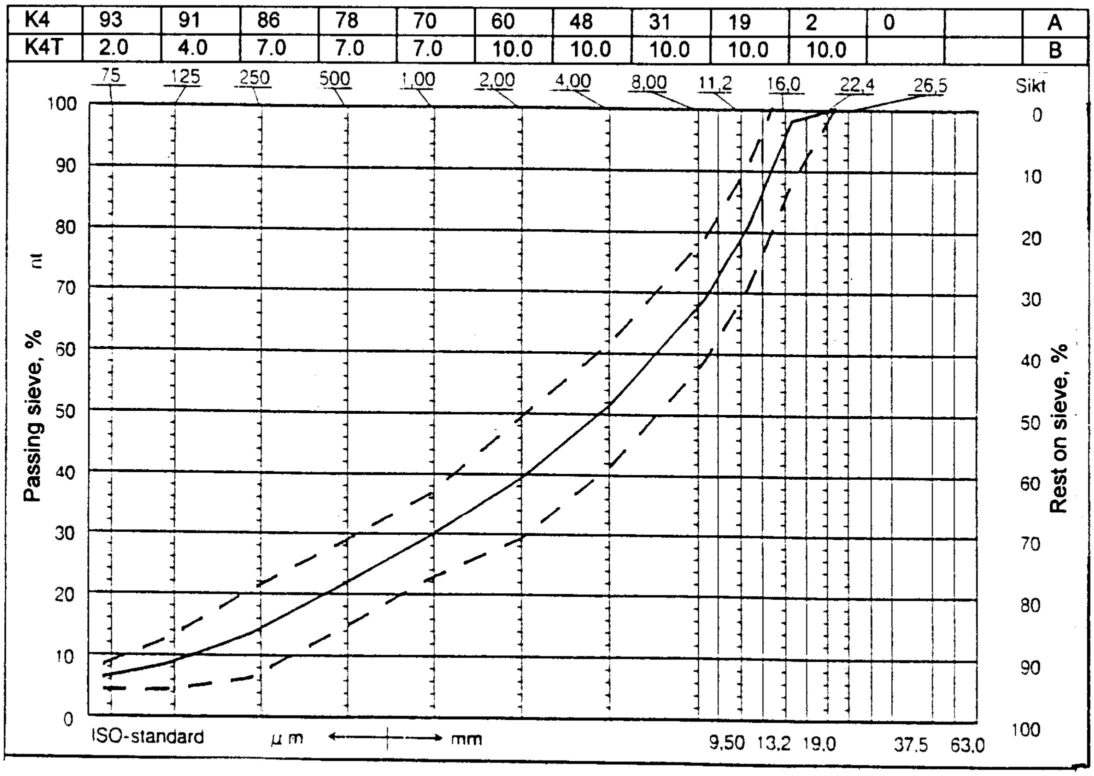


Figure 4.2 Sketch of the stowage of the freighter; a) plan, b) athwart.

4.3.2 Materials

<u>Agb 16 (asphalt concrete)</u>

This asphalt-mix was produced with a binder content of 5.2 percent. The mixing temperature was about 162°C.
The aggregate grading is shown in Figure 4.3.



K4	93	91	86	78	70	60	48	31	19	2	0		A
K4T	2.0	4.0	7.0	7.0	7.0	10.0	10.0	10.0	10.0	10.0			B

Figure 4.3 Aggregate grading Agb 16 (asphalt concrete, d_{max}=16mm).

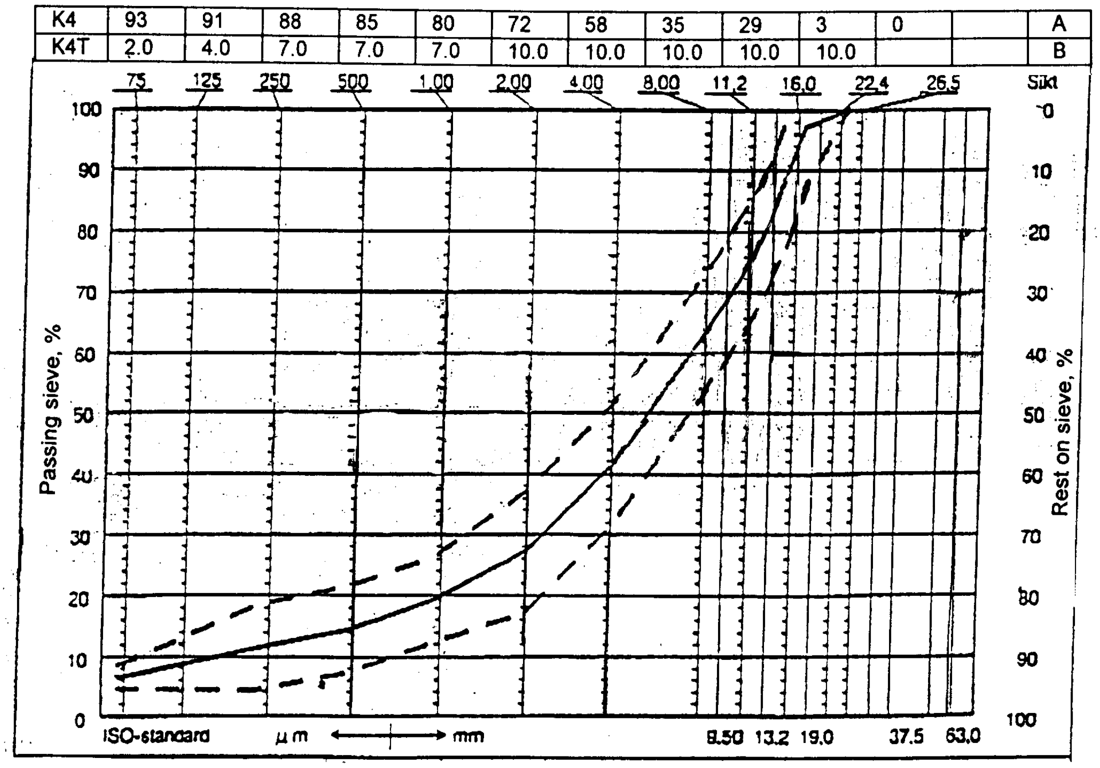
The bitumen type used was Pen 180 (penetration grade 180). Typical characteristics of the binder are given in Table 4.1.

Table 4.1 Characteristics of Pen180 taken from tank at the mixing-plant.

	Bitumen taken from tank at mixing-plant
Penetration, 25°C	196
Viscosity, 90°C (cPoise)	3180
Viscosity, 135°C (cPoise)	212
Softening point °C	37.5

Ma 16 (asphalt concrete, d_{max}=16mm)

This asphalt-mixture was produced with a binder content of 4 percent. The mixing temperature was about 125°C.
The aggregate grading is shown in Figure 4.4.



K4	93	91	88	85	80	72	58	35	29	3	0		A
K4T	2.0	4.0	7.0	7.0	7.0	10.0	10.0	10.0	10.0	10.0			B

Figure 4.4 Aggregate grading Ma 16,(d_{max}=16mm).

The bitumen type used was MB10.000 (soft binder). Typical characteristics of the binder are given in Table 4.2.

Table 4.2 Characteristics of MB10.000 (soft bitumen) taken from tank at the mixing-plant.

	Bitumen taken from tank at mixing-plant
Penetration, 10°C	154
Viscosity, 60°C (cPoise)	10.800
Viscosity, 90°C (cPoise)	915
Softening point °C	23,7

4.3.3 Test equipment

Temperature measurements

During transportation of the Agb 16, temperature sensors were placed in the mixture at three different locations.
The location of the temperature sensors is sketched in Figure 4.5.

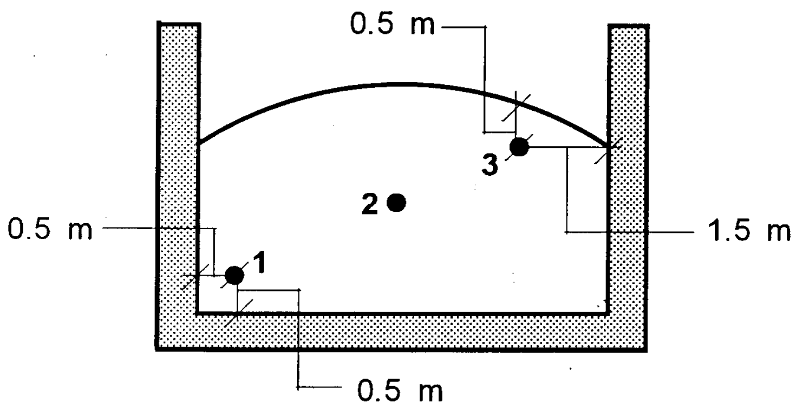


Figure 4.5 Location of temperature sensors.

Temperature sensor number 1 was placed 0.5 meters from the bottom, number 2 was in the middle of the mixture, and number 3 was 0, 5 meters from the top. In addition, one temperature sensor was used to record the air temperature in the stowage. The temperature sensors were connected to a signal logger, which logged the temperature every 15-minutes.

Bitumen analysis

Selected laboratory tests were performed on recovered, original and laboratory aged bitumen. In the following, these tests and test conditions are described.

Extraction

To extract the bitumen from the aggregates, methylenecloride was used as dissolvent medium. After the extraction, the binder was in a solution with the solvent.

Bitumen recovery

A rotavapor was used to recover the bitumen from the solution. The rotavapor used was a BÜCHI rotavapor model EL S.
The recovery process consists of two stages:

- Evaporation of solvent at atmospheric pressure.
- Evaporation of solvent under vacuum.

The Thin Film Oven Test (TFOT)

Artificial short-term ageing of bitumen in the laboratory was performed by the TFOT. The TFOT ageing temperature was 163°C and 120°C for Pen180 and MB 10.000 respectively. This test is expected to simulate the ageing during hot-mixture production.

The Pressure Ageing Vessel Test (PAV)

Artificially long-term ageing of bitumen in the laboratory was conducted by using PAV. This test is expected to simulate the ageing occurring in service. The ageing temperature was 90°C.

Penetration
The penetration test was carried out for recovered, original, and laboratory aged bitumen. On Pen180 the test was conducted at 25°C and for MB10.000 (soft binder) at 10°C.

The softening point test
The ASTM version of the softening point test was used. The test was carried out for all samples.

Viscosity
The viscosity was measured for all samples with the Brookfield rotational viscometer, model DV-II. The measurements were conducted at 90°C and 135°C for Pen180. For MB 10.000 the measurements were conducted at 60°C and 90°C.

Dynamic Shear Rheometer (DSR)
DSR-measurements were carried out for all samples at different temperatures (10,16, 22, 28 and 34°C). Measurements were conducted at strain levels within the linear viscoelastic range at a frequency of 10 rad/sec. The DSR equipment used was a Bohlin Dynamic Shear Rheometer.

The Bending Beam Rheometer (BBR)
BBR measurements were conducted on PAV-aged samples and on recovered binder.

4.4 Results

4.4.1 Temperature measurements

Results from the temperature measurements during the transportation of the Agb 16-mixture are shown in Figure 4.6.

As can bee seen from the Figure 4.6, the temperature in the middle of the asphalt mixture was almost constant. The temperature 0,5 meters from the bottom of the freighter was about 98°C after 40 hours. 0,5 meters from the top the temperature was about 113°C. The decrease rate in temperature was about 1,2°C/hour in the top and 1,34°C/hour in the bottom.
The air temperature within the stowage was about 13°C during the transportation.

The temperature on the surface was measured to be 90°C at the end of the registration period. At the end of the transportation, the asphalt mixture on the surface had cooled down to a too low temperature according to the specifications for compacting. Hence the asphalt in this part of the freighter was very stiff.

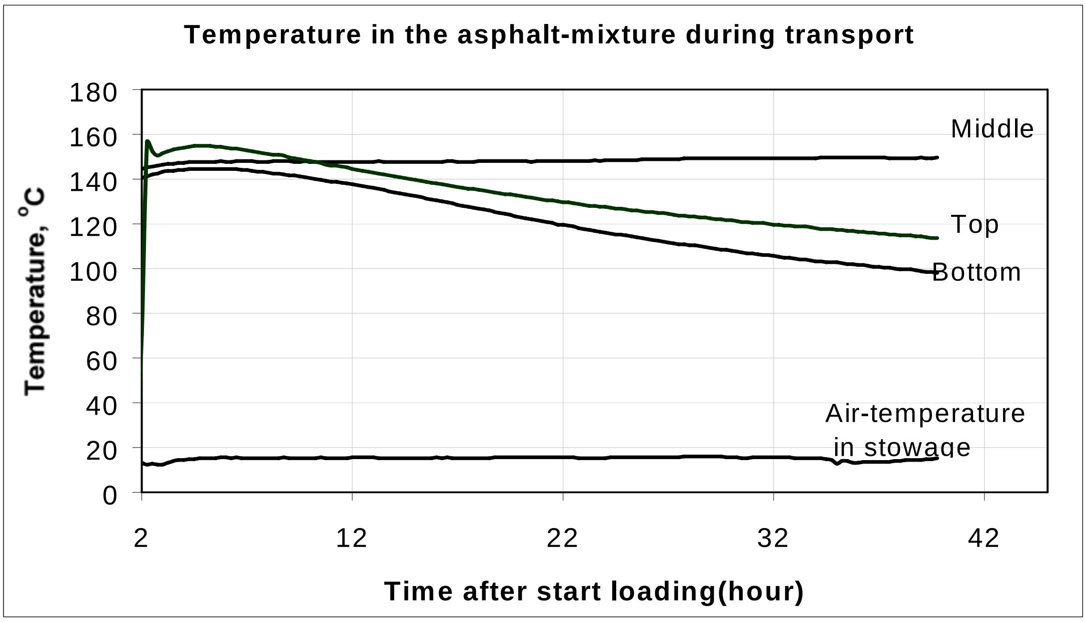


Figure 4.6 *Temperature in the asphalt-mixture during transportation on the freighter. The time-scale is in relation to when loading of the freighter was started.*

4.4.2 Bitumen analyses

Table 4.3 gives the sample identification used in the following results.

Table 4.3 *Sample identification.*

Sample no	**Bitumen**	**Treatment**	**Time for sampling (time after start of transportation), hours**	**Temp in mix at sampling, °C**	**Samples are taken**
A1	Pen180	Original			Tank in plant
A2	Pen180	TFOT			
A3	Pen180	PAV			
A4	Pen180	Recovered	0	160	In plant
A5	Pen180	Recovered	0.50	147	When loading the freigther
A6	Pen180	Recovered	37	109	When starting unloading the freighter
A7	Pen180	Recovered	41.25	113	After paver
A8	Pen180	Recovered	52.25	143	After paver
B1	MB 10.000	Original			Tank in plant
B2	MB 10.000	TFOT			
B3	MB 10.000	PAV			
B4	MB 10.000	Recovered	0	130	In plant
B5	MB 10.000	Recovered	0.50	128	When loading the freighter
B6	MB 10.000	Recovered	15.50	108	After paver
B7	MB 10.000	Recovered	23.25	84	After paver

Pen 180 (B180)

The results from analyses of Pen180 are given in Figures 4.7 – 4.11.

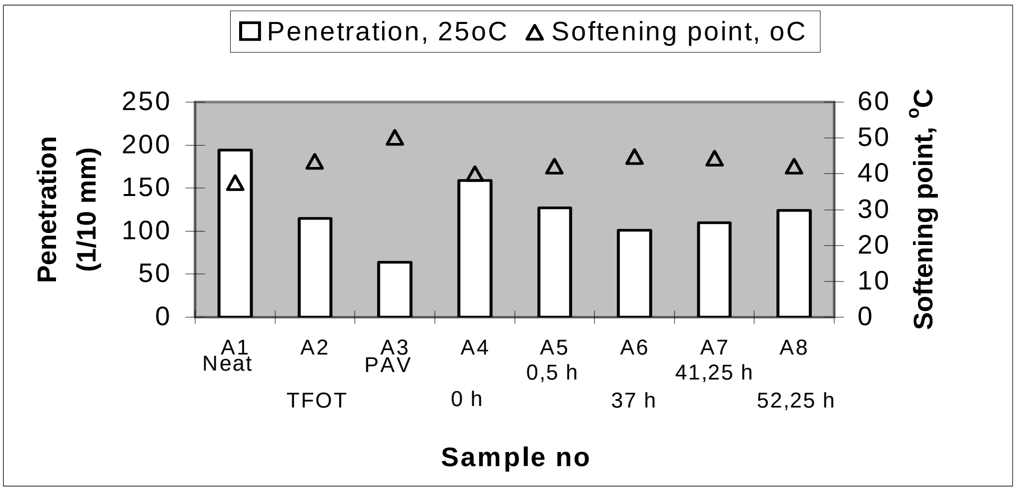


Figure 4.7 *Results from penetration and softening point tests on Pen180. Samples noted with hours (h) are recovered samples (A4-A8). h indicates time in hours after transportation started from the mixing-plant.*

As it can be seen in Figure 4.7, recovered samples taken after laying (A7-A8) have penetration and softening point values similar to those of TFOT-aged binder. Sample A6 was sampled from the top of the asphalt-mixture, while A7 and A8 were stored deeper in the freighter. Asphalt-mixture in the top of the freighter (sample A6) was more exposed to ageing than sample A7 and A8. The viscosity measurements shown in Figure 4.8 also show this effect.

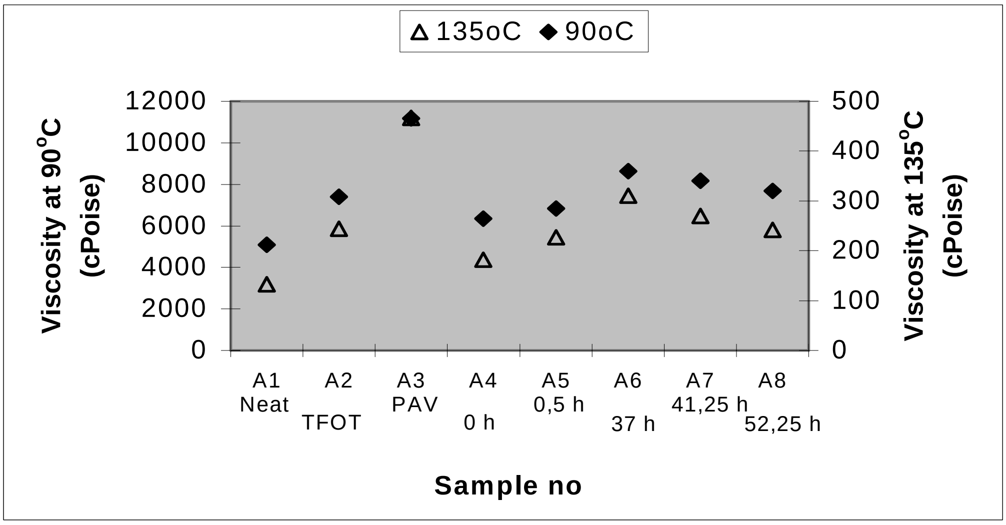


Figure 4.8 *Viscosity measured on Pen 180 at 90°C and 135°C. Samples noted with hour's (h) are recovered samples. h indicates time in hours after transportation started from mixing-plant.*

In Figure 4.9 and 4.10, results from DSR-measurements are shown. The TFOT-aged sample has about the same value for G* and phase angle as samples taken out after 0,5 hour (A5) and after 52,25 hours (A8). First sample taken after the freighter transport (A6) has the highest G*-value and the lowest phase angle-value. These results also show that sample A6 was most exposed to ageing and sample A4 least exposed to ageing.

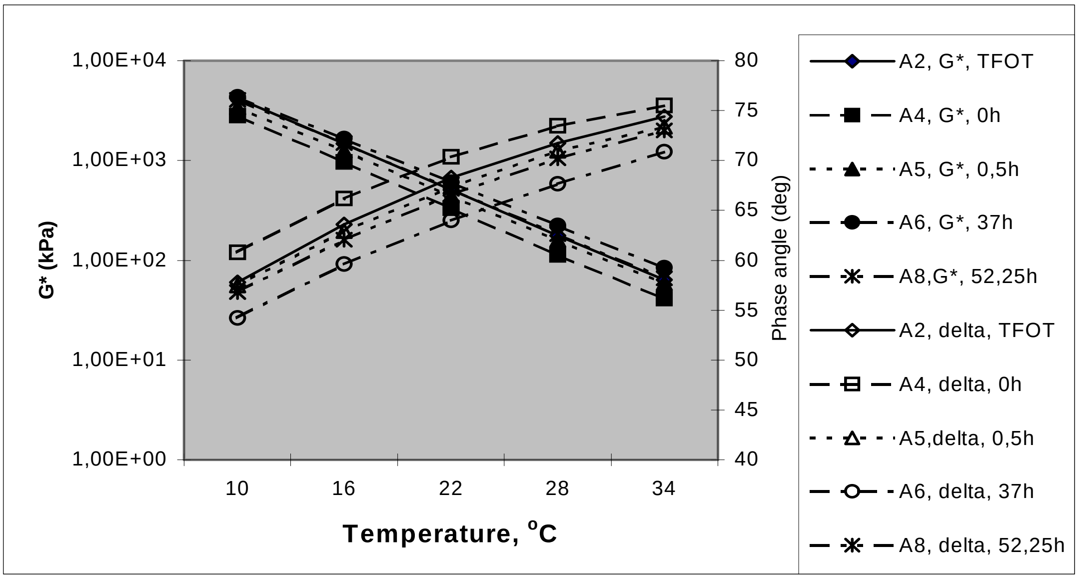


Figure 4.9 *Complex modulus and phase angle, Pen180, recovered and TFOT treated. Samples noted with hours (h) are recovered samples (A4-A8). h indicates time in hours after transportation started from mixing-plant.*

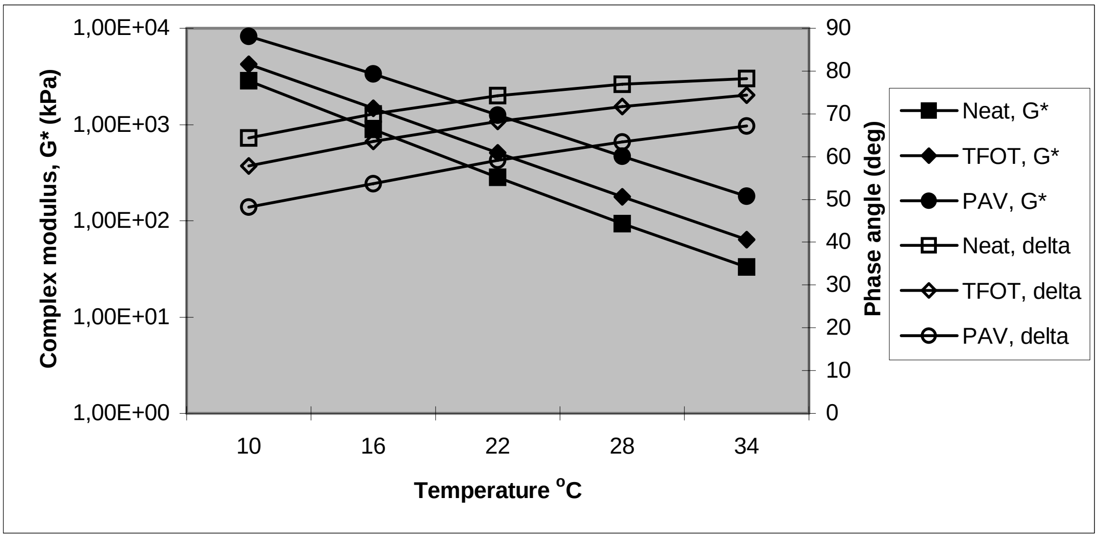


Figure 4.10 *Complex modulus and phase angle of Pen 180, original, TFOT- and PAV-aged.*

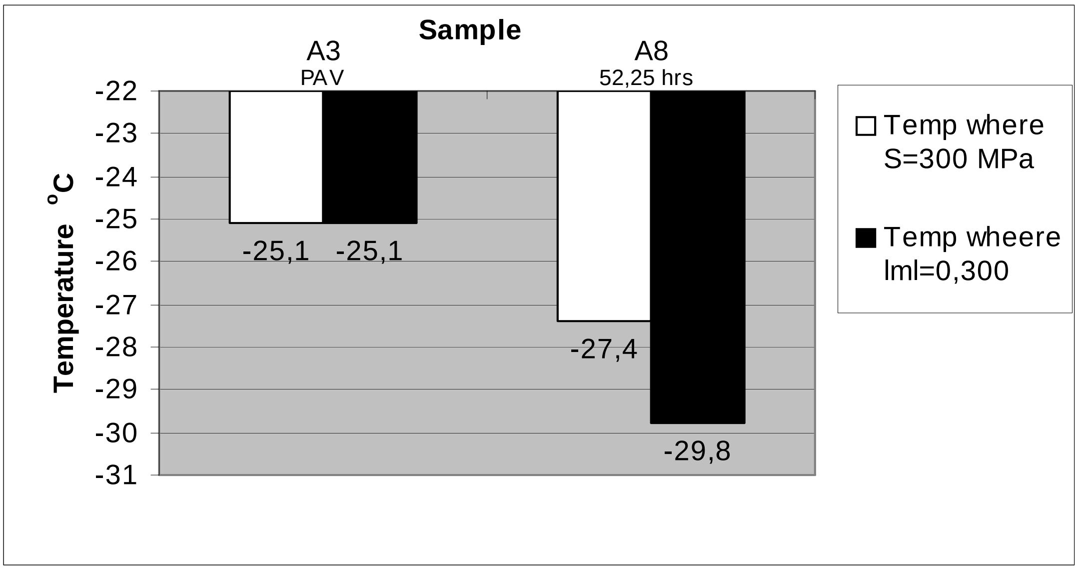


Figure 4.11 BBR analyses of Pen180.

As can be seen from Figure 4.11, the PAV aged sample, based on the SHRP performance grade asphalt binder specification, has a low temperature limit at –25°C (BBR-measurements). Sample A8 is still on the "right side" of this limit.

MB 10000 (soft binder)

Results from analyses on MB10000 are given in Figure 4.12 – 4.16.

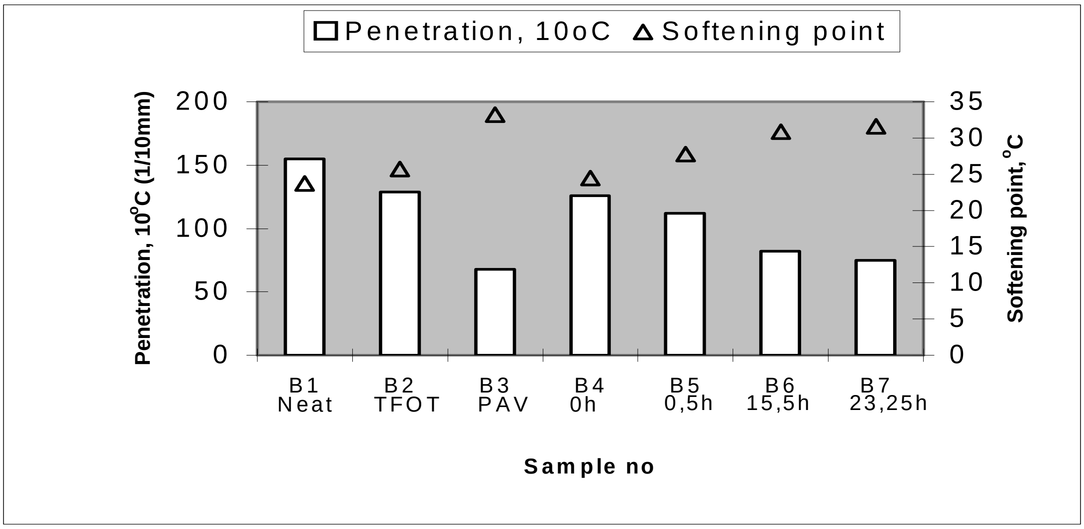


Figure 4.12 Results from penetration and softening point test on MB10000. Samples noted with hours (h) are recovered samples (B4-B7). h indicates time in hours after transportation started from the mixing-plant.

As can be seen in Figure 4.12, recovered binder sampled after laying have penetration values considerably lower than TFOT aged samples. Softening point values are somewhat higher. In this case hardening of the binder increases considerably with increasing time after production. The same trend is seen in Figure 4.13, with viscosity measurements.

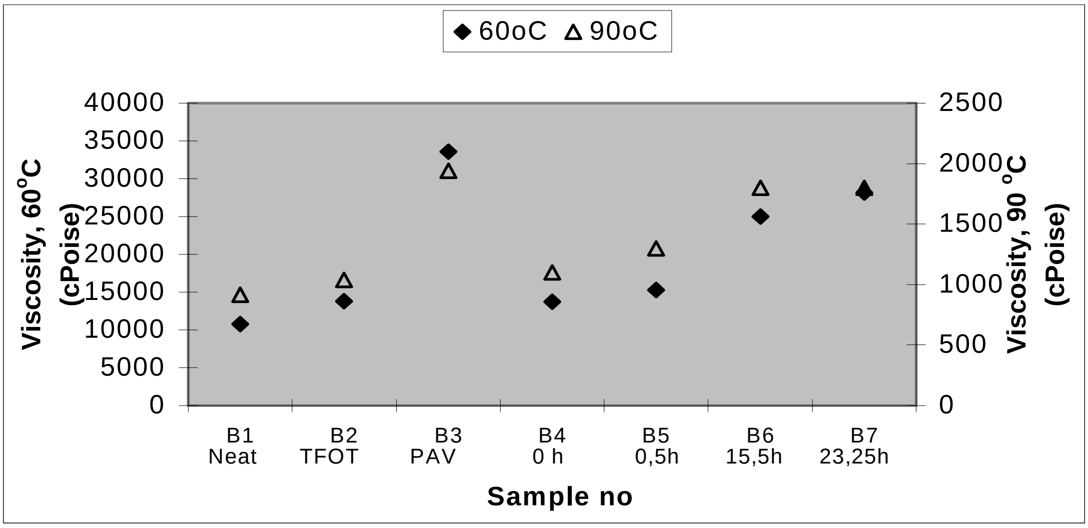


Figure 4.13 Viscosity measured on MB10000 at 60℃ and 90℃. Samples noted with hours (h) are recovered samples (B4-B7). h indicates time in hours after transportation started from mixing-plant.

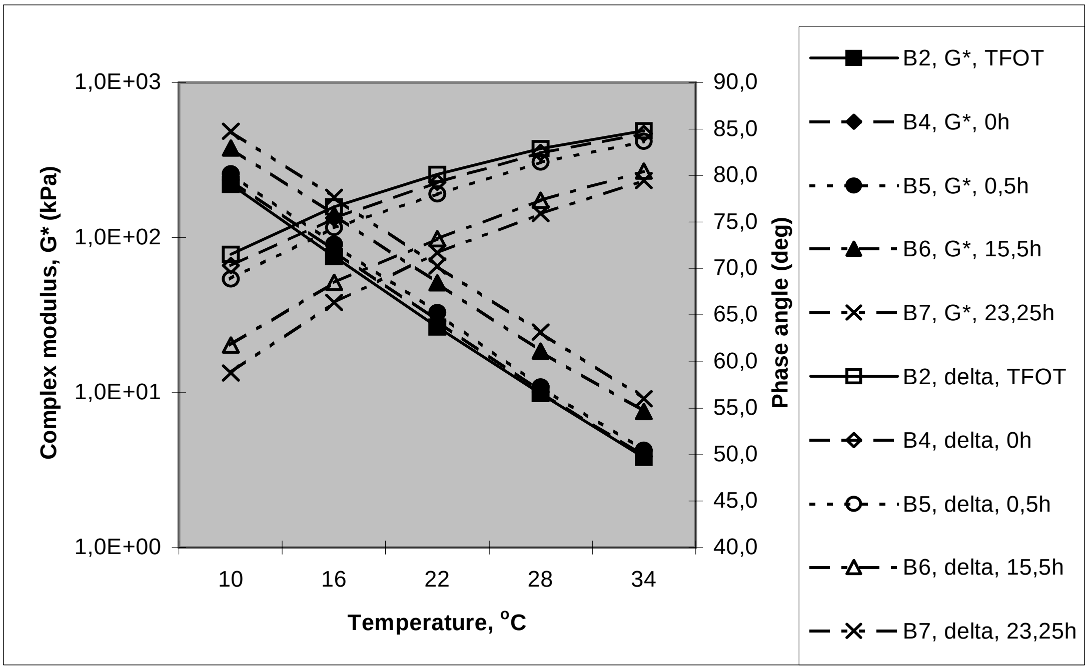


Figure 4.14 Complex modulus and phase angle, MB10000, recovered and TFOT treated. Samples noted with hours (h) are recovered samples (B4-B7). h indicates time in hours after transportation started from mixing-plant.

In Figure 4.14 it can be seen that sample B7 has the highest complex modulus, G*, and the lowest phase angle, δ. The complex modulus is increasing and the phase angle is decreasing with increasing time after production.

As can be seen in Figure 4.14, the TFOT aged sample has lower complex modulus and larger phase angle than all other samples, but very similar to samples B4 and B5.

This is an indication that TFOT-ageing procedure for MB10000 (soft binder) in this case does not cover the ageing effect during transportation. A considerable amount of hardening occurs during the transportation.
In Figure 4.15, the complex modulus and phase angle for laboratory aged binder is shown. In this figure the hardening of binder after TFOT seems to be rather low compared to the hardening after PAV and also compared to original binder.

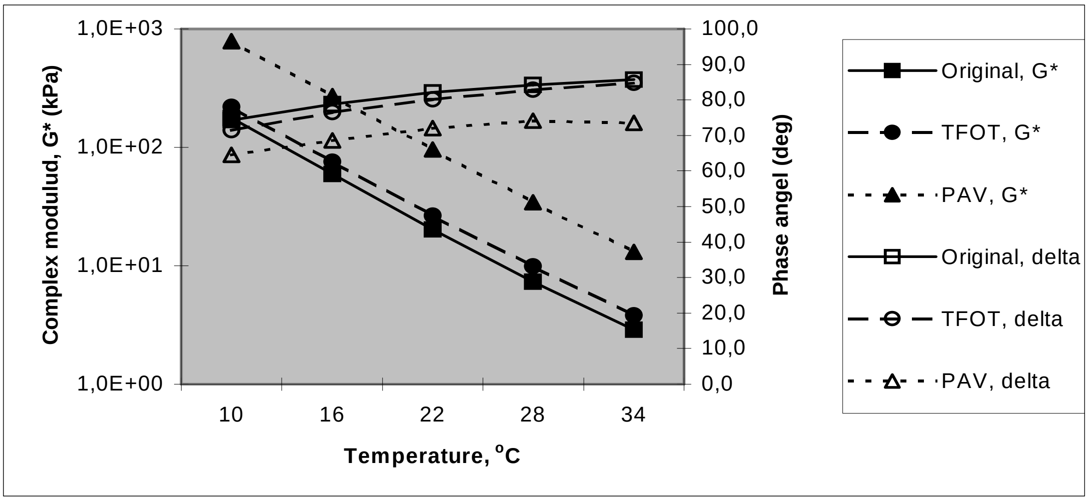


Figure 4.15 DSR measurements on MB10000, original and laboratory aged.

As can be seen in Figure 4.16, the PAV aged sample, based on the SHRP performance grade asphalt binder specification, has a low-temperature limit at –34°C, measured with the BBR. Sample B7 is at the same level as the PAV aged sample. This indicates that long transportation time of asphalt concrete containing MB10000 (soft binder) in some cases can give more hardening of the binder than laboratory ageing. PAV- aged samples are expected to simulate long-time performance on the road.

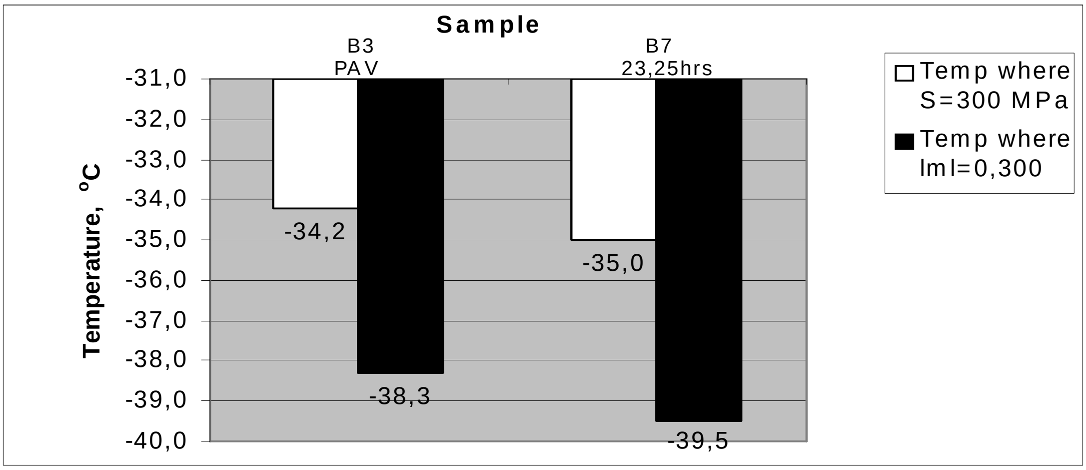


Figure 4.16 BBR analyses of MB10000.

4.5 Discussion

Temperature measurements

Asphalt mixtures with binder Pen180 has a recommended mixing temperature interval in the range 150 – 160°C according to the Norwegian guidelines. The average production temperature was recorded to 162 °C, which is slightly above the upper limit for the recommended production temperature. Generally, the temperature should be in the lower range to prevent hardening. In this case the temperature probably was raised as high as possible to avoid compaction problems after transportation.

Compaction is a critical stage in the process of paving. The compaction brings the constituents of the mixture into intimate contact. Where this does not occur, air voids will remain. The compaction temperature is of great importance for the final result. According to the Norwegian guidelines, the compaction temperature for Agb (asphalt concrete) should be 130 °C. The temperature measurements at sampling show that the temperatures varied between 109 °C and 143 °C. When the temperature is varying the compaction energy has to be adjusted. At low temperatures the compaction energy has to be increased to avoid high void content in the pavement. At high temperatures the compaction energy should be decreased to avoid bleeding.

According to Figure 4.6, the temperature varies with depth within the asphalt-mix during transportation. In the core, the temperature is constant between 147 °C and 149°C. At top and bottom the temperature decreases during transportation. After 40 hours in the freighter, the temperature at 0,5 m from the surface is 113 °C and 0,5 m from the bottom the temperature is 98 °C. This indicates that variations in the void content in the final pavement caused by temperature variations in the mixture can occur depending on the location of the asphalt mixture in the freighter during transportation. High void content in the final pavement will cause faster bitumen hardening. In Figure 4.17, the effect of void content on the hardening of bitumen on the road after 5 years in service is shown [54].

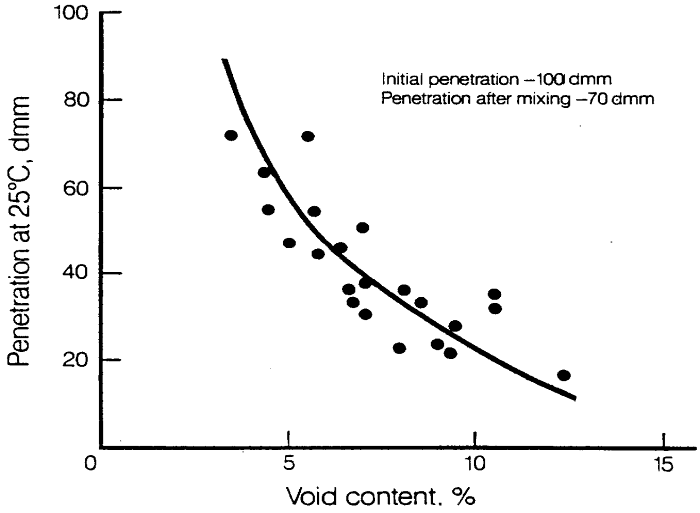


Figure 4.17 The effect of void content on the in-situ hardening of bitumen in five-year-old asphalt concrete [54].

When the transportation has a duration of more than 24 hours, it may be recommend to blend the asphalt mixture from different levels in the stowage in order to get a homogenous pavement. The need for this will also depend on the type of freighter and how the stowage is insulated. This was not done in this actual case.

Bitumen analyses

Pen180

It can be seen that the stiffness of the recovered binder is about the same as that of TFOT treated binder (Ref. figures 4.7 – 4.9). It seems like the transportation of the asphalt-mix (Agb16) with Pen 180 doesn't cause excessive hardening of the bitumen. Figure 4.18 shows a possible increase in viscosity for binders in different levels in the load (A6 from top, A7 from middle and A8 from bottom).

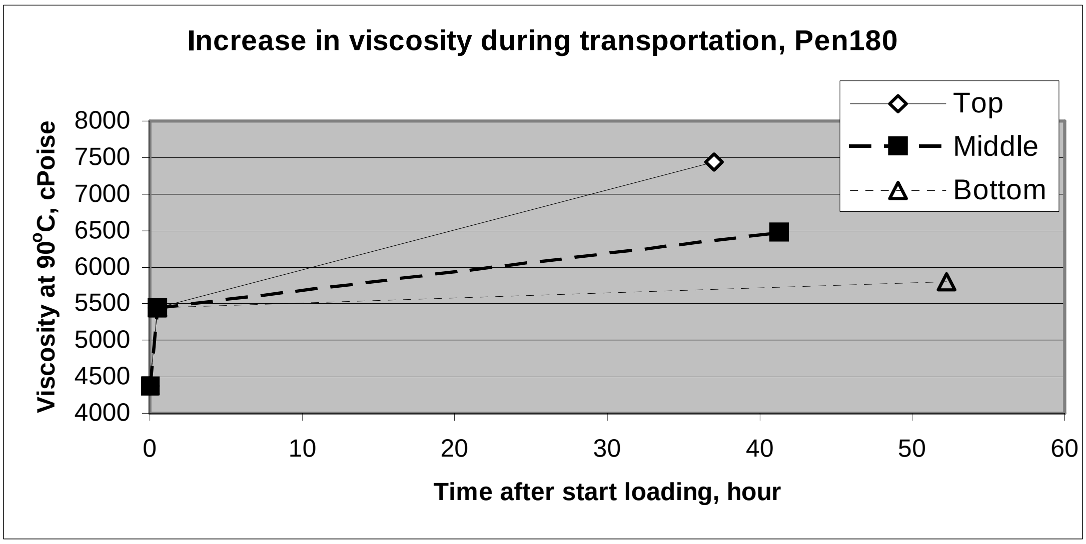


Figure 4.18 Increase in viscosity for binder Pen180 at different levels in the stowage.

Sample A6 has the highest increase in stiffness. This sample is taken from the surface of the asphalt-mixture in the stowage. This could be explained from the fact that this sample has continuously been exposed to air during transportation, and thus has been oxidised more severely.

Some hardening occurs during loading, due to evaporation. Figure 4.19 shows a situation where the evaporation is evident.

Figure 4.19 Evaporation from asphalt-mix during loading.

An ageing index is calculated using the complex modulus (at 10, 16, 22, 28 and 34 °C) and viscosity measurements (at 90°C and 135°C). Figure 4.20 shows that the ageing index differs quite a lot between different samples and are also depending on the test temperature.

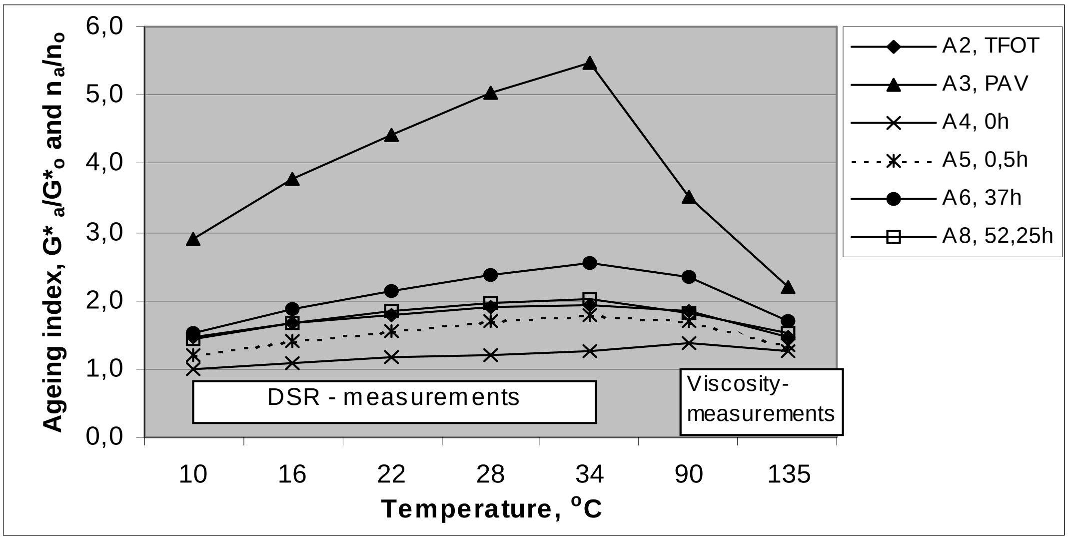


Figure 4.20 Ageing indices for Pen180. Ageing indices are calculated as the ratio of G^*_a (aged) to G^*_o (original) at 10, 16, 22, 28 and 34°C. At 90 and 135°C the ageing indices are calculated as ratio of η_a (aged) to η_o (original). G is based on DSR-measurements and η is based on viscosity measurements. Samples noted with hours (h) are recovered samples (A4-A8). h indicates time in hours after transportation started from mixing-plant.*

MB 10000 (Soft binder)

When using MB10000, the temperature at production should be between 110 and 130°C according to the Norwegian guidelines for asphalt production. The average production temperature for this mixture was 125 °C. The temperature recorded during the paving operation was between 84 °C and 108 °C, which is too much variation to get a homogeneous result.

As can be seen in Figure 4.12 – 4.14, the hardening of the binder increases in severity from sample B4 to sample B7.

The ageing indices calculated for MB 10000 are shown in Figure 4.21.

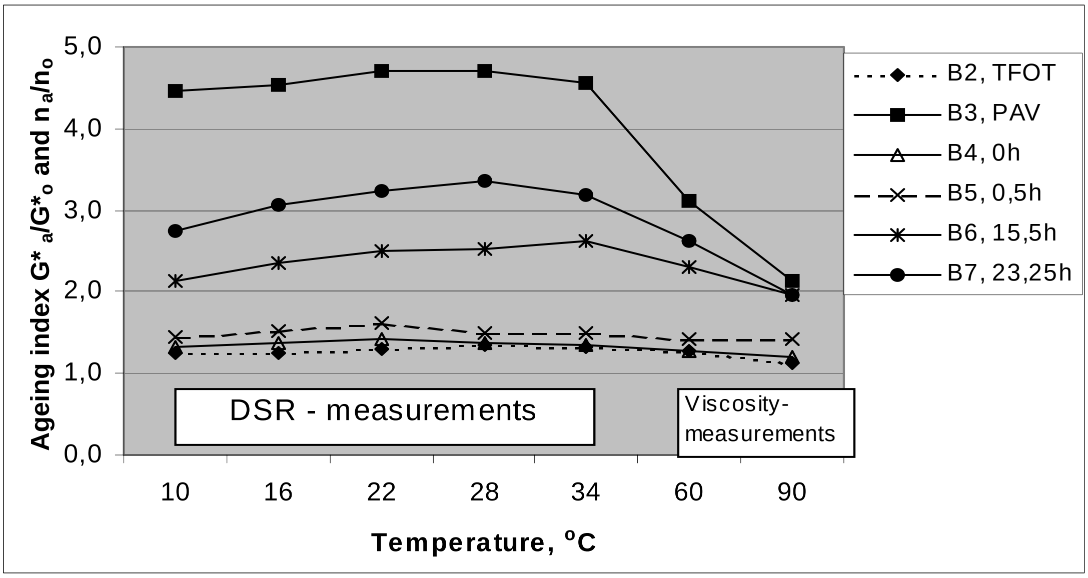


Figure 4.21 Ageing indices for MB 10000. Ageing indices are calculated as the ratio of G^*_a (aged) to G^*_o (original) at 10, 16, 22, 28 and 34 ℃. At 60 and 90 ℃ the ageing indices are calculated as the ratio of η_a (aged) to η_o (original). G is based on DSR-measurements and η is based on viscosity measurements. Samples noted with hours (h) are recovered samples (B4-B7). h indicates time in hours after transportation started from mixing-plant.*

As can be seen, the ageing index in situ for MB 10000 is somewhat higher than that of Pen 180 at most temperature levels.

4.6 Conclusions

Logging of temperatures over time in asphalt-mixtures during transportation in a freighter has given these observations, suggestions and recommendations:

- The temperature almost remained constant in the core of the asphalt-mixture up to as long as 40 hours of transport.
- The temperature 0,5 m over the bottom and 0,5 m below the top of the asphalt-mixture decreased by 1,34°C/hour and 1,2°C/hour, respectively.

- To get a homogenous pavement, it is recommended to mix the asphalt from different levels in the load. This is due to the fact that the temperature at compaction will differ significantly when taking the asphalt-mixture directly from the load without any mixing. High and varying void content in the pavement could be the result when the compaction temperature is too low. High void content could increase the ageing process in service.

From the results of the binder analyses we will conclude:

- For the Pen180 the long transportation time does not seem to contribute to excessive hardening. An increased hardening seems to occur mainly in the top of the load. This is due to the fact that the asphalt-mixture in the top is most exposed to oxidation and evaporation.
- MB10000 has hardened significantly during the transportation. The ageing index was higher at most temperature levels for MB 10000 compared to Pen180. This was the case although the transportation time was almost twice as long for the Pen180 compared to the MB10000.
- If the TFOT should simulate the binder properties in the pavement after production, the testing conditions does not seem to be rough enough compared with hardening of MB10000 during mixing and transportation; that is the situation in situ right after paving. The hardening of recovered binder from the sample taken in the mixing-plant is about the same as the TFOT-treated binder. It will not be practical to adjust the TFOT conditions to varying transportation lengths. In principal, the choice of MB-binders can be simplified as shown in Table 4.4. Here the choice of binder should be dependent on the transportation time. At long transportation time it is recommended to use softer binders to compensate for the excessive hardening effect during transportation.

Table 4.4 Selection of MB-graded binders in principal for long transportation time of asphalt mixture.

MB-graded binder at “normal transportation time”	**MB-graded binder at “long transportation time” (several hours)**
MB 10000	MB 8000
MB 8000	MB 6000
MB 6000	MB 3000

CHAPTER 5 *A LABORATORY STUDY OF FILLER-BITUMEN MIXES*

5.1 Background

It is well known that age hardening of asphalt mixes is a factor contributing to the deterioration of asphalt pavements. This is a major problem on airfields and low-volume roads in Norway.

Filler is the most fine-grained mineral in asphalt mixes. The surface area of the filler material represents a major part of the total surface area of the aggregate in an asphalt mixture. The filler material has a large contact area to the binder and the filler material can therefore influence considerably on the binder properties.

5.2 Objective

The objective of this investigation has been to evaluate different fillers and their contribution to the performance of bitumen during pavement lifetime. The primary goal of the study is to:

- Evaluate the properties of different fillers.
- Examine rheological properties of different filler-/bitumen mixes. In particular make comparative evaluation of mixes containing hydrated lime filler and mixes containing limestone and two other baghouse fillers from Norway. Hydrated lime is in this investigation evaluated as “ordinary filler” and not as additive to the binder.
- Examine rheological properties in filler-bitumen mixes with different filler contents.
- Examine ageing properties of different filler-bitumen mixes with different filler contents.
- Use of the SHRP-binder test equipment in the laboratory analyses and interpretation.

5.3 Test program

5.3.1 Materials

Two bitumen qualities and four different types of fillers are used in this study.

The bitumen used in the laboratory tests was obtained from Nynäs, Sweden. The bitumen is produced from Laguna crude oil. Bitumen used in the investigation consists of Pen180 (penetration grade 180) and MB10000 (viscosity at 60ºC in mm^2/s). These are bitumen types commonly used in Norway. Some selected binder characteristics for these two materials are given in Table 5.1.

Table 5.1 Characteristics of binder used in analysing filler-bitumen mixes.

Binder	**Pen (25°C)**	**Viscosity 60 °C (cPoise)**	**Softening point, °C**	**Specific density G/cm^3**
B180	158		37,4	1,011
MB10000		11200	24,2	1,002

The filler materials used in this investigation consists of two commercial fillers, limestone and hydrated lime, and two baghouse fillers from two Norwegian mixing plants, Lia (central Norway) and Norstone from Tau (western Norway). Limestone is commonly used as filler in asphalt mixtures in Norway. In other countries, hydrated lime has been used as additive to the binder. In the literature, hydrated lime has been referred to as an ageing inhibitor. In this investigation hydrated lime is used as "ordinary filler". Two baghouse fillers from different parts of Norway have been used in the investigation to compare baghouse fillers to limestone and hydrated lime, and to compare properties of different baghouse fillers.

5.3.2 Production of filler-bitumen-mixes

5.3.2.1 Calculation of filler content

It is well known that the viscosity of a bitumen/filler-mastic is a function of filler concentration, filler type, and viscosity of the bitumen. In this work it has been an objective to produce samples of the same stiffness with different fillers and binders. It is a well established correlation between Rigden void content and stiffness in the bitumen/filler mix. Hence "effective filler concentration" has been calculated using Rigdens method.

<u>Effective filler concentration</u> can be defined as the concentration of filler in a bitumen/filler mix, which gives the same stiffness for different fillers. The apparent filler concentration can be calculated using equation 5.2. The Rigden test enables determination of the air voids in dry compacted filler and the filler density, ρ_d, can be calculated using equation 5.4. The concentration of compacted filler, C_{vmax}, can be calculated using equation 5.3. At last the <u>effective filler concentration, C_e</u>, can be calculated using equation 5.1

The filler-content in the mix was calculated using the following equations:

Effective filler concentration: $C_e = C_v/C_{vmax}$ *(5-1)*

Volume concentration: $C_v = V_{filler}/(V_{filler}+V_{binder})$ *(5-2)*

V_{filler} = volume of filler
V_{binder} = volume of bitumen

Concentration compacted filler: $C_{vmax} = \rho_d/\rho_s$ *(5-3)*
$\rho_d = \rho_s\,(100-V_o)/100$ *(5-4)*

ρ_s = specific density
V_o = void in compacted filler (Rigden)

Volume of filler, $V_{filler} = C_v * V_{binder}/(1-C_v)$ *(5-5)*

Mass of filler, $m_{filler} = V_{filler} * \rho_s$ *(5-6)*

The mass of filler in a sample, which consists of 65 gram MB10000 is given in Table 5.2 for different types of fillers and different effective filler concentrations.

Table 5.2 Mass of filler [g] in a sample, which consist of 65 g MB10000. Effective filler concentration, C_e, is 0,1, 0,3 and 0,6.

Filler material	**C_e = 0,1**	**C_e = 0,3**	**C_e = 0,6**
Hydrated lime	6,1	19,8	45,8
Limestone	13,1	47,0	123,0
Lia	11,5	38,2	100,4
Tau	12,4	42,4	110,8

As can be seen in Table 5.2, the contents of filler materials expressed by mass (g) vary considerably in a given mass of bitumen for the same mix stiffness. It can be seen that the mass content of hydrated lime is very low. This is directly linked to the fact that hydrated lime has a high Rigden void content.
An effective filler concentration in a typical Norwegian asphalt mixture is close to 0,6.

5.3.2.2 Mixing process

In this investigation, an attempt has been made to simulate the plant mixing process. In a plant the mixing takes 30 – 60 seconds and aggregates are mixed before the binder is added.

In the laboratory the mixing was performed using a cup and a glass stick as shown in Figure 5.1. The mixing process was defined as finished when the mix visually became homogenous. In this case this normally took from 1 – 3 minutes. The homogeneity is controlled as shown in chapter 5.4.2.6.

Figure 5.1 Mixing of filler and bitumen using cup and glass stick.

The treatment of the binder and the filler before sample preparation is described in Appendix 8.

5.3.3 Tests on filler materials

The purpose of the tests on the pure filler materials is to characterise and classify the materials. The results from the pure filler analyses will also be used in evaluating the result from the binder-filler analyses. The tests performed on pure filler material are shown in Table 5.3.
The Rigden test was performed at department of Road and Railway Engineering at NTNU. The rest of the tests were performed at SINTEF, Civil and Environmental Engineering, department of Rock and Mineral Engineering.

Table 5.3 Tests on pure filler material.

Analysis	**Hydrated lime**	**Lime stone**	**Filler from Lia**	**Filler from Tau**
Filler grading curves	Coulter LS230	Coulter LS230	Coulter LS230	Coulter LS230
Mineral-composition	Roentgen diffraction – XRD.	Roentgen diffraction – XRD.	Roentgen diffraction – XRD.	Roentgen diffraction – XRD.
Particle-shape	Microscopic investigation.	Microscopic investigation.	Microscopic investigation.	Microscopic investigation.
Surface area	BET-analyses (Flowsorb II 2300) Blain-test	BET-analyses (Flowsorb II 2300) Blain-test	BET-analyses (Flowsorb II 2300)	BET-analyses (Flowsorb II 2300)
Specific density	Accupyc 1330	Accupyc 1330	Accupyc 1330	Accupyc 1330
Void content	Rigden-method (prEN 1097-4).	Rigden-method (prEN 1097-4).	Rigden-method (prEN 1097-4).	Rigden-method (prEN 1097-4).

Filler grading curves

The filler materials are first sieved out on a 75μm Derrick-sieve. Filler from Tau had to be crushed before sieving. Grading was performed on both total samples and samples sieved on a 75μm sieve, using Coulter LS 230. Using Coulter LS 230, laser light is used and light scatter patterns indicate different sized particles. The samples in this investigation are produced of filler less than 75μm.

Mineral composition

The mineral composition is identified using roentgen diffraction, XRD-analyses.

Particle shape

Using microscope, the particle shape of the filler materials was studied. Pictures of the samples were taken in the microscope with an objective of illustrating the difference in particle shape of the various samples.

Surface area

Specific surface area was measured using FlowSorb II 2300 after the BET-method. (DIN 66131, ASTM D 4567-86).
Specific surfaces for hydrated lime and limestone were also analysed using Blaine.

The FlowSorb 2300 (BET-method) is designed to measure the surface area on the molecular level of powdered materials and the total pore volume as well as the wall area. In the BET-method the surface area is measured as absorbed nitrogen (gas) on the surface. Nitrogen-gas consists of small particles and will therefore penetrate into small pores in the sample. Measuring the surface area using this method will normally give large surface areas.
In the Blain-method the air-permeability is measured. The air will pass at the sides of the particles. The air will not penetrate into pores and the surface area in the pores will therefore not be measured.
For porous material the surface area measured by the BET-method will give greater values than measured by the Blain-method.

Specific density

Specific density was analysed using Accupyc 1330. The Accupyc 1330 pycnometer is a gas displacement pycnometer, a type of instrument, which measures the volume of solid objects of irregular or regular shape whether powdered, or in one piece. The apparatus is shown in Figure 5.2.

Figure 5.2 Accupyc 1330, apparatus used to determine specific density for filler materials.

Void Content (Rigden-method)

The void content was measured using the modified Rigden–method as described in the Norwegian standards [90]. The test is based on prEN 1097-4: *Tests for mechanical and physical properties of aggregates – Determination of the voids of dry compacted filler*.

5.3.4 Tests on filler-bitumen mixes

In this investigation the primary goal was to evaluate the ageing properties of different filler-bitumen-mixes. The mixes have been tested at a temperature range relevant for the field situation. At critical high temperature situation the Ring and Ball-test has been used. At critical low temperature situation the SHRP-test methods, Direct Tension Test (DTT) and Bending Beam Rheometer (BBR) have been used. The Dynamic Shear Rheometer (DSR) has been used in the intermediate temperature range (10, 20 and 30°C) with a frequency of 1,5 Hz. This frequency is close to the frequency in the SHRP Performance Grade Asphalt Binder Specification. The test program is shown in Table 5.4.

Table 5.4 Testing program of filler-bitumen-mixes.

Material/ Ageing condition	Effective filler concentration, C_e	Softening point (C^o)	DSR (1.5Hz) 10 ^oC G^*/δ	DSR (1.5Hz) 20 ^oC G^*/δ	DSR (1.5Hz) 30^oC G^*/δ	Direct tension,*) -25/-20^oC	BBR*) -25/-20 ^oC S/m
MB		X	X	X	X	X	X
MB TFOT		X	X	X	X		
MB PAV		X	X	X	X	X	X
MB-KS	0,3	X	X	X	X	X	X
MB-KS TFOT	0,3	X	X	X	X		
MB-KS PAV	0,3	X	X	X	X	X	X
MB-KS	0,6	X	X	X	X	X	X
MB-KS TFOT	0,6	X	X	X	X		
MB-KS PAV	0,6	X	X	X	X	X	X
MB-HL	0,3	X	X	X	X	X	X
MB-HL TFOT	0,3	X	X	X	X		
MB-HL PAV	0,3	X	X	X	X	X	X
MB-HL	0,6	X	X	X	X	X	X
MB-HL TFOT	0,6	X	X	X	X		
MB-HL PAV	0,6	X	X	X	X	X	X
MB-FL	0,3	X	X	X	X	X	X
MB-FL TFOT	0,3	X	X	X	X		
MB-FL PAV	0,3	X	X	X	X	X	X
MB-FL	0,6	X	X	X	X	X	X
MB-FL TFOT	0,6	X	X	X	X		
MB-FL PAV	0,6	X	X	X	X	X	X
MB-FT	0,3	X	X	X	X	X	X
MB-FT TFOT	0,3	X	X	X	X		
MB-FT PAV	0,3	X	X	X	X	X	X
MB-FT	0,6	X	X	X	X	X	X
MB-FT TFOT	0,6	X	X	X	X		
MB-FT PAV	0,6	X	X	X	X	X	X
B		X	X	X	X	X	X
B TFOT		X	X	X	X		
B PAV		X	X	X	X	X	X
B-KS	0,3	X	X	X	X	X	X
B-KS TFOT	0,3	X	X	X	X		
B-KS PAV	0,3	X	X	X	X	X	X
B-HL	0,3	X	X	X	X	X	X
B-HL TFOT	0,3	X	X	X	X		
B-HL PAV	0,3	X	X	X	X	X	X
B-FL	0,3	X	X	X	X	X	X
B-FL TFOT	0,3	X	X	X	X		
B-FL PAV	0,3	X	X	X	X	X	X
B-FT	0,3	X	X	X	X	X	X
B-FT TFOT	0,3	X	X	X	X		
B-FT PAV	0,3	X	X	X	X	X	X

MB = MB10000, B = Pen180, KS =Limestone, HL=hydrated lime, FL =filler from Lia, FT=filler from Tau, TFOT = after Thin Film Oven Test, PAV= after TFOT + Pressure Ageing Vessel, *) Pen180:-20°C and MB10000=-25°C

Laboratory ageing

Ageing of filler-bitumen-mixes in laboratory was performed using standardised TFOT and modified PAV-tests.

TFOT

For mixes containing Pen180 the TFOT was performed at 163°C, while TFOT was performed at 120°C for mixes containing MB10000. The TFOT pans were filled with 50 cm^3 of filler-bitumen-mix, a similar volume of the "standard" TFOT-test for bitumen. Loss of mass was measured after TFOT.

PAV

PAV was performed in 7 days at 60°C. The pressure in the PAV was 2,1 MPa. PAV was performed at these conditions due to the fact that the temperature in the pavement on Norwegian roads will never or very seldom reach a temperature above this level. Another important reason for the choise of these test conditions is the fact that different reactions could occur between filler material and bitumen at higher temperatures, temperatures that do not occur under field conditions.

Softening point (Ring and ball)

The ring and ball test was performed in a standardised way. Water was used in the bath. The test equipment used is provided with a stirrer.

Dynamic Shear Rheometer(DSR)

In this investigation a 15-mm spindle was used with 1,5 mm gap. Measurements were made at 10, 20 and 30°C. The gap was set at 20°C. An attempt was made to perform all the testing at sufficiently low strains so that linear visco-elastic-behaviour was obtained. To be sure that measuring took place in the linear region, stress sweep were performed at all measurements. A frequency of 1,5 Hz was used during all measurements. An example of results from a stress sweep is shown in Figure 5.3.

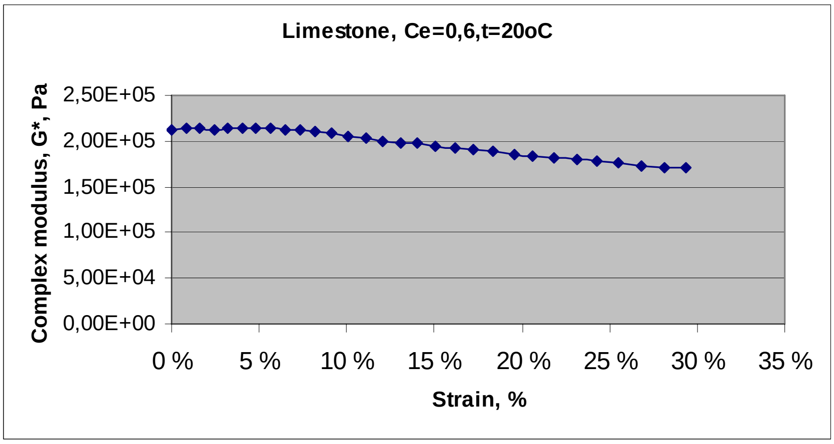


Figure 5.3 Stress sweep, DSR at 20℃, MB10000 with limestone and effective filler concentration, $C_e = 0,6$.

Bending Beam Rheometer (BBR)

The Bending Beam Rheometer (BBR) was used to characterise the low temperature properties of the filler-bitumen mixes. The BBR was performed on original (no artificial ageing) and after PAV ageing of the mixes. For mixes with Pen 180 the test was performed at -20°C and for mixes with MB10000 at -25°C.

Direct Tension Test (DTT)

Like the bending beam rheometer (BBR), the direct tension test (DTT) was used to characterise the low temperature properties of the filler-bitumen mixes. The DTT was performed on original (no laboratory ageing) and after PAV ageing of the mixes. For mixes with Pen 180 the test was performed at -20°C and for mixes with MB10000 at -25°C. Preparation of DTT-samples is shown in Figure 5.4.

Figure 5.4 Preparation of DTT-samples.

5.4 Results

5.4.1 Tests on filler materials

5.4.1.1 Filler grading curves

Figure 5.5 shows grading curves of total samples for hydrated lime, limestone and filler from Lia.

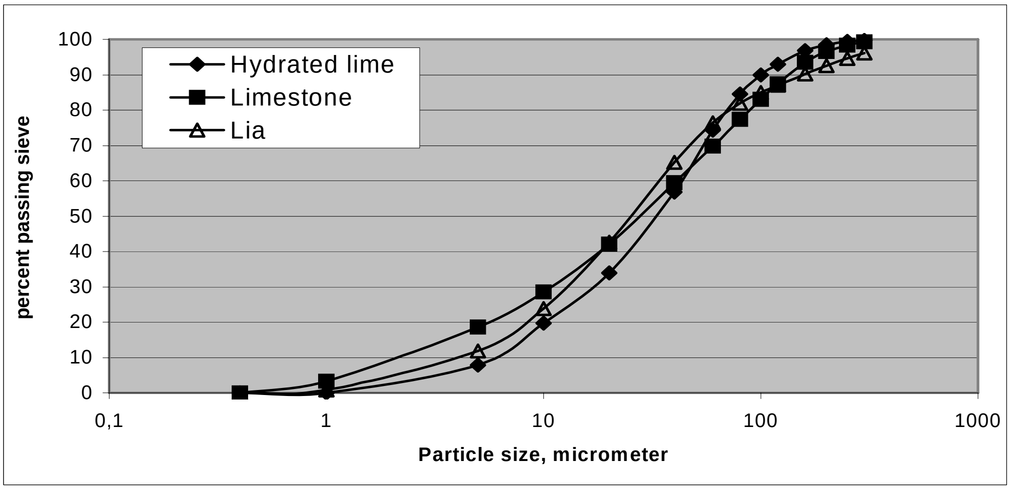


Figure 5.5 Grading curves on total samples of hydrated lime, limestone and filler from Lia.

Dry sieving was performed on total sample from Tau. Grading curve on total sample of filler from Tau is shown in Figure 5.6.

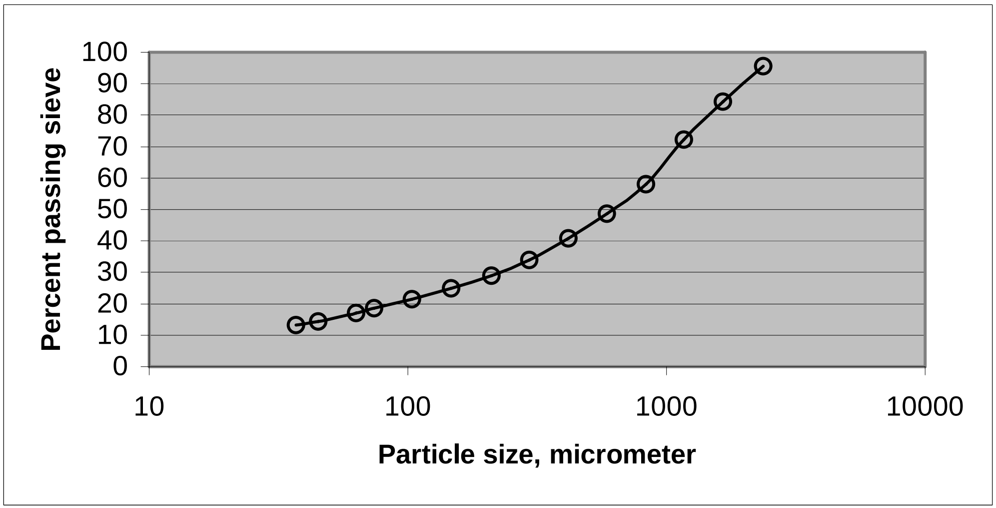


Figure 5.6 Grading curve of total sample from Tau.

All filler materials were sieved on a 75 micrometer sieve to satisfy the upper criterion of filler material given in the Norwegian standard for filler material. The Norwegian Standard filler criterion is given in chapter 1.5.7.

As can be seen in Figure 5.7, the grading curves for sieved material are similar for limestone, filler from Lia and Tau. The grading curve for hydrated lime is somewhat different from that of the other three, in that it contains higher proportions of the larger size fractions.

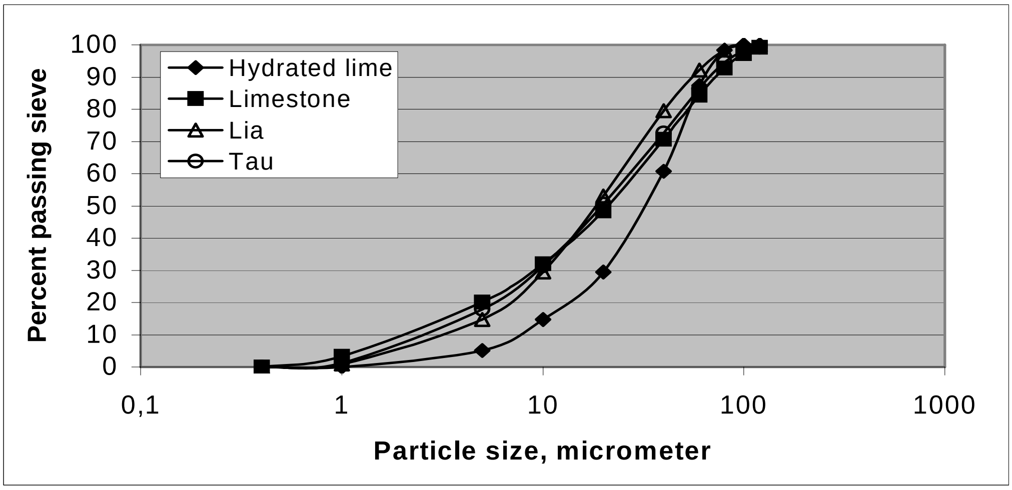


Figure 5.7 Grading curves of material sieved on a 75μm sieve.

The particle distribution is almost the same for limestone and Tau, with D_{50} and D_{90} of about 20 and 70 micrometer, respectively. For Lia, D_{50} is about 19 micrometer and D90 about 55 micrometer. The hydrated lime differs considerably from the other fillers with D_{50} about 33 micrometer and D_{90} about 62 micrometer. Also at $D_{10,}$ hydrated lime differs significantly from the other fillers. Some of these variations could be due to agglomeration of small hydrated lime particles. The "dark shadows" on the picture of hydrated lime probably illustrates the agglomeration (Figure 5.9). All types of fillers satisfy the filler grading criterion given in The Norwegian Standard.

5.4.1.2 Mineral composition

The mineral composition varies as can be seen in Table 5.5.

Table 5.5 Mineral composition of fillers in percent.

Mineral	Limestone	Hydrated lime	Lia	Tau
Calcite	99	<1	4	1
Portlandite		99		
Chlorite			47	25
Amphibole			23	
Plagioclase			16	46
Quartz	<1		4	14
Mica	<1		6	5
Epidote				9
Feldspar	<1			

5.4.1.3 Particle shape

The particle shape can affect the increase of viscosity in the mastic and also the void content in the asphalt mixture.
The particle shape of the materials is shown in Figure 5.8 to 5.11.

Limestone has the most regular particle shape. The sample from Lia is more elongated than the sample from Tau. The hydrated lime has an obvious different particle shape than the other materials. It consists of porous particles with irregular shape.

The hydrated limes particle-morphology is a function of the production process. The production process consists of calcination of $CaCO_3$ to CaO by removing CO_2 and stabilisation by addition of water to form hydrated lime, $CaO(OH)_2$.

Scale: 0.05 mm

Figure 5.8 Limestone in microscope.

Scale: 0.05 mm

Figure 5.9 Hydrated lime in microscope.

Scale: 0.05 mm

Figure 5.10 Filler from Lia in microscope.

Scale: 0.05 mm

Figure 5.11 Filler from Tau in microscope.

5.4.1.4 Surface area

Adsorption of components from the binder is mentioned as a reason for increasing viscosity caused by filler materials. Large surface area will require higher binder content. The specific surface area is an important parameter in evaluation of filler properties in asphalt mix design. Increased specific surface area can contribute to increased adsorption. The results from the BET test and the Blaine test are shown in Table 5.6.

Table 5.6 Specific surface for filler materials measured after the BET-method and Blaine.

Sample	Specific surface, m^2/g (BET-method)	Specific surface, m^2/g (Blaine)
HYDRATED LIME	11,65	1,22
LIMESTONE	0,88	0,44
LIA	1,69	
TAU	2,22	

The difference between hydrated lime and limestone are less obvious from the results of the Blaine test. The measurement of surface area indicates that hydrated lime is a porous material.

5.4.1.5 Specific density

The results from measurements of specific density are given in Table 5.7.

Table 5.7 Specific density measured using Accupyc 1330.

Sample	Specific density, g/cm^3
HYDRATED LIME	2,2371
LIMESTONE	2,7277
LIA	2,8731
TAU	2,7857

Limestone, Lia and Tau have specific densities in the same level of magnitude. Hydrated lime has considerably lower specific density.

5.4.1.6 Void content (Rigden)

In the literature, it is well documented that there is a correlation between Rigden void content of the filler and the change in viscosity in the filler-/bitumen-mastic. It is also necessary to know the Rigden void content of the filler material to be able to calculate the effective filler concentration in the mix. The results of the void content measurements are given in Table 5.8.

Table 5.8 Void content in filler-materials measured using the Rigden method.

Sample	Rigden void content (%)
HYDRATED LIME	59,83
LIMESTONE	31,15
LIA	42,23
TAU	36,53

Hydrated lime has a considerably higher void content then the other materials. There are also differences in void contents between the three others. As mentioned before, the void content is an important parameter when evaluating the stiffening effect of filler in a filler-bitumen mix. When producing a mix with a certain stiffness, filler with high void content will require lower content (mass content) in the mix compared to filler with low void content.

5.4.2 Tests on filler-bitumen mixes

5.4.2.1 Laboratory ageing

TFOT

Loss of mass was measured after TFOT and the results are given in Figures 5.12 and 5.13 for samples containing Pen180 and MB10000, respectively.

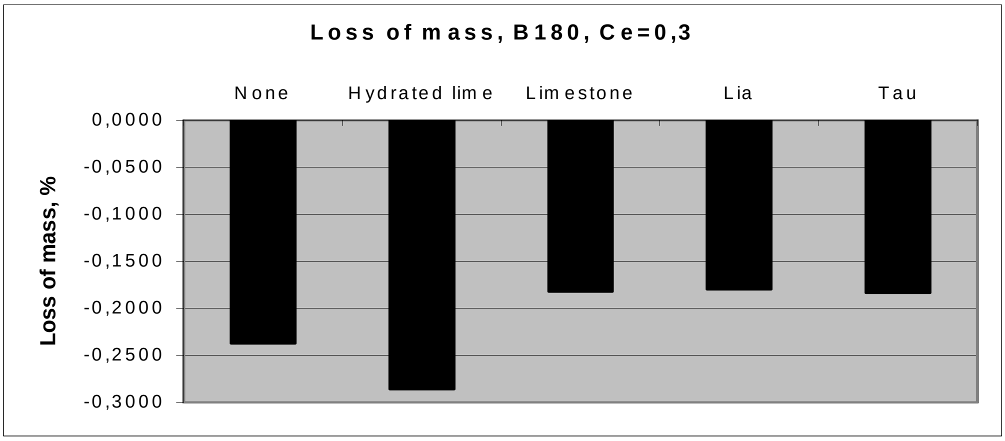


Figure 5.12 Loss of mass after TFOT for samples containing B180 with effective filler concentration, C_e=0,3.

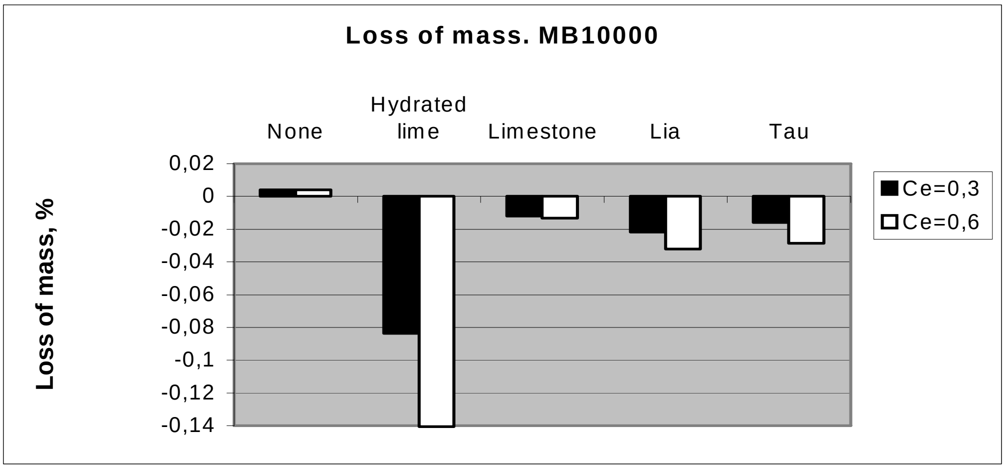


Figure 5.13 Loss of mass after TFOT for samples containing MB10000.

Figures 5.12 and 5.13 show considerably higher loss of mass for mixes containing hydrated lime. This indicates that there is a different reaction in mixes containing hydrated lime compared to the other mixes.

PAV-test

A visual description of samples after PAV ageing for MB10000 with effective filler concentration, C_e=0,6, is shown in Table 5.9.

Table 5.9 Visual description of samples after PAV with binder MB10000 and effective filler concentration, C_e=0,6.

Mastic	**MB10000 + limestone**	**MB10000 + hydrated lime**	**MB10000 + filler from Lia**	**MB10000 + filler from Tau**
Visual evaluation	Much bubbles on surface. Thin layer of binder in top. Much of filler seems to have sedimented. Like a “plate” in bottom	Little bubbles on surface. Homogenous sample.	Big bubbles on surface, not many. Thin layer of binder on surface but more homogenous than limestone and Tau.	Very much bubbles on surface. The sample is like sample with limestone.

A sample of MB10000 with filler from Tau (C_e=0,3) is shown in Figure 5.14.

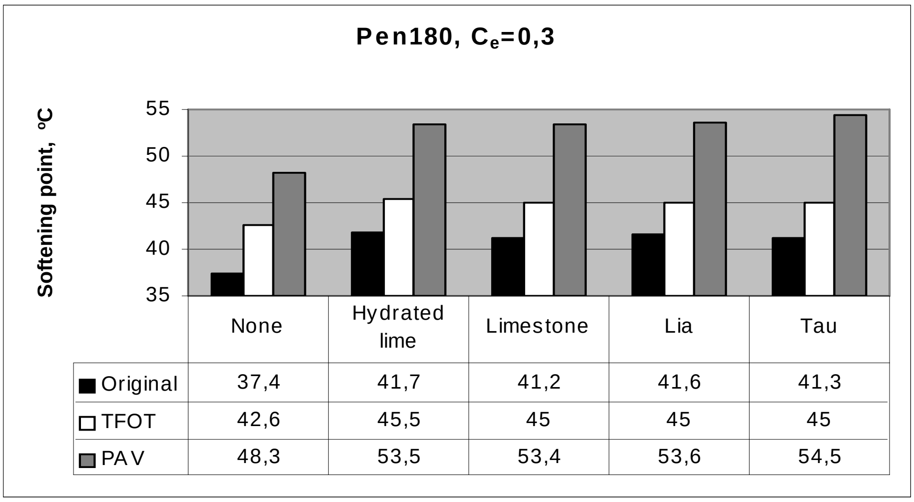
Figure 5.14 MB10000 with filler from Tau (C_e=0,3) after PAV.

5.4.2.2 Softening point (Ring and ball)

Results from analyses of different mixes are given in the following:

Pen180 with an effective filler concentration , C_e = 0,3

Figure 5.15 shows results from the analyses of Pen180 with C_e = 0,3.



	None	Hydrated lime	Limestone	Lia	Tau
■ Original	37,4	41,7	41,2	41,6	41,3
☐ TFOT	42,6	45,5	45	45	45
▨ PAV	48,3	53,5	53,4	53,6	54,5

Figure 5.15 Softening point bitumen-filler mixes containing Pen180 with an effective filler concentration, C_e = 0,3. The softening point is measured on original, TFOT-aged and PAV-aged mixes.

It can be seen from Figure 5.15 that there are small differences between softening point values for various mixes.

Figure 5.16 shows stiffness/ageing index, which is calculated as softening point-value of aged mix divided by softening point-value of unaged mix.

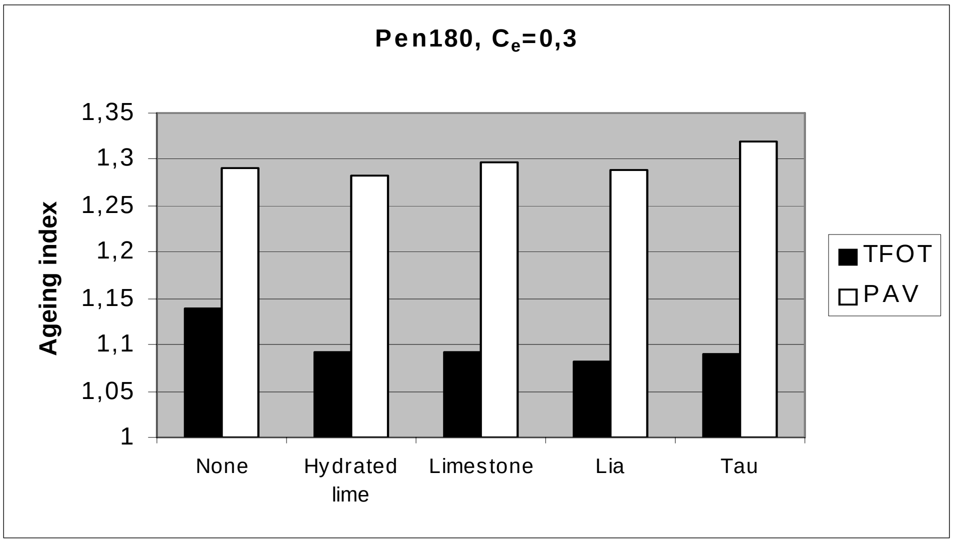
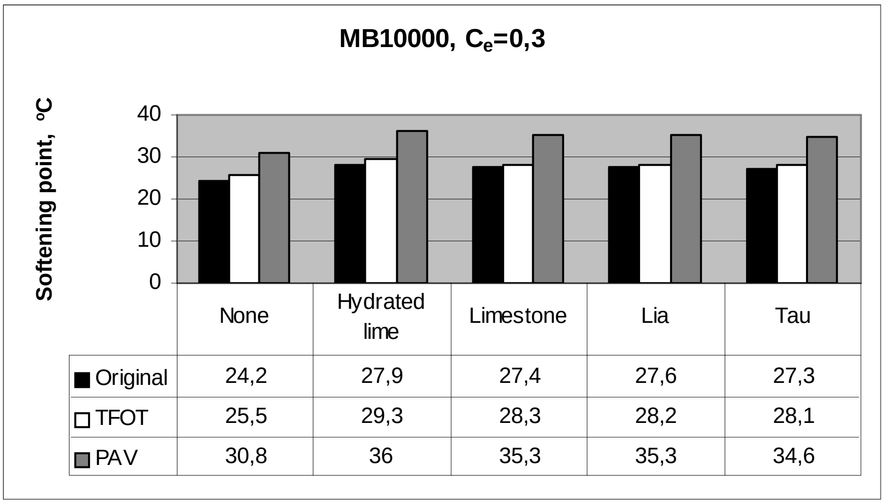


Figure 5.16 Ageing index for mixes containing Pen180 with effective filler concentration, C_e = 0,3. The ageing index is calculated after TFOT and after PAV.

Figure 5.16 indicates that there are small differences in the ageing index between different types of filler.

MB10000 with an effective filler concentration , C_e = 0,3

Results from the analyses are given in Figure 5.17.



	None	Hydrated lime	Limestone	Lia	Tau
■ Original	24,2	27,9	27,4	27,6	27,3
□ TFOT	25,5	29,3	28,3	28,2	28,1
▨ PAV	30,8	36	35,3	35,3	34,6

Figure 5.17 Softening point bitumen-filler mixes containing MB10000 with effective filler concentration, Ce = 0,3. The softening point was measured on original, TFOT-aged and PAV-aged mixes.

Figure 5.17 shows little difference between various types of mixes containing MB10000. Figure 5.18 shows ageing index, which is calculated as softening point-value of aged mix divided by softening point-value of unaged mix.

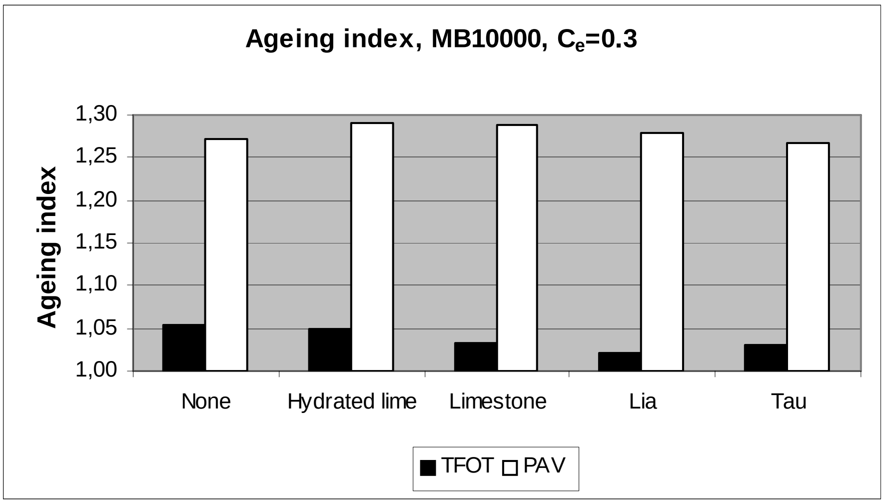
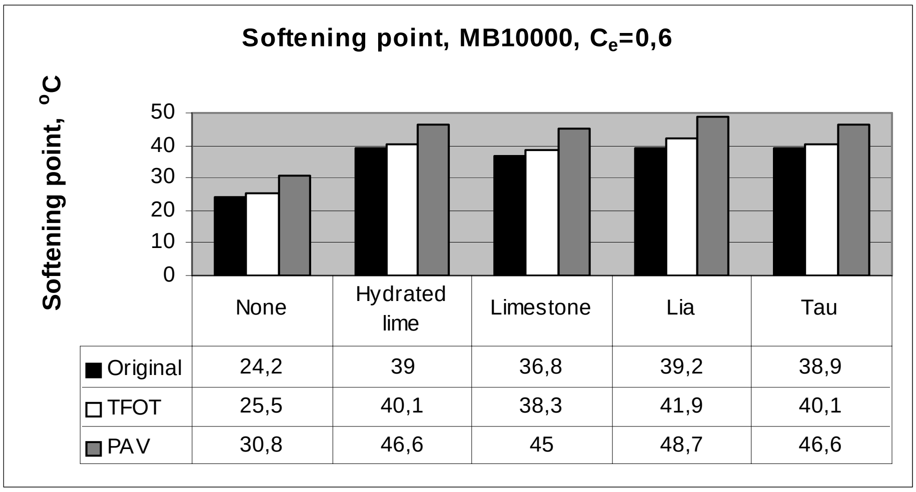


Figure 5.18 Ageing index for mixes containing MB10000 with effective filler concentration, C_e = 0,3. The ageing index was calculated after TFOT and after PAV.

The results shown in Figure 5.18 indicate small differences between various types of filler in the mix.

MB10000 with an effective filler concentration , C_e = 0,6

Results from the analyses are given in Figure 5.19.



	None	Hydrated lime	Limestone	Lia	Tau
■ Original	24,2	39	36,8	39,2	38,9
□ TFOT	25,5	40,1	38,3	41,9	40,1
▨ PAV	30,8	46,6	45	48,7	46,6

Figure 5.19 Softening point bitumen-filler mixes containing MB10000 with effective filler concentration, C_e = 0,6. The softening point was measured on original, TFOT-aged and PAV-aged mixes.

As for effective filler concentration C_e=0,3, Figure 5.19 also indicates little difference between various types of mixes containing MB10000 with effective filler concentration, C_e=0,6.
Figure 5.20 shows ageing index, which is calculated as softening point-value of aged mix divided by softening point-value of unaged mix.

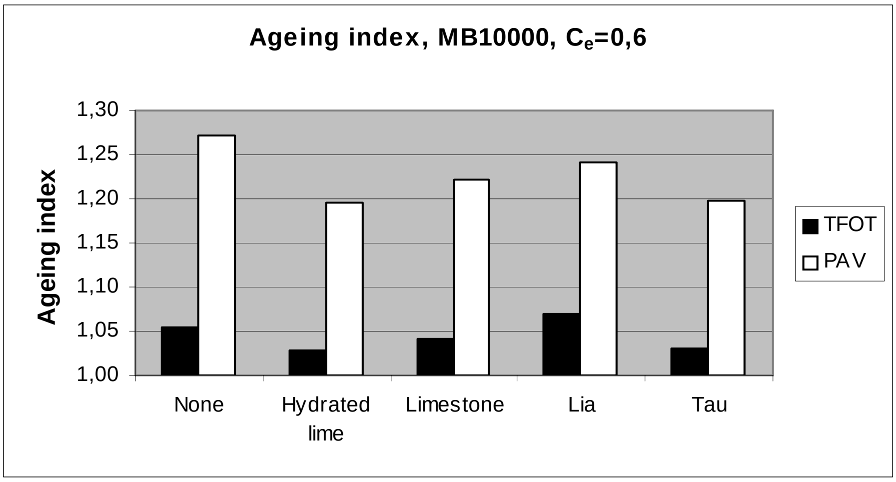


Figure 5.20 Ageing index for mastic containing MB10000 with effective filler concentration, C_e = 0,6. The ageing index was calculated after TFOT and after PAV.

Figure 5.20 shows that the ageing index of mastic containing hydrated lime is slightly lower after TFOT- as well as PAV-ageing. The differences are however small, specially compared to baghouse filler from Tau.

5.4.2.3 Dynamic Shear Rheometer (DSR)

The results from the DSR-measurements are given in Appendix 9.
In the following some selected results are given.

Pen 180–binder with an effective filler concentration, C_e = 0,3

In Figures 5.21 and 5.22 G*/sin δ and G* × sin δ for the various filer-bitumen mixes as a function of temperature is given. In the SHRP Performance Grade Binder Spesification, G*/sin δ and G* × sin δ is the rutting parameter and fatigue parameter, respectively.

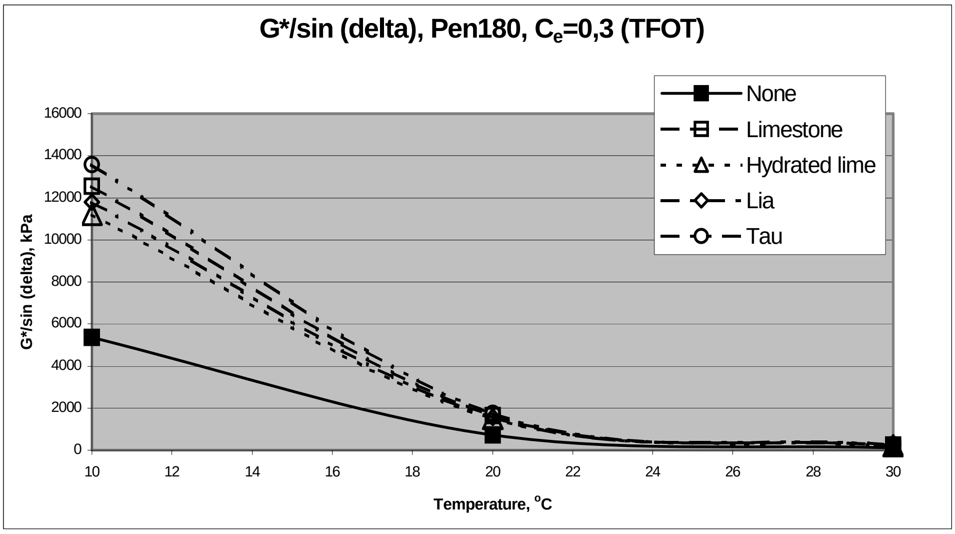


Figure 5.21 The "rutting parameter*", $G^*/\sin\delta$, Pen180, effective filler concentration C_e=0,3, TFOT.*

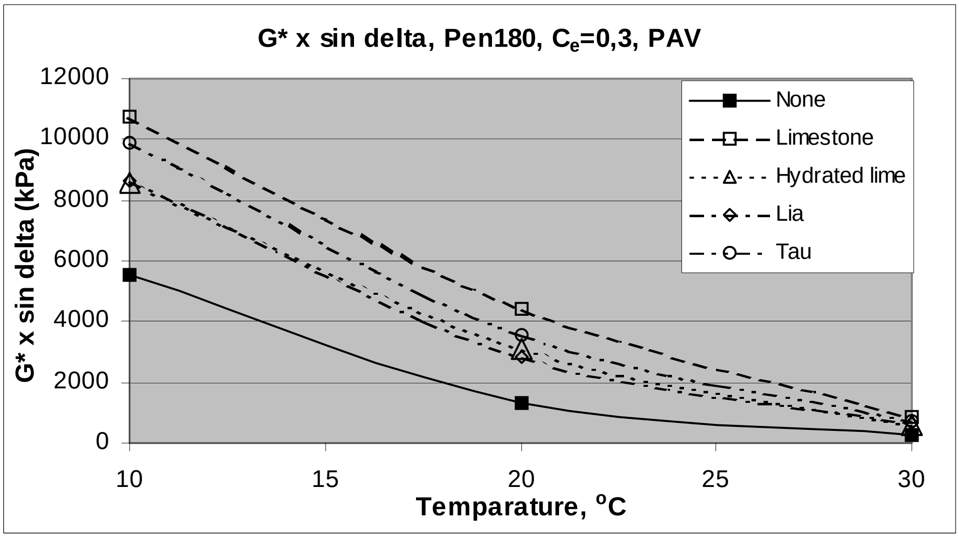
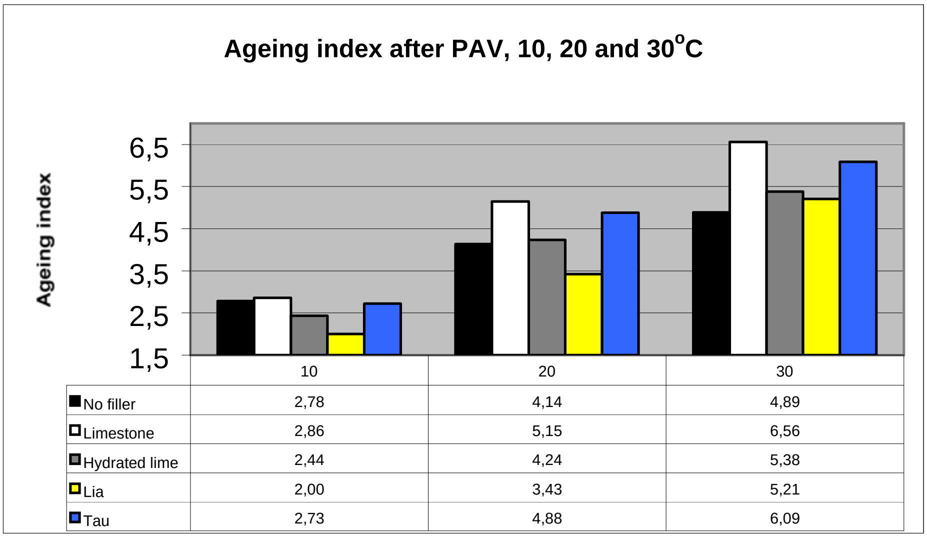


Figure 5.22 The "fatigue parameter*", $G^* \times \sin\delta$, Pen180, effective filler concentration C_e=0,3, PAV-aged.*

As can be seen from Figures 5.21 and 5.22, there are small differences in the rutting properties and some differences in the fatigue properties. Filler from Lia and Hydrated lime have the best fatigue properties, but they have also lower resistance against rutting.
In Figure 5.23 the ageing indices are calculated as the ratio of G* for PAV-aged material to G* for original material.



	10	20	30
■ No filler	2,78	4,14	4,89
□ Limestone	2,86	5,15	6,56
▨ Hydrated lime	2,44	4,24	5,38
🟨 Lia	2,00	3,43	5,21
🟦 Tau	2,73	4,88	6,09

Figure 5.23 Ageing index for Pen180, effective filler concentration, C_e=0,3.The ageing index is calculated as the ratio of G for PAV-aged material to G* original material.*

MB10000 with an effective filler concentration , C_e = 0,3

In Figure 5.24 and 5.25 the “SHRP rutting parameter” and the “SHRP fatigue parameter” are given, respectively.

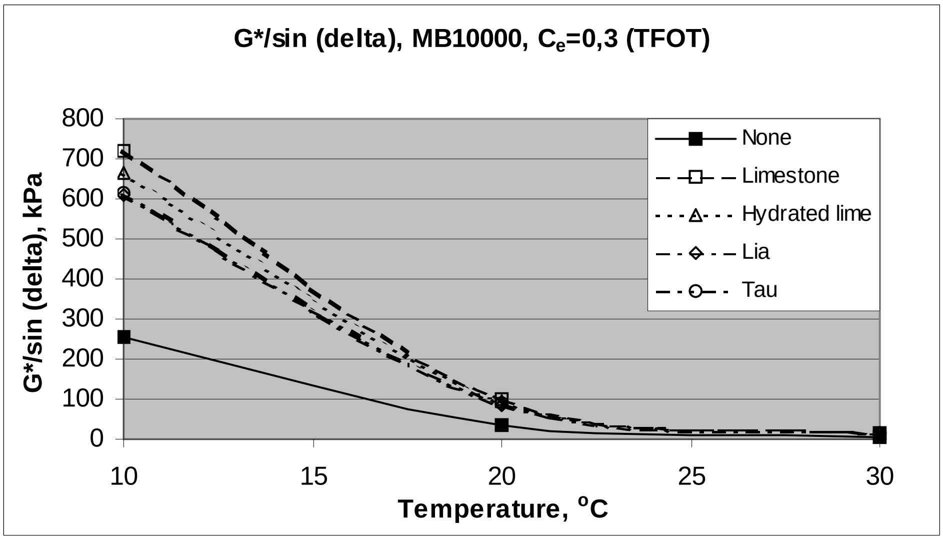


Figure 5.24 The “rutting parameter”, G/sin δ, MB10000, effective filler concentration, C_e=0,3.*

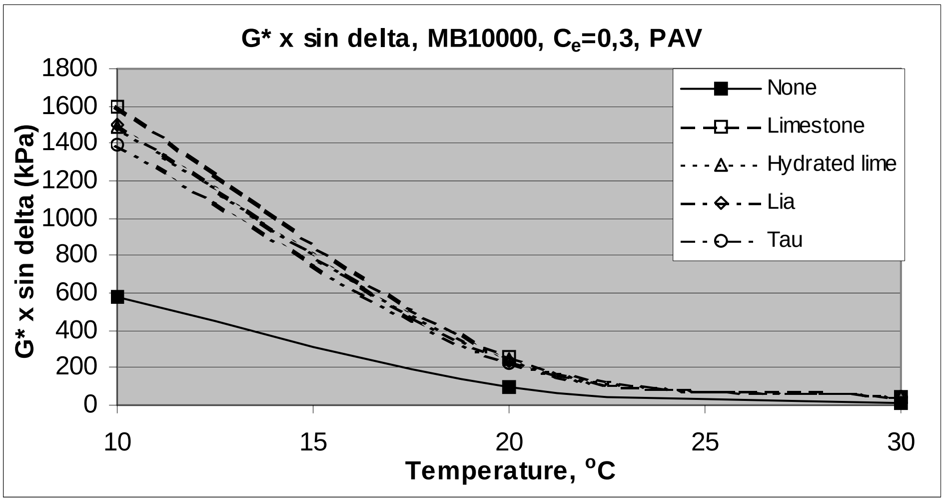
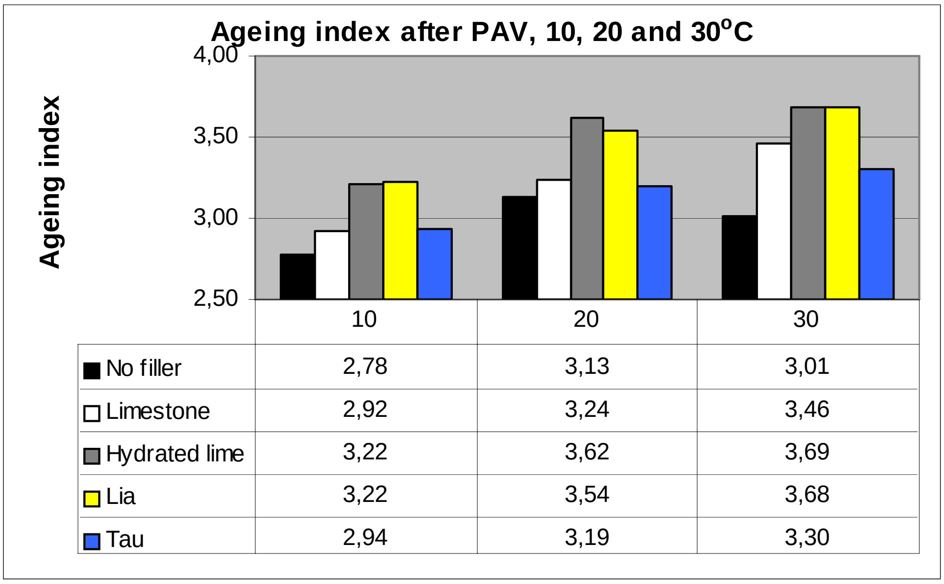


Figure 5.25 The "fatigue parameter", G × sin δ, MB10000, effective filler concentration C_e=0,3, PAV-aged.*

There are small differences in the G*/sin δ-values and the G* ×sin δ-values, for mixes containing MB10000 with an effective filler concentration, C_e=0,3.
The ageing index is calculated as G*-value after PAV divided by G*-value of original material. The ageing indices are shown in Figure 5.26.



	10	20	30
■ No filler	2,78	3,13	3,01
□ Limestone	2,92	3,24	3,46
▨ Hydrated lime	3,22	3,62	3,69
🟨 Lia	3,22	3,54	3,68
🟦 Tau	2,94	3,19	3,30

Figure 5.26 Ageing index for mixes containing MB10000 and effective filler concentration, Ce=0,3. Ageing index is calculated as G-value after PAV divided by G*-value of original material.*

As can be seen in Figure 5.26, the ageing index varies between materials and temperatures. According to Figure 5.26 mastic containing hydrated lime have the highest ageing index at all temperatures, and mix containing filler from Tau seems to have the best ageing properties.

MB10000 with an effective filler concentration , C_e = 0,6

In Figures 5.27 and 5.28 the "SHRP rutting parameter" and the "SHRP fatigue parameter" are given, respectively.

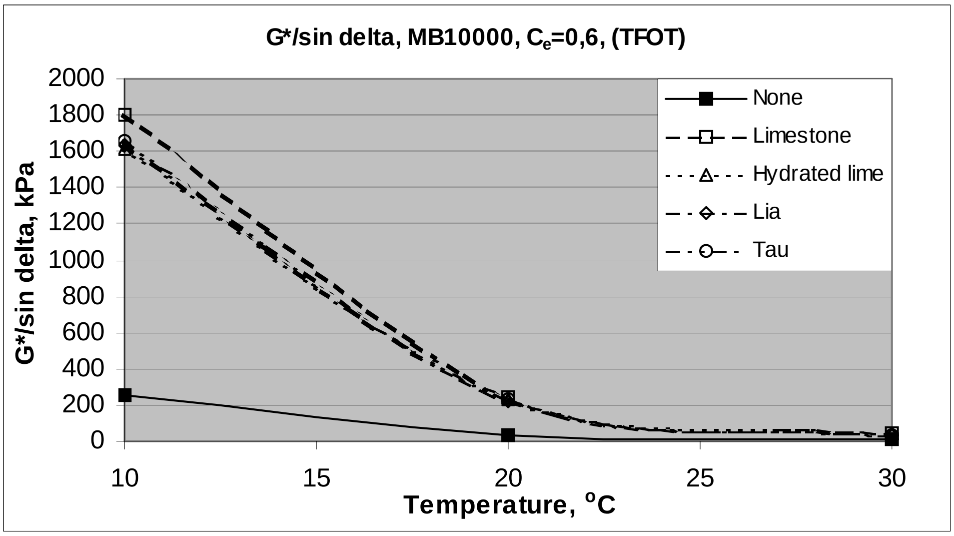


Figure 5.27 *The "rutting parameter", G*/sin δ, MB10000, effective filler concentration, C_e=0,6.*

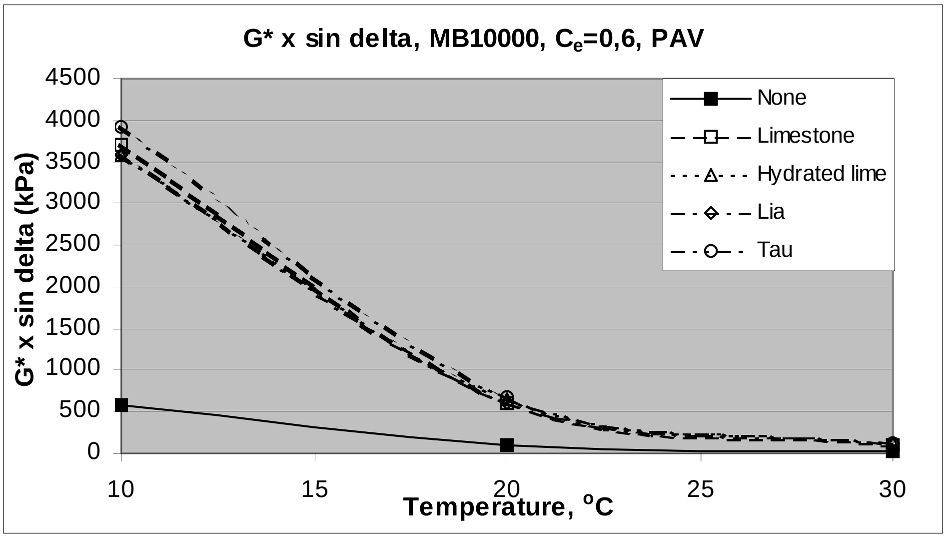
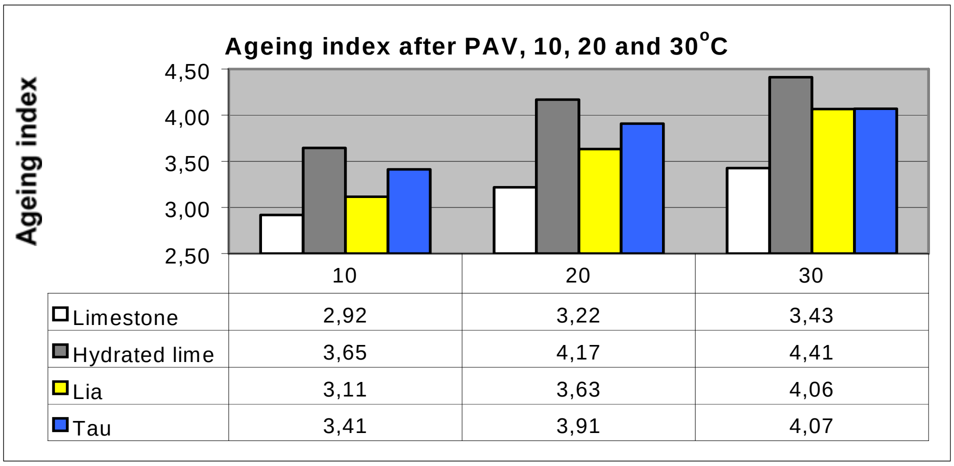


Figure 5.28 *The "fatigue parameter", G* $\times$ sin δ, MB10000, effective filler concentration C_e=0,6, PAV-aged.*

Figures 5.27 and 5.28 shows that there is small difference between the various mixes. The differences are even less than for mixes containing Pen180 and MB10000 with effective filler concentration, C_e=0,3.
The ageing indices are shown in Figure 5.29.



	10	20	30
☐ Limestone	2,92	3,22	3,43
■ Hydrated lime	3,65	4,17	4,41
🟨 Lia	3,11	3,63	4,06
🟦 Tau	3,41	3,91	4,07

Figure 5.29 Ageing index for mixes containing MB10000 and effective filler concentration, C_e=0,6. Ageing index is calculated as G-value after PAV divided by G*-value for original material.*

As can be seen in Figure 5.29, the ageing indices vary with temperature. As for mastics with effective filler concentration, C_e=0,3,mastic containing hydrated lime have the highest ageing index at all temperatures. In this case, mastic containing limestone seems to have the best ageing properties. This is contrary to mastic with C_e=0,3, where baghouse filler from Tau had the best ageing properties. Nevertheless, the results show rather small differences in ageing indices.

In Figures 5.30-5.32, the complex modulus for all mixes are shown for original, TFOT- and PAV-aged samples at 10, 20 and 30°C.

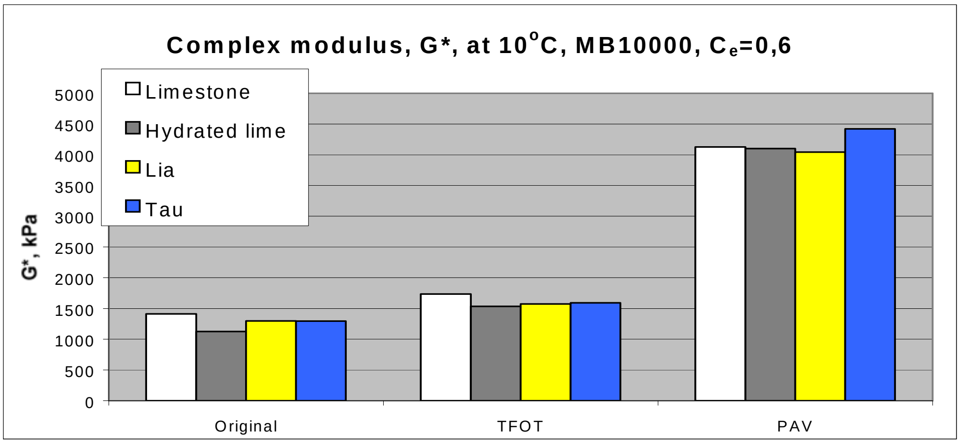


Figure 5.30 Complex modulus, G, measured with DSR, on original, TFOT, and PAV-aged materials at 10°C.*

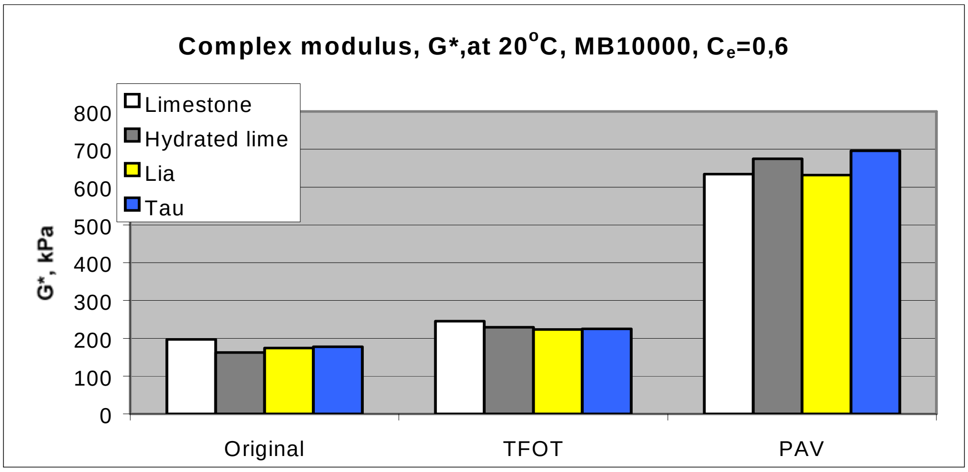


Figure 5.31 Complex modulus, G, measured with DSR, on original, TFOT, and PAV-aged materials at 20°C.*

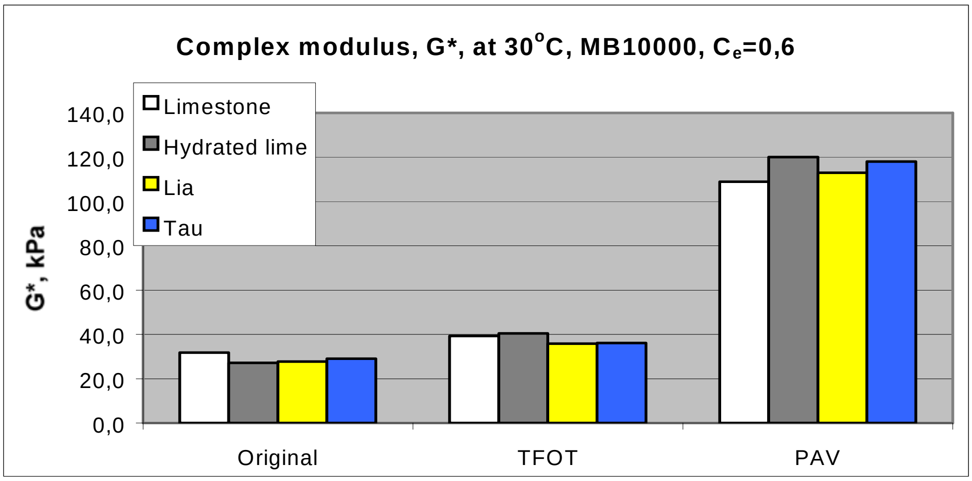


Figure 5.32 Complex modulus, G, measured with DSR, on original, TFOT, and PAV-aged materials at 30°C.*

Figures 5.30-5.32 shows that there are small variations in the value of the complex modulus between different mixes. This together with the softening points results shows that the concept of “effective filler concentration” based on Rigden voids is successful at producing filler-bitumen mixes of the same stiffness.

5.4.2.4 Bending Beam Rheometer (BBR)

Pen 180 –binder with an effective filler concentration, $C_e = 0,3$

The results from the BBR-measurements are given in Figure 5.33 and 5.34.
In Figure 5.35 stiffness-/ageing indices based on the BBR measurements are shown. Stiffness index is calculated as the ratio of S-value of filler-bitumen mix to the S-value of pure bitumen. Ageing index is calculated as the ratio of S-value after and before ageing.

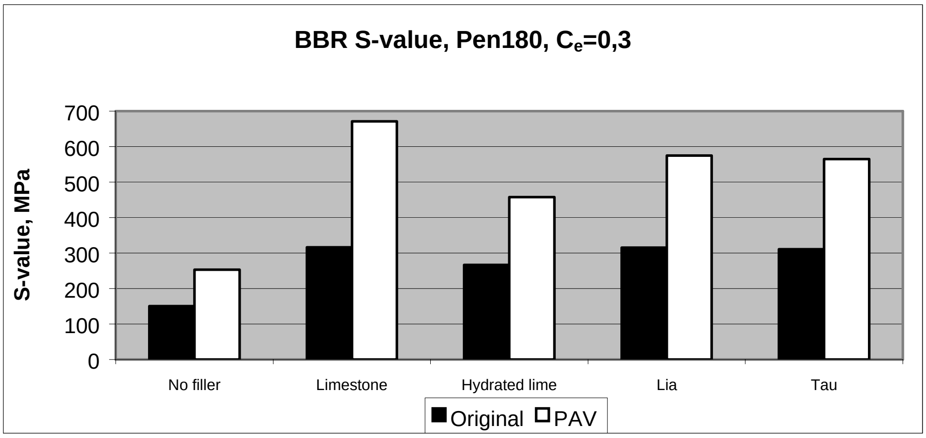


Figure 5.33 S-value from BBR-measurements for Pen 180 and effective filler concentration, C_e = 0,3. Test temperature; -20°C.

S-value for mix containing hydrated lime is somewhat lower for both original and PAV-aged mixes, as can be seen in Figure 5.33. This indicates better low temperature properties. Between other mastics there are small differences for original material. Mix containing limestone is noticeably stiffer than other mixes after PAV.

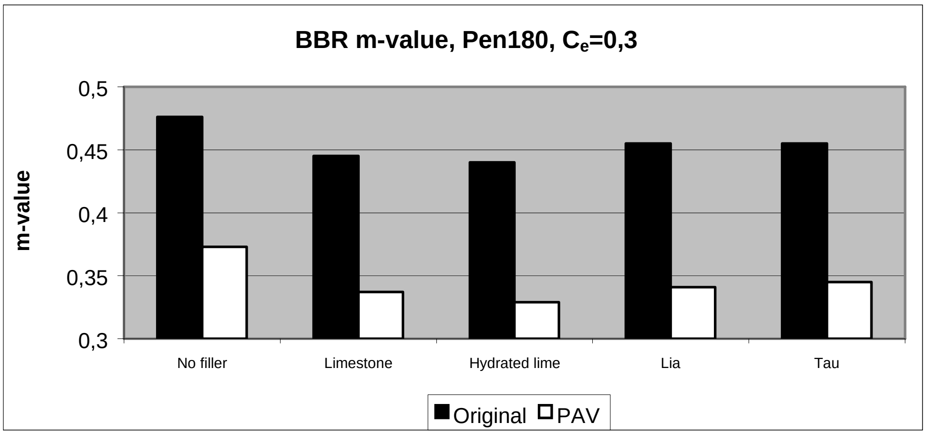
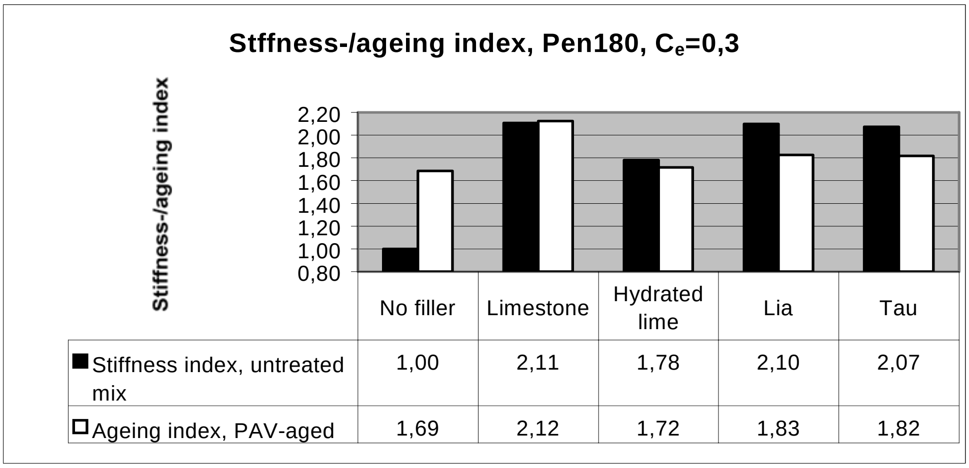


Figure 5.34 m-value from BBR-measurements for Pen 180 and effective filler concentration, C_e = 0,3. Test temperature; -20°C.

Figure 5.34 shows that there are small differences between m-values for all mixes. Nevertheless, mix containing hydrated lime has somewhat lower m-value. Low m-value indicates less stress relaxation properties.



	No filler	Limestone	Hydrated lime	Lia	Tau
■ Stiffness index, untreated mix	1,00	2,11	1,78	2,10	2,07
□ Ageing index, PAV-aged	1,69	2,12	1,72	1,83	1,82

Figure 5.35 Stiffness-/ageing index based on S-value from BBR-measurements for Pen 180 and effective filler concentration, Ce = 0,3. Test temperature; -20°C.

Figure 5.35 shows that the stiffness and ageing index is somewhat lower in the mix containing hydrated lime.

MB10000 with an effective filler concentration , C_e = 0,3

The results from the BBR-measurements are given in Figures 5.36 and 5.37.
In Figure 5.38 stiffness-/ageing index based on the BBR measurements are shown. Stiffness index is calculated as the ratio of S-value of filler-bitumen mix to that of pure bitumen. Ageing index is calculated as the ratio of S-value after and before ageing.

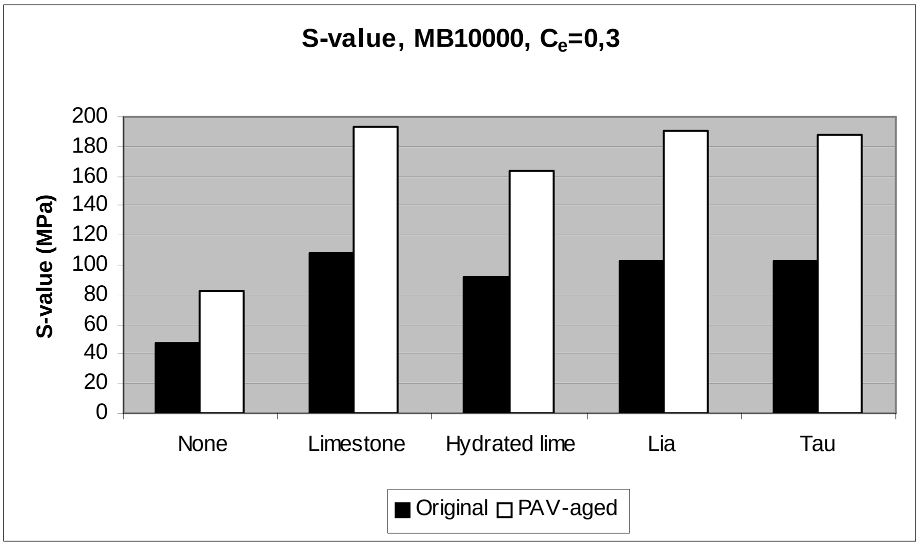
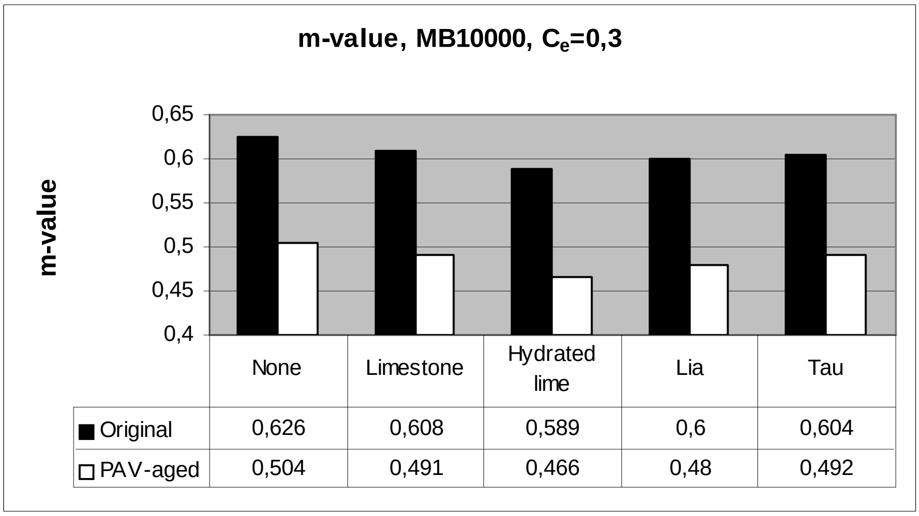


Figure 5.36 S-value from BBR-measurements for MB10000 and effective filler concentration, C_e= 0,3. Test temperature; -25°C.

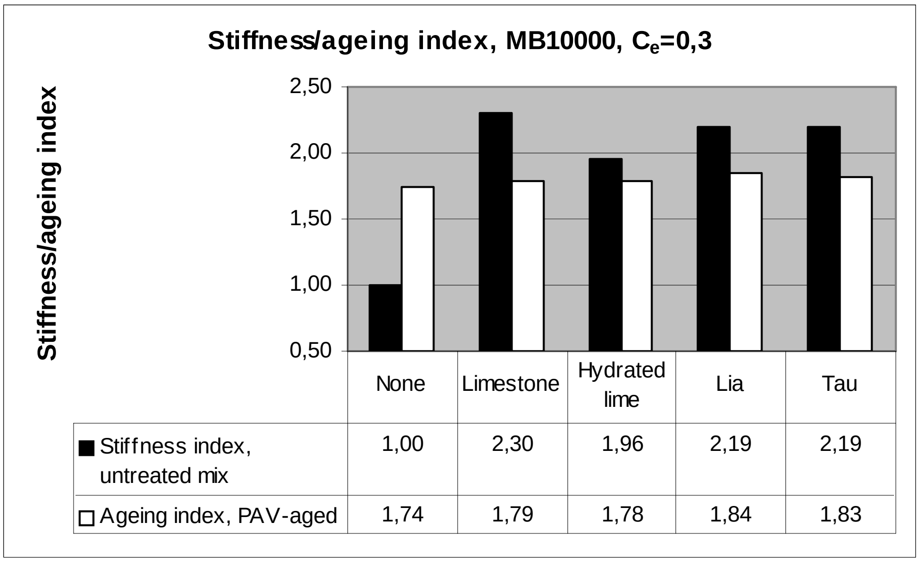
Mix containing MB10000 and hydrated lime (C_e=0,3) have somewhat lower S-value than other mastics, but the difference is less obvious than with binder Pen180 (Figure 5.36).



	None	Limestone	Hydrated lime	Lia	Tau
■ Original	0,626	0,608	0,589	0,6	0,604
□ PAV-aged	0,504	0,491	0,466	0,48	0,492

Figure 5.37 *m-value from BBR-measurements for MB10000 and effective filler concentration, C_e = 0,3. Test temperature; -25°C.*

Also for mixes containing MB10000 (C_e=0,3) there are small differences between m-values for all mixes (Figure 5.37).



	None	Limestone	Hydrated lime	Lia	Tau
■ Stiffness index, untreated mix	1,00	2,30	1,96	2,19	2,19
□ Ageing index, PAV-aged	1,74	1,79	1,78	1,84	1,83

Figure 5.38 *Stiffness-/ageing index based on S-value from BBR-measurements for MB10000 and effective filler concentration, C_e = 0,3. Test temperature; -25°C.*

From Figure 5.38 it can be seen that stiffness indices are somewhat lower for mastics containing hydrated lime. There are no obvious differences in ageing indices.

MB10000 with an effective filler concentration , C_e = 0,6

Figures 5.39 and 5.40 show the results from the BBR-measurements. In Figure 5.41 stiffness-/ageing index based on the BBR measurements are shown. Stiffness index is calculated as the ratio of S-value of filler-bitumen mix to S-value of pure bitumen. Ageing index is calculated as the ratio of S-value after and before ageing.

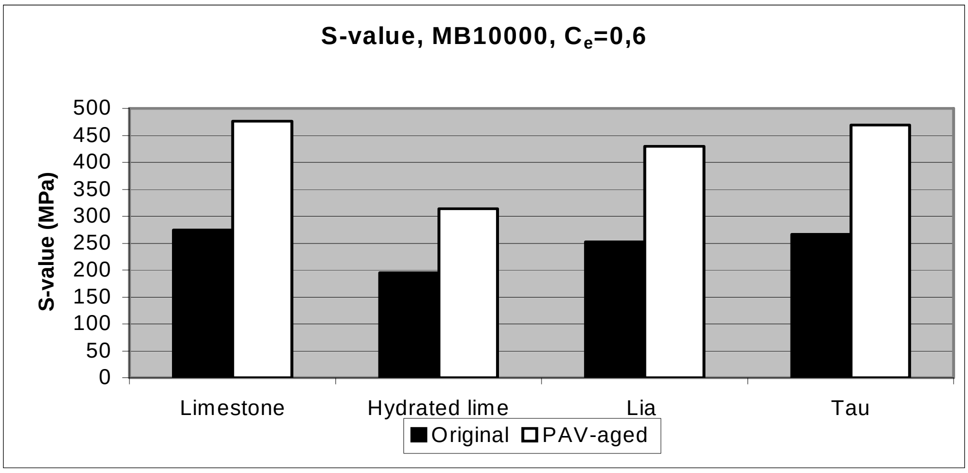
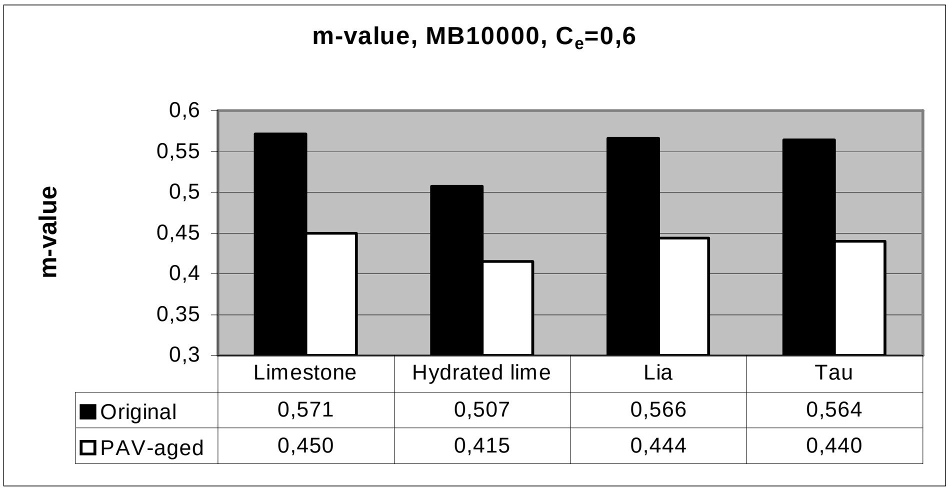


Figure 5.39 S-value from BBR-measurements for MB10000 and effective filler concentration, C_e=0,6. Test temperature; -25°C.

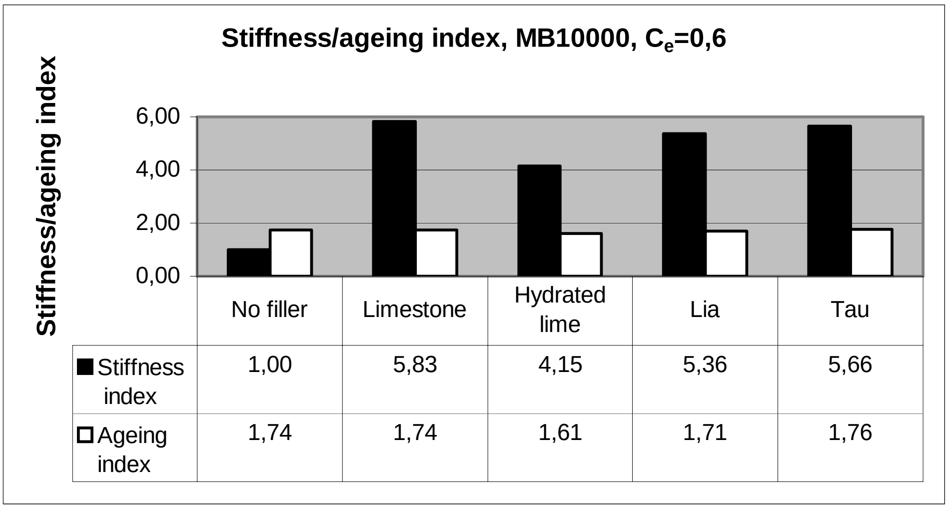
S-values for mixes containing limestone, filler from Lia and Tau are at similar level for both original and PAV-aged material. Mix containing hydrated lime has considerably lower S-value for both original and PAV aged material (Figure 5.39).



	Limestone	Hydrated lime	Lia	Tau
■ Original	0,571	0,507	0,566	0,564
□ PAV-aged	0,450	0,415	0,444	0,440

Figure 5.40 m-value from BBR-measurements for MB10000 and effective filler concentration, C_e=0,6. Test temperature; -25°C.

As previously shown from the BBR-measurements on mixes containing Pen 180 binder and MB10000, there are, relatively small differences between the m-values for all the mixes of MB10000 with effective filler concentration, C_e=0,6. In particular the differences for the PAV-aged material are small (Figure 5.40).



	No filler	Limestone	Hydrated lime	Lia	Tau
■ Stiffness index	1,00	5,83	4,15	5,36	5,66
□ Ageing index	1,74	1,74	1,61	1,71	1,76

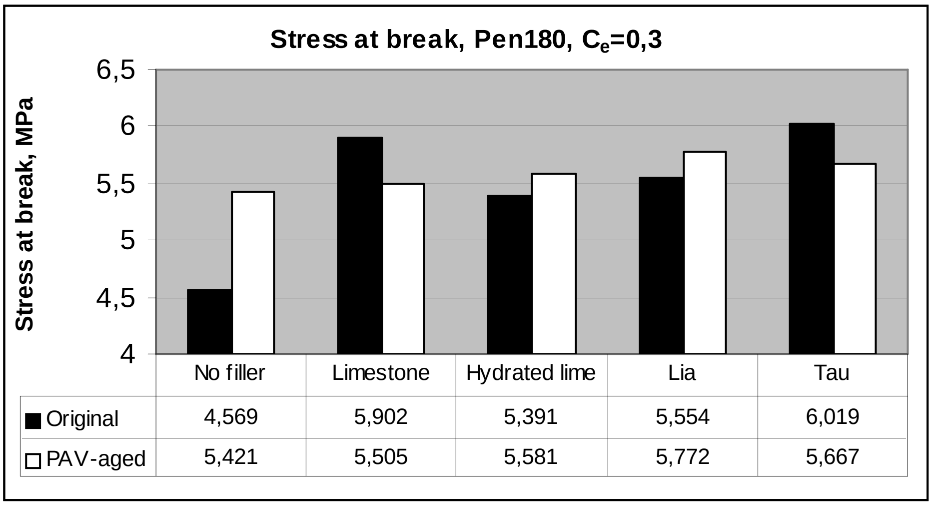
Figure 5.41 Stiffness-/ageing index based on S-value from BBR-measurements for MB10000 and effective filler concentration, C_e=0,6. Test temperature; -25°C.

As can be seen in Figure 5.41, the stiffness index is obviously lower for the mix containing hydrated lime than mixes with other filler types. The ageing index is also lower, but less obvious than the stiffness index.

5.4.2.5 Direct Tension Test (DTT)

Pen 180 –binder with an effective filler concentration, C_e = 0,3

The results from the DTT-measurements are given in Figures 5.42 and 5.43.



	No filler	Limestone	Hydrated lime	Lia	Tau
■ Original	4,569	5,902	5,391	5,554	6,019
□ PAV-aged	5,421	5,505	5,581	5,772	5,667

Figure 5.42 Stress at break from DTT-measurements for Pen 180 with effective filler concentration, C_e = 0,3. Test temperature, -20°C.

As can be seen in Figure 5.42, the unaged mix containing hydrated lime has the lowest stress at break. For PAV mixes the differences are rather small.

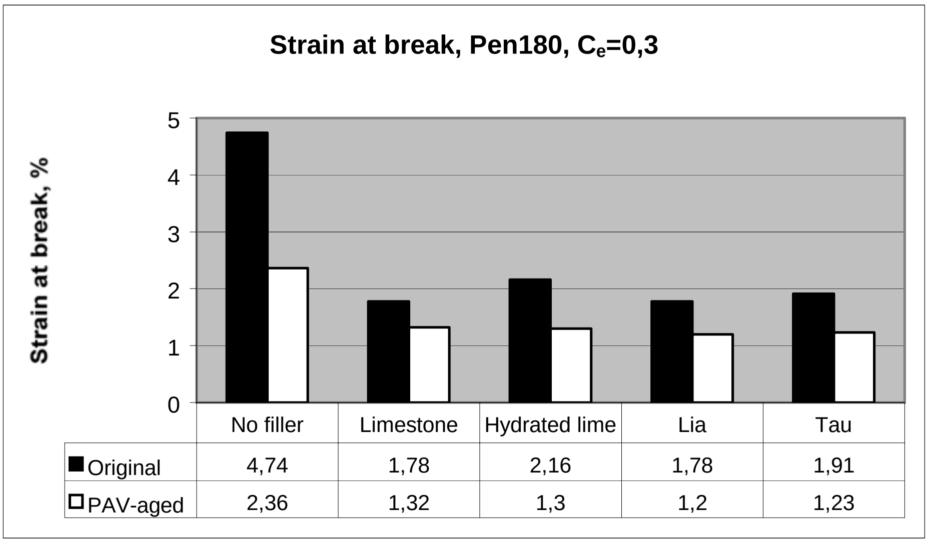


Figure 5.43 Strain at break from DTT-measurements for Pen 180 with effective filler concentration, C_e = 0,3. Test temperature, -20°C.

For original material the mix containing hydrated lime have a slightly higher strain at break than mixes with other filler types (Figure 5.43). For PAV-aged material the differences are rather small.

<u>MB10000 with an effective filler concentration , C_e = 0,3</u>

The results from the DTT-measurements are given in Figures 5.44 and 5.45. None of the specimens broke during the test (no break before 10 % elongation).

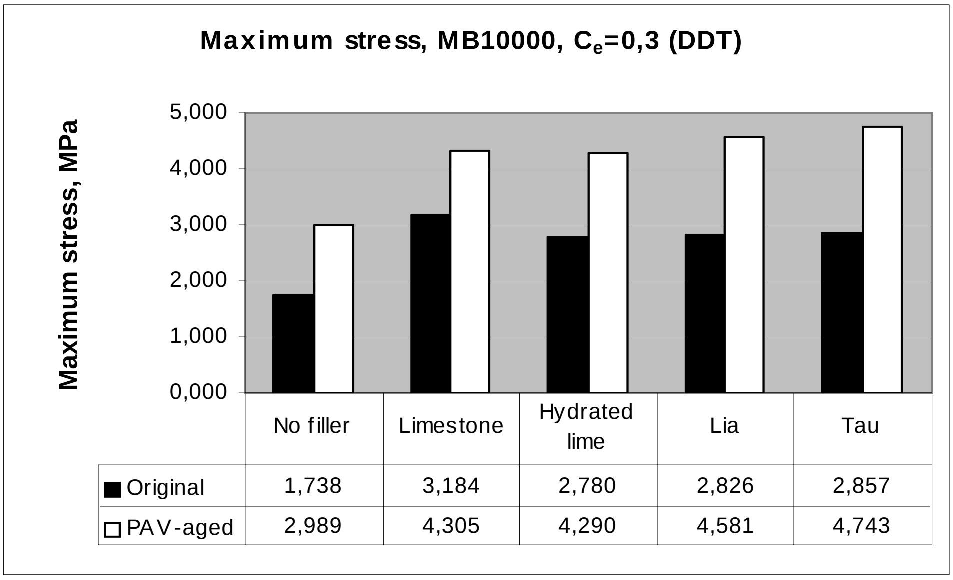


Figure 5.44 Maximum stress after DTT-measurements for MB10000 with effective filler concentration, C_e = 0,3. Test temperature, -25°C.

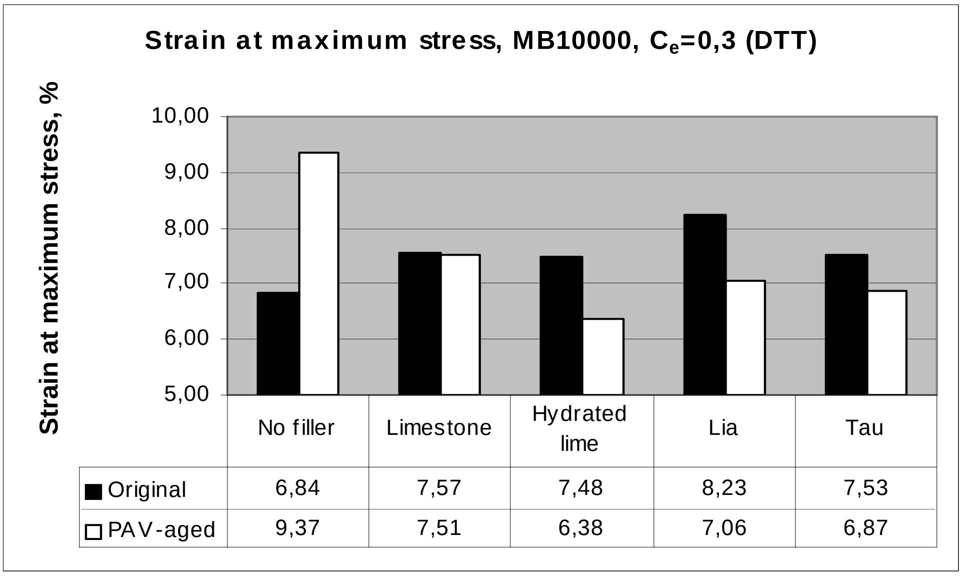


Figure 5.45 Strain at maximum stress from DTT-measurements for MB10000 with effective filler concentration, C_e = 0,3. Test temperature, -25°C.

As we see in Figures 5.44 and 5.45 there are relatively small differences between different types of mixes with respect to both maximum stress and strain at maximum stress. The mix containing hydrated lime has the lowest strain at maximum stress for both original and PAV-aged mixes.

MB10000 with an effective filler concentration, C_e = 0,6

The results from the DTT-measurements are given in Figures 5.46 and 5.47. None of the specimens broke during the test (no break before 10 % elongation).

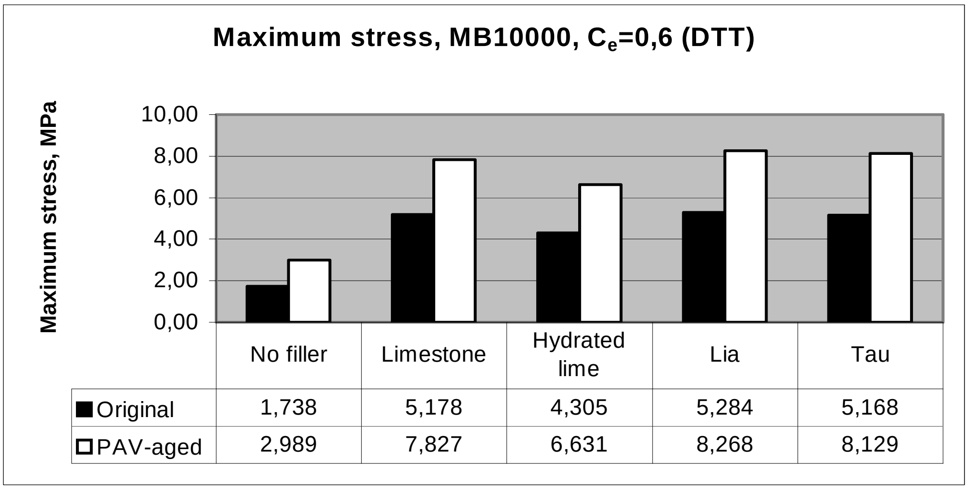
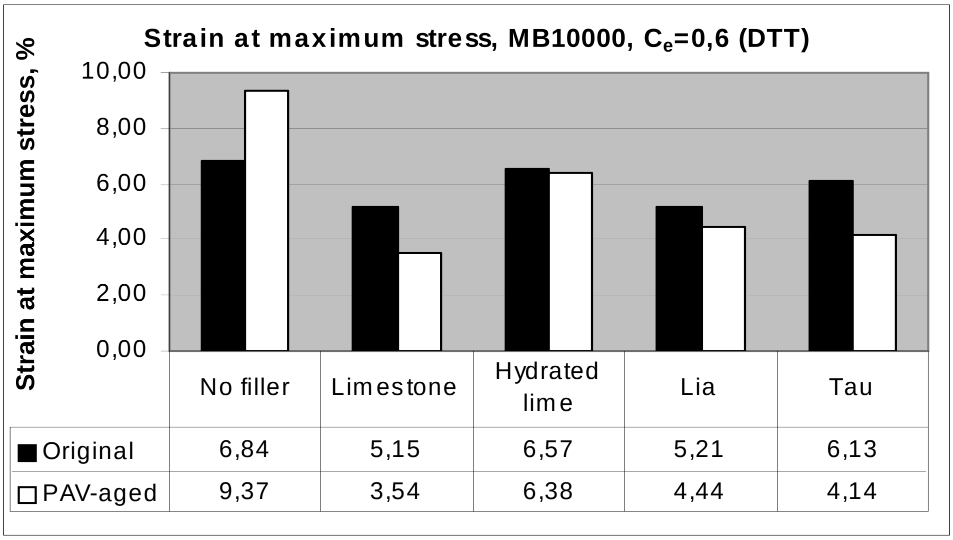


Figure 5.46 Maximum stress from DTT-measurements for MB10000 with effective filler concentration, C_e = 0,6. Test temperature, -25°C.

Maximum stress is somewhat lower for the mix containing hydrated lime. For other mixes the stress values are at about the same level (Figure 5.46).



	No filler	Limestone	Hydrated lime	Lia	Tau
■ Original	6,84	5,15	6,57	5,21	6,13
□ PAV-aged	9,37	3,54	6,38	4,44	4,14

Figure 5.47 Strain at maximum stress from DTT-measurements for MB10000 with effective filler concentration, C_e = 0,6. Test temperature, -25°C.

Figure 5.47 shows that the mix containing hydrated lime has somewhat higher strain at maximum stress than the other mixes, while mix containing limestone has the lowest strain at maximum stress, especially after PAV ageing.

5.4.2.6 Determination of filler content

The objective of checking the filler content in the filler-bitumen mixes is to check if there was any loss of filler during the preparation and to see if there was any sedimentation after the sample preparation. Especially it is of interest to see if the filler content vary in different levels of the BBR-samples.
Loss of filler can give lower viscosity and stiffness values. Also the ageing properties can be influenced by the content of filler.

The methods used are ASTM D 70-82 for measuring density of the mix. Filler-content is also measured using centrifugation of solvent-dissolved samples.
The results are given in Table 5.10.

Table 5.10 Theoretical density and density measured after ASTM D 70-82(1990) and filler content in samples measured using solvent and centrifuge. All measurements are performed on PAV-aged mixes.

Binder-filler-mix			**Theoretical density, g/cm^3**	**Measured density, g/cm^3**	**Theoretical filler content, mass %**	**Measured filler content, mass %**
C_e	**Binder**	**Filler*)**				
0,3	Pen180	L	1,37	1,44	42	42
		HL	1,16	1,17	23	
		Lia	1,33	1,36	37	38
		Tau	1,35	1,37	39	41
	MB	L	1,36	1,35	42	45
		HL	1,15	1,20	23	20
		Lia	1,32	1,32	37	37
		Tau	1,34	1,31	39	39
0,6	MB	L	1,71	1,71	65	66
		HL	1,30	1,31	41	40
		Lia	1,66	1,65	61	61
		Tau	1,68		63	63

*) L = limestone, HL = hydrated lime

To check if there is any sedimentation of filler in the bending beam samples, two random checks were performed. A sample with MB10000 and limestone with effective filler concentration 0,6 and a sample with MB 10000 and hydrated lime with effective filler concentration 0,3 were analysed. The bending beam sample was split in two and specific density was measured on the top layer and the bottom layer. The results are given in Table 5.11.

Table 5.11 Results from density in top and bottom layer in BBR-samples.

Sample	**Density, top layer, g/cm^3**	**Density, bottom layer, g/cm^3**	**Theoretical density, g/cm^3**
MB10000 and limestone with effective filler concentration, C_e=0,6	1,703	1,703	1,71
MB10000 and hydrated lime with effective filler concentration, C_e=0,3	1,146	1,148	1,15

As can be seen in Tables 5.10 and 5.11, there was not any loss of fillers in the sample preparing and there seems not to be any problem with sedimentation of filler in the preparation of BBR-samples.

5.5 Discussion

5.5.1 Filler materials

The particle distribution is approximately identical for limestone and filler from Tau, with D_{50} and D_{90} on 20 and 70 micrometer respectively. For filler from Lia D_{50} is about 19 micrometer and D90 about 55 micrometer. The particle distribution for hydrated lime vary considerably from the other samples with D_{50} of about 33 micrometer and D_{90} about 62 micrometer. Some of these variations could be due to agglomeration of small hydrated lime particles. The “dark shadows” on the pictures of hydrated lime probably illustrate the agglomeration.

Studies in microscope shows that the limestone has the most regular particle shape. The particle shape in the sample from Lia is more longitudinal than the sample from Tau. The hydrated lime has a significantly different particle shape than the other materials. It consists of porous particles with irregular shape.

Mineral composition varies significantly between different types of fillers. This variation can contribute to different behaviour in an asphalt mixture. It is difficult to draw any clear conclusions about correlation between mineral composition and performance from this limited investigation.

The specific surface area of hydrated lime is much larger than that of the other filler types even when the size distribution curve is courser for hydrated lime. A reason for this could be that hydrated lime is a porous (and aggregated) material. The specific surface area also varies between the other filler materials.

The Rigden void content varies considerably between all filler materials. The lowest void content is that of limestone with a value of 31,2 %. Hydrated lime has the highest void content of 59,8 %.

The specific densities also vary, and is lowest for hydrated lime with a value of 2,237 g/cm^3. Limestone, Lia and Tau have specific densities of 2,728, 2,873 and 2,786 g/cm^3, respectively.

5.5.2 Filler-bitumen mixes

An overview of the result from the analyses is given in Table 5.12. In this table the “best” filler and the “worst” filler is indicated according to selected performance factors.

Table 5.12 An overwiev of how bitumen-filler-mixes behave according to selected performance factors.

Performance factors	Criteria	Binder	Pen 180				MB 10000							
		C_e	0,3				0,3				0,6			
		Filler*)	L	HL	Li	T	L	HL	Li	T	L	HL	Li	T
Rutting	DSR***)			0		I	I		0	0	I	0	0	0
Fatigue	DSR**)		0	I	I		0			I	I	I	I	0
Low temp. (S-value)	BBR**)		0	I			0	I			0	I		
Low temp. (m-value)	BBR**)			0		I		0		I	I	0		
Low temp. (strain at break/maximum stress)	DTT**)		I		0		I	0			0	I		
Rutting	Softening point***)		0	I	0	0		I		0	0		I	

*) L=limestone, HL=hydrated lime, Li=filler from Lia, T=filler from Tau
**) After PAV ageing
***) After TFOT ageing
I = “best” properties
0 = “worst” properties

Table 5.12 shows that it is difficult to argue for a special type of filler when evaluating all the performance factors. The ranking of the fillers will vary with the different performance factors. It can also be seen that different filler concentrations influence on the ranking of the fillers. For some of the performance factors several fillers are classified as “best”, as they have almost identical values. This is also the case when several fillers are classified as “worst”.

The values from BBR measurements give almost the same ranking of filler materials for different types of binders and different filler concentrations. In this case, hydrated lime is best for S-value (lowest S-value) and worst for m-value (lowest m-value, which indicate low resistance to thermal fatigue). Limestone is worst for S-value. For m-value, filler from Tau is best in 2 of 3 measurements.

In the following chapters, a more detailed discussion of the results from this study is given. Ageing properties are evaluated in chapter 5.5.2.5.

5.5.2.1 Ring and ball

For all types of mixes, there are small differences in the softening point value between various types of filler. Based on the ring and ball test it is difficult to draw conclusions with respect to type of filler and the influence on the binder properties.

5.5.2.2 Dynamic Shear Rheometer (DSR)

Pen 180 with C_e=0,3

Mix containing hydrated lime has the lowest value of G*/sinδ (Figure 5.21). G*/sinδ is the rutting parameter in the SHRP-binder specifications. When mix containing hydrated lime has the lowest value, it indicates that this is the softest mix in this temperature range.

According to Figure 5.22, mix containing filler from Lia and hydrated lime have the best fatigue properties.

Ageing indices vary with temperature, as shown in Figure 5.23. Mix containing limestone has the highest ageing index at all temperatures, while mix containing filler from Lia has the lowest ageing index.

MB10000 with C_e=0,3

As can be seen in Figure 5.24, mix containing filler from Lia and Tau has the lowest value of G*/sinδ. Mix containing limestone has the highest value.
Mix containing filler from Tau has the best fatigue properties (the lowest value of G* x sinδ), as can be seen in Figure 5.25.
In Figure 5.26 it can be seen that mix containing hydrated lime has the highest ageing index. Mix containing filler from Tau has the best ageing properties.

MB10000 with C_e=0,6

In Figure 5.27 it can be seen that there is small differences in the "rutting parameter" between mixes containing filler from Lia, Tau and hydrated lime. Mix containing limestone is somewhat stiffer than the other mixes.
Also according to the fatigue criterion, there are small differences between the various mixes (Figure 5.28). Here mix containing filler from Tau is somewhat poorer than the other mixes.
As for mix with effective filler concentration C_e=0,3, mix containing hydrated lime has the highest ageing index. Mix containing limestone has considerably lower ageing index than the other mixes. This is contrary to mix with C_e=0,3, where baghouse filler from Tau had the best ageing properties.
As can be seen in Figures 5.30-5.32, there are small variations in the value of the complex modulus between different mixes.

5.5.2.3 Bending Beam Rheometer (BBR)

The tendencies after BBR-measurements are the same for mixes containing both binder Pen 180 and MB10000:

- S-value for mixes containing hydrated lime is lower than for the other mixes both before and after ageing. The S-value for mixes containing limestone, filler from Lia and Tau are at similar levels of magnitude.
- The m-value is about the same for mixes containing limestone, filler from Lia and from Tau. The m-value for mixes containing hydrated lime is somewhat lower than for other mixes. The binder with low m-values has less reserve strength than high m-value binder and thus has less resistance to thermal fatigue, as shown schematic in Figure 5.48.

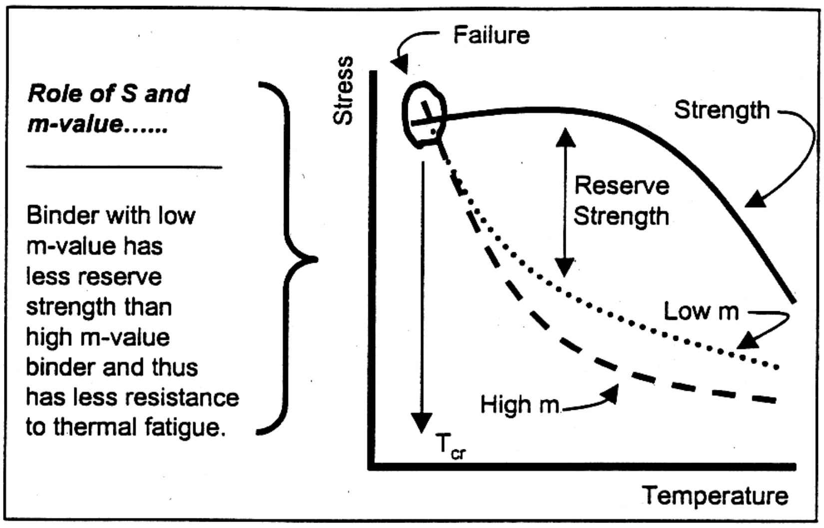


Figure 5.48 Schematic of the thermal shrinkage stress criterion. S and the m-value (from BBR) are used to calculate thermal shrinkage stresses and the strength is estimated from the direct tension test. The temperature where they are equal is the critical cracking temperature, T_{cr} [91].

- The stiffness index for mixes containing hydrated lime is somewhat lower than for the other mixes. The stiffness index is calculated as the ratio of the S-value of filler-bitumen mix to the S-value of pure bitumen.
- The ageing index is somewhat lower for mixes containing hydrated lime. However the differences between mixes are rather small. The ageing index is calculated as the ratio of S-value after and before ageing.

The BBR test shows a positive influence of hydrated lime on the S-value (lower S-values). The m-value is however also lower for the hydrated lime mixes compared to the others. Based on the BBR-results alone, the net effect of hydrated lime on low temperature cracking is uncertain. The low temperature stiffness is improved, but the stress relaxing properties seem to become poorer.

5.5.2.4 Direct Tension Test (DTT)

The following could be observed after DTT;

- There are small differences for mixes containing Pen180 and MB10000 with effective filler concentration, C_e=0,3. Strain at break is somewhat higher for original (not aged) mix containing hydrated lime with binder Pen 180. For mixes containing MB10000 (C_e=0,3) it is opposite with strain at maximum stress somewhat less for mixes containing hydrated lime, especially for PAV-aged mixes. There are small differences between other mixes.
- Mix containing hydrated lime with effective filler concentration, C_e=0,6, differ considerably from other mixes, both for maximum stress and strain at maximum stress. Mix containing hydrated lime has lower maximum stress and higher strain at maximum stress. There are small differences between other mixes.

5.5.2.5 Ageing and stiffness indices

The ageing index is calculated as the ratio of mix condition after PAV ageing to unaged mix.
The stiffness index is calculated as the ratio of mix condition of none aged filler-bitumen mix to pure bitumen (none aged).
This is calculated based on BBR, DSR and softening point results.

The ageing indices and the stiffness indices for mixes containing binder Pen 180 are shown in Figures 5.49 and 5.50 respectively.

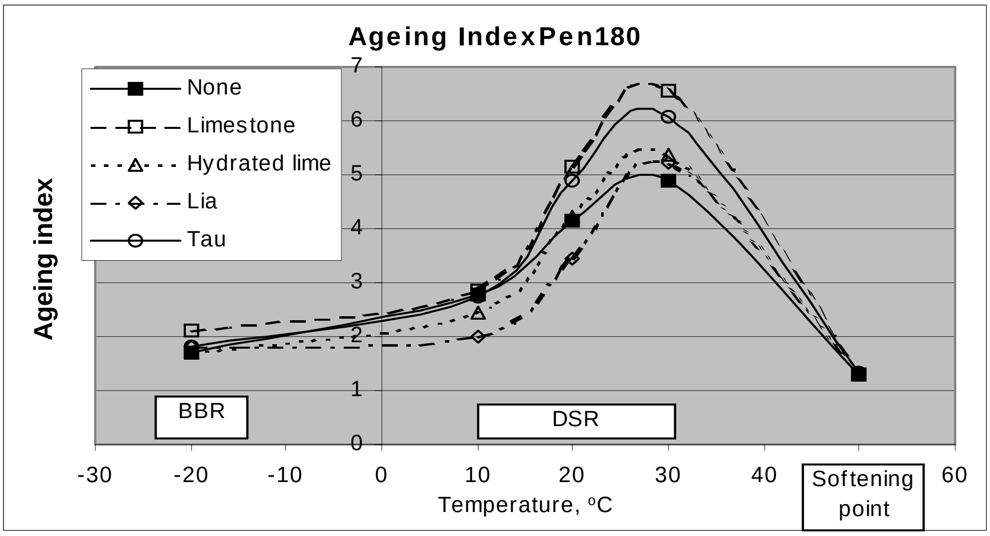


Figure 5.49 Ageing index for mix containing binder Pen180. The ageing index is calculated after PAV-ageing. (-20^{o}C after BBR-measurements, 10, 20 and 30^{o}C after DSR-measurements and softening point after ring and ball).

Figure 5.49 shows that limestone filler gives mix with highest ageing index at all temperatures. The curve for the ageing index of the mix with baghouse filler from Tau is fairly close to that of mix with limestone. At –20^{o}C mix with hydrated lime has the lowest ageing index. At the softening point the ageing index coincides for all mixes. In the intermediate temperature range, the mix containing filler from Lia has the lowest ageing index, but the mix containing hydrated lime is close to this curve also in this area.

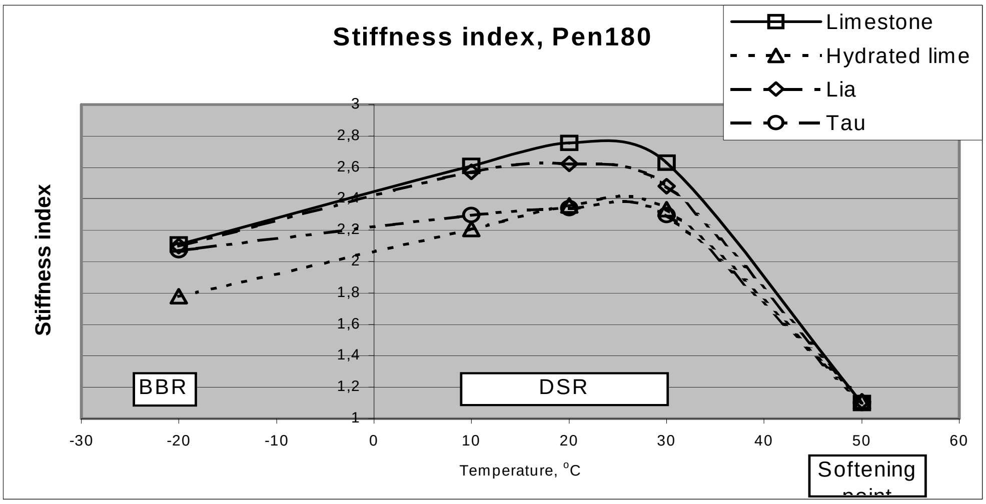


Figure 5.50 Stiffness index for mix containing binder Pen 180. The stiffness index is calculated for non-aged mixes. (-20°C after BBR-measurements, 10, 20 and 30 °C after DSR-measurements and softening point after ring and ball).

In this testing program, mixes with different filler types and the same effective filler concentration have been analysed. Theoretically the stiffness index should be the same for all mixes. As can be seen in Figures 5.50, 5.52 and 5.54, the stiffness index is the same for all the mixes in the high temperature range (softening point). However, at lower temperatures the stiffness index is different for the various mixes. Figure 5.50 shows that at –20°C the stiffness index for mix containing hydrated lime is considerably lower than for the other mixes. In the intermediate temperature range, mixes containing limestone and baghouse filler from Lia are very close and the stiffness indices are somewhat higher for these two fillers than mixes containing hydrated lime and baghouse filler from Tau.

As can be seen in Figures 5.49 and 5.50, ageing and stiffness index vary with type of filler. Hydrated lime has the lowest stiffness index at low temperature and also the lowest ageing index at this temperature. This indicates that the mix containing hydrated lime has better low-temperature stiffness than other mixes. Baghouse filler from Lia also shows good ageing properties, but the stiffness index is obviously higher at low temperatures compared to mix containing hydrated lime.

The ageing indices and the stiffness indices for mixes containing binder MB10000 with effective filler concentration, C_e=0,3, are shown in Figures 5.51 and 5.52 respectively.

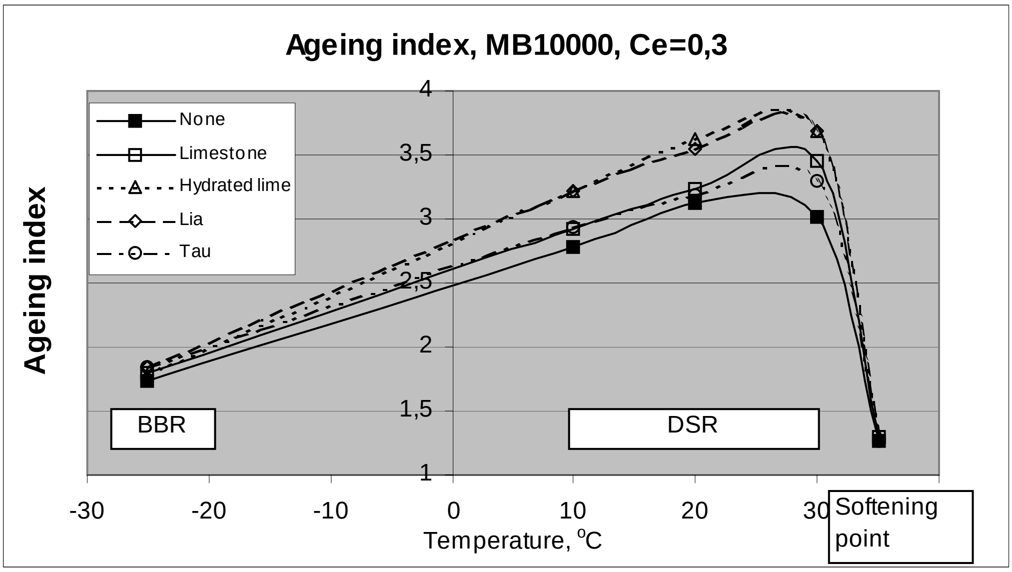


Figure 5.51 Ageing index for mix containing binder MB10000 with effective filler concentration, C_e=0,3. The ageing index is calculated after PAV-ageing. (-25°C after BBR-measurements, 10, 20 and 30 °C after DSR-measurements and softening point after ring and ball).

Figure 5.51 shows that there are some differences in ageing index depending on the filler type. At –25°C and at the softening point, ageing indices are similar for all mixes. In the intermediate temperature range, mixes containing limestone and baghouse filler from Tau are in about the same level. The ageing indices for mixes containing hydrated lime and filler from Lia are very similar and somewhat higher than that of limestone and baghouse filler from Tau.

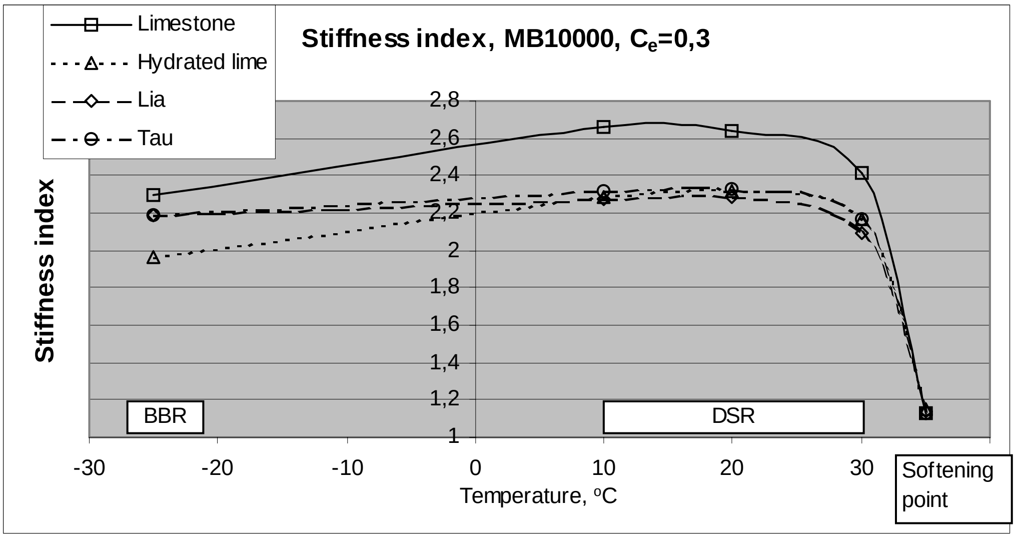


Figure 5.52 Stiffness index for mix containing binder MB10000 with effective filler concentration, C_e=0,3. The stiffness index is calculated for non-aged mixes. (-25°C after BBR-measurements, 10, 20 and 30 °C after DSR-measurements and softening point after ring and ball).

There are also some differences in the stiffness index for mixes containing MB10000 with effective filler concentration, C_e=0,3. The mix with limestone has highest stiffness index at all temperatures. At –25°C the mix containing hydrated lime has a stiffness index considerably lower than other mixes. Filler from Lia and Tau has near the same stiffness indices at all temperatures. At softening point the stiffness indices are about the same for all samples.

The ageing indices and the stiffness indices for mixes containing binder MB10000 with effective filler concentration C_e=0,6, are shown in Figures 5.53 and 5.54, respectively.

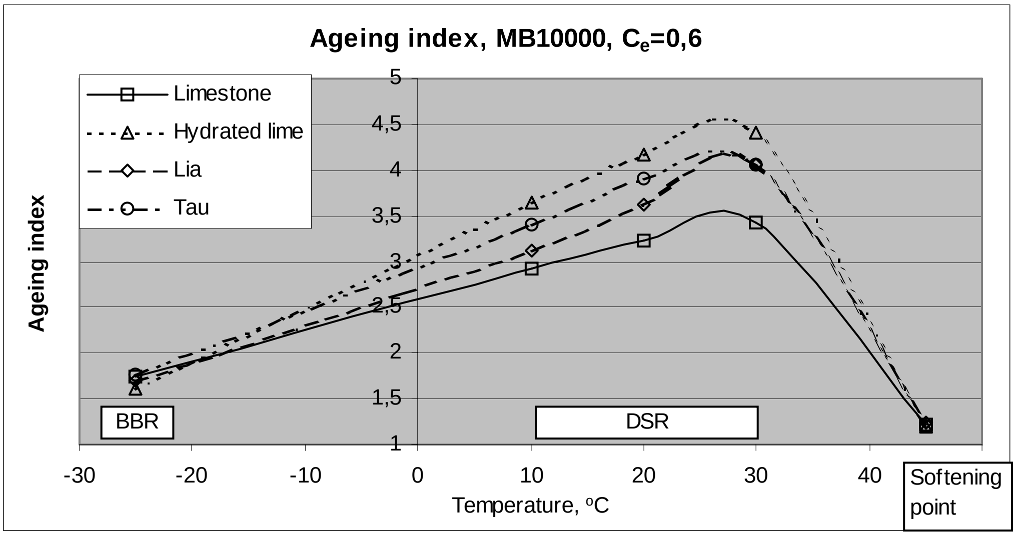


Figure 5.53 Ageing index for mix containing binder MB10000 with effective filler concentration, C_e=0,6. The ageing index is calculated after PAV-ageing. (-25°C after BBR-measurements, 10, 20 and 30 °C after DSR-measurements and softening point after ring and ball).

In Figure 5.53 we see that ageing indices for all mixes almost coincides at –25°C and at softening point. In the intermediate temperature range, the mix containing limestone filler has the lowest ageing index. In the intermediate temperature range (as for mix containing MB10000 and effective filler concentration, C_e=0,3), the mix containing hydrated lime has the highest ageing index.

In Figure 5.54 it can be seen that the stiffness index for the mix containing hydrated lime is considerably lower than for other mixes at –25°C. At the softening point, the stiffness indices for all mixes are the same. In the intermediate temperature range, the mix containing limestone has the highest stiffness index and the mix containing hydrated lime has the lowest index. The indices for mixes containing filler from Lia and Tau have values between the previous mentioned. The variation in stiffness index is decreasing with increasing temperature.

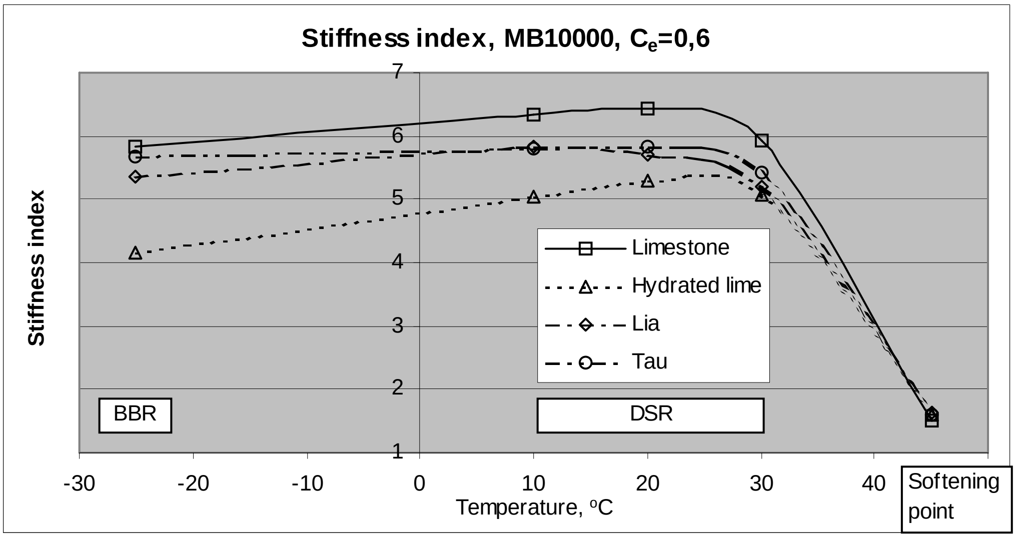


Figure 5.54 Stiffness index for mix containing binder MB10000 with effective filler concentration, C_e=0,6. The stiffness index is calculated for non-aged mixes. (-25°C after BBR-measurements, 10, 20 and 30°C after DSR-measurements and softening point after ring and ball).

5.5.2.6 Laboratory ageing

From the results of laboratory ageing, the following can be observed:

- The loss of mass is considerably higher for mixes containing hydrated lime, for Pen 180 as well as for MB10000. There are small differences between other mixes.
- The loss of mass is increasing with increasing effective filler concentration (MB10000).
- There are some differences in visual surface condition after PAV. Mixes containing hydrated lime seems to be most homogenous after PAV.

Questions could be raised whether or not the ageing tests and conditions used are suitable for filler-/bitumen mixes. However, based on the loss of mass there are differences between types of filler. Also the visual evaluation after PAV-ageing differs between different types of fillers.

5.5.2.7 Evaluation the SHRP-binder equipment used for analysis of filler/bitumen mixes

In this investigation analyses were carried out using several equipment. No scientific analysis was made on repeatability.
Here only experience from the work is discussed.

The sample preparation for all equipments was conducted in a satisfactory manner.

The direct tension samples did not break for samples containing binder MB10000. A regression analysis was carried out to relate BBR results to DTT results for samples containing MB10000. Values from the DTT are taken at maximum stress. The results are shown in Figure 5.55.

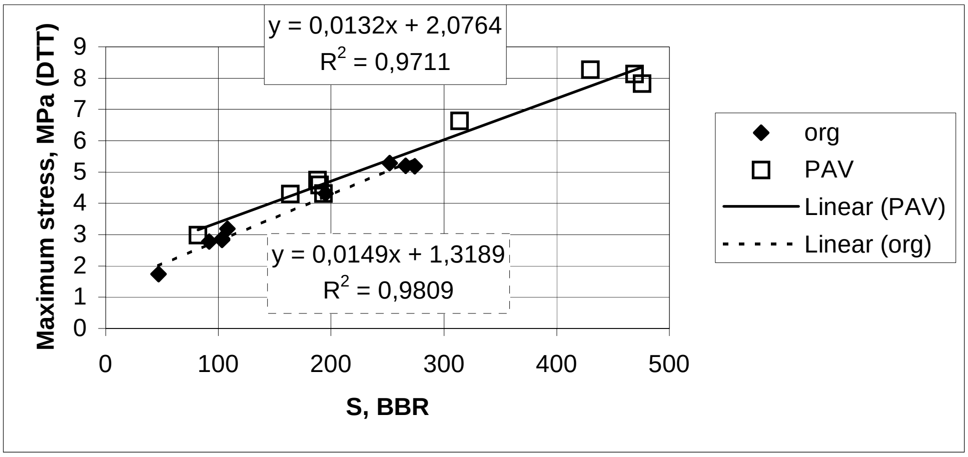


Figure 5.55 Comparison of results from BBR-test and Direct Tension test for mixes containing MB10000.

Figure 5.55 shows that for these fillers and binder, MB10000, you really do not need a direct tension test because the correlation between "stiffness" and "failure" is very good.

5.6 Conclusions

Based on the results from this laboratory investigation, the main conclusions from this research work are:

1. Hydrated lime seems to positively influence the low temperature stiffness (BBR S-value), but has a negative influence on the low temperature stress relaxing properties (BBR m-value). Measurements of failure properties using DTT do not show any clear improvements from hydrated lime. Based on these results, the net effect of hydrated lime on low temperature cracking properties seems to be very small.
2. No clear variation in ageing properties for mixes containing different types of fillers can be seen. However, there seems to be some differences between the two types of binders used in this investigation. There are some differences in ageing indices based on DSR-measurements. For samples containing MB10000, hydrated lime has the highest ageing index. For samples containing binder Pen 180 filler from Lia has the lowest ageing index and limestone the highest ageing index.
3. The “effective filler concentration concept”, based on Rigden void content, seems to be correct at high temperatures. At low temperature (-20/-25°C) there are some differences in stiffness between various fillers. Based on DSR-measurements there are also differences in stiffness index. The difference is decreasing with increasing temperature. The stiffness index at the softening point is about the same for all mixes.
4. To classify filler materials, it is not sufficient to give requirement to mass-% on sieve, which is done in Norwegian standards and guidelines. The quantity of filler required to get the same stiffness in the filler/bitumen mix varies for different types of fillers, and should be incorporated in mix design procedures.
5. The void content in filler is an important factor, which should be evaluated in asphalt mix-design in a more comprehensive way than today.
6. The SHRP-binder equipment seems to be a step forward in evaluating the properties of filler-bitumen mixes. Results from laboratory work have to be correlated with field data.

CHAPTER 6 *CONCLUSIONS AND RECOMMENDATIONS FOR FURTHER RESEARCH*

6.1 Conclusions

An important object of this research work has been to obtain more knowledge concerning mix design and maintenance of low volume roads with asphalt pavements in Norway.

A literature review on bitumen, filler, rheology and ageing has been carried out as part of this work. A choice was made to focus on bitumen and filler because of their primary importance in asphalt mixtures. Bitumen as the binder in the asphalt pavement, and filler because of its relatively large surface area compared to the rest of the aggregate. Filler will therefor have a relatively large contact area to the bitumen and thus being an important factor for the binder properties in the asphalt pavement.

The following findings from the literature formed the background for this work and the choice of the final laboratory program.

Bitumen consists mainly of carbon and hydrogen (95 %). In addition there are heteroatoms (nitrogen, oxygen, sulphur) and metals (vanadium, nickel, iron). This makes bitumen to a chemically very complex material, with physical properties dependent of temperature and time of loading.

During the Strategic Highway Research Program (SHRP) a new bitumen model was developed. This is claimed to be the most consistent description of bitumen in terms of chemistry. In this model, bitumen is supposed to be a collection of polar and non-polar molecules. However, it is still difficult to find good relationships between chemistry composition and pavement performance.

Bitumen is often characterised as a viscoelastic material. Bahia and Anderson [18] claim that dynamic (oscillatory) testing is the best technique to measure and also to explain the viscoelastic properties of bituminous materials.

With time, bitumen ages in the asphalt pavement and becomes harder. Among several mechanisms contributing to bitumen hardening, the following are the most important:

- Oxidation
- Evaporation
- Exudation
- Physical hardening

Two main functional theories of filler are mentioned in the literature:

- Filler is that portion of the mineral aggregate generally passing the number 200 sieve (0,074 mm). This portion occupies void space between coarser aggregate particles and reduces the size of these voids and hence increases the density and stability of the aggregate.
- The filler is an integrated part of the binder, and the filler-bitumen mix acts as a binder for the aggregate.

The void content based on the Rigden method has been shown to be of major importance for the behaviour of fillers and for the rheological properties of the filler-bitumen mix [62].

Hydrated lime is claimed to affect the rheological properties of the filler-bitumen mix in a positive manner. It is also claimed that hydrated lime affects the ageing properties of the bitumen, but some questions have been raised about this effect.

As a basis for further research on problems related to low volume roads in Norway, distress types occurring on selected low volume roads were studied. The most frequent defects on these roads are:

- Edge cracks
- Cracks due to frost heave
- Rutting
- Transverse cracks
- Ageing

Ageing is not a defect by itself, but several pavement properties are influenced by the ageing process. When the binder is aged, it becomes more susceptible to development of cracks and ravelling. It will be more sensitive to development of cracking due to rutting, low temperature, deformation and frost heave. The service life of an asphalt pavement on low volume roads is highly dependent on the ageing properties of the binder and the asphalt mixture. The challenge is to find the right binder and a proper asphalt mixture according to local conditions, such as temperature, structural conditions and bearing capacity.

Ageing of bitumen is a very complex process with many factors contributing to the age hardening. Among the most important factors contributing to the ageing process in the field are the climatic conditions. In a field investigation in this work, samples from different climatic conditions have been studied. Here asphalt core samples, recovered binders and laboratory aged binders have been studied. The SHRP binder specification tests and conventional binder tests have been used in the analyses of the binders. From the analyses of these asphalt mixtures the following was found:

- The calculated load distribution factors (a-factor) are higher than the standard load distribution factors used in the Norwegian Road Design Manual for Ma-mixtures (mixtures containing soft binders, MB6000 – MB10000). It have to be remarked that values calculated from the indirect tensile test are uncertain in this case due to the fact that some of the samples were rather thin.
- The E-modulus has a tendency to increase with age for Agb-mixtures. For Ma-mixtures there are a tendency of larger E-modulus in inland compared to those found in coastal areas.
- The void contents in the investigated field samples are slightly higher than in the corresponding mixture formulations. One reason for this could be decrease in temperature before compaction due to long transportation.

Based on the binder analyses the following main conclusions could be drawn:

- The ageing of the bitumen is more severe in the top layer and decreases with depth.
- There seems to be a tendency of higher rate of binder hardening in inland compared to coastal areas. One reason for this could be higher temperatures in the summer in inland compared to coastal areas.
- The climatic conditions should be taken into account when choosing binder for asphalt pavements. In this work, it was found that Pen 180 is not suitable for inland climatic conditions when evaluating low temperature properties. This conclusion was based on visual registration of pavement conditions and from laboratory results using the BBR and corresponding SHRP binder specification criterion.
- From this very limited investigation it was not possible to find any good model for the development of ageing in pavements with soft binders. Some younger pavements have age hardened more than older ones. One reason for this could be that several factors affect the ageing. One important factor could be the influence of different transportation length.
- Field samples with soft binders have age hardened more than bitumen aged by TFOT and PAV.

A relatively large part of the age hardening occurs during production, transportation and the paving process. To investigate the magnitude of the age hardening during transportation, two transports of asphalt mixtures with freighter was studied. The main conclusions from this study are summarised as follows:

- The temperature almost remained constant in the core of the asphalt mixture up to as long as 40 hours of transport. The temperature in the bottom and the top of the asphalt mixture decreases by 1,34°C/hour and 1,2°C/hour, respectively. Varying temperatures in the asphalt mixture during compaction could contribute to varying void content in the final pavement. High void content in the final pavement is known to cause faster bitumen hardening.
- For the Pen 180 binder, the long transportation time does not seem to contribute to excessive hardening. An increased hardening seems to occur mainly in the top of the load. The soft binder MB1000 however, did harden significantly during the transportation

Since the surface area of the filler material in an asphalt mixture represents a major part of the total surface area of the aggregate, a study of how different types of fillers affect the rheological and ageing properties of filler-bitumen mixes was carried out. Four different types of fillers were used, among them hydrated lime, which is claimed to have positive influence on the ageing properties of binders. Two types of binder were used in the study. Test equipment from the SHRP binder specification was used to analyse different filler-bitumen mixes. From this study, the following conclusions can be drawn:

- When producing filler-bitumen mixes of equal stiffness, the “effective filler concentration concept”, based on Rigden void content seems to correlate with the high temperature stiffness. At low temperatures (-20/-25°C) and intermediate temperatures (10-30°C) however, these same mixtures were found to vary in stiffness.

- Based on BBR and DTT measurements of filler-bitumen mixes, the net effect of hydrated lime on low temperature cracking properties seems to be very small.
- No clear variation in ageing properties for mixes containing different types of fillers can be seen. However, there seems to be some differences between the two types of binders used in the investigation.
- The void content of filler is an important factor, which should be evaluated in the asphalt mix-design in a more comprehensive way than today.
- The SHRP-binder equipment seems to be a step forward in evaluating the properties of filler-bitumen mixes.

6.2 Recommendations for further research

Additional investigations are needed to verify the conclusions of this research work. The following recommendations are given:

- To study the development of age hardening in pavements with time, it is necessary to obtain and test samples from the same pavements over several years. To study the effect of different climates on age hardening, this should be done in areas with clear different climatic conditions.
- Evaluation of the effect of different fillers on the final asphalt pavement should be studied further. A development of mix design procedures, which incorporate the filler-bitumen properties, seems to be important. The SHRP binder equipment could be used to evaluate the properties of filler-bitumen mixes. Results from laboratory investigations should be compared with field data.
- To develop better long term performance properties for asphalt pavements on low volume roads it is necessary to improve binder specifications to distinguish between different climatic zones. It is also a need for better mix-design procedures and mixing and paving guidelines which also incorporates the effect of transportation length and transportation time.

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APPENDIX 1

Proposed new filler specification from CEN TC 154/SC 3,
11. Nov. -95.

4.3 Filler aggregate

The properties and specifications indicated in this clause apply to added filler. The requirements of the following clauses:

- 4.3.3 Particle density,
- 4.3.4.1 Compactibility,
- 4.3.4.2 "Delta Ring and Ball",
- 4.3.6 Water susceptibility,
- 4.3.7 Clay activity,
- 4.3.8 Water solubility,

also apply to the 0 mm to 0,125 mm fraction taken from fine aggregate (sand) containing ≥ 10 % of fines by mass.

4.3.1 Particle size distribution

The particle size distribution shall be determined in accordance with EN 933-10. The requirements shall comply with the values given in table 15.

Table 15: Grading requirements for added fillers

Sieve size (mm)	Overall range for individual results. Percent passing by mass	Maximum SDR*) for 90 % of results
2	100	-
0,125	85 – 100	10
0,063	70 - 100	10

*) SDR is the supplier's declared grading range which shall be declared and be within the overall range.

4.3.2 Water content

The water content of the added filler determined in accordance with EN 1097-5 shall be ≤ 1 % by mass.

4.3.3 Particle density

The particle density of the fraction 0 mm to 0,125 mm shall be determined in accordance with EN 1097-7.

4.3.4 Stiffening properties

4.3.4.1 Compactibility (Rigden)

The compactibility shall be determined in accordance with EN 1097-4. The requirements shall comply with the values given in table 16.

Table 16: Requirements for compactibility (Rigden)

Overall range for indivdual results. Percentage by volume	Maximum SDR*) for 90 % of results	Category
28 – 44	4 %	A
28 – 36	4 %	B
37 – 44	4 %	C
44 – 55	4 %	D
No requirement	-	E

*) SDR is the supplier's declared compactibility range which shall be declared and be within the overall range.

4.3.4.2 "Delta Ring and Ball" of bituminous bound fillers

The "Delta Ring and Ball" shall be determined in accordance with EN… The requirements shall comply with the values given in table 17.

Table 17: Requirements for "Delta Ring and Ball" of bituminous bound fillers

"Delta Ring and Ball	Category
8 – 22	A
8 – 16	B
17 – 22	C
≥ 23	D
No requirement	E

4.3.5 Apparent density

The apparent density shall be determined in accordance with EN 1097-3. The requirements shall comply with the values given in table 18.

Table 18: Requirements for apparent density

Apparent density, Mg/m^3	Category
0,5 – 0,9	A
No requirement	B

4.3.6 Water susceptibility

The water susceptibility of the fraction 0 mm to 0,125 mm shall be determined in accordance with EN… The requirements shall comply with the values in table 19.

Table 19: Requirements for water susceptibility

Water susceptibility	Category
≤ 10	A
No requirement	B

4.3.7 Clay activity

The clay activity of the fraction 0 mm to 0,125 mm shall be determined in accordance with EN 933-9. The requirements shall comply with the values given in table 20.

Table 20: Requirements for clay activity

Clay activity, g/kg	Category
0 – 10	A
11 – 25	B
No requirement	C

4.3.8 Water solubility

The water solubility of the fracytion 0 mm to 0,125 mm shall be determined in accordance with clause 15 of EN 1744-1. The requirements shall comply with the values given in table 21.

Table 21: Requirements for water solubility

Water solubility	Category
≤ 10	A
No requirement	B

4.3.9 Consistency of filler production

The consistency of filler production shall be measured on at least on e of the following properties.

4.3.9.1 "Bitumen number" of added filler

The "bitumen number" of added filler shall be determined in accordance with EN… The suppliers declared range shall be 6.

4.3.9.2 Loss on ignition of fly ash

The loss on ignition of pulverized coal fly ash used as filler shall be determined in accordance with clause 16 of EN 17744-1. The suppliers declared range shall be 6 % by mass.

4.3.9.3 Particle density of added filler

The particle density of added filler shall be determined in accordance with EN 1097-7. The suppliers declared range shall be 0,2 Mg/m^3.

4.3.10 Carbonate content

The carbonate content shall be determined in accordance with EN 196-2. The requirements shall comply with values given in table 22.

Table 22: Requirements for carbonate content

Carbonate content. Percentage by mass	Category
≥ 90	A
≥ 80	B
≥ 70	C
No requirement	D

4.3.11 Calcium oxide and dihydroxide content

The calcium oxide and dihydroxide content shall be determined in accordance with EN 459-2. The requirements shall comply with the values in table 23.

Table 23: Requirements for calcium oxide and dihydroxide content

Calcium oxide and dihydroxide content. Percentage by mass	Category
≥ 25	A
≥ 10	B
No requirement	C

APPENDIX 2

Results from visual inspection of pavement surface conditions (chapter 2)

Results from visual registration on pavement surface conditions.

Investigation number 1.

The investigation was carried out in central Norway in 1991.
The results from the investigation are given in Table A2.1. In Table A2.1 the extent of the cracks is given as percent of total registration length. On some distances there can be more than one damage occurring at the same time. Total registration length was 662.27 km.

Table A2.1 Result from registration of cracks on low-volume roads in central Norway.

Type of crack	Extent of total registration length (%)
Crack due to frost heave	7.16
Edge cracks	6.10
Cracks due to plastic deformation	1.13
Fatigue cracks	0.99

Investigation number 2.

The investigation was carried out in central Norway in 1992.
The results from the investigation are given in Table A2.2. In Table A2.2 the extent of the defects is given as percent of total registration length. On some sections more than one damage occurs at the same time. For defects occurring only at one single point, the frequency of defects is given as number pr. km. Total registration length was 328.97 km. This was 268,294 km governmental roads and 60,680 km local roads.

Table A2.2 Result from registration of defects on low-volume roads in central Norway.

Type of defect	Extent of total registration length (%)	Number pr. km.
Edge crack	26,2	
Crack due to frost heave	19,0	
Rutting	5,5	
Crocodile cracks	4,6	
Cracks due to rutting	2.4	
Stripping	1,9	
Ageing	1,8	
Separating in asphalt-mix	0,6	
Defect after patch	0,5	
Corrugation	0,2	
Local shear-rupture	0,1	
Shear cracks	0,02	
Transverse crack		1,8
Pothole		0,85
Diagonal crack		0,17

Table A2.3 gives the result divided into governmental- and local roads

Table A2.3 *Result from registration of cracks on low-volume roads in central Norway divided into governmental- and local roads.*

Type of defect	**Extent of total registration length, governmental roads (%)**	**Extent of total registration length, local roads (%)**
Edge crack	26,8	23,6
Crack due to frost heave	20,8	13,3
Rutting	5,5	5,6
Crocodile cracks	3,7	8,3
Cracks due to rutting	2,8	0,7
Stripping	2,2	1,0
Ageing	0,8	6,2
Separating in asphalt-mix	0,7	0,4
Defect after patch	0,2	1,9
Corrugation	0,1	0.7
Local shear-rupture	0,1	0

Investigation number 3.

The investigation was carried out in western Norway in 1992.
The results from the investigation are given in Table A2.4. In Table A2.4 the extent of the defects is given as percent of total registration length. On some distances there can be more than one damage occurring at the same time. For defects occurring only at a point, number pr. km. is given. Total registration length was 20.4 km.

Table A2.4 *Result from registration of defects on low-volume roads in western Norway.*

Type of defect	**Extent of total registration length (%)**	**Number pr. km.**
Rutting	11.0	
Stripping	10.6	
Crocodile cracks	10.4	
Edge crack	10.3	
Ageing	7.7	
Defect after patch	7.4	
Edge defect	4.5	
Cracks in wheel path	4.3	
Cracks due to frost heave	3.5	
Deformation	2.8	
Other defects	1.2	
Pothole		6.8
Transverse crack		2.4

Investigation number 4.

The investigation was carried out in northern Norway in 1993.
The results from the investigation are given in Table A2.5. In Table A2.5 the extent of the defects is given as percent of total registration length. On some distances there can be more than one damage occurring at different zone. For defects occurring only at a point, number pr. km. is given. Total registration length was 283,442 km. There was 168,528 km on E6, 92,595 km governmental roads and 22,319 km local roads.

Table A2.5 *Result from registration of defects on low-volume roads in northern Norway, Finnmark.*

	Extent of total registration length (%)		
Type of defect	**Left zone**	**Median zone**	**Right zone**
Ageing	12.34	12.53	12.09
Edge crack	2.96		5.03
Longitudinal crack	1.42	2.39	1.96
Crocodile cracks	0.09	0.34	0.67
Defect after patch		0.31	
Diagonal crack	0.11	0.05	0.09
No defect	82.64	84.19	77.71
	Number pr. km.		
Type of defect	**Left zone**	**Median zone**	**Right zone**
Transverse crack	0.14	17.31	0.11
Pothole	0.54	1.10	0.55
Local shear-rupture	0.23		0.59
Defect after patch		0.01	

In Table A2.6 the defects are divided into road in the coastal area and the inland area.

Table A2.6 *Result from registration of defects on low-volume roads in northern Norway, Finnmark, divided into roads in the coastal area and in the inland area.*

	Extent of total registration length (%)					
Type of defect	**Left zone**		**Median zone**		**Right zone**	
	Coast	**Inland**	**Coast**	**Inland**	**Coast**	**Inland**
Ageing	30.75	0.22	30.68	0.59	29.76	0.45
Longitudinal crack	2.25	1.34	2.90	2.05	2.36	1.70
Edge crack	0.81	4.38			1.51	11.18
Crocodile cracks	0.07	0.09	0.31	0.35	0.33	0.90
Defect after patch			0.10	0.44		
Diagonal crack	0.11	0.10		0.08	0.13	0.05
No defect	66.04	93.58	65.93	96.22	65.89	85.49
	Number pr. km.					
Type of defect	**Left zone**		**Median zone**		**Right zone**	
	Coast	**Inland**	**Coast**	**Inland**	**Coast**	**Inland**
Transverse crack	0.14	0.14	10.98	21.49	0.14	0.08
Pothole	0.63	0.49	1.67	0.73	0.71	0.44
Local shear-rupture	0.19	0.26			0.55	0.62
Defect after patch				0.02		

Investigation number 5.

The investigation was carried out in northern Norway in 1994.
The results from the investigation are given in Table A2.7. In Table A2.7 the extent of the cracks is given as percent of total registration length. On some distances there can be more than one damage occurring at the same time. Totally 28.4 km was registrated.

Table A2.7 Result from registration of defects on low-volume roads in northern Norway, Nordland.

Type of defect	**Extent of total registration length (%)**
Edge crack	29
Rutting	14.2
Flushing	8.4
Defect after patch	6.7
Crack due to joints	6.4
Crack due to frost heave	5.5
Other cracks	5.3
Deformation	4.1
Cracks in wheel path	3.9
Crocodile cracks	2.0
Separation in asphalt mix	2.0
Shear cracks	0.1
No defects	44.7

APPENDIX 3

Temperature maps for Norway (summer and winther)
(chapter 3)

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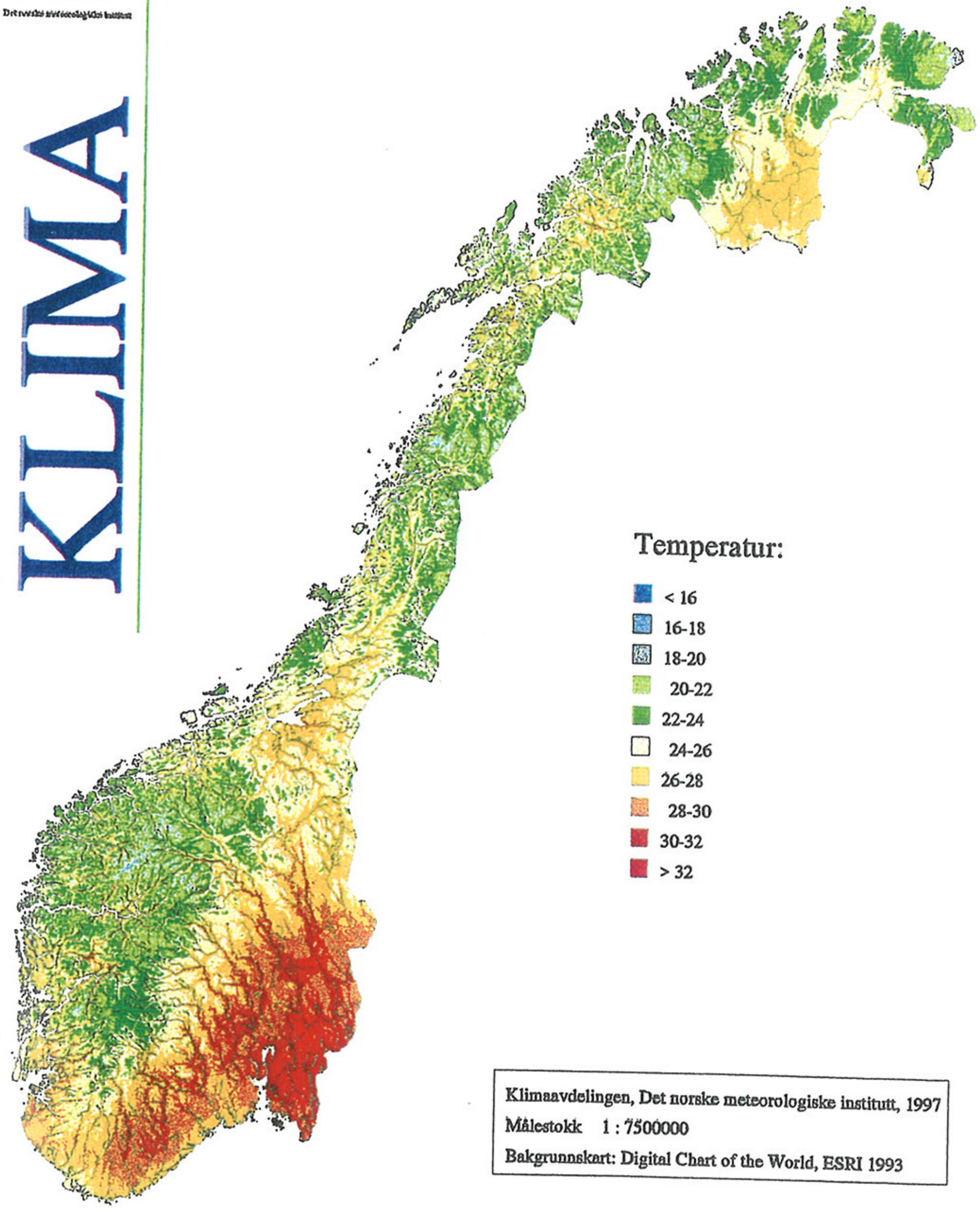
TEMPERATURKART

Høyeste lufttemperatur (7 døgn, T_{max}, 98 %)



Klimaavdelingen, Det norske meteorologiske institutt, 1997
Målestokk 1 : 7500000
Bakgrunnskart: Digital Chart of the World, ESRI 1993

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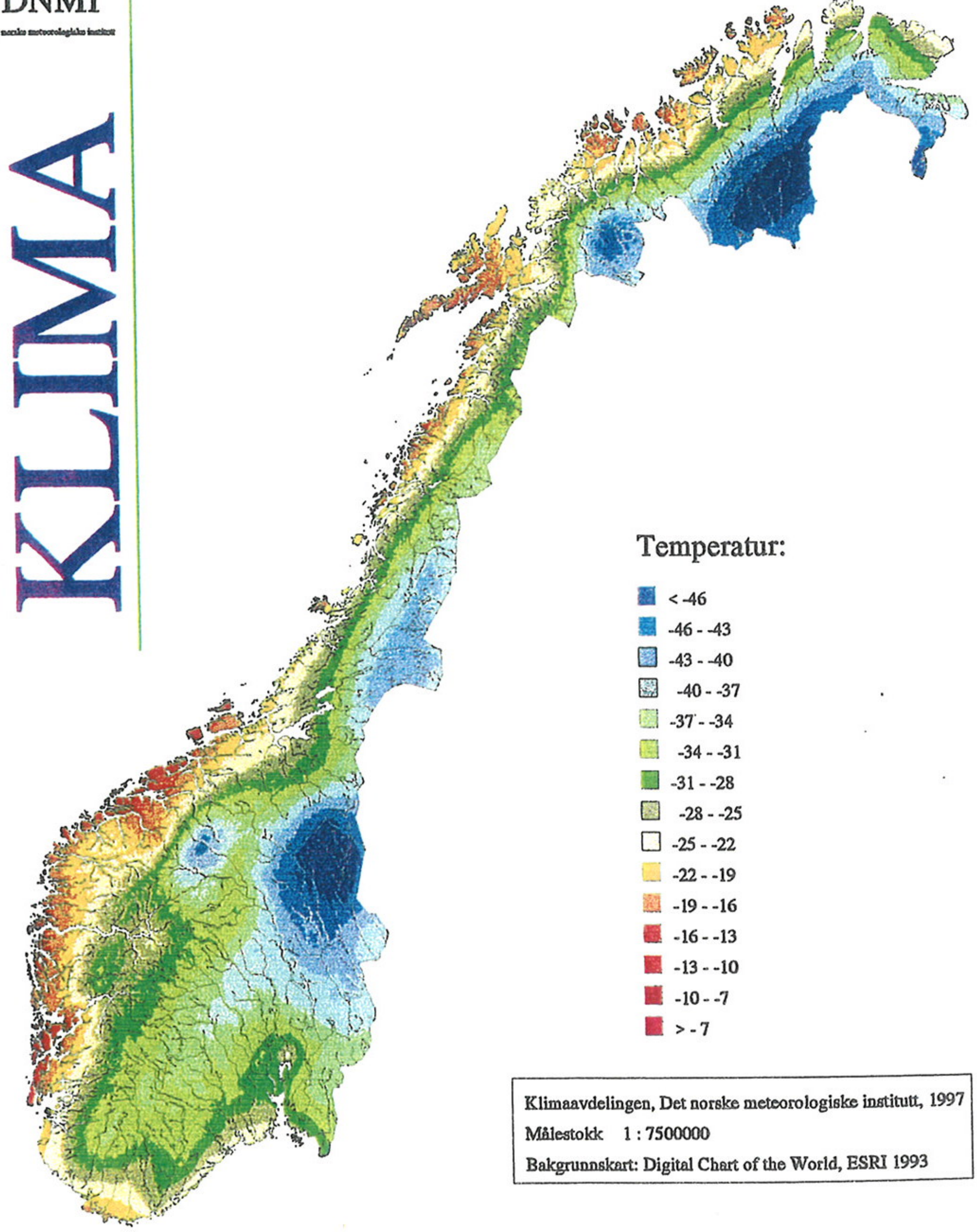
Det norske meteorologiske institutt

KLIMA

TEMPERATURKART

Laveste lufttemperatur (T_{min}, 98 %)



Klimaavdelingen, Det norske meteorologiske institutt, 1997
Målestokk 1 : 7500000
Bakgrunnskart: Digital Chart of the World, ESRI 1993

APPENDIX 4

Detailed results from asphalt-mix analyses (chapter 3)

Sample nr.	Binder	Pavement age (year)	Aggregate density (g/cm3)	Binder content (%)	Void content (%)	Binder in void (%)	Strength (kN)	Defor-mation (mm)	Tensile strength (kPa)	E-modulus (MPa)	Load distribution coeff. (a)	Normal value (a)
1	MB 10000	4	2,815	5,24	10,3	53,8	0,99	2,33	221	1446	2,30	1,5
3	BE70S	1	2,675	3,18	12,7	35,6	0,12	1,60	24	248	1,10	1,25
7	VO550	11	2,633	3,43	9,8	43,8	0,91	1,62	150	1017	2,02	1,25
8	VO550	11	2,697	3,87	11,5	42,7	1,17	1,52	206	1355	2,24	1,25
9	VO550	12	2,669	3,06	7,2	49,8	0,43	1,52	72	539	1,58	1,25
11	MB 10000	3	2,890	5,01	6,1	67,0	0,55	1,96	199	1316	2,22	1,5
12	MB 10000	1	2,920	5,58	4,5	75,6	1,29	2,77	334	2137	2,64	1,5
13	MB 8000	7	2,702	4,01	8,8	50,9	1,09	1,60	224	1464	2,31	1,5
14	MB 10000	3	2,932	5,22	5,6	70,0	1,50	2,95	378	2408	2,75	1,5
15	B 180	1	2,919	4,45	9,5	53,3	0,53	1,91	139	946	1,97	3
16	MB 10000	4	2,758	4,94	9,8	53,5	1,05	2,33	264	1712	2,44	1,5
17	MB 8000	8	2,705	5,28	3,0	80,8	0,72	1,95	171	1141	2,11	1,5
18	MB 10000	1	2,736	4,92	9,3	54,6	1,03	1,99	238	1554	2,36	1,5
19	MB 6000	9	2,708	4,86	5,7	66,5	1,02	2,15	159	1068	2,06	1,5
20	MB 10000	1	2,748	4,77	8,3	56,8	1,02	1,82	185	1231	2,17	1,5
21	MB 6000	9	2,698	4,52	9,3	52,4	0,56	1,72	106	747	1,80	1,5
22	B180	11	2,719	5,46	3,8	77,3	2,36	3,07	323	2070	2,61	3
26	B180	3	2,914	5,81	4,2	77,3	2,00	3,58	300	1932	2,54	3
27	B180	3	2,924	5,67	4,8	74,9	1,82	3,52	305	1958	2,56	3
28	B180	1	2,907	5,71	1,8	88,9	1,74	3,29	312	2001	2,58	3
29	B180	1	2,869	4,26	6,9	60,1	2,68	2,82	581	3644	3,17	3
30	MB 6000	14	2,615	4,25	5,0	66,2	0,85	2,07	153	1031	2,03	1,5
31	MB 6000	14	2,665	4,58	7,4	58,6	1,18	2,39	258	1676	2,42	1,5
32	MB 8000	7	2,675	5,16	6,8	63,3	0,77	2,01	160	1074	2,06	1,5

Sample nr.	Asphalt mix	Binder	Inland/ coast	Pavement age (year)	Aggregate density (g/cm3)	Heights in mm				Binder content (%)	Weight (g)	Volume (cm3)	Compact density (g/cm3)	Density (g/cm3)	Void content (%)	Binder in void (%)	Strength (kN)	Defor-mation (mm)	Tensile strength (kPa)	E-modulus (MPa)	Load distribution coeff. (a)	Normal value (a)
						Height-1	Height-2	Height-3	Medium height													
1A	Ma16	MB 10000	Coast	4	2,815	32,10	31,80	31,10	31,67	5,24	550,9	237,9	2,574	2,309	10,3	53,8	0,92	2,30	184	1222	2,16	1,5
1B	Ma16	MB 10000	Coast	4	2,815	26,10	26,60	25,70	26,13	5,24	461,0	197,3	2,574	2,330	9,5	56,0	1,10	2,40	268	1735	2,45	1,5
1C	Ma16	MB 10000	Coast	4	2,815	29,50	28,40	28,70	28,87	5,24	490,9	213,9	2,574	2,288	11,1	51,7	0,96	2,30	210	1381	2,26	1,5
1	Ma16	MB 10000	Coast	4	2,815					5,24					10,3	53,8	0,99	2,33	221	1446	2,30	1,5
3A	Eg11	BE70S	Inland	1	2,675	31,20	33,00	31,10	31,77	3,18	499,9	225,5	2,542	2,210	13,0	34,8	0,14	1,30	27	265	1,14	1,25
3B	Eg11	BE70S	Inland	1	2,675	30,10	30,40	29,90	30,13	3,18	478,6	213,8	2,542	2,232	12,2	36,6	0,09	1,40	20	222	1,03	1,25
3C	Eg11	BE70S	Inland	1	2,675	31,00	31,40	30,30	30,90	3,18	474,2	213,2	2,542	2,218	12,7	35,4	0,13	2,09	26	259	1,13	1,25
3	Eg11	BE70S	Inland	1	2,675					3,18					12,7	35,6	0,12	1,60	24	248	1,10	1,25
7A	Og16	VO550	Inland	11	2,633	40,40	39,90	39,20	39,83	3,43	675,5	297,9	2,495	2,261	9,4	45,0	1,00	1,50	160	1076	2,06	1,25
7B	Og16	VO550	Inland	11	2,633	38,70	40,30	38,70	39,23	3,43	649,7	290,0	2,495	2,234	10,5	42,0	0,86	1,48	140	954	1,97	1,25
7C	Og16	VO550	Inland	11	2,633	37,60	37,50	37,20	37,43	3,43	641,9	283,7	2,495	2,256	9,6	44,4	0,88	1,89	151	1021	2,02	1,25
7	Og16	VO550	Inland	11	2,633					3,43					9,8	43,8	0,91	1,62	150	1017	2,02	1,25
8A	Og16	VO550	Coast	11	2,697	36,10	36,10	36,80	36,33	3,87	601,4	268,0	2,533	2,238	11,7	42,3	1,40	1,52	249	1619	2,39	1,25
8B	Og16	VO550	Coast	11	2,697	36,60	36,10	36,70	36,47	3,87	603,2	270,4	2,533	2,224	12,2	41,1	1,10	1,56	194	1283	2,20	1,25
8C	Og16	VO550	Coast	11	2,697	37,80	38,60	36,60	37,67	3,87	624,3	275,3	2,533	2,261	10,7	44,7	1,00	1,48	174	1161	2,12	1,25
8	Og16	VO550	Coast	11	2,697					3,87					11,5	42,7	1,17	1,52	206	1355	2,24	1,25
9A	Og16	VO550	Coast	12	2,669	37,50	38,10	36,90	37,50	3,06	626,4	266,8	2,541	2,341	7,9	47,4	0,40	1,70	70	527	1,57	1,25
9B	Og16	VO550	Coast	12	2,669	39,50	39,90	39,30	39,57	3,06	674,6	283,9	2,541	2,369	6,8	51,5	0,48	1,39	79	582	1,63	1,25
9C	Og16	VO550	Coast	12	2,669	40,30	40,10	39,50	39,97	3,06	680,9	287,2	2,541	2,364	7,0	50,7	0,42	1,48	67	509	1,54	1,25
9	Og16	VO550	Coast	12	2,669					3,06					7,2	49,8	0,43	1,52	72	539	1,58	1,25
11A	Ma16	MB 10000	Inland	3	2,890	16,70	17,10	16,50	16,77	5,01	311,2	126,4	2,643	2,455	7,1	63,1	0,56	2,48	210	1381	2,26	1,5
11B	Ma16	MB 10000	Inland	3	2,890	16,60	15,50	16,80	16,30	5,01	305,0	121,5	2,643	2,503	5,3	70,0	0,36	1,48	142	966	1,98	1,5
11C	Ma16	MB 10000	Inland	3	2,890	18,70	19,20	19,20	19,03	5,01	360,4	144,4	2,643	2,489	5,9	67,8	0,73	1,91	246	1601	2,38	1,5
11	Ma16	MB 10000	Inland	3	2,890					5,01					6,1	67,0	0,55	1,96	199	1316	2,22	1,5
12A	Ma14	MB 10000	Inland	1	2,920	26,20	25,35	26,30	25,95	5,58	490,9	194,9	2,641	2,511	4,9	73,8	1,35	2,91	329	2107	2,62	1,5
12B	Ma14	MB 10000	Inland	1	2,920	19,20	19,60	19,20	19,33	5,58	375,9	146,6	2,641	2,557	3,2	81,5	1,20	2,74	335	2144	2,64	1,5
12C	Ma14	MB 10000	Inland	1	2,920	24,90	25,50	24,70	25,03	5,58	475,0	189,8	2,641	2,495	5,5	71,4	1,33	2,65	338	2162	2,65	1,5
12	Ma14	MB 10000	Inland	1	2,920					5,58					4,5	75,6	1,29	2,77	334	2137	2,64	1,5
13A	Ma16	MB 8000	Inland	7	2,702	25,00	24,80	25,00	24,93	4,01	418,2	181,8	2,532	2,294	9,4	49,2	0,78	1,70	200	1320	2,22	1,5
13B	Ma16	MB 8000	Inland	7	2,702	26,40	28,20	27,40	27,33	4,01	481,0	206,9	2,532	2,318	8,4	52,1	1,20	1,39	239	1558	2,36	1,5
13C	Ma16	MB 8000	Inland	7	2,702	28,50	28,00	28,85	28,45	4,01	495,8	213,8	2,532	2,312	8,7	51,4	1,30	1,70	232	1515	2,33	1,5
13	Ma16	MB 8000	Inland	7	2,702					4,01					8,8	50,9	1,09	1,60	224	1464	2,31	1,5
14A	Ma16	MB 10000	Inland	3	2,932	26,10	26,20	26,00	26,10	5,22	482,3	190,6	2,667	2,523	5,4	70,7	1,40	3,17	343	2192	2,66	1,5
14B	Ma16	MB 10000	Inland	3	2,932	28,00	24,60	26,50	26,37	5,22	484,1	193,8	2,667	2,491	6,6	66,1	1,62	2,98	391	2485	2,78	1,5
14C	Ma16	MB 10000	Inland	3	2,932	22,90	24,40	23,90	23,73	5,22	457,2	179,6	2,667	2,538	4,8	73,1	1,48	2,70	401	2546	2,80	1,5
14	Ma16	MB 10000	Inland	3	2,932					5,22					5,6	70,0	1,50	2,95	378	2408	2,75	1,5
15A	Agb14	B 180	Inland	1	2,919	27,30	27,70	27,00	27,33	4,45	488,0	202,2	2,693	2,406	10,6	50,0	0,54	2,11	128	881	1,92	3
15B	Agb14	B 180	Inland	1	2,919	25,70	24,70	25,40	25,27	4,45	467,2	188,6	2,693	2,470	8,3	56,8	0,60	2,09	151	1021	2,02	3
15C	Agb14	B 180	Inland	1	2,919	21,30	21,50	21,20	21,33	4,45	383,6	157,0	2,693	2,436	9,5	53,0	0,46	1,54	137	936	1,96	3
15	Agb14	B 180	Inland	1	2,919					4,45					9,5	53,3	0,53	1,91	139	946	1,97	3
16A	Ma16	MB 10000	Coast	4	2,758	26,10	25,80	25,80	25,90	4,94	439,2	191,9	2,541	2,282	10,2	52,3	1,06	2,00	264	1710	2,44	1,5
16B	Ma16	MB 10000	Coast	4	2,758	26,80	26,90	27,50	27,07	4,94	447,1	195,5	2,541	2,280	10,2	52,1	1,16	2,43	276	1784	2,47	1,5
16C	Ma16	MB 10000	Coast	4	2,758	22,90	24,40	23,20	23,50	4,94	398,7	171,7	2,541	2,315	8,9	56,1	0,92	2,57	253	1643	2,40	1,5
16	Ma16	MB 10000	Coast	4	2,758					4,94					9,8	53,5	1,05	2,33	264	1712	2,44	1,5
17A	Ma16	MB 8000	Coast	8	2,705	26,60	26,20	26,90	26,57	5,28	479,1	198,0	2,485	2,413	2,9	81,3	0,69	1,78	169	1131	2,10	1,5
17B	Ma16	MB 8000	Coast	8	2,705	26,50	26,80	27,60	26,97	5,28	487,0	201,7	2,485	2,408	3,1	80,2	0,80	2,04	193	1277	2,20	1,5
17C	Ma16	MB 8000	Coast	8	2,705	27,70	28,60	28,80	28,37	5,28	512,6	212,0	2,485	2,411	3,0	80,9	0,66	2,04	150	1015	2,02	1,5
17	Ma16	MB 8000	Coast	8	2,705					5,28					3,0	80,8	0,72	1,95	171	1141	2,11	1,5
18A	Ma11	MB 10000	Coast	1	2,736	27,40	27,60	27,10	27,37	4,92	456,6	200,1	2,524	2,275	9,8	53,0	0,87	1,83	204	1344	2,24	1,5
18B	Ma11	MB 10000	Coast	1	2,736	29,00	29,20	29,70	29,30	4,92	504,7	217,3	2,524	2,316	8,2	57,8	1,25	2,37	278	1796	2,48	1,5
18C	Ma11	MB 10000	Coast	1	2,736	27,40	26,50	26,10	26,67	4,92	444,4	194,6	2,524	2,277	9,8	53,2	0,96	1,78	233	1521	2,34	1,5
18	Ma11	MB 10000	Coast	1	2,736					4,92					9,3	54,6	1,03	1,99	238	1554	2,36	1,5

19A	Ma16	MB 6000	Coast	9	2,708	38,70	38,20	37,60	38,17	4,86	672,1	283,3	2,503	2,366	5,5	67,4	1,30	1,80	175	1168	2,13	1,5
19B	Ma16	MB 6000	Coast	9	2,708	38,60	37,00	38,60	38,07	4,86	652,9	275,3	2,503	2,365	5,5	67,3	0,82	2,43	141	960	1,98	1,5
19C	Ma16	MB 6000	Coast	9	2,708	38,60	39,20	39,10	38,97	4,86	639,1	271,1	2,503	2,351	6,1	64,9	0,95	2,22	160	1076	2,06	1,5
19	Ma16	MB 6000	Coast	9	2,708					4,86					5,7	66,5	1,02	2,15	159	1068	2,06	1,5
20A	Ma11	MB 10000	Coast	1	2,748	37,20	38,40	37,50	37,70	4,77	641,2	274,5	2,540	2,329	8,3	57,0	1,14	1,65	196	1296	2,21	1,5
20B	Ma11	MB 10000	Coast	1	2,748	36,80	34,90	34,60	35,43	4,77	598,5	256,1	2,540	2,330	8,2	57,2	1,00	1,72	182	1210	2,15	1,5
20C	Ma11	MB 10000	Coast	1	2,748	34,40	33,10	33,20	33,57	4,77	577,1	247,7	2,540	2,323	8,5	56,3	0,93	2,09	178	1186	2,14	1,5
20	Ma11	MB 10000	Coast	1	2,748					4,77					8,3	56,8	1,02	1,82	185	1231	2,17	1,5
21A	Ma16	MB 6000	Coast	9	2,698	34,50	33,80	34,80	34,37	4,52	603,6	265,1	2,509	2,270	9,5	51,7	0,56	1,63	102	722	1,78	1,5
21B	Ma16	MB 6000	Coast	9	2,698	32,70	33,40	32,60	32,90	4,52	585,6	256,5	2,509	2,276	9,3	52,4	0,61	1,63	119	826	1,87	1,5
21C	Ma16	MB 6000	Coast	9	2,698	33,10	33,50	33,60	33,40	4,52	563,4	246,2	2,509	2,282	9,0	53,0	0,51	1,91	97	692	1,75	1,5
21	Ma16	MB 6000	Coast	9	2,698					4,52					9,3	52,4	0,56	1,72	106	747	1,80	1,5
22A	Agb16	B180	Inland	11	2,719	47,00	46,80	47,20	47,00	5,46	864,2	359,8	2,489	2,395	3,8	77,4	1,86	3,04	253	1643	2,40	3
22B	Agb16	B180	Inland	11	2,719	47,50	46,80	46,30	46,87	5,46	873,6	362,8	2,489	2,401	3,5	78,6	2,46	3,09	341	2180	2,65	3
22C	Agb16	B180	Inland	11	2,719	46,80	47,20	46,60	46,87	5,46	866,4	362,0	2,489	2,386	4,1	75,8	2,76	3,09	375	2388	2,74	3
22	Agb16	B180	Inland	11	2,719					5,46					3,8	77,3	2,36	3,07	323	2070	2,61	3
26A	Agb16	B180	Inland	3	2,914	40,20	41,60	41,90	41,23	5,81	796,7	314,7	2,626	2,524	3,9	78,9	2,01	3,74	312	2003	2,58	3
26B	Agb16	B180	Inland	3	2,914	45,00	44,10	45,00	44,70	5,81	878,4	347,6	2,626	2,520	4,1	78,1	2,02	3,91	289	1863	2,51	3
26C	Agb16	B180	Inland	3	2,914	41,90	42,20	41,80	41,97	5,81	802,9	320,2	2,626	2,500	4,8	75,0	1,98	3,09	300	1930	2,54	3
26	Agb16	B180	Inland	3	2,914					5,81					4,2	77,3	2,00	3,58	300	1932	2,54	3
27A	Agb16	B180	Inland	3	2,924	37,30	35,60	36,20	36,37	5,67	700,4	277,4	2,640	2,518	4,6	75,3	1,66	3,65	292	1881	2,52	3
27B	Agb16	B180	Inland	3	2,924	38,60	38,50	38,00	38,37	5,67	719,6	288,6	2,640	2,486	5,8	70,5	1,76	3,61	294	1893	2,53	3
27C	Agb16	B180	Inland	3	2,924	39,50	40,80	40,30	40,20	5,67	781,5	306,8	2,640	2,540	3,8	78,9	2,05	3,30	328	2101	2,62	3
27	Agb16	B180	Inland	3	2,924					5,67					4,8	74,9	1,82	3,52	305	1958	2,56	3
28A	Agb14	B180	Inland	1	2,907	38,40	37,20	38,00	37,87	5,71	746,8	289,3	2,625	2,574	2,0	88,1	1,75	3,41	295	1900	2,53	3
28B	Agb14	B180	Inland	1	2,907	33,60	34,10	34,70	34,13	5,71	678,8	262,2	2,625	2,581	1,7	89,7	1,61	3,41	302	1942	2,55	3
28C	Agb14	B180	Inland	1	2,907	34,90	35,40	35,00	35,10	5,71	693,7	268,3	2,625	2,578	1,8	89,0	1,85	3,04	338	2162	2,65	3
28	Ag14	B180	Inland	1	2,907					5,71					1,8	88,9	1,74	3,29	312	2001	2,58	3
29A	Ag14	B180	Inland	1	2,869	30,70	30,70	31,20	30,87	4,26	560,7	224,8	2,660	2,487	6,5	61,7	2,71	2,98	560	3516	3,13	3
29B	Ag14	B180	Inland	1	2,869	30,50	30,50	30,50	30,50	4,26	546,5	221,9	2,660	2,456	7,7	57,4	2,57	2,57	537	3376	3,09	3
29C	Ag14	B180	Inland	1	2,869	27,10	27,62	27,10	27,27	4,26	506,1	203,1	2,660	2,485	6,6	61,3	2,76	2,91	646	4041	3,28	3
29	Ag14	B180	Inland	1	2,869					4,26					6,9	60,1	2,68	2,82	581	3644	3,17	3
30A	Ma16	MB 6000	Inland	14	2,615	35,70	37,00	37,70	36,80	4,25	633,8	271,4	2,450	2,329	4,9	66,5	0,91	2,04	159	1070	2,06	1,5
30B	Ma16	MB 6000	Inland	14	2,615	37,20	35,50	34,20	35,63	4,25	623,9	267,4	2,450	2,326	5,0	66,1	0,91	2,04	163	1094	2,08	1,5
30C	Ma16	MB 6000	Inland	14	2,615	34,40	35,60	34,30	34,77	4,25	604,3	259,0	2,450	2,326	5,0	66,1	0,74	2,13	136	930	1,95	1,5
30	Ma16	MB 6000	Inland	14	2,615					4,25					5,0	66,2	0,85	2,07	153	1031	2,03	1,5
31A	Ma16	MB 6000	Inland	14	2,665	30,20	29,60	28,50	29,43	4,58	502,7	221,2	2,479	2,266	8,6	54,5	1,32	2,67	288	1857	2,51	1,5
31B	Ma16	MB 6000	Inland	14	2,665	29,60	28,50	30,10	29,40	4,58	515,3	223,0	2,479	2,304	7,1	59,7	1,13	1,97	246	1601	2,38	1,5
31C	Ma16	MB 6000	Inland	14	2,665	29,40	29,20	28,70	29,10	4,58	510,8	219,8	2,479	2,317	6,5	61,7	1,10	2,52	241	1570	2,36	1,5
31	Ma16	MB 6000	Inland	14	2,665					4,58					7,4	58,6	1,18	2,39	258	1676	2,42	1,5
32A	Ma16	MB 8000	Inland	7	2,675	28,30	27,60	28,50	28,13	5,16	500,2	215,9	2,465	2,310	6,3	65,2	0,73	1,72	167	1119	2,09	1,5
32B	Ma16	MB 8000	Inland	7	2,675	31,80	32,50	33,00	32,43	5,16	556,7	243,1	2,465	2,283	7,4	61,3	0,76	2,30	151	1021	2,02	1,5
32C	Ma16	MB 8000	Inland	7	2,675	33,00	32,70	33,00	32,90	5,16	574,8	249,3	2,465	2,299	6,7	63,5	0,82	2,02	161	1082	2,07	1,5
32	Ma16	MB 8000	Inland	7	2,675					5,16					6,8	63,3	0,77	2,01	160	1074	2,06	1,5

APPENDIX 5

Detailed results from analysis on recovered bitumen (chapter 3)

Result from analysis of recovered binder

Sample no	Binder	Inland/coast	Pavement age (year)	Temp where S=300 MPa (°C)	Temp where lml=0,300 MPa (°C)	Layer 1, temp where G* x sin d = 5000 kPa (°C)	Layer 2, temp where G* x sin d = 5000 kPa (°C)	Layer 1, Pen, 10°C	Layer 2, Pen, 10°C	Layer 1, Pen, 25°C	Layer 2, Pen, 25°C	Layer 1, Softning point, °C	Layer 2, Softning point, °C	Layer 1, viscosity 60°C (cPoise)	Layer 1, viscosity 90°C (cPoise)	Layer 2, viscosity 60°C (cPoise)	Layer 2, viscosity 90°C (cPoise)
29	B180	Inland	1	-21,8	-24,6	13,3				53		49,9		442 000	10 800		
27	B180	Inland	3	-24,1	-27,4	10,0	9,6			82	93	45,4	43,5	196 000	7 260	168 000	6 140
22	B180	Inland	11	-25,8	-28,1	9,6	5,3			76	137	46,6	40,9	281 000	6 400	119 000	5 140
11	MB 10000	Inland	1	-30,6	-34,8	-2,0		60				32,3		38 200	1 940		
12	MB 10000	Inland	1	-24,5	-28,5	9,2	5,0	24	33			42,8	39,8	133 000	5 480	97 600	4 120
18	MB 10000	Coast	1	-31,9	-33,8	2,3		53				34,0		47 200	2 200		
1	MB 10000	Coast	4	-30,0	-31,5	4,5	-4,0	39	54			37,5	34,5	62 200	2 920	39 400	2 160
32	MB 8000	Inland	7	-32,8	-36,2	-3,8		84				29,8		26 800	1 460		
13	MB 8000	Inland	7	-32,5	-36,5	-3,0	-2,6	68	92			31,8	29,1	38 000	1 840	24 600	1 460
17	MB 8000	Coast	8	-33,9	-39,1	-4,1	-6,2	97	125			28,4	26,3	25 400	1 450	17 000	1 110
30	MB 6000	Inland	14	-34,0	-37,5	-10,5	-5,1	51	59			36,0	32,6	56 200	2 560	52 200	2 300
31	MB 6000	Inland	14	-34,7	-37,1	-5,4		52				35,4		60 400	2 420		
19	MB 6000	Coast	9	-35,3	-40,3	-5,4	-4,1	93	91			29,2	29,4	23 000	1 490	25 200	1 470
3	MB 1500	Inland	1	-43,2	-38,9	-20,0						10,5		2 940	324		
7	VO 550	Inland	11	-45,9	-42,8	-15,2						21,4		8 340	695		
8	VO 550	Coast	11	-34,7	-37,4	-5,7						38,3		64 000	3 220		
9	VO 550	Coast	12	-43,1	-38,3	-17,0						17,0		5 200	500		

Sample 29, layer 1

B180

Dynamic shear rheometer, RECOVERED BINDER

Temp where G* x sind = 5000 kPa (°C) **13,3**

Temp (°C)	10	10	16	16	22	22	28	28	34	34	**10**	**16**	**22**	**28**	**34**
G* (kPa)	10834,0	11212,0	3995,0	4190,0	1407,4	1475,2	514,0	511,0	187,7	185,1	**11023,0**	**4092,500**	**1441,300**	**512,500**	**186,4**
δ (°)	48,5	48,2	55,4	55,2	60,9	61,3	65,7	65,8	69,6	69,5	**48,4**	**55,300**	**61,100**	**65,750**	**69,6**
G* x sinδ (kPa)	8117,1	8360,1	3286,5	3440,6	1230,4	1293,6	468,4	466,1	176,0	173,4	**8238,6**	**3363,550**	**1262,000**	**467,265**	**174,7**
γ (%)	0,79	0,76	1,00	0,96	1,43	1,36	1,96	1,97	2,71	2,75	**0,78**	**0,980**	**1,395**	**1,965**	**2,73**

Bending Beam rheometer

Temp where S = 300 MPa (°C) **-21,8**

Temp where |m| = 0,300 (°C) **-24,6**

Temp (°C)	-30	-30	-20	-20	**-30**	**-20**
S(60 sek) (MPa)	899,8	925,3	230,2	237,6	**912,6**	**233,9**
|m| (60 sek)	0,221	0,223	0,359	0,374	**0,222**	**0,367**

25°C	**Penetrasjon ($^{1}/_{10}$ mm)**			
Binder	**1**	**2**	**3**	**Average**
Revovered	52,6	52,6	52,5	**53**

	Dynamic viscosity (cPoise)		
Binder	60°C	90°C	Skjærhast.
Recovered	442000	10800	

Measure nr.	**Softning point,°C**
1	49,5
2	50,3
Average	**49,9**

Sample 27, layer 1

B180

8mm spindel

Dynamic shear rheometer, RECOVERED BINDER

Temp where G* x sind = 5000 kPa (°C) **10,0**

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	5955,7	6344,8	2060,0	2211,4	727,8	776,8	258,2	268,1	95,2	98,2	**6150,3**	**2135,700**	**752,300**	**263,150**	**96,7**
δ (°)	54,2	53,9	60,8	60,4	65,9	65,5	69,6	69,5	72,5	72,6	**54,1**	**60,600**	**65,700**	**69,550**	**72,6**
G* x sinδ (kPa)	4830,6	5128,3	1798,4	1923,8	664,6	707,1	242,0	251,1	90,8	93,7	**4979,5**	**1861,100**	**685,850**	**246,550**	**92,3**
γ (%)	0,92	0,95	1,31	1,27	1,80	1,75	2,39	2,38	3,18	3,18	**0,94**	**1,290**	**1,775**	**2,385**	**3,18**

Bending Beam rheometer

Temp where S = 300 MPa (°C) **-24,1**

Temp where |m| = 0,300 (°C) **-27,4**

Temp (°C)	-30	-30	-20	-20	-30	-20
S(60 sek) (MPa)	701,2	733,5	163,2	160,3	**717,4**	**161,8**
|m| (60 sek)	0,249	0,260	0,426	0,428	**0,255**	**0,427**

10°C	Penetrasjon ($^1/_{10}$ mm)			
Binder	1	2	3	Average
Recovered	82,3	81,9	81,6	**82**

	Dynamic viscosity (cPoise)		
Binder	60°C	90°C	Skjærhast.
Recovered	196000	7260	

Measure nr.	Softning point,°C
1	45,2
2	45,5
Average	**45,4**

Sample 27, layer 2

B180

Spindel 8mm.

Dynamic shear rheometer, RECOVERED BINDER

Temp where G* x sind = 5000 kPa (°C)

9,6

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	5673,6	5607,6	1839,2	1832,1	628,1	645,4	211,8	216,5	77,5	78,5	5640,6	1835,650	636,780	214,145	78,0
δ (°)	56,5	56,3	63,0	63,0	67,9	67,6	71,8	71,5	74,4	74,0	56,4	63,000	67,750	71,650	74,2
G* x sinδ (kPa)	4733,3	4666,7	1638,6	1632,6	581,9	596,9	201,2	205,3	74,7	75,4	4700,0	1635,600	589,395	203,250	75,1
γ (%)	0,94	0,95	1,39	1,40	1,92	1,87	2,58	2,53	3,44	3,40	0,95	1,395	1,895	2,555	3,42

25°C	Penetrasjon ($^1/_{10}$ mm)			
Binder	1	2	3	Average
Revovered	92,3	92,8	92,6	93

	Dynamic viscosity (cPoise)		
Binder	60°C	90°C	Skjærhast.
Recovered	168000	6140	

Measure nr.	Softning point,°C
1	43,5
2	43,5
Average	43,5

Sample 22, layer 1

B180

Dynamic shear rheometer, RECOVERED BINDER

Temp where G* x sind = 5000 kPa (°C)

9,6

Temp (°C)	10	10	16	16	22	22	28	28	34	34	**10**	**16**	**22**	**28**	**34**
G* (kPa)	5999,5	5896,0	2200,0	2177,7	786,1	802,9	290,3	292,9	111,9	116,1	**5947,8**	**2188,850**	**794,475**	**291,565**	**114,0**
δ (°)	52,6	52,2	58,3	57,9	62,9	62,4	66,7	66,3	69,6	69,3	**52,4**	**58,100**	**62,650**	**66,500**	**69,5**
G* x sinδ (kPa)	4764,6	4656,0	1871,7	1844,1	699,5	711,3	266,7	268,1	104,8	108,6	**4710,3**	**1857,900**	**705,390**	**267,360**	**106,7**
γ (%)	0,92	0,93	1,23	1,24	1,73	1,69	2,27	2,25	2,97	3,09	**0,93**	**1,235**	**1,710**	**2,260**	**3,03**

Bending Beam rheometer

Temp where S = 300 MPa (°C) **-25,8**

Temp where |m| = 0,300 (°C) **-28,1**

Temp (°C)	-30	-30	-20	-20	**-30**	**-20**
S(60 sek) (MPa)	570,1	555,3	130,2	122,3	**562,7**	**126,3**
|m| (60 sek)	0,272	0,267	0,429	0,431	**0,270**	**0,430**

25°C	Penetrasjon ($^1/_{10}$ mm)			
Binder	1	2	3	Average
Revovered	75,9	76,5	75,6	**76**

	Dynamic viscosity (cPoise)		
Binder	60°C	90°C	Skjærhast.
Recovered	281000	6400	

Measure nr.	Softning point,°C
1	46,4
2	46,7
Average	**46,6**

Sample 22, layer 2

B180

Spindel 8mm.

Dynamic shear rheometer, RECOVERED BINDER

Temp where G* x sind = 5000 kPa (°C)

5,3

Temp (°C)	10	10	16	16	22	22	28	28	34	34	**10**	**16**	**22**	**28**	**34**
G* (kPa)	2415,0	2587,2	804,7	839,3	289,9	299,6	102,9	107,5	39,4	41,4	**2501,1**	**821,965**	**294,740**	**105,200**	**40,4**
δ (°)	60,9	60,4	65,8	65,6	69,4	68,9	72,0	71,8	73,5	73,2	**60,7**	**65,700**	**69,150**	**71,900**	**73,4**
G* x sinδ (kPa)	2109,6	2249,4	734,1	764,3	271,4	279,5	97,9	102,1	37,8	39,6	**2179,5**	**749,195**	**275,440**	**100,019**	**38,7**
γ (%)	1,24	1,20	1,75	1,73	2,27	2,30	3,19	3,14	4,15	4,03	**1,22**	**1,740**	**2,285**	**3,165**	**4,09**

25°C	**Penetrasjon ($^1/_{10}$ mm)**			
Binder	**1**	**2**	**3**	**Average**
Recovered	137	137	137	**137**

	Dynamic viscosity (cPoise)		
Binder	60°C	90°C	Skjærhast.
Recovered	119000	5140	

Measure nr.	**Softning point,°C**
1	40,7
2	41,0
Average	**40,9**

Sample 11, layer 1

MB 10000

25mm spindel

Dynamic shear rheometer, RECOVERED BINDER

Temp where G* x sind = 5000 kPa (°C) -2,0

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	906,4	911,3	309,1	308,4	105,8	105,1	35,4	35,0	12,4	12,5	908,9	308,765	105,455	35,188	12,5
δ (°)	54,8	54,5	67,3	66,9	73,8	73,4	78,1	78,1	81,3	84,7	54,7	67,100	73,600	78,100	83,0
G* x sinδ (kPa)	740,8	741,7	285,2	283,7	101,6	100,7	34,6	34,2	12,3	12,5	741,3	284,450	101,155	34,436	12,4
γ (%)	1,65	1,65	2,27	2,27	3,12	3,14	4,24	4,29	5,79	5,76	1,65	2,270	3,130	4,265	5,78

Bending Beam rheometer

Temp where S = 300 MPa (°C) -30,6

Temp where |m| = 0,300 (°C) -34,8

Temp (°C)	-35	-35	-30	-30	-35	-30
S(60 sek) (MPa)	558,8	558,8	279,7	270,4	558,8	275,1
|m| (60 sek)	0,296	0,296	0,391	0,396	0,296	0,394

10°C	Penetrasjon ($^1/_{10}$ mm)			
Binder	1	2	3	Average
Recovered	59,7	60	60,4	60

	Dynamic viscosity (cPoise)		
Binder	60°C	90°C	Skjærhast.
Recovered	38200	1940	

Measure nr.	Softning point,°C
1	32,6
2	32,0
Average	32,3

Sample 12, layer 1

MB 10000

8mm spindel

Dynamic shear rheometer, RECOVERED BINDER

Temp where G* x sind = 5000 kPa (°C) 9,2

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	5295,4	5019,1	1762,2	1621,1	587,9	549,3	198,8	181,2	71,9	67,1	5157,3	1691,650	568,565	189,995	69,5
δ (°)	57,6	58,0	64,0	64,2	68,6	68,7	72,3	72,3	74,8	74,8	57,8	64,100	68,650	72,300	74,8
G* x sinδ (kPa)	4468,9	4257,4	1587,9	1459,7	547,4	511,8	189,4	172,6	69,4	64,7	4363,2	1523,800	529,590	180,990	67,1
γ (%)	0,98	1,04	1,36	1,48	1,88	1,92	2,53	2,74	3,43	3,53	1,01	1,420	1,900	2,635	3,48

Bending Beam rheometer

Temp where S = 300 MPa (°C) -24,5

Temp where |m| = 0,300 (°C) -28,5

Temp (°C)	-35	-35	-25	-25	-35	-25
S(60 sek) (MPa)	1149,0	1199,0	320,9	322,0	1174,0	321,5
|m| (60 sek)	0,187	0,184	0,362	0,363	0,186	0,363

10°C	Penetrasjon ($^1/_{10}$ mm)			
Binder	1	2	3	Average
Recovered	24,2	24,5	24	24

	Dynamic viscosity (cPoise)		
Binder	60°C	90°C	Skjærhast.
Recovered	133000	5480	

Measure nr.	Softning point,°C
1	42,6
2	43,0
Average	42,8

Average	42,8

Sample 12, layer 2

MB 10000

8mm spindel

Dynamic shear rheometer, RECOVERED BINDER

Temp where G* x sind = 5000 kPa (°C)

5,0

Temp (°C)	10	10	16	16	22	22	28	28	34	34	**10**	**16**	**22**	**28**	**34**
G* (kPa)	2069,8	2712,3	718,4	874,9	265,6	301,8	92,1	102,7	35,0	37,9	**2391,1**	**796,665**	**283,710**	**97,388**	**36,5**
δ (°)	61,2	61,5	66,9	67,0	71,1	71,2	74,1	73,9	75,6	75,8	**61,4**	**66,950**	**71,150**	**74,000**	**75,7**
G* x sinδ (kPa)	1813,0	2384,1	660,9	805,6	251,2	285,8	88,6	98,7	33,9	36,8	**2098,6**	**733,205**	**268,495**	**93,610**	**35,4**
γ (%)	1,31	1,15	1,73	1,72	2,32	2,34	3,28	3,08	4,27	4,22	**1,23**	**1,725**	**2,330**	**3,180**	**4,25**

10°C	Penetrasjon ($^1/_{10}$ mm)			
Binder	1	2	3	Average
Recovered	32,8	33,4	33,8	33

	Dynamic viscosity (cPoise)		
Binder	60°C	90°C	Skjærhast.
Recovered	97600	4120	

Measure nr.	Softning point,°C
1	39,9
2	39,7
Average	39,8

Sample 18, layer 1

MB 10000

25mm spindel

Dynamic shear rheometer, RECOVERED BINDER

Temp where G* x sind = 5000 kPa (°C) 2,3

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	1382,1	1359,6	435,9	470,5	147,2	159,4	48,1	52,5	16,9	18,1	1370,9	453,225	153,255	50,281	17,5
δ (°)	66,6	63,3	68,7	68,6	73,1	72,9	76,2	75,8	76,4	76,7	65,0	68,650	73,000	76,000	76,6
G* x sinδ (kPa)	1268,4	1214,6	406,1	438,0	140,8	152,3	46,7	50,9	16,5	17,6	1241,5	422,045	146,550	48,786	17,0
γ (%)	0,87	1,48	2,08	1,93	2,76	2,55	4,28	3,92	5,19	4,91	1,18	2,005	2,655	4,100	5,05

Bending Beam rheometer

Temp where S = 300 MPa (°C) -31,9

Temp where |m| = 0,300 (°C) -33,8

Temp (°C)	-35	-30	-35	-30
S(60 sek) (MPa)	481,2	221,8	481,2	221,8
|m| (60 sek)	0,281	0,360	0,281	0,360

10°C	Penetrasjon ($^1/_{10}$ mm)			
Binder	1	2	3	Average
Recovered	53,3	52,5	52,3	53

	Dynamic viscosity (cPoise)		
Binder	60°C	90°C	Skjærhast.
Recovered	47200	2200	

Measure nr.	Softning point,°C
1	33,9
2	34,1
3	34,0
Average	34,0

2	34,1
3	34,0
Average	34,0

Sample 1, layer 1

MB 10000

8 mm spindel

Dynamic shear rheometer, RECOVERED BINDER

Temp where G* x sind = 5000 kPa (°C) **4,5**

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	2292,6	2425,9	796,2	558,3	271,8	291,6	91,0	99,6	31,2	34,3	2359,3	677,260	281,655	95,308	32,8
δ (°)	58,2	58,1	64,3	64,1	69,3	69,3	73,2	73,2	75,7	75,8	58,2	64,200	69,300	73,200	75,8
G* x sinδ (kPa)	1948,6	2060,5	717,4	772,0	254,2	272,7	87,1	85,3	30,3	33,3	2004,6	744,735	263,475	86,236	31,8
γ (%)	1,31	1,24	1,64	1,52	2,42	2,25	3,38	3,08	4,62	4,20	1,28	1,580	2,335	3,230	4,41

Bending Beam rheometer

Temp where S = 300 MPa (°C) **-30,0**

Temp where |m| = 0,300 (°C) **-31,5**

Temp (°C)	-35	-35	-30	-30	-35	-30
S(60 sek) (MPa)	598,0	532,4	294,7	302,0	565,2	298,4
|m| (60 sek)	0,259	0,242	0,321	0,322	0,251	0,322

10°C	Penetrasjon ($^1/_{10}$ mm)			
Binder	1	2	3	Average
Recovered	38,9	38	39	39

	Dynamic viscosity (cPoise)		
Binder	60°C	90°C	Skjærhast.
Recovered	62200	2920	

Measure nr.	Softning point,°C
1	37,5
2	37,6
Average	37,5

Sample 1, layer 2

MB 10000
25mm spindel
Dynamic shear rheometer, RECOVERED BINDER
Temp where G* x sind = 5000 kPa (°C)

											-4,0				
Temp (°C)	10	10	16	16	22	22	28	28	34	34	**10**	**16**	**22**	**28**	**34**
G* (kPa)	914,5	894,8	342,4	339,1	115,3	117,0	39,2	40,5	14,5	14,5	**904,6**	**340,705**	**116,165**	**39,846**	**14,5**
δ (°)	51,7	52,0	64,9	64,9	72,8	72,5	77,8	77,8	81,7	81,3	**51,9**	**64,900**	**72,650**	**77,800**	**81,5**
G* x sinδ (kPa)	717,9	705,4	310,1	306,9	110,2	111,6	38,3	39,6	14,3	14,3	**711,7**	**308,500**	**110,885**	**38,946**	**14,3**
γ (%)	1,75	1,79	2,19	2,21	3,04	2,99	4,08	3,95	5,54	5,53	**1,77**	**2,200**	**3,015**	**4,015**	**5,54**

10°C	**Penetrasjon ($^1/_{10}$ mm)**			
Binder	**1**	**2**	**3**	**Average**
Recovered	53,9	55	54,2	54

	Dynamic viscosity (cPoise)		
Binder	60°C	90°C	Skjærhast.
Recovered	39400	2160	

Measure nr.	**Softning point,°C**
1	34,8
2	34,3
Average	**34,5**

Sample 32, layer 1

MB 8000

25mm spindel

Dynamic shear rheometer, RECOVERED BINDER

Temp where G* x sind = 5000 kPa (°C)

-3,8

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	495,2	527,9	165,8	174,8	57,1	58,7	20,1	20,0	7,2	7,1	511,5	170,335	57,881	20,078	7,2
δ (°)	63,6	63,0	71,9	71,9	77,1	76,7	80,7	80,3	83,2	83,2	63,3	71,900	76,900	80,500	83,2
G* x sinδ (kPa)	443,3	470,6	157,7	166,2	55,7	57,1	19,9	19,7	7,2	7,1	456,9	161,900	56,372	19,801	7,1
γ (%)	1,82	1,70	2,71	2,63	3,85	3,75	4,97	5,24	6,93	7,02	1,76	2,670	3,800	5,105	6,98

Bending Beam rheometer

Temp where S = 300 MPa (°C) **-32,8**

Temp where |m| = 0,300 (°C) **-36,2**

Temp (°C)	-35	-35	-30	-30	-35	-30
S(60 sek) (MPa)	435,2	453,7	180,4	190,2	444,5	185,3
|m| (60 sek)	0,314	0,339	0,441	0,441	0,327	0,441

10°C	Penetrasjon ($^1/_{10}$ mm)			
Binder	1	2	3	Average
Recovered	83,3	84,5	85	84

	Dynamic viscosity (cPoise)		
Binder	60°C	90°C	Skjærhast.
Recovered	26800	1460	

Measure nr.	Softning point,°C
1	29,8
2	29,8
Average	29,8

Sample 13, layer 1

MB 8000
25mm spindel
Dynamic shear rheometer, RECOVERED BINDER
Temp where G* x sind = 5000 kPa (°C) **-3,0**

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	667,7	736,7	223,6	254,7	75,8	86,3	26,9	30,1	10,5	11,2	702,2	239,140	81,028	28,500	10,8
δ (°)	57,8	57,0	68,2	67,8	74,2	73,8	78,1	77,8	81,2	81,2	57,4	68,000	74,000	77,950	81,2
G* x sinδ (kPa)	565,1	618,1	207,7	235,7	72,9	82,9	26,3	29,4	10,4	11,0	591,6	221,685	77,880	27,873	10,7
γ (%)	1,80	1,63	2,46	2,36	3,30	3,13	4,46	4,32	6,17	6,28	1,72	2,410	3,215	4,390	6,23

Bending Beam rheometer
Temp where S = 300 MPa (°C) **-32,5**
Temp where |m| = 0,300 (°C) **-36,5**

Temp (°C)	-35	-30	-35	-30
S(60 sek) (MPa)	427,6	209,8	427,6	209,8
|m| (60 sek)	0,327	0,420	0,327	0,420

10°C	Penetrasjon ($^1/_{10}$ mm)			
Binder	1	2	3	Average
Recovered	67,8	68,4	68	68

	Dynamic viscosity (cPoise)		
Binder	60°C	90°C	Skjærhast.
Recovered	38000	1840	

Measure nr.	Softning point,°C
1	31,6
2	32,0
Average	31,8

Measure nr.	Softning point,°C
1	31,6
2	32,0
Average	31,8

Sample 13, layer 2

MB 8000

25mm spindel

Dynamic shear rheometer, RECOVERED BINDER

Temp where G* x sind = 5000 kPa (°C)

-2,6

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	457,7	476,6	151,2	119,2	49,9	51,9	17,7	19,1	6,7	6,8	467,1	135,235	50,915	18,418	6,7
δ (°)	64,0	64,2	72,5	74,6	77,6	77,1	80,8	84,8	83,3	83,5	64,1	73,550	77,350	82,800	83,4
G* x sinδ (kPa)	411,5	429,0	144,2	115,0	48,7	50,6	17,5	19,0	6,6	6,7	420,2	129,595	49,685	18,264	6,7
γ (%)	1,97	1,89	2,65	3,35	3,51	3,56	5,08	4,45	7,52	6,65	1,93	3,000	3,535	4,765	7,09

10°C	Penetrasjon ($^1/_{10}$ mm)			
Binder	1	2	3	Average
Recovered	92,6	91,2	91	92

	Dynamic viscosity (cPoise)		
Binder	60°C	90°C	Skjærhast.
Recovered	24600	1460	

Measure nr.	Softning point,°C
1	28,9
2	28,9
3	29,4
Average	29,1

Sample 17, layer 1

MB 8000

25mm spindel

Dynamic shear rheometer, RECOVERED BINDER

Temp where G* x sind = 5000 kPa (°C) -4,1

Temp (°C)	10	10	16	16	22	22	28	28	34	34	**10**	**16**	**22**	**28**	**34**
G* (kPa)	437,4	443,5	141,5	143,3	48,2	48,5	17,0	17,0	6,3	6,2	**440,5**	**142,415**	**48,374**	**16,981**	**6,2**
δ (°)	65,2	65,2	73,3	72,9	77,9	77,7	81,1	81,1	83,8	83,5	**65,2**	**73,100**	**77,800**	**81,100**	**83,7**
G* x sinδ (kPa)	397,0	402,5	135,5	136,4	47,1	47,4	16,8	16,8	6,2	6,1	**399,7**	**135,960**	**47,277**	**16,778**	**6,2**
γ (%)	2,06	2,03	2,83	2,79	3,94	3,92	5,30	5,31	7,20	6,49	**2,05**	**2,810**	**3,930**	**5,305**	**6,85**

Bending Beam rheometer

Temp where S = 300 MPa (°C) -33,9

Temp where |m| = 0,300 (°C) -39,1

Temp (°C)	-35	-30	**-35**	**-30**
S(60 sek) (MPa)	365,4	153,7	**365,4**	**153,7**
|m| (60 sek)	0,375	0,467	**0,375**	**0,467**

10°C	**Penetrasjon (¹/₁₀ mm)**			
Binder	**1**	**2**	**3**	**Average**
Recovered	97,2	97,9	96,9	**97**

	Dynamic viscosity (cPoise)		
Binder	60°C	90°C	Skjærhast.
Recovered	25400	1450	

Measure nr.	**Softning point,°C**
1	28,4
2	28,5
3	28,2
Average	**28,4**

Average	28,4

Sample 17, layer 2

MB 8000

25mm spindel

Dynamic shear rheometer, RECOVERED BINDER

Temp where G* x sind = 5000 kPa (°C)

-6,2

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	289,2	289,0	96,5	89,9	30,9	29,4	10,6	10,5	4,1	4,3	289,1	93,193	30,127	10,588	4,2
δ (°)	69,6	69,9	76,1	75,9	79,9	79,7	82,6	82,7	84,6	84,9	69,8	76,000	79,800	82,650	84,8
G* x sinδ (kPa)	271,1	263,0	93,6	87,2	30,4	28,9	10,6	10,5	4,1	4,2	267,1	90,417	29,651	10,501	4,2
γ (%)	2,42	2,50	3,11	3,34	4,54	4,77	6,11	6,17	8,07	7,75	2,46	3,225	4,655	6,140	7,91

10°C	Penetrasjon ($^1/_{10}$ mm)			
Binder	1	2	3	Average
Recovered	125	125	126	125

	Dynamic viscosity (cPoise)		
Binder	60°C	90°C	Skjærhast.
Recovered	17000	1110	

Measure nr.	Softning point,°C
1	26,1
2	26,1
3	26,7
Average	26,3

Sample 30, layer 1

MB 6000
25mm spindel
Dynamic shear rheometer, RECOVERED BINDER
Temp where G* x sind = 5000 kPa (°C) -10,5

Temp (°C)	10	10	16	16	22	22	28	28	34	34	**10**	**16**	**22**	**28**	**34**
G* (kPa)	447,0	487,8	175,1	194,3	63,7	70,8	23,2	25,5	9,2	9,5	**467,4**	**184,700**	**67,222**	**24,360**	**9,4**
δ (°)	48,8	48,6	61,4	60,6	68,8	68,2	74,1	73,9	78,0	77,8	**48,7**	**61,000**	**68,500**	**74,000**	**77,9**
G* x sinδ (kPa)	336,5	365,8	153,7	169,3	59,3	65,7	22,3	24,5	9,0	9,3	**351,2**	**161,515**	**62,548**	**23,414**	**9,1**
γ (%)	2,24	2,05	2,57	2,32	3,61	3,25	4,75	4,70	6,22	6,30	**2,15**	**2,445**	**3,430**	**4,725**	**6,26**

Bending Beam rheometer
Temp where S = 300 MPa (°C) **-34,0**
Temp where |m| = 0,300 (°C) **-37,5**

Temp (°C)	-35	-30	**-35**	**-30**
S(60 sek) (MPa)	343,7	171,0	**343,7**	**171,0**
|m| (60 sek)	0,330	0,389	**0,330**	**0,389**

10°C	**Penetrasjon (¹/₁₀ mm)**			
Binder	**1**	**2**	**3**	**Average**
Recovered	50,6	50,7	50,5	**51**

	Dynamic viscosity (cPoise)		
Binder	60°C	90°C	Skjærhast.
Recovered	56200	2560	

Measure nr.	**Softning point,°C**
1	35,8
2	35,9
3	36,2
Average	**36,0**

1	35,8
2	35,9
3	36,2
Average	36,0

Sample 30, layer 2

MB 6000

25mm spindel

***Dynamic shear rheometer*, RECOVERED BINDER**

Temp where G* x sind = 5000 kPa (°C)

-5,1

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	776,1	733,1	290,5	284,1	104,7	100,4	37,5	36,9	13,8	13,9	754,6	287,260	102,535	37,223	13,8
δ (°)	52,2	52,1	63,1	62,9	69,8	69,6	74,8	74,8	78,8	78,6	52,2	63,000	69,700	74,800	78,7
G* x sinδ (kPa)	613,5	578,6	259,0	252,8	98,2	94,1	36,2	35,7	13,5	13,6	596,0	255,890	96,168	35,916	13,6
γ (%)	1,91	1,98	2,41	2,36	3,25	3,19	4,00	4,06	5,44	5,41	1,95	2,385	3,220	4,030	5,43

10°C	Penetrasjon ($^1/_{10}$ mm)			
Binder	**1**	**2**	**3**	**Average**
Recovered	59	59,3	58,7	59

	Dynamic viscosity (cPoise)		
Binder	60°C	90°C	Skjærhast.
Recovered	52200	2300	

Measure nr.	Softning point,°C
1	34,3
2	33,9
3	29,7
Average	32,6

Sample 31, layer 1

MB 6000

25mm spindel

Dynamic shear rheometer, RECOVERED BINDER

Temp where G* x sind = 5000 kPa (^oC)

-5,4

Temp (^oC)	10	10	16	16	22	22	28	28	34	34	**10**	**16**	**22**	**28**	**34**
G* (kPa)	887,7	869,9	349,4	346,8	127,8	128,8	47,6	47,3	16,6	17,0	**878,8**	**348,110**	**128,330**	**47,455**	**16,8**
δ (°)	49,7	49,5	61,8	61,4	69,1	68,8	74,4	74,3	78,5	78,6	**49,6**	**61,600**	**68,950**	**74,350**	**78,6**
G* x sinδ (kPa)	676,7	661,8	308,0	304,4	119,4	120,1	45,8	45,6	16,2	16,7	**669,3**	**306,210**	**119,760**	**45,703**	**16,5**
γ (%)	1,58	1,61	2,09	2,11	2,97	2,95	3,78	4,02	5,25	5,11	**1,60**	**2,100**	**2,960**	**3,900**	**5,18**

Bending Beam rheometer

Temp where S = 300 MPa (^oC) **-34,7**

Temp where |m| = 0,300 (^oC) **-37,1**

Temp (^oC)	-35	-35	-30	-30	**-35**	**-30**
S(60 sek) (MPa)	312,9	312,9	158,7	163,9	**312,9**	**161,3**
|m| (60 sek)	0,327	0,327	0,392	0,391	**0,327**	**0,392**

10°C	**Penetrasjon ($^1/_{10}$ mm)**			
Binder	**1**	**2**	**3**	**Average**
Recovered	51,6	51,8	52,1	**52**

	Dynamic viscosity (cPoise)		
Binder	60°C	90°C	Skjærhast.
Recovered	60400	2420	

Measure nr.	**Softning point,°C**
1	35,2
2	35,5
Average	**35,4**

Average	35,4

Sample 19, layer 1

MB 6000

25mm spindel

Dynamic shear rheometer, RECOVERED BINDER

Temp where G* x sind = 5000 kPa (°C) -5,4

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	431,1	387,2	147,2	132,4	50,6	45,7	16,9	16,4	6,5	6,4	409,1	139,810	48,151	16,633	6,5
δ (°)	63,5	63,6	71,2	71,2	76,1	76,0	79,8	79,6	82,7	82,6	63,6	71,200	76,050	79,700	82,7
G* x sinδ (kPa)	385,9	346,7	139,3	125,4	49,1	44,4	16,6	16,1	6,5	6,4	366,3	132,330	46,726	16,365	6,4
γ (%)	2,09	2,20	2,72	3,02	3,96	3,94	5,33	5,19	7,03	7,02	2,15	2,870	3,950	5,260	7,03

Bending Beam rheometer

Temp where S = 300 MPa (°C) -35,3

Temp where |m| = 0,300 (°C) -40,3

Temp (°C)	-35	-30	-35	-30
S(60 sek) (MPa)	282,8	110,6	282,8	110,6
|m| (60 sek)	0,395	0,484	0,395	0,484

10°C	Penetrasjon ($^1/_{10}$ mm)			
Binder	1	2	3	Average
Recovered	93	92,9	94,2	93

	Dynamic viscosity (cPoise)		
Binder	60°C	90°C	Skjærhast.
Recovered	23000	1490	

Measure nr.	Softning point,°C
1	28,9
2	29,5
Average	29,2

Sample 19, layer 2

MB 6000

25mm spindel

Dynamic shear rheometer, RECOVERED BINDER

Temp where G* x sind = 5000 kPa (°C) -4,1

Temp (°C)	10	10	16	16	22	22	28	28	34	34	**10**	**16**	**22**	**28**	**34**
G* (kPa)	452,6	442,6	148,9	141,7	49,7	48,9	17,3	16,9	6,4	6,5	**447,6**	**145,305**	**49,286**	**17,081**	**6,4**
δ (°)	64,8	64,7	72,9	72,9	77,6	77,4	81,0	81,0	86,1	83,6	**64,8**	**72,900**	**77,500**	**81,000**	**84,9**
G* x sinδ (kPa)	409,6	400,1	142,4	135,4	48,5	47,7	17,0	16,7	6,4	6,5	**404,8**	**138,880**	**48,124**	**16,872**	**6,4**
γ (%)	1,99	2,03	2,69	2,82	3,82	3,89	5,22	5,32	7,07	6,93	**2,01**	**2,755**	**3,855**	**5,270**	**7,00**

10°C	**Penetrasjon (¹/₁₀ mm)**			
Binder	**1**	**2**	**3**	**Average**
Recovered	91,2	91,2	91,4	**91**

	Dynamic viscosity (cPoise)		
Binder	60°C	90°C	Skjærhast.
Recovered	25200	1470	

Measure nr.	**Softning point,°C**
1	29,3
2	29,1
3	29,7
Average	**29,4**

Sample 3, layer 1

BE70S (MB1500)

25mm spindel

Dynamic shear rheometer, RECOVERED BINDER

Temp where G* x sind = 5000 kPa (°C)

Temp (°C)	10	10	16	16	22	22	28	28	34	34	-20,0				
G* (kPa)	23,6	23,7	8,0	8,1	2,8	2,9	1,1	1,1	0,5	0,5	10	16	22	28	34
δ (°)	81,5	81,5	84,2	83,9	85,9	85,9	87,4	87,4	89,3	87,8	23,7	8,054	2,825	1,088	0,5
G* x sinδ (kPa)	23,4	23,4	7,9	8,1	2,7	2,9	1,1	1,1	0,5	0,5	81,5	84,050	85,900	87,400	88,6
γ (%)	4,23	4,64	6,90	6,51	9,06	8,68	12,08	11,83	15,38	15,08	23,4	7,995	2,807	1,087	0,5
											4,44	6,705	8,870	11,955	15,23

Bending Beam rheometer

Temp where S = 300 MPa (°C) -43,2

Temp where |m| = 0,300 (°C) -38,9

Temp (°C)	-35	-30	-35	-30
S(60 sek) (MPa)	41,0	12,2	41,0	12,2
|m| (60 sek)	0,613	1,010	0,613	1,010

Binder	Dynamic viscosity (cPoise)		
	60°C	90°C	Skjærhast.
Recovered	2940	324	

Measure nr.	Softning point,°C
1	10,4
2	10,6
Average	10,5

Sample 7, layer 1

VO 550

25mm spindel

Dynamic shear rheometer, RECOVERED BINDER

Temp where G* x sind = 5000 kPa (°C) -15,2

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	74,7	79,4	26,7	28,5	9,9	10,5	3,8	3,9	1,6	1,6	77,1	27,608	10,178	3,863	1,6
δ (°)	73,0	72,5	77,2	76,9	80,8	80,5	83,5	83,1	85,5	85,1	72,8	77,050	80,650	83,300	85,3
G* x sinδ (kPa)	71,5	75,7	26,1	27,7	9,7	10,3	3,8	4,0	1,6	1,6	73,6	26,900	10,043	3,881	1,6
γ (%)	3,61	3,40	4,86	4,57	6,08	6,20	7,90	8,42	11,05	10,86	3,51	4,715	6,140	8,160	10,96

Bending Beam rheometer

Temp where S = 300 MPa (°C) -45,9

Temp where |m| = 0,300 (°C) -42,8

Temp (°C)	-35	-30	-35	-30
S(60 sek) (MPa)	43,9	18,1	43,9	18,1
|m| (60 sek)	0,551	0,712	0,551	0,712

	Dynamic viscosity (cPoise)		
Binder	60°C	90°C	Skjærhast.
Recovered	8340	695	

Measure nr.	Softning point,°C
1	21,1
2	21,6
Average	21,4

Sample 8, layer 1

VO 550

25mm spindel

Dynamic shear rheometer, RECOVERED BINDER

Temp where G* x sind = 5000 kPa (°C) -5,7

Temp (°C)	10	10	16	16	22	22	28	28	34	34	**10**	**16**	**22**	**28**	**34**
G* (kPa)	935,1	960,2	387,8	392,4	148,9	148,6	55,8	56,6	20,6	21,5	**947,6**	**390,080**	**148,720**	**56,194**	**21,1**
δ (°)	49,2	48,9	60,8	60,5	77,1	76,4	73,0	72,9	77,1	81,4	**49,1**	**60,650**	**76,750**	**72,950**	**79,3**
G* x sinδ (kPa)	707,5	723,6	338,5	341,5	145,1	144,4	53,3	54,1	20,1	21,3	**715,5**	**339,980**	**144,755**	**53,728**	**20,7**
γ (%)	1,73	1,67	2,06	2,04	2,69	2,76	3,59	3,71	4,86	5,11	**1,70**	**2,050**	**2,725**	**3,650**	**4,99**

Bending Beam rheometer

Temp where S = 300 MPa (°C) -34,7

Temp where |m| = 0,300 (°C) -37,4

Temp (°C)	-35	-30	**-35**	**-30**
S(60 sek) (MPa)	317,3	131,7	**317,3**	**131,7**
|m| (60 sek)	0,332	0,400	**0,332**	**0,400**

	Dynamic viscosity (cPoise)		
Binder	60°C	90°C	Skjærhast.
Recovered	64000	3220	

Measure nr.	Softning point,°C
1	37,8
2	38,8
Average	**38,3**

Sample 9, layer 1

VO 550

25mm spindel

Dynamic shear rheometer, RECOVERED BINDER

Temp where G* x sind = 5000 kPa (°C) -17,0

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	43,0	47,4	15,3	15,8	5,7	6,1	2,2	2,2	0,9	0,9	45,2	15,526	5,917	2,205	0,9
δ (°)	76,2	76,0	80,0	79,7	82,7	82,7	84,9	85,0	86,7	86,7	76,1	79,850	82,700	84,950	86,7
G* x sinδ (kPa)	41,8	45,9	15,0	15,5	5,7	6,1	2,2	2,2	0,9	0,9	43,9	15,283	5,869	2,197	0,9
γ (%)	4,42	4,01	5,57	5,39	6,99	6,55	9,28	8,91	10,93	12,76	4,22	5,480	6,770	9,095	11,85

Bending Beam rheometer

Temp where S = 300 MPa (°C) -43,1

Temp where |m| = 0,300 (°C) -38,3

Temp (°C)	-35	-30	-35	-30
S(60 sek) (MPa)	40,3	11,7	40,3	11,7
|m| (60 sek)	0,573	0,986	0,573	0,986

	Dynamic viscosity (cPoise)		
Binder	60°C	90°C	Skjærhast.
Recovered	5200	500	

Measure nr.	Softning point,°C
1	16,9
2	17,1
Average	17,0

APPENDIX 6

Detailed results from analysis on original and laboratory aged bitumen
(chapter 3)

DSR and BBR

Sample no	Bitumen	Treatment	G* 10°C, (kPa)	G* 16°C, (kPa)	G* 22°C, (kPa)	G* 28°C, (kPa)	G* 34°C, (kPa)	Delta 10°C	Delta 16°C	Delta 22°C	Delta 28°C	Delta 34°C	Temp where G*/sin delta = 1.00 kPa	Temp. where G*/sin delta=2,20 kPa	Temp. where G*xsin delta=5000 kPa	Temp where S=300 MPa	Temp where Iml=0,300 MPa
1	B180	Neat	3730,5	1038,6	321,7	100,3	35,3	65,1	71,2	75,3	77,8	78,7	54,5				
2	B180	TFOT	5604,4	1733,9	569,5	188,3	68,1	58,7	65,2	69,6	72,8	75,1		54,2			
3	B180	PAV	11296,5	4156,6	1533,8	542,3	201,8	48,4	55,1	60,4	65,0	68,4			13,5	-20,0	-23,2
4	MB 10000	Neat	206,0	61,4	20,8	7,3	2,8	75,2	80,2	83,0	84,8	86,4	40,5				
5	MB 10000	TFOT	266,6	78,9	26,3	9,0	3,4	72,9	78,5	81,6	83,9	85,5		36,7			
6	MB 10000	PAV	673,6	211,7	71,7	25,4	9,7	68,7	72,3	74,9	76,0	73,3			-0,3	-31,5	-35,7
7	MB 8000	Neat	147,8	44,7	14,6	5,2	2,0	77,4	81,2	83,7	85,5	86,8	38,5				
8	MB 8000	TFOT	190,4	57,4	19,4	6,8	2,6	75,0	79,5	82,4	84,6	85,9		35,1			
9	MB 8000	PAV	436,2	143,3	48,5	17,2	6,6	64,2	71,9	76,5	80,0	82,6			-4,4	-33,4	-36,8
10	MB 6000	Neat	94,5	28,0	9,3	3,4	1,3	79,8	82,9	84,9	86,4	87,2	35,9				
11	MB 6000	TFOT	109,5	34,5	11,6	4,3	1,7	77,3	81,2	83,4	85,3	86,6		32,4			
12	MB 6000	PAV	266,2	84,7	28,9	10,5	3,9	69,1	74,7	78,6	81,7	83,8			-6,0	-34,3	-38,9

Concventional tests

Sample no	Bitumen	Treatment	Pen 25°C	Pen 10°C	Viscosity 60°C CPS	Viscosity 90°C CPS	Softening point, °C	Loss of mass, %	Ageing Index, Viscosity 60°C, n_a/n_o
1	B180	Neat	158		85 400	3 520	38,2		
2	B180	TFOT	100		166 000	5 760	43,3	-0,2153	1,94
3	B180	PAV	51		504 000	13 300	51,6		5,90
4	MB 10000	Neat		154	12 200	885	23,8		
5	MB 10000	TFOT		134	13 600	995	25,1	0,0003	1,11
6	MB 10000	PAV		74	49 600	1 480	30,8		4,07
7	MB 8000	Neat		193	8 640	680	20,9		
8	MB 8000	TFOT		158	11 400	820	23,5	-0,0037	1,32
9	MB 8000	PAV		87	22 600	1 390	29,4		2,62
10	MB 6000	Neat		259	6 740	588	18,8		
11	MB 6000	TFOT		219	8 200	620	20,3	0,0144	1,22
12	MB 6000	PAV		118	15 800	1 030	26,5		2,34

B 180

Spindel 8mm

Dynamic shear rheometer, ORIGINAL

Temp where G*/sinδ = 1,00 kPa (°C)

54,5

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	3796,100	3664,800	1041,600	1035,500	324,320	319,130	101,010	99,619	35,630	34,952	3730,450	1038,550	321,725	100,315	35,291
δ (°)	65,4	64,7	71,4	71,0	75,5	75,1	78,1	77,5	78,9	78,4	65,1	71,2	75,300	77,800	78,7
G*/sinδ (k	4176,000	4053,500	1098,800	1094,900	334,960	330,280	103,220	102,040	36,304	35,681	4114,750	1096,9	332,620	102,630	35,993
γ (%)	1,08	1,09	1,64	1,65	2,24	2,27	3,11	3,16	4,47	4,37	1,09	1,6	2,255	3,135	4,42

Dynamic shear rheometer, TFOT (8mm)

Temp where G*/sinδ = 2,20 kPa (°C)

54,2

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	5667,100	5541,700	1766,700	1701,000	585,750	553,150	191,490	185,140	69,747	66,472	5604,400	1733,850	569,450	188,315	68,110
δ (°)	58,4	59,0	65,0	65,4	69,3	69,8	72,7	72,8	75,0	75,2	58,7	65,200	69,550	72,750	75,1
G*/sinδ (k	6650,100	6463,000	1949,900	1871,500	626,050	589,470	200,570	193,790	72,207	68,744	6556,550	1910,700	607,760	197,180	70,476
γ (%)	0,97	0,99	1,42	1,44	1,89	1,91	2,65	2,63	3,46	3,56	0,98	1,430	1,900	2,640	3,51

Dynamic shear rheometer, PAV (8mm)

Temp where G* x sind = 5000 kPa (°C)

13,5

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	11258,0	11335,0	4167,3	4145,9	1525,4	1542,2	544,6	540,1	202,9	200,8	11295,5	4156,600	1533,800	542,345	201,8
δ (°)	48,3	48,5	55,1	55,1	60,4	60.7	64,7	65,2	68,4	68,3	48,4	55,100	60,400	64,950	68,4
G* x sinδ	8405,2	8486,5	3417,1	3402,0	1326,9	1344,7	492,5	490,2	188,6	186,6	8445,9	3409,550	1335,800	491,365	187,6
γ (%)	0.80	0,79	1,08	1,09	1,45	1,43	1,94	1,96	2,56	2,58	0,79	1,086	1,440	1,960	2,67

Bending Beam rheometer

Temp where S = 300 MPa (°C) -20,0

Temp where |m| = 0,300 (°C) -23,2

Temp (°C)	-30	-30	-20	-20	-30	-20
S(60 sek)	901,2	992,1	299,9	299,9	946,7	299,9
|m| (60 se	0.191	0.212	0,347	0,347	0,202	0,347

25°C	Penetrasjon (1/10 mm)			
Binder	1	2	3	Average
Original	158	159	157	158
TFOT	101	101	99	100
PAV	51,6	51,8	50,8	61

Binder	Dynamic viscosity (cPoise)		
	60°C	90°C	Skjærhast.
Original	85400	3520	
TFOT	166000	5760	
PAV	504000	13300	

Measure nr.	Softning point.°C		
	Original	TFOT	PAV
1	38,3	43,1	51,7
2	38,0	43,4	51,4
Average	38,2	43,3	51,6

Measure nr.	Loss of mass after TFOT, %			Loss, %
	Bowl	Before	After	
1	141,5753	176,472	176,3989	-0,2095
2	147,1838	182,391	182,3132	-0,221
Average				-0,21525

MB 10000

Dynamic shear rheometer, ORIGINAL
Temp where G*/sinδ = 1,00 kPa (°C) 40,5

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	206,050	205,930	63,220	59,560	21,240	20,259	7,391	7,298	2,827	2,804	205,990	61,390	20,750	7,345	2,816
δ (°)	75,2	75,1	80,4	80,0	83,0	82,9	84,8	84,8	86,4	86,3	75,2	80,2	82,950	84,800	86,4
G*/sinδ (k	213,120	213,050	64,126	60,470	21,406	20,416	7,422	7,328	2,833	2,810	213,085	62,3	20,911	7,375	2,821
γ (%)	2,57	2,57	3,56	3,70	4,94	4,94	6,77	6,78	8,86	8,93	2,57	3,6	4,940	6,775	8,90

Dynamic shear rheometer, TFOT
Temp where G*/sinδ = 2,20 kPa (°C) 36,7

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	265,390	267,820	77,737	80,139	26,166	26,480	8,846	9,200	3,326	3,471	266,605	78,938	26,323	9,023	3,398
δ (°)	72,9	72,9	78,5	78,4	81,6	81,6	83,8	84,0	85,6	85,4	72,9	78,450	81,600	83,900	85,5
G*/sinδ (k	277,720	280,190	79,333	81,818	26,448	26,769	8,897	9,251	3,336	3,486	278,956	80,576	26,609	9,074	3,411
γ (%)	2,34	2,32	3,47	3,37	4,59	4,65	6,45	6,31	8,58	8,36	2,33	3,420	4,620	6,380	8,47

Dynamic shear rheometer, PAV
Temp where G* x sind = 5000 kPa (°C) -0,3

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	691,4	655,8	216,9	206,5	74,2	69,2	26,0	24,7	10,0	9,5	673,6	211,655	71,685	25,386	9,7
δ (°)	68,7	68,6	72,4	72,2	75,0	74,8	76,0	75,9	73,4	73,2	68,7	72,300	74,900	75,950	73,3
G* x sinδ	644,1	610,4	206,8	169,6	71,7	66,7	25,3	24,0	9,6	9,1	627,3	188,175	69,216	24,647	9,3
γ (%)	1,74	1,84	2,34	2,45	3,45	3,71	4,70	4,95	6,01	6,20	1,79	2,396	3,680	4,826	6,11

Bending Beam rheometer
Temp where S = 300 MPa (°C) -31,5
Temp where |m| = 0,300 (°C) -35,7

Temp (°C)	-36	-36	-30	-30	-36	-30
S(60 sek)	639,6	656,5	234,0	229,9	648,1	232,0
|m| (60 se	0,293	0,294	0,418	0,416	0,294	0,417

10°C	Penetrasjon ($^1/_{10}$ mm)			
Binder	1	2	3	Average
Original	154	153	156	154
TFOT	133	135	135	134
PAV	73,4	73,6	73,5	74

	Dynamic viscosity (cPoise)		
Binder	60°C	90°C	Skjærhast.
Original	12200	885	
TFOT	13600	995	
PAV	49600	1480	

Measure nr.	Softning point.°C		
	Original	TFOT	PAV
1	24,2	25,0	30,7
2	23,3	25,2	30,9
Average	23,8	25,1	30,8

Measure nr.	Loss of mass after TFOT, %			
	Bowl	Before	After	Loss, %
1	89,2443	138,9908	138,9913	0,001
2	85,1871	135,4613	135,4611	-0,0004
3	88,9125	139,1682	139,1684	0,0004
Average				0,000333

MB 8000

Dynamic shear rheometer, ORIGINAL
Temp where G*/sinδ = 1.00 kPa (°C)
38,5

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	146,100	149,490	44,617	44,868	14,575	14,562	5,139	5,356	2,014	2,073	147,795	44,743	14,569	5,248	2,043
δ (°)	77,4	77,4	81,0	81,4	83,9	83,5	85.6	85.3	86,7	86,8	77,4	81,2	83,700	85,450	86,8
G*/sinδ (k	149.740	153,180	45,169	45,380	14,657	14,655	5,155	5,374	2,017	2,076	151,460	45,3	14,656	5,264	2,047
γ (%)	2,81	2,74	4,03	4,01	5,49	5,50	7,40	7.28	9,69	9,66	2,78	4,0	5,496	7,340	9,68

Dynamic shear rheometer, TFOT
Temp where G*/sinδ = 2.20 kPa (°C)
35,1

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	192,310	188,550	55,654	59,209	19,246	19,531	6,853	6,735	2.530	2,672	190,430	57,432	19,389	6,794	2,601
δ (°)	74,6	75,4	79,7	79,3	82,1	82,6	84.6	84,5	85,7	86,1	75,0	79,500	82,350	84,650	85,9
G*/sinδ (k	199,440	194,800	56,564	60,265	19,430	19,697	6,884	6,766	2,537	2,678	197,120	58,415	19,564	6,825	2,608
γ (%)	2.60	2,65	3,77	3,72	5,20	4,99	6,72	6.98	9,10	8,99	2,63	3,745	5,096	6,860	9,05

Dynamic shear rheometer, PAV
Temp where G* x sind = 5000 kPa (°C)
-4,4

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	437,6	434,9	145,4	141,3	49,8	47,1	17,3	17,0	6,6	6,6	436,2	143,330	48,474	17,163	6,6
δ (°)	64,2	64,1	71,9	71,8	76,5	76,5	80.0	79,9	82,5	82,7	64,2	71,850	76,500	79,950	82,6
G* x sinδ (	394,0	391,1	138,1	134,2	48,4	45,8	17,1	16,7	6.5	6,5	392,6	136,175	47,133	16,900	6,5
γ (%)	2,06	2,07	2,82	2,90	3,82	3,93	5,20	5,29	6,98	6,99	2.07	2,860	3,875	5,246	6,99

Bending Beam rheometer
Temp where S = 300 MPa (°C) -33,4
Temp where |m| = 0,300 (°C) -36,8

Temp (°C)	-36	-36	-20	-20	-36	-20
S(60 sek)	524,0	512,9	17,1	17,2	518,5	17,2
$\lvert m \rvert$ (60 se	0.325	0.326	0.854	0.831	0.326	0,843

10°C	Penetrasjon ($^1/_{10}$ mm)			
Binder	1	2	3	Average
Original	192	193	193	193
TFOT	157	159	159	158
PAV	86.1	87.1	86.7	87

Binder	Dynamic viscosity (cPoise)		
	60°C	90°C	Skjærhast.
Original	8640	680	
TFOT	11400	820	
PAV	22600	1390	

Measure nr.	Softning point °C		
	Original	TFOT	PAV
1	20,6	23.6	29,4
2	21,2	23,4	29.4
Average	20,9	23,5	29,4

Measure nr.	Loss of mass after TFOT, %			Loss, %
	Bowl	Before	After	
1	84,9917	135,2441	135,2429	-0,0024
2	84,4491	134,6528	134,6503	-0,005
Average				-0,0037

MB 6000

Dynamic shear rheometer, ORIGINAL
Temp where G*/sinδ = 1.00 kPa (°C) 35,9

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	94,051	94,951	27,716	28,207	8,887	9,676	3,266	3,507	1,324	1,350	94,501	27,962	9,282	3,386	1,337
δ (°)	79,8	79,8	83,0	82,7	85,1	84,7	86,4	86,3	87,2	87,1	79,8	82,9	84,900	86,350	87,2
G*/sinδ (k	92,553	92,553	27,512	27,979	8,854	9,635	3,254	3,500	1,323	1,348	92,553	27,7	9,245	3,377	1,335
γ (%)	3,21	3,21	4,58	4,56	6,37	6,21	8,51	8,39	11,06	11,00	3,21	4,6	6,290	8,450	11,03

Dynamic shear rheometer, TFOT
Temp where G*/sinδ = 2.20 kPa (°C) 32,4

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	110,060	108,970	34,846	34,242	11,469	11,647	4,377	4,313	1,689	1,726	109,515	34,544	11,558	4,345	1,708
δ (°)	77,4	77,2	81,2	81,1	83,5	83,3	85,3	85,3	86,5	86,7	77,3	81,150	83,400	85,300	86,6
G*/sinδ (k	112,800	111,760	35,261	34,658	11,544	11,727	4,392	4,328	1,692	1,729	112,280	34,960	11,636	4,360	1,711
γ (%)	3,09	3,12	4,31	4,38	5,93	5,93	7,66	7,78	10,38	10,16	3,11	4,345	5,930	7,720	10,27

Dynamic shear rheometer, PAV
Temp where G* x sind = 5000 kPa (°C) -6,0

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	272,,39	266,2	85,3	84,2	29,1	28,8	10,2	10,8	3,9	4,0	266,2	84,740	28,902	10,526	3,9
δ (°)	69,5	68,7	74,7	74,6	78,6	78,6	81,8	81,5	83,8	83,8	69,1	74,650	78,600	81,650	83,8
G* x sinδ (	255,1	248,1	82,3	81,2	28,5	28,2	10,1	10,7	3,9	3,9	251,6	81,720	28,333	10,414	3,9
γ (%)	2,31	2,37	3,28	3,33	4,48	4,52	6,17	6,09	8,07	8,07	2,34	3,305	4,500	6,130	8,07

Bending Beam rheometer
Temp where S = 300 MPa (°C) -34,3
Temp where |m| = 0,300 (°C) -38,9

Temp (°C)	-36	-36	-30	-30	-36	-30
S(60 sek)	436,8	408,8	122,8	122,8	422,8	122,8
|m| (60 se	0,369	0,360	0,496	0,496	0,365	0,496

10°C	Penetrasjon ($^1/_{10}$ mm)			
Binder	1	2	3	Average
Original	255	261	262	259
TFOT	216	220	222	219
PAV	117	118	118	118

	Dynamic viscosity (cPoise)		
Binder	60°C	90°C	Skjærhast.
Original	6740	588	
TFOT	8200	620	
PAV	15800	1030	

Measure nr.	Softning point.°C		
	Original	TFOT	PAV
1	18,3	19,8	26,4
2	19.2	20.7	26.5
Average	18,8	20,3	26,5

Measure nr.	Loss of mass after TFOT, %			Loss, %
	Bowl	Before	After	
1	84,9918	135,3483	135,3405	0,0155
2	86,0386	136,3421	136,335	0,0141
3	84,4489	134,5649	134,335	0,0136
Average				0.0144

APPENDIX 7

Results from binder analysis
(chapter 4)

Results from analysis on Pen 180 (B180)

Conventional tests

Sample no	Sampling time	Pen. 25°C	Pen. 10°C	Viscosity 60°C CPS	Viscosity 90°C CPS	Viscosity 135°C CPS	Softening point, °C	Loss of mass, %
A1	Neat	194			3180	212	37,5	
A2	TFOT	115			5849	308	43,5	-0,26083
A3	PAV	64			11200	466	50,2	
A4	0	159			4370	264	40	
A5	0,5	127			5440	285	42,2	
A6	37	101			7440	360	44,8	
A7	41,25	110			6480	340	44,3	
A8	52,25	124			5800	320	42,1	

DSR and BBR

10 16 22 28 34

Sample no	Sampling time	G* 10°C, (kPa)	G* 16°C, (kPa)	G* 22°C, (kPa)	G* 28°C, (kPa)	G* 34°C, (kPa)	Delta 10°C	Delta 16°C	Delta 22°C	Delta 28°C	Delta 34°C	Temp. where G*xsin delta=5000kPa	Temp. where G*/sin delta=2,20 kPa	Temp where S=300 MPa	Temp where lml=0,300 MPa
A1	Neat	2849,75	891,995	284,09	93,599	33,005	64,4	70	74,25	76,9	78,3				
A2	TFOT	4206,3	1483	508,455	177,27	63,606	57,8	63,6	68,3	71,75	74,4		53,6		
A3	PAV	8279	3360,4	1254,8	469,975	180,7	48,2	53,7	59,15	63,4	67,1	11,5		-25,1	-25,1
A4	0	2804,05	968,355	333,975	112,51	41,095	60,8	66,2	70,35	73,5	75,5		51,5		
A5	0,5	3414,35	1256	442,265	159,01	58,498	57,5	62,95	67,35	71	73,4		53,7		
A6	37	4331,8	1662,1	605,2	222,205	83,98	54,2	59,6	64	67,65	70,9		56,3		
A8	52,25	4071,5	1494,45	521,775	182,495	66,208	56,8	62,05	66,6	70,2	73		54,1	-27,4	-29,8

Sample A1, A2 and A3, B180, Hærøy

Dynamic shear rheometer, ORIGINAL Spindel 8 mm
Temp where G*/sinδ = 1,00 kPa (°C) 54,2

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	2987,600	2711,900	926,890	857,100	296,660	271,520	97,376	89,821	34,026	31,983	2849,750	891,995	284,090	93,599	33,005
δ (°)	64,4	64,4	69,9	70,0	74,1	74,4	76,8	77,0	78,4	78,1	64,4	70,0	74,250	76,900	78,3
G*/sinδ (kPa)	3313,200	3007,800	987,120	912,130	308,520	281,910	100,000	92,202	34,739	32,691	3160,500	949,6	295,215	96,101	33,715
γ (%)	1,18	1,21	1,65	1,69	2,30	2,36	3,18	3,26	4,31	4,39	1,20	1,7	2,330	3,220	4,35

Dynamic shear rheometer, TFOT Spindel 8 mm
Temp where G*/sinδ = 2,20 kPa (°C) 53,6

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	4262,000	4150,600	1489,700	1476,300	514,440	502,470	178,530	176,010	63,179	64,032	4206,300	1483,000	508,455	177,270	63,606
δ (°)	57,8	57,7	63,6	63,6	68,1	68,5	71,6	71,9	74,4	74,4	57,8	63,600	68,300	71,750	74,4
G*/sinδ (kPa)	5039,200	4910,800	1663,000	1648,400	554,460	540,230	188,130	185,200	65,609	66,489	4975,000	1655,700	547,345	186,685	68,049
γ (%)	1,06	1,07	1,44	1,45	1,96	1,98	2,67	2,68	3,61	3,59	1,07	1,445	1,970	2,675	3,60

Dynamic shear rheometer, PAV Spindel 8 mm
Temp where G* x sind = 5000 kPa (°C) 11,5

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	8232,7	8325,3	3299,3	3421,5	1239,0	1270,6	467,8	472,2	180,7	180,7	8279,0	3360,400	1254,800	469,975	180,7
δ (°)	48,4	48,0	54,1	53,3	59,3	59,0	63,6	63,2	67,4	66,8	48,2	53,700	59,150	63,400	67,1
G* x sinδ (kPa)	6152,8	6183,1	2672,9	2742,8	1065,7	1089,0	419,0	421,4	166,8	166,1	6168,0	2707,850	1077,350	420,210	166,5
γ (%)	0,88	0,88	1,15	1,13	1,52	1,51	2,02	2,01	2,66	2,66	0,88	1,140	1,515	2,015	2,66

Bending Beam rheometer
Temp where S = 300 MPa (°C) -25,1
Temp where |m| = 0,300 (°C) -25,1

Temp (°C)	-30	-30	-20	-20	-30	-20
S(60 sek) (MP	531,0	531,0	167,0	164,0	531,0	165,5
|m| (60 sek)	0,227	0,227	0,375	0,375	0,227	0,375

25°C	Penetrasjon ($^1/_{10}$ mm)			
Binder	1	2	3	Average
Original	196	192	193	194
TFOT	116	114	114	115
PAV	64.4	64	64,2	64

	Dynamic viscosity (cPoise)		
Binder	90°C	135°C	Skjærhast.
Original	3180	212	
TFOT	5849	308	
PAV	11200	466	

Measure nr.	Softning point,°C		
	Original	TFOT	PAV
1	37,2	43,3	50,0
2	37,9	43,7	50,5
Average	37,5	43,5	50,2

Measure nr.	Loss of mass after TFOT, %			Loss, %
	Bowl	Before	After	
1				-0,2543
2				-0,2562
3				-0,272
Average				-0,26083

Sample A4, B180, Hærøy

Dynamic shear rheometer, TFOT Spindel 8 mm
Temp where G*/sinδ = 2.20 kPa (°C)

51,5

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	2818,800	2789,300	951,060	985,650	333,730	334,220	107,890	117,130	39,994	42,195	2804,050	968,355	333,975	112,510	41,095
δ (°)	60,8	60,7	66,4	66,0	70,3	70,4	73,5	73,5	75,4	75,5	60,8	66,200	70,350	73,500	75,5
G*/sinδ (kPa)	3227,800	3197,400	1038,000	1078,600	354,550	354,880	112,510	122,150	41,329	43,586	3212,600	1058,300	354,715	117,330	42,458
γ (%)	1,20	1,20	1,64	1,63	2,23	2,22	3,09	3,01	4,12	4,05	1,20	1,635	2,225	3,050	4,09

Pen.($^1/_{10}$ mm), 25°C, average of 3	159
Dynamic viscosity (cPoise), 90°C	4370
Dynamic viscosity (cPoise), 135°C	264
Softning point,°C, average of 3	40

Sample A5, B180, Hærøy

Dynamic shear rheometer, TFOT Spindel 8 mm
Temp where G*/sinδ = 2.20 kPa (°C)

53,7

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	3521,000	3307,700	1282,300	1229,700	448,130	436,400	159,890	158,130	57,881	59,115	3414,350	1256,000	442,265	159,010	58,498
δ (°)	57,4	57,5	63,0	62,9	67,5	67,2	71,2	70,8	73,3	73,4	57,5	62,950	67,350	71,000	73,4
G*/sinδ (kPa)	4181,200	3921,000	1439,000	1380,800	485,080	473,320	168,870	167,400	60,444	61,687	4051,100	1409,900	479,200	168,135	61,066
γ (%)	1,12	1,14	1,51	1,52	2,04	2,06	2,75	2,76	3,70	3,68	1,13	1,515	2,050	2,755	3,69

Pen.($^1/_{10}$ mm), 25°C, average of 3	127
Dynamic viscosity (cPoise), 90°C	5440
Dynamic viscosity (cPoise), 135°C	285
Softning point,°C, average of 3	42,2

Sample A6, B180, Hærøy

Dynamic shear rheometer, TFOT Spindel 8 mm

Temp where G*/sinδ = 2.20 kPa (°C) 56,3

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	4196,300	4467,300	1642,000	1682,200	580,860	629,540	216,820	227,590	82,090	85,869	4331,800	1662,100	605,200	222,205	83,980
δ (°)	54,4	53,9	59,7	59,5	64,2	63,8	67,7	67,6	70,9	70,9	54,2	59,600	64,000	67,650	70,9
G*/sinδ (kPa)	5159,700	5529,800	1902,100	1952,200	645,180	701,350	234,430	246,180	86,884	90,891	5344,750	1927,150	673,265	240,305	88,888
γ (%)	1,07	1,05	1,40	1,39	1,90	1,85	2,52	2,49	3,34	3,30	1,06	1,395	1,875	2,505	3,32

Pen.($^1/_{10}$ mm), 25°C, average of 3	101
Dynamic viscosity (cPoise), 90°C	7440
Dynamic viscosity (cPoise), 135°C	360
Softning point,°C, average of 3	44,8

Sample A7, B180, Hærøy

Pen.($^1/_{10}$ mm), 25°C, average of 3	110
Dynamic viscosity (cPoise), 90°C	6480
Dynamic viscosity (cPoise), 135°C	340
Softning point,°C, average of 3	44,3

Sample A8, B180, Hærøy

Dynamic shear rheometer, TFOT Spindel 8 mm

Temp where G*/sinδ = 2.20 kPa (°C) 54,1

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	4086,400	4056,600	1528,900	1460,000	536,810	506,740	188,330	176,660	68,313	64,102	4071,500	1494,450	521,775	182,495	66,208
δ (°)	56,8	56,8	62,0	62,1	66,5	66,7	70,3	70,1	72,8	73,1	56,8	62,050	66,600	70,200	73,0
G*/sinδ (kPa)	4885,000	4848,600	1731,100	1652,000	585,270	551,750	200,030	187,820	71,526	67,003	4866,800	1691,550	568,510	193,925	69,265
γ (%)	1,08	1,08	1,43	1,45	1,94	1,97	2,63	2,68	3,53	3,59	1,08	1,440	1,955	2,655	3,56

Bending Beam rheometer

Temp where S = 300 MPa (°C) -27,4

Temp where |m| = 0,300 (°C) -29,8

Temp (°C)	-30	-30	-20	-20	-30	-20
S(60 sek) (MPa)	453,0	455,0	93,1	90,2	454,0	91,7
|m| (60 sek)	0,292	0,302	0,451	0,455	0,297	0,453

Pen.($^1/_{10}$ mm), 25°C, average of 3	124
Dynamic viscosity (cPoise), 90°C	5800
Dynamic viscosity (cPoise), 135°C	320
Softning point,°C, average of 3	42,1

Results from analysis on binder MB10000

Conventional tests

Sample no	Sampling time	en. 25°	Pen. 10°C	Viscosity 60°C CPS	Viscosity 90°C CPS	Viscosity 135°C	Softening point, °C	Loss of mass, %
B1	Neat		155	10800	915		23,7	
B2	TFOT		129	13800	1040		25,7	-0,0116
B3	PAV		68	33600	1940		33,2	
B4	0		126	13700	1100		24,5	
B5	0,3		112	15300	1300		27,8	
B6	15,3		82	25000	1800		30,9	
B7	23,15		75	28200	1800		31,6	

DSR and BBR

							10	16	22	28	34				
Sample no	Sampling time	G* 10°C, (kPa)	G* 16°C, (kPa)	G* 22°C, (kPa)	G* 28°C, (kPa)	G* 34°C, (kPa)	Delta 10°C	Delta 16°C	Delta 22°C	Delta 28°C	Delta 34°C	Temp. where G*xsin delta=5000kPa	Temp. where G*/sin delta=2,20 kPa	Temp where S=300 MPa	Temp where lml=0,300 MPa
B1	Neat	177,2	59,8	20,3	7,3	2,9	74,3	78,9	82,2	84,3	85,8				
B2	TFOT	221,0	75,3	26,4	9,9	3,8	71,5	76,7	80,2	82,9	84,8		37,4		
B3	PAV	789,1	272,0	95,6	34,4	13,1	64,5	68,7	72,0	74,2	73,6	-1,3		-34,2	-38,3
B4	0	236,3	82,2	28,9	10,0	3,9	70,3	75,5	79,3	82,5	84,5		37,6		
B5	0,3	258,3	90,7	32,8	10,9	4,3	68,9	74,4	78,1	81,5	83,7		38,2		
B6	15,3	376,9	140,6	50,9	18,5	7,6	61,8	68,5	73,2	77,4	80,5		42,3		
B7	23,15	487,0	182,6	65,3	24,6	9,1	58,8	66,4	71,7	75,9	79,5		42,6	-35,0	-39,5

Sample B1, B2, and B3, MB 10.000 HITRA

Dynamic shear rheometer, ORIGINAL Spindel 25 mm
Temp where G*/sinδ = 1.00 kPa (°C) 40,8

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	183,730	170,640	61,268	58,389	20,857	20,069	7,539	7,098	2,934	2,808	177,185	59,829	20,463	7,319	2,871
δ (°)	74,3	74,2	79,0	78,8	82,2	81,8	84,1	84,4	85,8	85,8	74,3	78,9	82,000	84,250	85,8
G*/sinδ (kPa)	190,830	177,350	62,414	59,523	21,053	20,277	7,579	7,132	2,942	2,816	184,090	61,0	20,665	7,356	2,879
γ (%)	2,65	2,70	3,64	3,69	4,97	5,03	6,68	6,80	8,78	8,90	2,68	3,7	5,000	6,740	8,84

Dynamic shear rheometer, TFOT Spindel 25 mm
Temp where G*/sinδ = 2.20 kPa (°C) 37,4

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	221,680	220,400	75,644	75,031	26,250	26,726	10,050	9,692	3,756	3,842	221,040	75,338	26,486	9,866	3,799
δ (°)	71,5	71,4	76,8	76,5	80,3	80,0	82,8	83,0	84,9	84,7	71,5	76,650	80,150	82,900	84,8
G*/sinδ (kPa)	233,720	232,510	77,707	77,177	26,628	27,142	10,130	9,756	3,771	3,858	233,115	77,442	26,885	9,943	3,814
γ (%)	2,51	2,51	3,52	3,43	4,65	4,63	6,15	6,21	8,16	8,12	2,51	3,475	4,640	6,180	8,15

Dynamic shear rheometer, PAV Spindel 8 mm
Temp where G* x sind = 5000 kPa (°C) -1,3

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	797,8	780,5	274,7	269,2	96,5	94,6	34,7	34,0	13,1	13,2	789,1	271,955	95,564	34,374	13,1
δ (°)	64,3	64,6	68,6	68,7	72,0	72,0	74,1	74,2	73,6	73,5	64,5	68,650	72,000	74,150	73,6
G* x sinδ (kPa)	718,8	704,8	255,7	250,9	91,8	90,0	33,4	32,7	12,6	12,6	711,8	253,280	90,905	33,066	12,6
γ (%)	1,73	1,74	2,35	2,37	3,19	3,21	4,29	4,32	5,69	5,68	1,74	2,360	3,200	4,305	5,69

Bending Beam rheometer
Temp where S = 300 MPa (°C) -34,2
Temp where |m| = 0,300 (°C) -38,3

Temp (°C)	-35	-35	-30	-30	-35	-30
S(60 sek) (MP	351,0	341,0	149,0	146,0	346,0	147,5
|m| (60 sek)	0,341	0,347	0,408	0,414	0,344	0,411

10°C	Penetrasjon ($^1/_{10}$ mm)			
Binder	1	2	3	Average
Original	154	155	155	155
TFOT	128	129	129	129
PAV	66,8	67,8	68,1	68

	Dynamic viscosity (cPoise)		
Binder	60°C	90°C	Skjærhast.
Original	10800	915	
TFOT	13800	1040	
PAV	33600	1940	

Measure nr.	Softning point,°C		
	Original	TFOT	PAV
1	23,8	25,3	33,1
2	23,6	26,1	33,4
Average	23,7	25,7	33,2

Measure nr.	Loss of mass after TFOT, %			Loss, %
	Bowl	Before	After	
1				-0,0112
2				-0,0108
3				-0,0128
Average				-0,0116

Sample B4, MB 10.000, Hitra

Dynamic shear rheometer, TFOT		Spindel 25 mm
Temp where G*/sinδ = 2.20 kPa (°C)		37,6

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	234,120	238,380	78,561	85,748	28,431	29,418	10,281	9,811	3,779	4,004	**236,250**	**82,155**	**28,925**	**10,046**	**3,891**
δ (°)	70,2	70,4	75,5	75,4	79,4	79,2	82,5	82,4	84,5	84,5	**70,3**	**75,450**	**79,300**	**82,450**	**84,5**
G*/sinδ (kPa)	248,810	252,970	81,129	88,615	28,923	29,945	10,373	9,898	3,797	4,022	**250,890**	**84,872**	**29,434**	**10,135**	**3,909**
γ (%)	2,47	2,45	3,38	3,30	4,55	4,50	6,10	6,19	8,16	8,02	**2,46**	**3,340**	**4,525**	**6,145**	**8,09**

Pen.($^1/_{10}$ mm), 10°C, average of 3	126
Dynamic viscosity (cPoise), 60°C	13700
Dynamic viscosity (cPoise), 90°C	1100
Softning point,°C, average of 2	24,5

Sample B5, MB 10.000, Hitra

Dynamic shear rheometer, TFOT		Spindel 25 mm
Temp where G*/sinδ = 2.20 kPa (°C)		38,2

Temp (°C)	10	10	16	16	22	22	28	28	34	34	10	16	22	28	34
G* (kPa)	262,520	254,010	90,911	90,494	32,745	32,877	10,666	11,084	4,342	4,180	**258,265**	**90,703**	**32,811**	**10,875**	**4,261**
δ (°)	69,1	68,7	74,4	74,4	78,2	77,9	81,6	81,3	83,6	83,7	**68,9**	**74,400**	**78,050**	**81,450**	**83,7**
G*/sinδ (kPa)	281,040	272,540	94,405	93,935	33,448	33,619	10,783	11,214	4,369	4,206	**276,790**	**94,170**	**33,534**	**10,999**	**4,288**
γ (%)	2,39	2,41	3,24	3,25	4,36	4,36	6,04	5,97	7,83	7,93	**2,40**	**3,245**	**4,360**	**6,005**	**7,88**

Pen.($^1/_{10}$ mm), 10°C, average of 3	112
Dynamic viscosity (cPoise), 60°C	15300
Dynamic viscosity (cPoise), 90°C	1300
Softning point,°C, average of 2	27,8

Sample B6, MB10.000, Hitra

Dynamic shear rheometer, TFOT Spindel 25 mm

Temp where G*/sinδ = 2.20 kPa (°C) 42,3

Temp (°C)	10	10	16	16	22	22	28	28	34	34	**10**	**16**	**22**	**28**	**34**
G* (kPa)	379,220	374,480	144,120	137,120	51,539	50,225	18,777	18,218	7,536	7,591	**376,850**	**140,620**	**50,882**	**18,498**	**7,564**
δ (°)	61,8	61,7	68,6	68,4	73,1	73,3	77,3	77,4	80,5	80,5	**61,8**	**68,500**	**73,200**	**77,350**	**80,5**
G*/sinδ (kPa)	430,600	425,400	154,840	147,510	53,877	52,429	19,246	18,668	7,641	7,698	**428,000**	**151,175**	**53,153**	**18,957**	**7,669**
γ (%)	2,14	2,15	2,84	2,88	3,83	3,85	5,13	5,17	6,68	6,67	**2,15**	**2,860**	**3,840**	**5,150**	**6,68**

Pen.($^1/_{10}$ mm), 10°C, average of 3	81,8
Dynamic viscosity (cPoise), 60°C	25000
Dynamic viscosity (cPoise), 90°C	1800
Softning point,°C, average of 2	30,9

Sample B7, MB10.000, Hitra

Dynamic shear rheometer, TFOT Spindel 25 mm

Temp where G*/sinδ = 2.20 kPa (°C) 42,6

Temp (°C)	10	10	16	16	22	22	28	28	34	34	**10**	**16**	**22**	**28**	**34**
G* (kPa)	517,800	456,250	191,370	173,890	67,374	63,299	24,519	24,700	9,065	9,217	**487,025**	**182,630**	**65,337**	**24,610**	**9,141**
δ (°)	58,5	59,0	66,2	66,5	71,8	71,6	76,0	75,8	79,7	79,3	**58,8**	**66,350**	**71,700**	**75,900**	**79,5**
G*/sinδ (kPa)	607,080	532,460	209,130	189,620	70,911	66,692	25,267	25,483	9,215	9,378	**569,770**	**199,375**	**68,802**	**25,375**	**9,296**
γ (%)	1,96	2,03	2,61	2,69	3,54	3,60	4,74	4,73	6,33	6,30	**2,00**	**2,650**	**3,570**	**4,735**	**6,32**

Bending Beam rheometer

Temp where S = 300 MPa (°C) -35,0

Temp where |m| = 0,300 (°C) -39,5

Temp (°C)	-35	-35	-30	-30	**-35**	**-30**
S(60 sek) (MPa)	301,0	295,0	117,0	127,0	**298,0**	**122,0**
|m| (60 sek)	0,368	0,363	0,446	0,431	**0,366**	**0,439**

Pen.($^1/_{10}$ mm), 10°C, average of 3	74,5
Dynamic viscosity (cPoise), 60°C	28200
Dynamic viscosity (cPoise), 90°C	1800
Softning point,°C, average of 2	31,6

APPENDIX 8

Procedures for sample preparation.
(chapter 5)

Procedure for preparation of filler-bitumen samples

The treatment of the binder in the sample preparation is described as follows. Temperatures given are used for MB10000 and temperature for B180 is given in parentheses.

Original

1. Bottle with binder was heated at 120/130°C (C_e=0,3/C_e=0,6)(150°C) until it is completely liquid, and then split in boxes for mixing.
2. Boxes with binder were cooled down to room temperature and stored.
3. Boxes with binder were heated at 120/130°C (C_e=0,3/C_e=0,6)(150°C). The filler was heated at the same temperature.
4. The filler and binder were mixed using a glass stick for about 1-3 minutes.
5. After mixing the samples were reheated for 10 minutes at 120/130°C (C_e=0,3/C_e=0,6) (150°C).
6. Short mixing before sample preparation.
7. Sample preparation.

TFOT-aged

1.- 6. The same procedure as for original samples.

7. Mix was transferred to TFOT-pans.
8. The standard TFOT was performed at 120°C (163°C).
9. TFOT-pans were scraped and the material poured into a container and cooled down.
10. The mix was heated at 120°C (150°C) and then homogenised.
11. The mix was heated at 120°C (160°C) for 15 minutes with a vacuum pressure of 150 millibar to remove entrapped air.
12. The mix was reheated at 130°C (150°C) for 10 minutes.
13. Short mixing before sample preparation.

PAV-aged

1. - 8. The same procedure as TFOT-aged.

9. TFOT-pans were cooled down to room temperature and stored.
10. PAV-test was performed in 7 days at 60°C, with a pressure of 2,1 MPa.
11. TFOT-pans were scraped and the material poured into a container and cooled down.
12. The mix was heated at 120°C (150°C) and homogenised.
13. The mix was heated at 120°C (160°C) for 15 minutes with a vacuum pressure of 150 millibar.
14. The mix was reheated at 130°C (150°C) for 10 minutes.
15. Short mixing before sample preparation.

APPENDIX 9

Results from DSR-measurements on filler-bitumen-mixes (chapter 5)

RESULTS FROM DSR-MEASUREMENTS, PEN180, Ce=0,3

Temperature	10				20				30			
Filler/treatment	G* (kPa)	G' (kPa)	G" (kPa)	phase angle (deg)	G* (kPa)	G' (kPa)	G" (kPa)	phase angle (deg)	G* (kPa)	G' (kPa)	G" (kPa)	phase angle (deg)
No filler, orig.	2885	1285	2580	63,6	372	97	360	75	54	9	53	80,3
" TFOT	4380	2510	3585	55	667	245	620	68,4	104	27	100	74,9
" PAV	8025	5795	5550	43,8	1540	761	1335	60,5	264	95	246	69
Limestone, orig.	7525	4025	6360	57,7	1025	288	987	73,7	142	25	140	80
" TFOT	9665	6185	7455	50,3	1530	556	1425	68,7	233	57	226	76
" PAV	21500	18650	10700	30	5275	2890	4415	56,8	931	339	867	68,7
Hydr. Lime, orig.	6365	3355	5410	58,2	877	250	841	73,5	126	23	124	79,7
" TFOT	8640	5500	6665	50,5	1400	525	1295	68	220	56	212	75,2
" PAV	15500	12900	8610	33,7	3715	2090	3070	55,8	678	267	623	66,8
Lia, orig.	7415	4015	6230	57,3	975	280	958	73,7	134	24	132	79,9
" TFOT	9070	5810	6965	50,2	1445	539	1340	68,2	224	56	217	75,5
" PAV	14850	12100	8625	35,6	3340	1800	2815	57,4	698	227	553	67,7
Tau, orig.	6620	3430	5660	58,8	871	237	838	74,2	124	21	122	80
" TFOT	10350	6720	7920	49,7	1640	603	1525	68,4	249	62	242	75,7
" PAV	18050	15150	9890	33,2	4250	2315	3575	57,1	755	284	700	68

RESULTS FROM DSR-MEASUREMENTS, MB10000, C_e=0,3

Temerature	10				20				30			
Filler/treatment	G* (kPa)	G' (kPa)	G" (kPa)	Phase angel (deg)	G* (kPa)	G' (kPa)	G" (kPa)	Phase angel (deg)	G* (kPa)	G' (kPa)	G" (kPa)	Phase angel (deg)
No filler, orig.	223	47	218	77,9	31	4	30	82,3	5,4	0,5	5,3	84,6
" TFOT	245	59	238	76,0	35	6	35	80,9	6,2	0,7	6,1	83,9
" PAV	620	232	575	68,0	96	25	93	75,1	16,1	2,8	15,9	80,2
Limestone, orig.	594	131	580	77,3	81	11	80	82,3	12,9	1,1	12,8	84,9
" TFOT	696	177	674	75,3	100	16	99	80,7	16,7	1,7	16,6	84,2
" PAV	1737	690	1595	66,6	262	68	253	75,0	44,6	7,4	44,0	80,4
Hydr.lime, orig	511	117	498	76,8	71	10	70	81,9	11,6	1,1	11,6	84,8
" TFOT	642	175	618	74,3	93	16	91	79,9	15,4	1,7	15,3	83,5
" PAV	1643	693	1489	65,1	257	73	247	73,4	42,8	7,9	42,0	79,4
Lia, orig.	509	115	496	77,0	70	10	70	82,1	11,2	1,0	11,1	85,0
" TFOT	591	148	572	75,5	84	14	83	80,8	13,4	1,4	13,3	84,1
" PAV	1638	658	1500	66,3	248	66	239	74,5	41,2	7,1	40,6	80,1
Tau, orig.	517	116	504	77,1	71	10	71	82,1	11,6	1,0	11,5	84,9
" TFOT	598	148	580	75,7	87	14	86	80,9	13,6	1,3	13,6	84,4
" PAV	1518	597	1396	66,8	228	59	220	74,9	38,3	6,5	37,7	80,2

RESULTS FROM DSR-MEASUREMENTS, MB10000, C_e=0,6

Temperature	10				20				30			
Filler/treatment	G* (kPa)	G' (kPa)	G" (kPa)	Phase angel (deg)	G* (kPa)	G' (kPa)	G" (kPa)	Phase angel (deg)	G* (kPa)	G' (kPa)	G" (kPa)	Phase angel (deg)
No filler, orig.	223	47	218	77,9	31	4	30	82,3	5,4	0,5	5,3	84,6
" TFOT	245	59	238	76,0	35	6	35	80,9	6,2	0,7	6,1	83,9
" PAV	620	232	575	68,0	96	25	93	75,1	16,1	2,8	15,9	80,2
Limestone, orig.	1414	334	1373	76,3	197	27	195	82,0	31,8	2,7	31,7	85,1
" TFOT	1736	478	1668	74,0	245	40	242	80,5	39,3	4,0	39,1	84,2
" PAV	4125	1829	3697	63,7	634	172	611	74,3	109,0	18,8	107,0	80,1
Hydr.lime, orig.	1125	286	1088	75,3	162	25	160	81,1	27,2	2,6	27,1	84,4
" TFOT	1533	475	1458	72,0	229	45	225	78,7	40,5	4,9	40,2	83,0
" PAV	4102	1994	3585	60,9	675	214	640	71,5	120,0	25,0	117,0	77,9
Lia, orig.	1298	314	1260	76,0	174	25	172	81,8	27,8	2,4	27,7	85,1
" TFOT	1573	434	1512	74,0	223	37	220	80,4	35,8	3,6	35,7	84,2
" PAV	4042	1861	3588	62,6	632	179	606	73,6	113,0	20,7	111,0	79,4
Tau, orig.	1295	308	1258	76,3	178	25	177	82,0	29,0	2,5	28,9	85,1
" TFOT	1591	436	1530	74,1	225	37	221	80,5	36,1	3,7	35,9	84,2
" PAV	4421	2033	3926	62,6	696	198	667	73,5	118,0	21,5	116,0	79,5