DOCTORAL THESIS

Development of Pavement Temperature Estimation Models and the Impact of Climate Change on Pavement Temperatures in Estonia

Karli Kontson

TALLINNA TEHNIKAÜLIKOOL
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KARLI KONTSON

TALLINN UNIVERSITY OF TECHNOLOGY
School of Engineering
Department of Civil Engineering and Architecture
This dissertation was accepted for the defence of the degree 09/04/2025

Supervisor: Assoc. Prof. Andrus Aavik
School of Engineering
Tallinn University of Technology
Tallinn, Estonia

Co-supervisor: Prof. Artu Ellmann
School of Engineering
Tallinn University of Technology
Tallinn, Estonia

Opponents: Assoc. Prof. DSc Viktors Haritonovs
Faculty of Civil and Mechanical Engineering
Institute of Civil Engineering
Riga Technical University
Riga, Latvia

Assoc. Prof. DSc habil.
Faculty of Civil and Environmental Engineering
Department of Transportation Engineering
Technology

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Declaration:
Hereby I declare that this doctoral thesis, my original investigation and achievement, submitted for the doctoral degree at Tallinn University of Technology has not been submitted for doctoral or equivalent academic degree.

Karli Kontson

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TALLINNA TEHNIKAÜLIKOOL
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Teekatte temperatuuride arvutamise mudelite arendamine ja kliimamuutuste mõju Eesti teekatete temperatuuridele

KARLI KONTSON

TALTECH
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Contents

List of Publications

The thesis has been prepared based on the following peer-reviewed journal (indexed by SCOPUS and WOS) papers:

I **Kontson, K.**, Lill, K., & Aavik, A. (2023). Superpave Pavement Design Temperatures in Estonia. The Baltic Journal of Road and Bridge Engineering, 18(2), 190–204. https://doi.org/10.7250/bjrbe.2023-18.603

II **Kontson, K.**, Lill, K., & Aavik, A. (2024). Statistical-Empirical Pavement Temperature Prediction Models Based on Data From Road Weather Stations in Estonia. Road Materials and Pavement Design, 1–11. https://doi.org/10.1080/14680629.2024.2415347

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Author’s Contribution to the Publications

Contribution to the papers in this thesis are:

I Conceptualization, data collection, analysis and visualisation. Drafting and writing the original manuscript. Reviewing the manuscript with the co-authors.

II Conceptualization, data analysis, writing – original draft and revision of final manuscript.

III Conceptualisation, data collection, data analysis and visualisation. Drafting and writing the manuscript with co-authors and reviewing the manuscript.

Introduction

In modern society, roads are indispensable and essential for various reasons, serving as critical infrastructure that supports economic, social, security and environmental functions. In Europe, the annual asphalt production volumes range between 250 and 300 million tons, with more than 50 million square meters of various bituminous surface treatments laid each year to support infrastructure development and maintenance (EAPA, 2023).

Asphalt mixtures typically comprise 94–96 wt.% aggregate and 4–6 wt.% bitumen. Despite its relatively low proportion, bitumen plays a crucial role in ensuring the performance and durability of asphalt pavements. As a thermoplastic and viscoelastic material, the in-service behaviour of bitumen is significantly affected by environmental factors and loading conditions, such as temperature and forces induced by traffic. Bitumen viscosity decreases at elevated temperatures, making asphalt pavements more susceptible to rutting under traffic-induced loads (Delgadillo & Bahia, 2010; Harvey et al., 2009; Sousa et al., 1991). Example of asphalt pavement rutting is illustrated in Figure 1.

Figure 1. Extreme case of asphalt rutting on road nr E67 (Pärnu bypass). Rutting occurred soon after paving in summer 2012.

At lower temperatures, the viscosity of bitumen increases, causing it to transition into a solid-elastic state which results in reduced strain tolerance and brittle behaviour (Harvey et al., 2009; Hunter et al., 2015). Brittleness combined with temperature-induced shrinkage at low temperatures in asphalt pavements can lead to premature pavement cracking, known as low temperature cracking (Figure 2) (Hesp, 2003; Hesp et al., 2009; Judycki et al., 2017; Rys et al., 2022; Yee et al., 2006).

Figure 2. Pavement low temperature transverse cracking with secondary fatigue cracking adjacent to crack. Fatigue cracking is most probably caused by discontinuity in pavement structural integrity and water penetration into the subbase and subgrade.

Rutting and low-temperature cracking are among the most significant types of pavement failures. Rutting shortens the service life of pavements and can compromise road safety by creating conditions for water pooling, which increases the risk of hydroplaning or ice formation in wheel paths (EC, 1999; Fares et al., 2024; Start et al., 1998). Low-temperature cracking is one of the primary failure modes in cold regions (Dore & Zubeck, 2009; Hesp, 2003; Hesp et al., 2009; Yee et al., 2006). Cracked asphalt pavements allow water to penetrate into the pavement, leading to progressive pavement deterioration (Birgisson & Ruth, 2003; Cedergren, 1988; Dawson, 2008; Dore & Zubeck, 2009; Wiman, 2006). This process is often additionally affected by specific seasonal problems, such as extended wet periods in spring or freeze-thaw cycles, which accelerate the deterioration and ultimately reduce the pavement's service life. This is especially important in Estonia where sedimentary aggregates susceptible to moisture and freeze-thaw cycles are used as primary base course material (Jefremova et al., 2009; Sillamäe et al., 2015; Truu et al., 2013).

Although it is difficult to provide exact quantitative estimation on how much bitumen properties affect the pavement performance, different studies indicate that approximately 30–40% of rutting and up to 90% of low-temperature cracking is estimated to be caused by bitumen properties (Dawley & Pulles, 1996; Hesp, 2003; Hesp et al., 2009; Saarela, 1992). All this leads to a conclusion that in order to ensure the durability of asphalt pavements, it's essential to carefully select bitumen properties to meet the environmental and loading conditions of the pavement.

In Europe, the properties of bitumen are determined by empirical specifications known as penetration grading (CEN, 2009, 2010, 2015d). The selection of bitumen grade for a specific road or region is typically made by the pavement designers or road agencies, relying on prior experience with the specific bitumen grade. However, the empirical specifications have shown to correlate weakly with in-service pavement performance (Asphalt Institute, 2011; Kennedy et al., 1994; Lill et al., 2020a). Criticism of penetration grading intensified following the crude oil crisis in the 1960s and 1970s in the USA, when the diversification of crude oil sources used for fuel and bitumen production was perceived by the road industry as a major contributor to variability in the performance of bitumen with similar penetration grades (Adams & Holmgreen, 1986; Petersen et al., 1993). Currently, similar issues may rise as the security situation and sanctions on Russia have caused variations in the traditional crude oil sources.

In the late 1980s, the U.S. government initiated the Strategic Highway Research Program (SHRP) which was aimed to improve the performance and durability of asphalt pavements against rutting, fatigue cracking and low temperature cracking (Kennedy et al., 1994; TRB, 1984). One of the many significant outcomes of the SHRP was the introduction of the Superpave (Superior Performing Asphalt Pavements) bitumen specification, known as Superpave Performance Grading (PG grading), which is now widely adopted in North America as the AASHTO M 320 specification (AASHTO, 2023; Asphalt Institute, 1996, 2011). SHRP researchers determined that bitumen properties should be selected based on two key factors: (1) traffic load characteristics, including traffic speed and equivalent single axle loads (ESALs), and (2) environmental conditions, such as the temperature range of the road (Asphalt Institute, 1996, 2011; Kennedy et al., 1994). This introduced a completely new approach for specifying bitumen properties and requirements, replacing traditionally used empirical specifications with newly developed test methods that were targeted to measure the fundamental properties of bitumen. The aim was to provide better correlation between measured properties and in-service pavement performance (Asphalt Institute, 1996, 2011; Kennedy et al., 1994).

The PG grading is based on the actual environmental conditions of the road, specifically the temperatures at which the pavement is expected to perform. According to the PG grading, bitumen grades are denoted as "PG HT-LT", where "PG" stands for *Performance Grade* and "HT-LT" represent the *high temperature grade* and *low temperature grade,* respectively. PG grade indicates the range of pavement temperatures within which the bitumen is assumed to provide adequate resistance to both environmental and loading conditions (Asphalt Institute, 1996, 2011; Kennedy et al., 1994).

The aim of this thesis is to analyse, which PG grades are applicable for Estonian climatic region. Since asphalt pavements are susceptible to temperature variations, the impact of climate change was also considered in the analysis. Results indicated that widely used PG grade estimation models could lead to erroneous results in Estonia. Therefore, pavement temperature estimation models were developed to better suit Estonian conditions.

Publication I investigated the Performance Grades suitable for Estonian conditions based on different pavement temperature calculation models. The comparison between those models showed that calculated pavement temperatures are significantly influenced by the selected model. Field validation was carried out based on road weather station data which indicated that pavement temperature calculation models developed in Canada and Norway resulted in reasonably close estimates with in-service pavement temperatures.

Publication II developed pavement temperature prediction models based on road weather stations installed in Estonian. Pavement temperature data extracted from road weather stations was used to develop statistical-empirical pavement temperature models to calculate pavement maximum and minimum temperatures with respect to depth from pavement surface. Field validation indicated that these models provide improved precision compared to other widely adopted pavement temperature estimation models.

Publication III investigated the potential impact of climate change to pavement temperatures in Estonia. The analysis was conducted based on pavement temperature models developed in **Publication II** and based on three different climate change scenarios. Analysis indicated that all observed scenarios lead to increased pavement maximum and minimum temperatures with more significant increases expected in pavement minimum temperatures.

The main novelties of the research are a) developing pavement temperature estimation models suitable for Estonian environmental conditions; b) introducing the Superpave PG grade suitable for use in Estonia; c) analysing the impact of climate change on pavement temperatures and bitumen grade selection in Estonia.

Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
ASTM	American Society for Testing and Materials
BBR	Bending Beam Rheometer
C3S	Copernicus Climate Change Service
CMIP6	Coupled Model Intercomparison Project Phase 6
C-SHRP	Canadian Strategic Highway Research Program
DENT	Double-Edge-Notched Tension
DSR	Dynamic Shear Rheometer
DTT	Direct Tension Test
EEA	Estonian Environment Agency
EN	European norm (standard)
ESAL	Equivalent Single-Axle Load
IPCC	Intergovernmental Panel on Climate Change
IPCC AR6	IPCC Sixth Assessment Report
LTPP	Long-Term Pavement Performance
LTPP-SMP	Long-Term Pavement Performance Seasonal Monitoring Program
MSCR	Multiple Stress Creep Recovery
PAV	Pressure Ageing Vessel
PG	Performance Grading
PG HT	PG high temperature
PG IT	PG intermediate temperature
PG LT	PG low temperature
RCP	Representative Concentration Pathways
RTFOT	Rolling Thin Film Oven Test
SARA	Saturates, Aromatics, Resins, Asphaltenes
SHRP	Strategic Highway Research Program
SSP	Shared Socioeconomic Pathway
SUPERPAVE	Superior Performing Asphalt Pavements

1 Literature overview

Bitumen is a crucial material for road construction and asphalt paving industry. It's a by-product of crude oil refining and is primarily produced by distillation processes. Key crude oil refining methods include atmospheric and vacuum distillation, with vacuum distillation being predominant for bitumen production (Hunter et al., 2015; Mund et al., 2009). Bitumen is the fraction that remains after distillation, offering characteristics highly sought after in road construction industry, such as thermoplastic properties, water resistance and adhesion. The amount of bitumen (residue) in a crude oil and the consistency and viscosity of the bitumen are influenced by crude oil source and refining processes (Behrenbruch & Dedigama, 2007; Hunter et al., 2015; Lesueur, 2009; Mund et al., 2009). The general overview of the crude oil refining process and oil fractionation is illustrated in Figure 3.

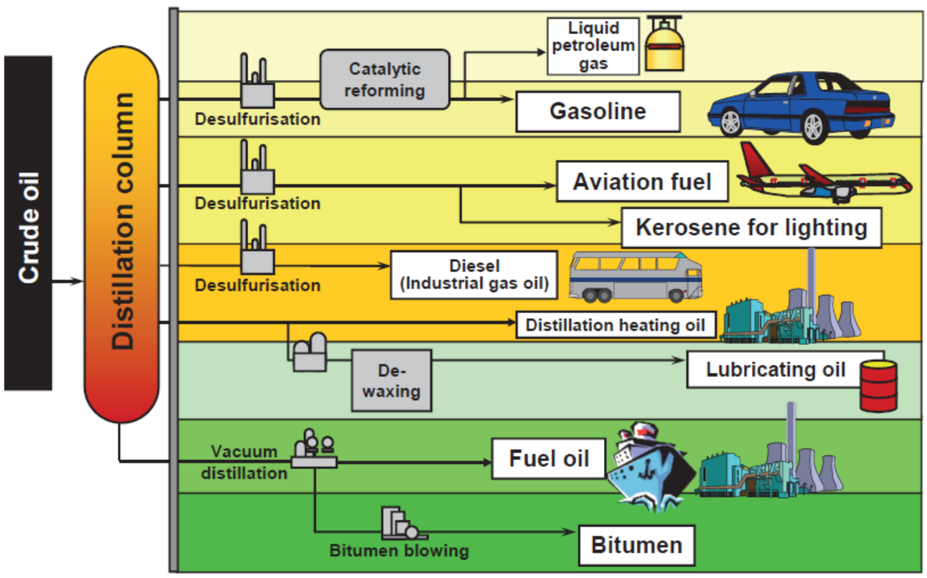

Figure 3. Schematic overview of crude oil refining process (Hunter et al., 2015).

Bitumen can flow like a viscous liquid at high temperatures (e.g., those associated with asphalt production) but behaves as a glass-like material at low temperatures (Harvey et al., 2009; Hunter et al., 2015; Lesueur, 2009; Robertson et al., 2001). At room temperature, bitumen is a very viscous liquid. In addition to temperature dependency, bitumen properties are affected also by loading characteristics, such as applied stress and loading duration (Asphalt Institute, 2011; Hunter et al., 2015; Lesueur, 2009).

To ensure that asphalt pavement performance and durability is going to meet the expectations of the road owners and users, it is important to select bitumen properties (i.e. viscosity, stiffness) to match closely with the environmental and loading conditions of the road or region (Asphalt Institute, 2011; Kennedy et al., 1994; Robertson et al., 2001).

1.1 Bitumen chemistry, ageing and its' impact to properties and durability

The unique properties of bitumen originate from its complex chemical composition, which is primarily made up of inhomogeneous mixture of hydrocarbons with varying structures, molecular weights, and polarities, along with heteroatoms such as oxygen, sulphur, nitrogen, and trace metals (Hunter et al., 2015; Lesueur, 2009; Li & Greenfield, 2014; Petersen, 2009; Robertson et al., 2001). Bitumen composition is determined by crude oil source and refining process (Branthaver et al., 1994; Mund et al., 2009; Petersen, 2009; Petersen et al., 1993). Due to the complexity of bitumen chemistry, it is virtually impossible to separate and determine all individual bitumen components and their impact to bitumen properties. However, the composition of bitumen can be broadly classified into separate fractions having similar properties. The most widely adopted approach is known as SARA fractions first introduced by Corbett (Corbett, 1969; Hunter et al., 2015; Lesueur, 2009). SARA refers to saturates, aromatics, resins and asphaltenes. The physical appearance of these groups is illustrated in Figure 4.

The complex interactions between SARA fractions can be used to explain the mechanical and rheological properties of bitumen to some degree. The characterisation of SARA fractions and their impact on its properties can be briefly summarized as follows (Fernandez-Gomez et al., 2014; Lesueur, 2009; Redelius & Soenen, 2015; Robertson et al., 2001; S. Weigel & Stephan, 2018):

- Saturates are non-polar hydrocarbon fraction with minimal functional groups, forming a colourless or lightly coloured liquid at room temperature. The glass transition temperature of saturates is around –70 ^{o}C. They have low polarity and molecular weight, contributing to the overall ductility of bitumen. The typical content of saturates in bitumen is 5–15%;
- Aromatics are slightly polar constituents with lightly condensed aromatic rings, forming a viscous yellow to red liquid at room temperature. Aromatics fraction is more viscous compared to saturates with a glass transition temperature around –20 ^{o}C. Aromatics are one the most abundant fraction in bitumen (30–45%) and have a significant impact on the the viscosity and flow characteristics of bitumen;
- Resins are more polar molecules than aromatics. They stabilise asphaltenes and form a bridge between saturates and asphaltenes and contribute to ductility and adhesive properties of bitumen. The content of resins in bitumen is usually similar to aromatics (30–45%). Resins form a dark solid at room temperature. Their composition is similar to asphaltenes, but with lower molecular weight;
- Asphaltenes are the heaviest in terms of molecular weight and also most polar fraction, responsible for the stiffness and viscosity of bitumen. They exist as micellar structures stabilised by resins. The general content of asphaltenes in bitumen is approximately 5–20%. They are in the form a dark powder at room temperature.

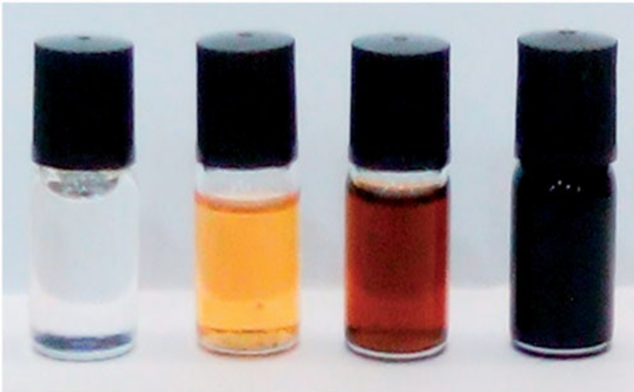

Figure 4. SARA fractions from left to right – saturates, aromatics, resins and asphaltenes (Gharbi et al., 2018).

These fractions form a colloidal structure of bitumen (Branthaver et al., 1994; Lesueur, 2009; Petersen et al., 1993). The description of bitumen as a colloidal structure was introduced already in 1914 by Rosinger (1914). According to colloidal model, bitumen is characterised as a dispersion of asphaltene micelles (solid particles) within an oily phase (fluid), facilitated by the presence of peptising agents (Hunter et al., 2015; Lesueur, 2009; Petersen et al., 1993; Robertson et al., 2001). The asphaltenes are surrounded with the high polarity aromatic component of the maltenes (resins), which serve as peptizing agents for the asphaltenes. This entire system is immersed in the oily fraction, which acts as a flocculating agent for the asphaltenes (Figure 5).

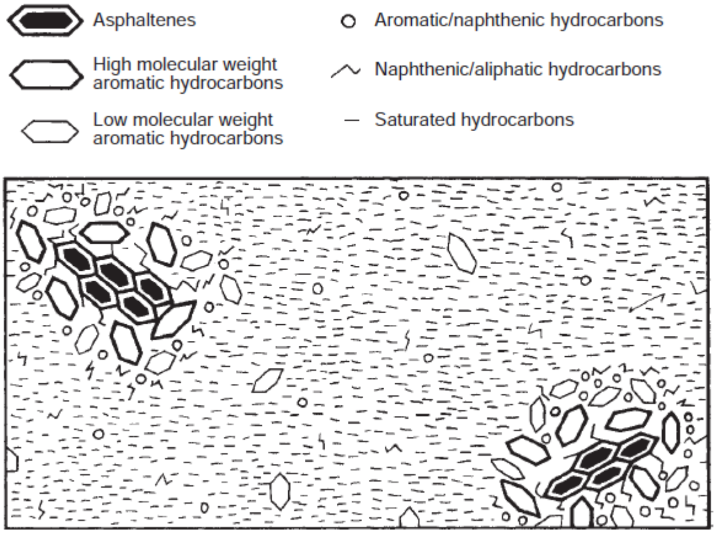

Figure 5. The colloidal model of bitumen and SARA fractions. The asphaltenes are surrounded by higher-polarity resins and lower-polarity aromatic compounds. These are, in turn, encased by a fraction of saturated compounds (Hunter et al., 2015).

Another dimension of complexity in determining bitumen suitability and durability is caused by the fact that bitumen is susceptible to ageing which causes different chemical changes to occur in the bitumen which causes bitumen properties to change (Branthaver et al., 1994; Fernandez-Gomez et al., 2014; Liang et al., 2019; Petersen, 2009; Petersen et al., 1993). It is known that ageing can vary significantly between bitumen from different refineries and crude oil sources (Behrenbruch & Dedigama, 2007; Petersen, 2009). The main bitumen ageing processes are (Hunter et al., 2015; Lu & Isacsson, 2002; Petersen, 2009; Prosperi & Bocci, 2021):

- Oxidation caused by oxygen in air reacting with the hydrocarbons in bitumen, forming oxygen-containing compounds like ketones, carboxylic acids, and sulfoxides. As a result, this increases bitumen stiffness and reduces its ductility, leading to brittleness;
- Volatilisation caused by evaporation of light and volatile compounds. This is mostly caused by high temperatures experienced during the asphalt mixture production process. The loss of lighter oily compounds increases bitumen stiffness and reduces flexibility;
- Physical hardening or steric hardening caused by molecular rearrangements and wax crystallisation at low temperatures. The process can usually be reversed by heating;
- Polymerisation caused by aggregation of hydrocarbons, leading to the formation of larger, more complex and polar molecules, e.g. asphaltenes.
- Ultraviolet (UV) ageing caused by solar radiation which breaks down the molecular structures and accelerates oxidation and polymerisation, further contributing to the ageing.

These ageing processes (excluding physical and steric hardening) cause shift in SARA fractions (Hu et al., 2023; Hunter et al., 2015; Lu & Isacsson, 2002; Petersen, 2009). During the ageing process of bitumen, the saturates fraction remains relatively unchanged (Fernandez-Gomez et al., 2014; Petersen, 2009). The content of saturates changes mainly during asphalt mixture production, transportation, and installation when loss of volatile component occurs. In service, the content of saturates remains nearly constant. A shift takes place in the SARA fractions related with aromatics and resins – aromatics and resins become more polar during ageing. This leads to aggregation with other polar compounds and as a result, the polarity and molecular weight of the aromatics and resins increase. This leads aromatics to become part of resins fraction and resins to become part of asphaltenes fractions, i.e. the content of asphaltenes is increasing. As a result, bitumen becomes more controlled by components with higher polarity and molecular weight which increases the stiffness and brittleness the of bitumen.

Bitumen ageing is generally divided into two separate stages – short-term ageing and long-term ageing (Asphalt Institute, 1996, 2011; Hu et al., 2023; Kennedy et al., 1994). Short-term ageing describes the ageing processes which occur when bitumen is exposed to elevated temperatures, i.e. during the asphalt mixing, transportation and paving process. Long-term aging occurs throughout the service life of the road pavement. Long-term ageing takes place at lower temperatures, and therefore the ageing processes are much slower (Fernandez-Gomez et al., 2014; Petersen, 2009).

1.2 Bitumen specifications and relevant test methods

Since the chemical composition of bitumen and its impact on properties and durability are highly complex, various bitumen specifications and test methods have been developed over time to select bitumen with suitable properties and ensure its durability in service. These test methods and specifications are mostly targeting bitumen properties at following temperatures and purposes (Asphalt Institute, 2011; Dore & Zubeck, 2009; Halstead & Welborn, 1974; Hunter et al., 2015; Kennedy et al., 1994):

- At high temperatures which are associated with asphalt mixture production and paving. This range of temperatures is necessary to make sure that bitumen has required viscosity to produce asphalt mixtures and facilitate paving;
- At elevated pavement temperatures experienced during the summer to ensure adequate resistance of bitumen to plastic deformations, e.g. rutting, shoving, etc.;
- At medium pavement temperatures, often to ensure that bitumen has necessary properties to avoid fatigue cracking;
- At low pavement temperatures, to ensure the bitumen's ability to withstand low temperature cracking which is caused by shrinking inherent to bitumen.

In addition, bitumen specifications include various additional requirements aimed at different properties, such as solubility (purity) or flash point (safety). In general, bitumen test methods and specifications can be classified as empirical or fundamental (Asphalt Institute, 1996, 2011; Kennedy et al., 1994). Empirical methods determine bitumen properties based on experience and are not based on stress and strain. Empirical test results are affected by sample geometry and test setup. Fundamental properties are related with stress and strain and bitumen's physical and rheological properties, such as viscosity, complex modulus and stiffness.

1.2.1 European bitumen specifications

In Europe, paving bitumen specifications and requirements are based on penetration grades (CEN, 2009, 2010, 2014, 2015d). Penetration grades refer to a range of acceptable penetration values which is determined with a penetration test, e.g. penetration grade 70/100 refers that bitumen must have a penetration value between 7–10 tenth of a millimetre. Penetration grade is supplemented with additional requirements which are aimed to provide overview about bitumen properties at elevated pavement temperatures. For example, the resistance to rutting is mostly assessed based on softening point test, although dynamic viscosity is also used, if required by regional regulations. In addition, specific tests related with bitumen purity (solubility in toluene) and safety (flash point) are also part of the European bitumen specifications. Additional properties related with workability (mixing and paving), such as kinematic viscosity, and low temperature performance based on Fraass breaking point can be specified, if required by regional regulations. For example, low temperature properties are not critical in all parts of Europe and therefore requirements for Fraass breaking point are not specified in all European countries.

Penetration grading is built around the needle penetration test which originates from 1888 (Halstead & Welborn, 1974). Penetration test methodology was further refined by Dow in 1903 by introducing the testing setup which is mostly adopted currently in all over the world (Halstead & Welborn, 1974). Penetration test methodology adopted in European bitumen specifications is described in EN 1426 (CEN, 2015a). Penetration test is an empirical method which measures the penetration depth of a standardised needle (with a specified shape and weight) into the bitumen sample after 5 seconds at temperature of 25 ºC. The result (penetration depth) is expressed in decimillimetres and higher penetration depth corresponds to softer consistency, i.e. lower viscosity. The most common bitumen penetration grade used in asphalt mixtures in Estonia is 70/100. Penetration grade 160/220 is often used for base course stabilization and surface dressing.

Softening point test is the main method adopted in Europe to assess the properties of bitumen at high temperatures (CEN, 2009, 2010, 2014, 2015d). The test method was adopted as ASTM standard in 1916 (Abson, 1959; Grotlisch & Burstein, 1945). Softening point testing procedure and methodology adopted in European bitumen specifications is described in EN 1427 (CEN, 2015b). The test method is an empirical method to determine the temperature when two steel balls with standardised properties (size and weight) placed on top of bitumen samples cause the samples to deform by 25 mm due to gravity and reduced viscosity of the bitumen. Test is conducted in liquid medium, usually in water. The liquid is heated uniformly at a rate of 5 ºC per minute and softening point is determined by the average temperature at which the two samples deformed by 25 mm.

Fraass breaking point test method is the only optional test method in the current European bitumen specifications to assess low temperature properties of bitumen (CEN, 2009, 2010, 2014, 2015d). The method was first introduced in 1937 by Fraass
& Vaitkus, 2017). During the test, a thin metal plate is coated with a thin layer bitumen and placed in the liquid cooling medium. The sample is cooled at a constant rate of 1 ºC per minute, and for every degree dropped, the plate is bent. After bending, the sample is visually inspected to determine whether a crack has appeared in the bitumen layer. If a crack is observed, the temperature at which it occurred is registered. This temperature is called the Fraass breaking point (CEN, 2015c).

Bitumen specifications and requirements are mostly applicable to unaged bitumen. These requirements are meant to set a limit to short-term ageing susceptibility of the bitumen. Long-term ageing susceptibility is not considered in the European bitumen specifications.

European bitumen specifications do not include requirements to measure long-term durability of bitumen and only certain requirements apply to assess the durability against short-term ageing (CEN, 2009, 2010, 2014, 2015d). The resistance to short-term ageing is assessed based on the Rolling Thin Film Oven Test (RTFOT) method (CEN, 2024). The method was first introduced in 1963 and was standardised as ASTM standard in 1970 (Airey, 2003; Farrar et al., 2012; Hveem et al., 1963). RTFOT is not a test in the traditional sense but rather a laboratory ageing protocol that provides bitumen samples for further testing. RTFOT simulates the ageing which takes place at high temperatures related with asphalt production, transportation, and installation and experience has shown that RTFOT correlates reasonably well with the actual ageing occurring during the asphalt production, transportation and paving (Airey, 2003; Farrar et al., 2012).

During the test, bitumen samples are placed into eight glass containers, which are horizontally attached to a rotating frame inside the RTFOT oven. During the test, the frame rotates, causing the bitumen samples to flow continuously. With each full rotation, hot air is blown into the glass containers to induce oxidation. The test is conducted at 163 ºC for a duration of 75 minutes (CEN, 2024).

In the European bitumen specifications, the following three properties are assessed after RTFOT ageing of the bitumen (CEN, 2009, 2010, 2014, 2015d):

- Change of mass, which provides an overview of how much the sample has lost or gained in weight after short-term ageing;
- Retained penetration, which provides an overview of how much the bitumen's stiffness has increased during short-term ageing;
- Change in softening point. Similar to retained penetration, this indicates how much the stiffness of the bitumen has increased during short-term ageing.

1.2.2 Superpave Performance Grading

In the 1980s, a comprehensive asphalt research programme known as the Strategic Highway Research Program (SHRP), was launched in the United States (Asphalt Institute, 1996; Kennedy et al., 1994). The research focused on redefining the principles of asphalt mix design and the selection criteria for bitumen (Kennedy et al., 1994). The main aim was to improve the performance and durability of asphalt pavements against rutting, fatigue cracking and low temperature cracking (Asphalt Institute, 1996, 2011; Kennedy et al., 1994). The research addressed concerns related with previously widely used asphalt mix design principles (e.g., Marshall and Hveem mix design methods) and empirical bitumen selection criteria (i.e., penetration or viscosity-based grading specifications). Road agencies and the asphalt industry in the United States indicated that these methods were outdated and would not ensure road pavements with the required durability (Kennedy et al., 1994; TRB, 1984).

Besides revising asphalt mixture design process, the SHRP project led to a completely new approach to determine and select suitable bitumen grade. This is known as Superpave (Superior Performing Asphalt Pavements) Performance Grading (PG grading) which is currently adopted as AASHTO standard M320 (AASHTO, 2023). PG grading is a framework which categorises bitumen based on their performance characteristics under specific environmental and loading conditions (Asphalt Institute, 1996, 2011; Harrigan et al., 1994; Kennedy et al., 1994).

According to Superpave PG grading, bitumen is classified using the PG designation, which includes two numbers indicating the temperature range for optimal performance (Asphalt Institute, 1996, 2011; Kennedy et al., 1994). The PG high temperature grade (PG HT) denotes resistance to rutting up to a certain pavement temperature (e.g., +58 °C), while the low-temperature PG grade (PG LT) indicates resistance to low temperature cracking down to a certain minimum pavement temperature (e.g., –28 °C). For example, a bitumen with PG 58–28 grade is suitable to be used as a binder in road pavements with a maximum and minimum in-service temperature of +58 °C and –28 °C, respectively. Grades are given with 6 °C increments (AASHTO, 2023; Asphalt Institute, 2011). Lowest and highest PG HT grades are 46 and 82, respectively. Lowest and highest PG LT grades are –40 to –10 °C, respectively.

This approach ensures that bitumen is selected based on climatic conditions and expected traffic loads rather than empirical properties such as penetration (Asphalt Institute, 1996, 2011). In addition, Superpave PG grading allows users to adopt different levels of reliability to ensure that these temperatures are not exceeded by certain selected probabilities (Asphalt Institute, 1996, 2011). This allows bitumen requirements to be reduced or increased by considering the priority of the road. For example, main roads could require higher reliability than secondary roads.

Superpave specifications adopted new testing methods such as Dynamic Shear Rheometer (DSR) and Bending Beam Rheometer (BBR) to measure the fundamental rheological properties of bitumen (Asphalt Institute, 1996, 2011; Harrigan et al., 1994;

Kennedy et al., 1994). In addition to short-term ageing protocol, a long-term ageing protocol was included in the specification to assess bitumen aging during the pavement service life (Asphalt Institute, 1996, 2011; Harrigan et al., 1994; Kennedy et al., 1994).

1.2.2.1 Resistance to rutting and fatigue cracking

Superpave specifications adopted DSR to assess the bitumen's resistance to deformation at high pavement temperatures by measuring the viscoelastic properties of bitumen, specifically the resistance to deformation under shear stress (Harrigan et al., 1994; Kennedy et al., 1994). It evaluates the bitumen's ability to withstand rutting by determining parameters such as the complex shear modulus (G^*) and phase angle (δ) (Asphalt Institute, 1996, 2011). These parameters characterise bitumen's viscous and elastic properties at selected temperatures. The resistance to rutting is determined based on two criteria (AASHTO, 2023; Asphalt Institute, 2011):

- $G^*/sin\delta$ 1.00 kPa for unaged bitumen sample;
- $G^*/sin\delta$ 2.20 kPa for short-term aged (RFTOT aged) bitumen sample.

For the unaged bitumen $G^*/sin\delta$ must be at least 1,00 kPa at the pavement maximum temperature (PG HT temperature), e.g. at 52, 58 or 64 °C. For the short-term aged bitumen, the $G^*/sin\delta$ must be at least 2.20 kPa at the specified PG HT temperature. The lowest temperature which meets these criteria is the limiting PG HT grade for the tested bitumen (AASHTO, 2023; Asphalt Institute, 2011). Testing unaged bitumen samples ensures that bitumen has adequate stiffness and performance characteristics in its original state, providing a baseline for quality control and material specification. Short-term aged bitumen testing ensures that the bitumen retains sufficient stiffness and performance characteristics after exposure to the high temperatures and oxidative conditions encountered during production and construction. This reflects the bitumen's ability to resist rutting during the early life of the pavement (Kennedy et al., 1994).

PG high temperature grade is suitable for regular highway loading conditions. However, to consider high traffic volumes and slow-moving traffic, the initial specification also recommended to increase PG HT grade by one (6 °C) or two grades (12 °C) depending on the traffic conditions of the road (Asphalt Institute, 1996, 2011). This approach is known as grade pumping. Grade pumping is now mostly replaced by Multiple Stress Creep and Recovery (MSCR) test method. MSCR is using the same DSR test setup as PG high temperature grading and is is part of the AASHTO M332 specification (AASHTO, 2021). As a result, PG HT grades are supplemented with an additional MSCR grades which are denoted with letters S (standard traffic), H (heavy traffic), V (very heavy traffic) or E (extremely heavy traffic). MSCR grade is selected based on traffic conditions. For example, PG HT grade 58V would be suitable for roads where pavement temperature would not exceed +58 °C and very heavy traffic conditions are expected. MSCR has shown correlate well with pavement rutting (D'Angelo, 2009; Walubita et al., 2022; J. Zhang et al., 2015).

DSR is also used to assess the bitumen's resistance to fatigue cracking based on Superpave fatigue criteria ($G^* \times sin\delta$ 5 MPa) (Kennedy et al., 1994). The test is done after short- and long-term ageing of the bitumen. The test temperature is calculated based on PG HT and LT temperatures based on Equation 1. This temperature is known as PG intermediate temperature grade (PG IT grade).

$$PG\ IT = \frac{PG\ HT - PG\ LT}{2} + 4 \qquad \textit{Equation 1}$$

where
PG IT – Superpave intermediate grade;
PG HT – Superpave high temperature grade;
PG LT – Superpave low temperature grade.

Superpave fatigue criteria after short- and long-term ageing was adopted to ensure that bitumen can withstand repeated loading without cracking over the pavement's design life and to minimise long-term maintenance costs associated with fatigue cracking (Asphalt Institute, 1996, 2011; Kennedy et al., 1994). Superpave fatigue criteria was derived based on data from Zaca-Wigmore road test sections built in California, USA in the 1950s (Gibson et al., 2012). The initial limiting fatigue criteria value was proposed to be 3 MPa but was increased to 5 MPa by the researchers (Gibson et al., 2012). However, Superpave fatigue criteria has been criticized for having low correlation with the actual pavement fatigue cracking (Anderson et al., 2001; Gibson et al., 2012). Since then, different new fundamental test methods have been proposed to improve this correlation. An improved correlation between bitumen tested properties and fatigue cracking detected on pavements has been observed with Double-Edge-Notched Tension (DENT) test method (Gibson et al., 2012; Hesp, 2004; Hesp et al., 2009). DENT test has been officially adopted in Ontario, Canada as a part of bitumen specifications (MTO, 2020).

1.2.2.2 Resistance to low temperature cracking

Superpave bitumen specification's approach to address bitumen properties at high and intermediate temperatures were described in the previous sections. However, bitumen properties at low temperatures are often critical in regions with cold climate, such as the northern states of the USA, Canada, and Northern and Northeastern Europe, including Estonia (Dore & Zubeck, 2009; Harvey et al., 2009; Hesp, 2003; Hesp et al., 2009; Saarela, 1992; Yee et al., 2006).

The Superpave specifications adopted two test methods to assess bitumen low temperature properties – Bending Beam Rheometer (BBR) and Direct Tension Test (DTT) (Asphalt Institute, 1996, 2011; Harrigan et al., 1994; Kennedy et al., 1994). The latter was not widely adopted and the BBR test is considered to be the main method to assess the low temperature properties of bitumen. Testing is done after short- and long-term ageing of the samples. BBR test measures the ability of bitumen to resist thermal cracking, which occurs when pavements contract due to cold temperatures. The test measures the stiffness (S-value) and the rate of stress relaxation (m-value) of a bitumen at in-service pavement low temperature (PG LT). The testing is usually conducted at 10 ^{o}C higher temperature compared to PG LT grade, taking advantage of time-temperature superposition principle to shorten the time needed for testing (Asphalt Institute, 1996, 2011; Harrigan et al., 1994).

The BBR testing is conducted at different temperatures to find the temperatures at which the bitumen meets two criteria:

- Stiffness (S-value) = 300 MPa;
- Stress relaxation (m-value) = 0.3.

Both of these properties are determined at 60 seconds after loading the specimen. The lowest temperature at which any of these criteria is met determines the PG low-temperature (LT) grade of the bitumen. The BBR test method refers to a 1-hour specimen conditioning time at testing temperature prior to testing. The original BBR test method indicates that strict adherence to the conditioning time is necessary to avoid variability in results, which can be caused by physical hardening (Harrigan et al., 1994). The initial Superpave specification recommended to report the degree of physical hardening after 24h conditioning at testing temperature (Kennedy et al., 1994).

Physical hardening is a phenomenon which can cause bitumen to become stiffer under isothermal conditioning at low temperatures (Hesp, 2003, 2004). Research conducted in Queen's University in Kingston, Canada have shown that BBR results correlate much better with low temperature cracking of asphalt pavements if PG LT grade is measured after isothermal conditioning of 72 hours and therefore allowing the physical hardening phenomenon to occur (Hesp, 2003, 2004; Hesp et al., 2009; Yee et al., 2006). The current specifications in Ontario require BBR testing to be conducted after 1 and 72 hours of conditioning (MTO, 2020). The change in PG LT grade is expressed as grade loss.

1.2.2.3 Pressure Aging Vessel

With the introduction of the Superpave PG specification, a test method for simulating the long-term ageing of bitumen was introduced known as pressure ageing vessel (PAV) which simulates 5 to 10 years of ageing on field (Airey, 2003; Kennedy et al., 1994; Petersen et al., 1993). The purpose of PAV is to provide an overview of how bitumen ages in service. Long-term laboratory aging is performed after short-term aging (RTFOT). During the test, bitumen samples are subjected to air pressure (2.1 MPa) and elevated temperatures (typically 90–110 ^{o}C). The test duration is usually 20 hours. In the PG specifications, the samples subjected to PAV are used for evaluating the resistance to fatigue cracking and low temperature cracking.

1.3 Temperature models for pavement temperature and PG grade estimation

Bitumen is temperature susceptible material and the properties of asphalt pavements are related with the pavement temperatures. Pavement temperature is a function of various environmental and material properties, such as weather (temperature, wind speed, etc.), solar adsorption and reflectivity, thermal conductivity, thermal emittance, specific heat, surface convection, etc. (Adwan et al., 2021; Mohseni, 1998; Sun, 2016). Historically, various pavement temperature estimation models have been developed and used as an input to back calculate pavement's or it's layers moduli with Falling Weight Deflectometer or to evaluate the pavement temperatures (Adwan et al., 2021; Sun, 2016). This thesis provides an overview of pavement temperature estimation models which serve as input for selecting the Performance Grade for a road or region.

1.3.1 SHRP models

One of the aims of SHRP research was to develop asphalt mixture and bitumen specifications which replicate field conditions as closely as possible. This includes both loads induced by traffic and environmental conditions, specifically pavement temperature. Regarding pavement temperatures, a dedicated model for calculating the maximum pavement temperature was developed. The model was developed based on theoretical energy balance (Equation 2) at the pavement surface (Huber, 1994; Kennedy et al., 1994; Mohseni, 1998; Solaimanian & Kennedy, 1993).

$$Net\ heat\ flow = direct\ solar\ radiation + diffuse\ radiation \mp conduction - black\ body\ radiation \quad \textit{Equation 2}$$

To calculate pavement surface temperature during the hottest 7-day period of the year, following properties and values were used (Huber, 1994):

- Solar absorption (0.90);
- Transmission through air (0.81);
- Atmospheric radiation (0.70);
- Wind speed (4.5 m/s).

Using theoretical analysis and trial and error analysis of five databases of actual pavement and air temperature data, Equation 3 was developed to correlate air and pavement temperatures (Huber, 1994).

$$T_{surf} - T_{air} = -0.00618\varphi^2 + 0.2289\varphi + 24.4 \quad \textit{Equation 3}$$

where
T_{surf} – Pavement surface temperature, ºC;
T_{air} – Air temperature, ºC;
φ – Latitude of the specific road section or region, degrees.

According to Superpave Performance Grading principles, the maximum temperature should correspond to pavement temperature at 20 mm depth from the surface of the layer which can be calculated by using Equation 4 (Huber, 1994; Mohseni, 1998; Sun, 2016). In this thesis, this temperature is denoted as SHRP PG HT.

$$T_{Pav20,max} = \left(T_{Air,max} - 0.00618\varphi^2 + 0.2289\varphi + 42.2\right) \times 0.9545 - 17.78 \quad \textit{Equation 4}$$

where
$T_{Pav20,max}$ – Pavement maximum temperature at 20 mm depth, ºC
$T_{Air,max}$ – 7-day average high air temperature, ºC
φ – Latitude of the specific road section or region, degrees;
h – Depth from surface, mm.

Based on analysis of observed pavement temperatures, the pavement minimum temperature was assumed to be equal to the daily minimum air temperature (Huber, 1994; Kennedy et al., 1994; Mohseni, 1998; Solaimanian & Kennedy, 1993). In this thesis, this temperature is denoted as SHRP PG LT. The assumption was done based on limited amount of data and lead to an overly conservative pavement minimum temperature estimation (Mohseni, 1998).

1.3.2 LTPP-SMP models

The SHRP methodology for determining pavement maximum and minimum temperatures was later revised based on air and pavement temperature data collected from 30 road test sections across North America as part of the Long Term Pavement Performance study's Seasonal Monitoring Program (LTPP-SMP) (Mohseni, 1998). The study made it possible to re-evaluate the SHRP pavement temperature model and assumptions related with pavement maximum and minimum temperatures. LTPP-SMP data indicated that there were significant differences between estimated temperatures according to SHRP methodology and registered pavement temperatures (Mohseni, 1998). This lead to developing LTPP pavement temperature models which were updated based on air and pavement temperature data obtained from test sections between 1993 and 1995.

Based on LTPP-SMP data, several variables were analysed to predict pavement temperature (Mohseni, 1998). The LTPP data indicated that the most important variables are the daily maximum or minimum air temperature, the location of the road (latitude), and the depth from the road surface. Daily air temperatures had the highest correlation with daily maximum and minimum pavement temperatures and the correlation was found to be linear. Latitude and pavement temperature relationship was also found to be rather strong. Data analysis indicated that latitude squared (φ^2) had better fit over $log(\varphi)$ and therefore the former was adopted in the models as an input variable. As expected, the depth from pavement surface (h) was also found to be strongly correlated with pavement temperature. Since the relationship was non-linear, depth squared (h^2) and $log_{10}(h+25)$ were considered. It turned out that $log_{10}(h+25)$ provided better fit over depth squared (h^2). New models for calculating pavement maximum and minimum temperatures were proposed as per Equation 5 and Equation 6 (Mohseni, 1998). In this thesis, these temperatures are referred to as LTPP PG HT and LTPP PG LT, respectively.

$$T_{Pav,max} = 54.32 + 0.78 \times T_{Air,max} - 0.0025 \times \varphi^2 - 15.14 \times log_{10}(h+25) + z\sqrt{9 + 0.61 \times \sigma^2_{Air,max}} \qquad \textit{Equation 5}$$

where

$T_{Pav,max}$ – Maximum pavement temperature (LTPP PG HT) at depth h, °C;

$T_{Air,max}$ – Daily maximum air temperature or 7-day average high temperature, °C;

φ – Latitude of the specific road section or region, degrees;

h – Depth from pavement surface, mm;

$_{Air,max}$ – Standard deviation of air temperature or 7-day average high temperature;

z – Variable from standard normal distribution table, z = 2.055 for 98% reliability.

$$T_{Pav,min} = -1.56 + 0.72 \times T_{Air,min} - 0.004 \times \varphi^2 + 6.26 \times log_{10}(h+25) - z\sqrt{4.4 + 0.52 \times \sigma^2_{Air,min}} \qquad \textit{Equation 6}$$

where

$T_{Pav,min}$ – Minimum pavement temperature (LTPP PG LT) at depth h, °C;

$T_{Air,min}$ – Daily minimum air temperature, °C;

φ – Latitude of the specific road section or region, degrees;

h – Depth from pavement surface, mm;

$_{Air,min}$ – Standard deviation of daily minimum air temperature;

z – Variable from standard normal distribution table, z = 2.055 for 98% reliability.

1.3.3 C-SHRP model

In parallel and in collaboration with the SHRP, similar research was carried out in Canada, known as Canadian Strategic Highway Research Project (C-SHRP). One of the outcomes of the C-SHRP was the realisation that the SHRP PG LT assumption – that the lowest pavement temperature equals the lowest air temperature – leads to too conservative pavement temperature estimation (Dawley & Pulles, 1996; Mohseni, 1998). Based on road weather station data, it was noted that pavement temperatures in winter are always somewhat higher than the daily minimum air temperatures. Based on air and pavement temperature data analysis conducted in Canada, an Equation 7 was proposed which was aimed to provide more realistic pavement temperatures for Canadian conditions (Dawley & Pulles, 1996; Mohseni, 1998). In this thesis, this is denoted as C-SHRP PG LT temperature.

$$T_{Pav,min} = 0.859 \times T_{Air,min} + \left(0.002 - 0.0007 \times T_{Air,min}\right) \times h + 1.7 \qquad \textit{Equation 7}$$

where
$T_{Pav,min}$ – Minimum pavement temperature (C-SHRP PG LT) at depth h, °C
$T_{Air,min}$ – Daily minimum air temperature, °C
h – Depth from pavement surface, mm.

Unlike the LTPP pavement minimum temperature equation (Equation 6), the C-SHRP PG LT model does not consider the location of road or region (latitude) as an input.

1.3.4 Norwegian model

In Norway, two comprehensive asphalt related studies were initiated: New Asphalt Technology (*Ny asfaltteknologi)* from 1994 to 1998 and PROKAS (*Proporsjonering og Kontroll av Asfalt*) from 1998 to 2004 (Andersen, 1998; Lerfald et al., 2004). One of the main objectives of these studies was to assist the Norwegian asphalt industry in transitioning to performance-based and fundamental requirements for asphalt mix design. Similarly to the SHRP research, the justification for these studies highlighted that traditional asphalt mix design principles, including bitumen grade selection, no longer ensure the desired durability in modern applications. The research was largely aligned with the SHRP and C-SHRP studies conducted in North America, placing significant emphasis on selecting bitumen grades (properties) based on the Superpave Performance Grading (PG) principles (Lerfald et al., 2004).

During the research, it became evident that by adopting the SHRP maximum pavement temperature prediction models to Norwegian conditions, the calculated pavement temperatures would be significantly lower compared to the actual temperatures experienced in Norway (Andersen, 1998; Jóhannesson, 2005). A separate PG HT calculation model (Equation 8) was developed which is more aligned with the environmental conditions experienced in Norway (Lerfald et al., 2004). In this thesis, this is denoted as NOR PG HT temperature or grade. Similarly to the SHRP pavement high design temperature calculation model, the inputs were 7-day average high air temperature and the location (latitude) of the road or region.

$$T_{Pav\ 20mm} = \left(T_{Air,max} - 0.0055\varphi^2 + 0.15\varphi + 36\right) \times 0.9545 - 0.8 \qquad \textit{Equation 8}$$

where
$T_{Pav\ 20mm}$ – Maximum pavement temperature at 20 mm below the surface, °C
$T_{Air,max}$– 7-day average high air temperature, °C
φ – Latitude of the specific road section or region, degrees

When comparing the pavement temperatures calculated according to SHRP, LTPP, and Norwegian PG HT models, it becomes clear that using the same input data, i.e. daily maximum air temperature and latitude, the Norwegian equation estimates the pavement temperatures to be significantly higher than the SHRP and LTPP PG HT equations (Jóhannesson, 2005; Kontson et al., 2023).

According to Jóhannesson (2005), the Norwegian researchers pointed out that the air-pavement temperature relationship described with Equation 8 was not based on strong foundation. Additionally, the equation's broader applicability is limited by the fact that it lacks a separate variable to calculate pavement temperature at different depths from surface. As a result, it is not possible to estimate pavement temperatures at depths other than 20 mm from the pavement surface. Regarding the minimum pavement temperatures for PG LT grade estimation, the C-SHRP model (Equation 7) was adopted in Norway (Lerfald et al., 2004; Vegvesen, 2014).

1.4 The impact on climate change to pavement temperatures and bitumen grade selection

Different studies and general consensus among climate scientists indicate that human activities, including the burning of fossil fuels (such as coal, oil, and natural gas), deforestation, and industrial processes, have substantially raised the levels of greenhouse gases in the atmosphere, such as carbon dioxide, methane, and nitrous oxide. This rise amplifies the natural greenhouse effect, driving global warming and contributing to climate change (IPCC, 2023b; Myers et al., 2015; Ripple et al., 2024).

Rising global temperatures contribute to extreme weather events, such as heatwaves and prolong seasonal periods which are causing majority of the pavement damages, leading to increasing damages to infrastructure like roads, bridges, and buildings (Delgadillo et al., 2020; FHWA, 2015; IPCC, 2022; Knott et al., 2019; Swarna & Hossain, 2020; Vincent et al., 2018; Viola & Celauro, 2015; Q. Zhang et al., 2024).

Bitumen bound pavement layers, including asphalt pavements are particularly susceptible to temperature, with bitumen playing a crucial role in determining pavement overall behaviour. Elevated air and pavement temperatures experienced during the summer can lead pavement deformations such as longitudinal rutting, particularly under heavy, slow-moving traffic (FHWA, 2015; Fletcher et al., 2016; Mills et al., 2007; Sousa et al., 1991; Q. Zhang et al., 2024). Pavement deterioration is expected to increase in the future due to extended fall season and shorter winters, which have implications to rutting and bearing capacity of the roads (Knott et al., 2019; Mills et al., 2007).

In the United States alone, climate change is projected to increase pavement costs by an estimated \$13.6 billion to \$35.8 billion, depending on various climate scenarios and based on the impact of temperature increase on bitumen grade selection (Underwood et al., 2017). In Italy, pavement temperatures are expected to increase which requires road agencies to investigate options to use polymer modified bitumen having double the price of regular bitumen (Viola & Celauro, 2015). Research conducted by Delgadillo et al., (2020) studied the impact of climate change to Superpave PG grades in Chile based on two Representative Concentration Pathway (RCP) scenarios – RCP-2.6 (most optimistic) and RCP-8.5 (most pessimistic). Both observed scenarios would lead to significant changes to bitumen grade selection in Chile with up to 40% of the observed stations being impacted.

Similar patterns have been predicted in Canada, leading to recommendations to take the climate change into account when selecting bitumen grades (City of Toronto, 2019; Swarna & Hossain, 2020). Mills et al., (2007) analysed the impact of climate change to pavements in Southern Canada. They concluded that over the next 50 years, issues related with pavement low temperature cracking is expected to decrease. They also noted that pavement structures will freeze later and thaw earlier, leading to reduced bearing capacity. This will increase fatigue cracking and rutting of the pavements leading to reduced service life of the pavements. A study conducted by Fletcher et al., (2016) concluded that pavement temperatures are expected to increase in all observed climate change scenarios in Southern Canada. This would lead to changes in bitumen Performance Grades in some of the regions. Under moderate and strong warming scenarios 7 and 9 out of 17 major cities would exhibit an increase in PG grades. The impact of climate change to pavement performance in Canada's Newfoundland Island was analysed by Swarna et al., (2023b). They concluded pavement rutting and fatigue cracking is expected to increase and low temperature cracking is expected to decrease in all observed locations. This could lead to reduced service life of the pavements in the province in Newfoundland (Swarna et al., 2023b).

1.4.1 Climate change modelling and climate change projections

Climate change models are tools used by scientists to analyse historical climate patterns and forecast future climate trends. These models simulate the physical, chemical, and biological processes of the atmosphere, land, and oceans (Eyring et al., 2016; IPCC, 2022, 2023a; Meehl et al., 1997; K. Weigel et al., 2021). The models are improved over time as scientists over the world integrate higher spatial resolutions, advanced physical processes, and biogeochemical cycles (Eyring et al., 2016). These enhancements are generally timed to align with the schedule of the Intergovernmental Panel on Climate Change (IPCC) assessment reports, with model outputs released in preparation for each report (IPCC, 2023b). The latest IPCC Sixth Assessment Report (IPCC AR6) uses Coupled Model Intercomparison Project Phase 6 (CMIP6) to predict climate change based on different scenarios (IPCC, 2022, 2023b, 2023a). The Coupled Model Intercomparison Project (CMIP) is an initiative by the World Climate Research Programme (WCRP) that offers climate data to analyse past, present, and future climate change scenarios (Eyring et al., 2016).

CMIP6 provides climate simulations that help researchers to provide climate change assessments and projections. These projections are based on scenarios previously known as Representative Concentration Pathways (RCPs). The latest IPCC report AR6 utilizes Shared Socioeconomic Pathway (SSP) scenarios, which have replaced RCPs (IPCC, 2022, 2023b). SSPs offer a more detailed framework by incorporating socioeconomic factors such as population growth, economic development, and technological advancements. This approach improves the understanding of how these factors affect greenhouse gas emissions and climate change, providing a broader perspective for climate modelling and policymaking (D. van Vuuren et al., 2021; D. P. van Vuuren & Carter, 2014).

The main SSP scenarios that are typically referred to are following (IPCC, 2023b; D. van Vuuren et al., 2021; D. P. van Vuuren & Carter, 2014):

- SSP1-1.9 (very low) represents a scenario where the world achieves net-zero global CO_2 emissions around 2050. This scenario aligns with the Paris Agreement's 1,5 °C warming limit, with temperatures peaking at 1,5 °C before decreasing to 1,4 °C by the end of the century. This is the most optimistic scenario where society globally transitions to sustainable practices, focusing on well-being rather than economic growth. In this scenario, it is assumed that investments in education and health increase, and inequality decreases.

Although extreme weather events become more frequent, significant impacts of climate change are mitigated.

- SSP1-2.6 (low) represents a scenario where the world achieves significant technological advancements, transitions to renewable energy, and addresses social and environmental issues, resulting in relatively low levels of climate change (with global warming likely staying below 2 ºC by 2100). The '2.6' refers to the level of radiative forcing (measured in watts per square meter) by 2100. This is considered to be an optimistic scenario.
- SSP2-4.5 is a "middle-of-the-road" scenario for socioeconomic development and climate change that projects a 2–3 ºC temperature increase by 2100. This scenario assumes higher usage of oil and coal compared to SSP1 scenarios.
- SSP3-7.0 (high) is a high-emissions, low-development pathway with slow economic growth, high inequality, and limited technological progress. The scenario represents a fragmented world with poor international cooperation, leading to high emissions and significant climate change impacts. By 2100, this scenario could raise global temperatures by 3.5–4 ºC.
- The SSP5-8.5 scenario represents a future characterised by rapid economic growth and heavy reliance on fossil fuels. This pathway results in the highest greenhouse gas emissions and the most severe climate impacts among the SSPs. Under this scenario, significant technological advancements and economic development occur, but they are primarily driven by high energy consumption and fossil fuel dependency. As a result, global temperatures could rise dramatically, potentially exceeding 4 ºC by 2100. The high emissions and limited mitigation efforts make this pathway one of the most challenging for climate adaptation and mitigation, leading to severe environmental and societal consequences. This is considered to be the most pessimistic scenario.

Although it is uncertain which SSP pathway will prevail, the SSP2-4.5 scenario is often regarded as the most probable scenario. This is because it represents a continuation of current global trends, including moderate socioeconomic development, gradual technological progress, and emissions reductions that are not sufficiently aggressive to limit global warming to 2 ºC or below (IPCC, 2023b; Scafetta, 2024).

2 Aims of the study

The primary aim of this study was to examine the current and projected pavement temperatures in Estonia in line with the Superpave Performance Grading principles. The initial intention was to use widely adopted pavement temperature calculation models for calculating pavement temperatures in Estonia. However, data and insights gathered during the study revealed that these models would produce inaccurate results for Estonia. This lead to an additional objective to develop pavement maximum and minimum temperature estimation models which would be more suitable for Estonian conditions. The impact of climate change was analysed based on developed pavement temperature estimation models and three different climate change scenarios.

3 Methodology and data

3.1 Geographical location and climate

This study focuses on Estonia, located in Northeastern Europe and bordered by the Baltic Sea to the west and the Gulf of Finland to the north (Figure 6). Estonia covers a total area of approximately 45 000 square kilometres, with geographical coordinates spanning between 57°30' N to 59°49' North latitude and 21°46' E to 28°13' East longitude. Estonia's location places it within a temperate climate zone, where the weather is influenced by both maritime and continental climate patterns. The average annual air temperature from 1991 to 2020 was 6,4 °C, with recorded extremes of 35,6 °C on 11 1992 ,5 °C on 17 January 1940. The warmest months are typically July and August, while the coldest months are January and February (Estonian Environmental Agency, 2021).

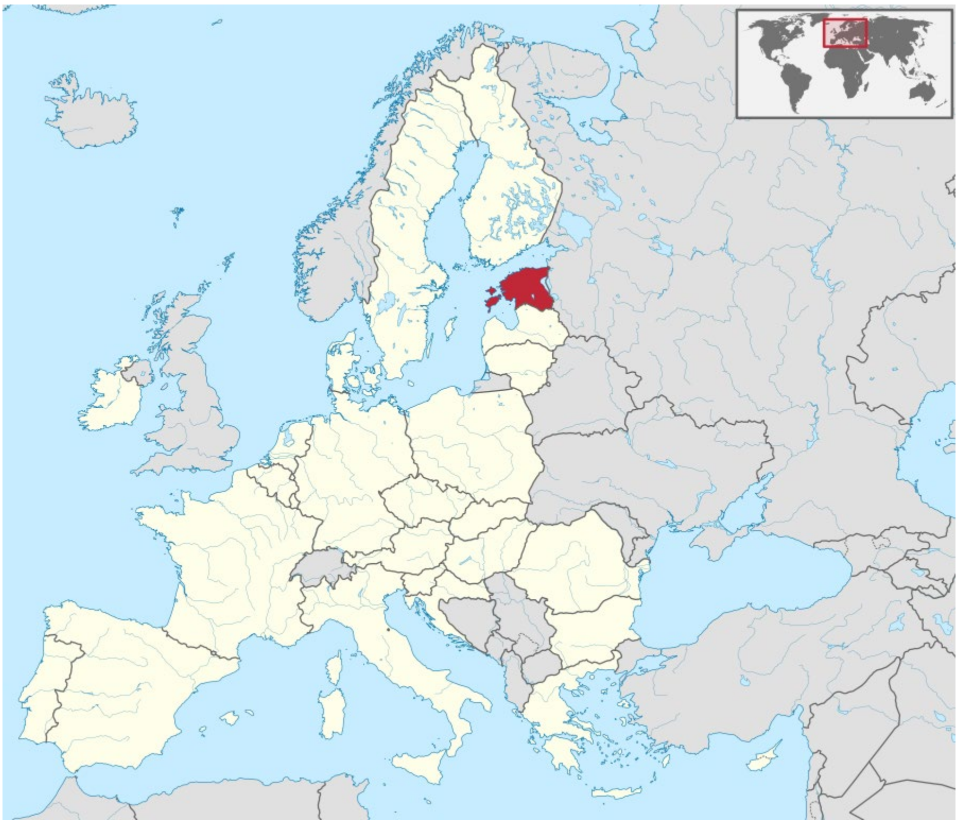

Figure 6. Location of Estonia (red). Light yellow countries are members of European Union (Including Estonia). Grey areas refer to rest of Europe and surrounding region. ("Estonia in European Union" by TUBS CC BY 3.0).

3.2 Temperature data from meteorological stations

Publication I and **Publication III** investigated the pavement temperatures based on historical air temperature data provided by Estonian Environment Agency (EEA). EEA provided access to historically registered daily maximum and minimum air temperatures for 37 different weather stations in Estonia. In **Publication I**, the observation period was selected to include years from 1992 to 2021 (30 years). This period meets the minimum requirement of 20 years for Superpave PG grade determination (Asphalt Institute, 1996, 2011; Kennedy et al., 1994). All meteorological stations with less than 30 years of daily temperature data were excluded from the analysis and the number of suitable stations reduced from 37 to 25.

The location of each included station is indicated in Figure 7. The selected stations are fairly evenly distributed over the territory of Estonia and temperature data density resolution is one station per approximately 1800 km^2. The same meteorological stations, as indicated in Figure 7, were used for pavement temperature analysis in **Publication III**. The observation period in **Publication III** was selected to be narrower and ranged from 2004 to 2024 (21 years).

Publications I and III focused on analysing the 7-day maximum air temperature and daily (1-day) minimum air temperatures of each included meteorological station. **Publication III** also included daily (1-day) maximum air temperatures to calculate daily maximum pavement temperatures. In **publications I and III** the annual mean and standard deviations were used as input to ensure reliability of at least 98%.

The daily maximum and minimum air temperature data from each included station underwent verification to ensure the data was correct and sufficient for pavement PG HT and PG LT grade calculations (**Publication I**) and maximum and minimum pavement temperature analysis (**Publication III**). The critical months for pavement PG HT calculations were identified to be June to September (included), while for PG LT calculations, it was October to March (included). Additionally, the standard deviation of the 7-day average high air temperature and the 1-day maximum and minimum air temperature was included to estimate the pavement temperatures with at least 98% reliability.

In rare cases where daily minimum and maximum temperatures were unavailable for a limited period, such as a week or a month, data from nearby weather stations were analysed to evaluate the impact of the missing data on the calculated pavement temperatures. For example, if a weather station was missing daily minimum air temperature records from December 1 to December 15, but air temperature data from nearby stations indicated that the coldest temperatures of the year did not occur within this period, then the data from the affected station was still included in the analysis.

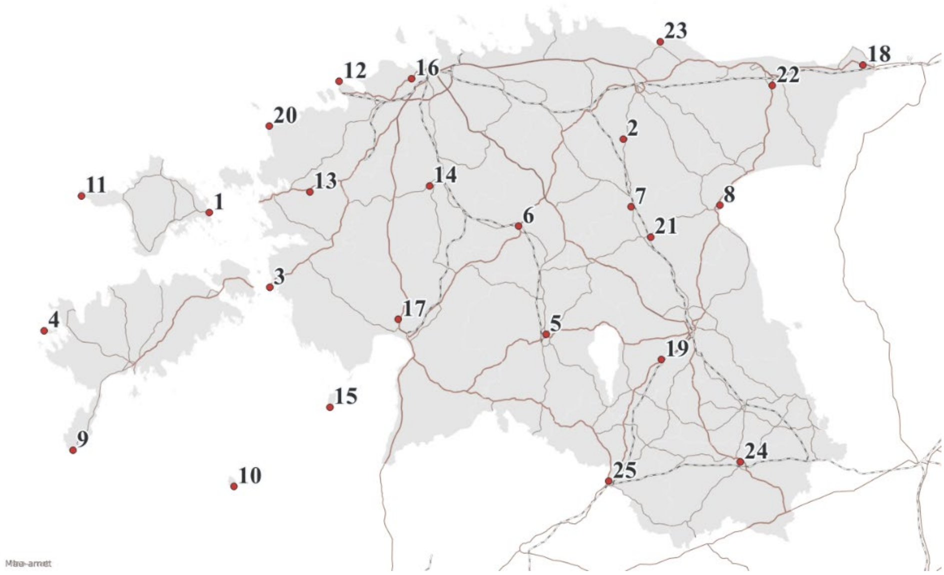

Figure 7. Locations and numbering of the 25 selected weather stations included in the analysis in Publications I and III.

3.3 Air and pavement temperature data from road weather stations

Publications I and II also relied on road pavement temperatures recorded by road weather stations (RWSs) installed in Estonia. The road weather stations referred to in **Publications I and II** are equipped with Vaisala DSR Road & Runway pavement temperature sensors with a temperature measurement range from –40 °C to +80 °C. Vaisala sensors are equipped with two PT-100 elements which register pavement surface temperature and pavement temperature at 60 mm depth from the pavement surface. The air and pavement temperature data are stored with a 10-minute interval to a Structured Query Language (SQL) database managed by the Estonian Transport Administration (previously Estonian Road Administration).

In **Publication I**, the registered daily maximum and minimum road surface and air temperatures ranging from 2020 to 2022 were collected from seven randomly selected road weather stations indicated in Figure 8. The aim was to compare the pavement temperatures calculated with SHRP, LTPP, C-SHRP and Norwegian models with the maximum and minimum pavement temperatures registered by the road weather stations.

Figure 8. Locations and designations of the road weather stations included in Publication I. These stations were used to compare the calculated and measured pavement temperatures.

Publication II focused on establishing the relationships between daily maximum and minimum air and pavement temperatures which were the basis of developed statistical-empirical pavement maximum and minimum temperature prediction models. Additionally, 12 road weather stations were included in the analysis to compare the calculated pavement temperatures with measured temperatures to check the validity of the established models. The location of the road weather stations used for air-temperature relationship analysis and validation in **Publication II** are indicated in Figure 9.

Pavement surface temperatures and the temperatures in the pavement at 60 mm depth from the surface were included in the analysis. Road weather stations located in Aegviidu, Mõisaküla and Käru were used for analysing and establishing the relationship between daily maximum air and pavement temperatures. The road weather stations located in Vodava, Sõmeru and Kauksi were used for analysing and establishing the relationship between daily minimum air and pavement temperatures.

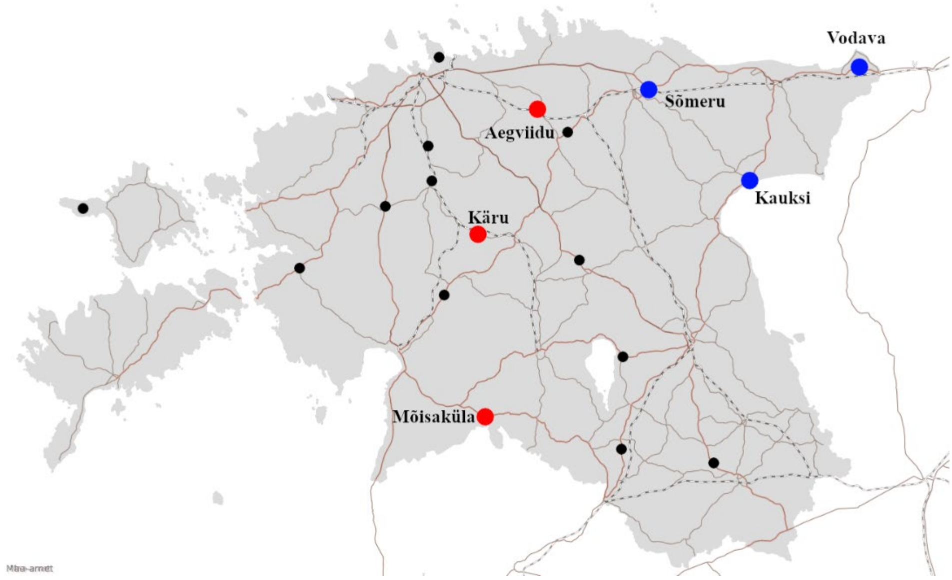

Figure 9. Locations of the road weather stations used for establishing and validating pavement and air temperature models; red dots indicate stations which were used for maximum temperature model; blue dots indicate stations which were used for minimum temperature model; black dots indicate stations which were used for field validation of the derived temperature models.

3.4 CMIP6 climate change model

Publication III analysed the impact of climate change to pavement temperatures in Estonia. For this purpose, the changes in daily maximum and minimum temperatures associated with climate change were used across the different time periods considered. In **Publication III**, the impact of climate change to pavement temperatures was analysed based on Coupled Model Intercomparison Project Phase 6 (CMIP6) model data for Estonia. To incorporate the impacts of climate change in pavement temperature calculations, changes in daily maximum temperatures (TXx) during the June–August (JJA) period and daily minimum temperatures (TNn) during the December–February (DJF) period (measured at 2 meters above ground level) relative to the reference period 2004–2024 were extracted from the CMIP6 model. The CMIP6 model average TXx and TNn changes for the Estonian region were derived using the Copernicus Climate Change Service (C3S) online interface, specifically through the Copernicus Interactive Climate Atlas. Climate change projections were derived for three 20-year periods indicated below:

- 2024–2044 (near term);
- 2045–2064 (medium term);
- 2065–2084 (long term).

The impact of climate change to pavement temperatures were analysed based on following three CMIP6 Socioeconomical Pathways:

- SSP1-2.6
- SSP2-4.5
- SSP3-7.0

4 Results and Discussion

4.1 The impact of selected model to calculated pavement design temperatures in Estonian

Publication I analysed the impact of different pavement temperature calculation models to calculated maximum and minimum pavement temperatures. The maximum pavement temperatures (PG HT) were calculated based on SHRP (Equation 4), LTPP (Equation 5), and Norwegian (Equation 8) pavement temperature models. The minimum pavement temperature (PG LT) analysis incorporated the SHRP assumption, which equated the pavement surface temperature to the daily minimum air temperature, as well as the LTPP (Equation 6) and C-SHRP (Equation 7) models.

Calculations were carried our based on 25 selected weather stations indicated in Figure 7. Estimated pavement maximum and minimum temperatures demonstrated high variability and dependence on the selected models and the calculated results were compared with pavement temperatures recorded by road weather stations to analyse which of the models would match the best with measured maximum and minimum pavement temperatures.

4.1.1 Estonian PG HT and LT design temperatures based on SHRP, LTPP, C-SHRP and Norwegian models

The pavement maximum temperatures (PG HT) at 20 mm depth from the road surface were calculated based air temperature data from 1992 to 2021. The results are indicated in Figure 10. The calculated pavement temperatures show variability between adopted models. The Norwegian PG HT model provides the highest predicted pavement temperatures at 20 mm depth from road surface and estimates the pavement PG HT temperatures to be about 4–6 °C higher than LTPP PG HT model. The LTPP PG HT model estimates the pavement temperatures to be approximately 3–5 °C higher compared to temperatures calculated with the SHRP model.

The pavement minimum temperatures (PG LT) were also calculated based on air temperature data from 1992 to 2021. The results are indicated in Figure 11. When comparing the minimum pavement temperatures calculated using the SHRP, LTPP, and C-SHRP models, it becomes evident that the difference between the SHRP assumptions and the LTPP PG LT calculated temperatures differed from each other from approximately +2 °C to –3 °C. The temperature difference between the LTPP and C-SHRP models is notably larger and leads to minimum pavement temperatures approximately 4 °C to 7 °C higher compared to LTPP model.

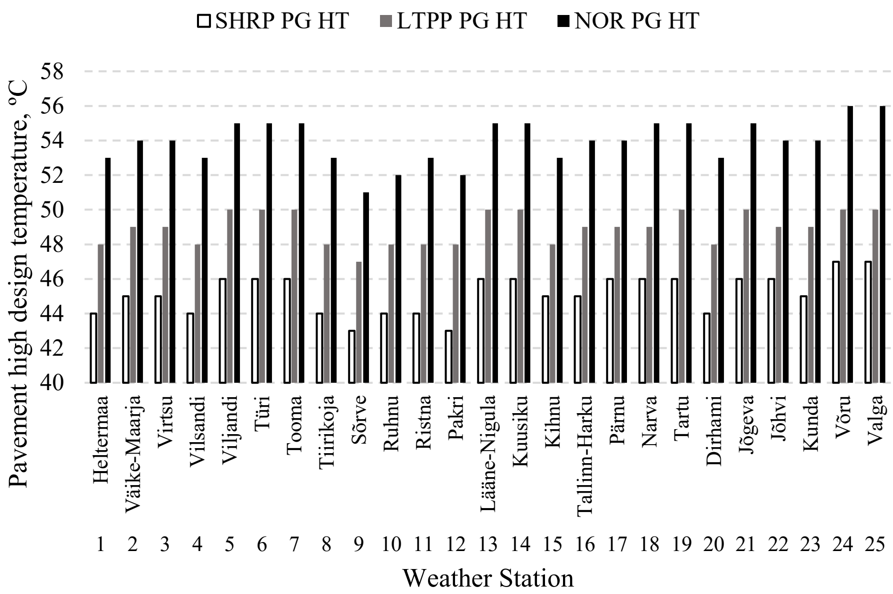

Figure 10. Pavement PG HT design temperatures according to SHRP, LTPP and Norwegian calculation models (with ≥ 98% reliability). Calculated results are based on mean of 7-day average high air temperature.

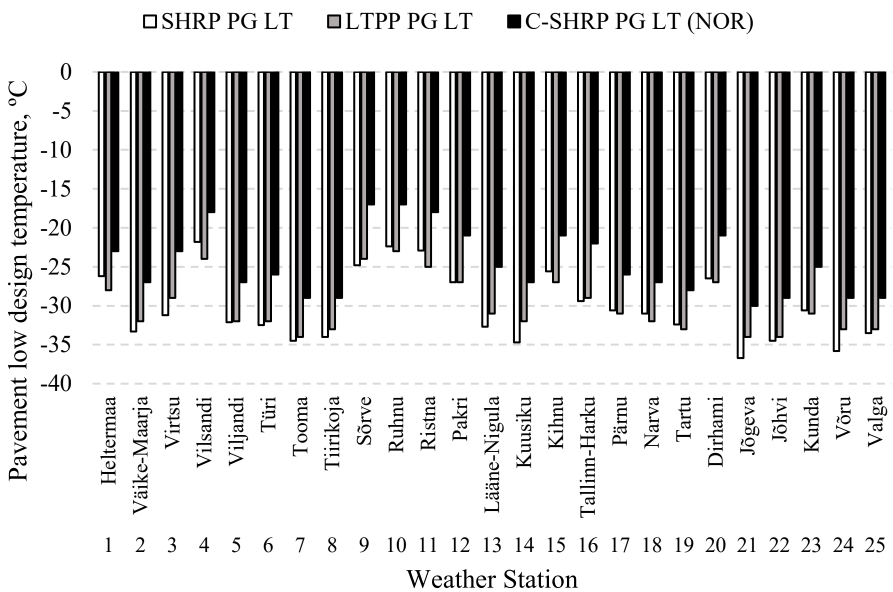

Figure 11. Low pavement low design temperatures according to LTPP and C-SHRP calculation models (with ≥ 98% reliability). SHRP PG LT temperature is assumed to be equal to lowest 1-day air temperature. C-SHRP model is also used in Norway.

4.1.2 Comparison with Road Weather Stations

Comparison between pavement temperature calculation models indicated that different models lead to varying estimated pavement temperatures. This highlighted the importance of validating the models against actual pavement temperatures to avoid using unsuitable models for specific regions. To determine which of the models aligns more closely with the actual pavement temperatures in Estonia, an additional analysis was carried out in **Publication I**. The analysis focused on comparing the calculated temperatures with registered maximum and minimum pavement temperatures from seven randomly selected road weather stations. The results of the analysis are presented in Figure 12 and Figure 13.

Road weather station temperature data describes the highest temperature at the pavement surface. Calculated temperatures describe the pavement temperature at a depth of 20 mm below the surface. The highest pavement surface temperatures recorded by road weather stations were adjusted to represent the temperature at a depth of 20 mm to make results comparable. This was done by assuming the pavement temperature at 20 mm from surface to be 4 ºC lower than the surface temperature. This assumption is based on the temperature-depth relationship used in the LTPP PG HT equation which showed good agreement with the actual registered pavement temperatures with respect to depth. This approach was not necessary for the analysis of the pavement minimum temperatures, as both the calculated and measured temperatures are based on the surface temperatures.

The comparison of calculated PG HT results with pavement temperatures show that the Norwegian model provides the closest approximation. The largest difference occurred at Aegviidu (RWS1), where the calculated and assumed temperatures differ by almost 8 ºC. However, the average difference between registered and estimated temperatures across all observed road weather stations is approximately 2–3 ºC. SHRP and LTPP models would lead to lower calculated pavement temperatures compared to registered pavement temperatures.

When examining the calculated PG LT temperatures with the lowest temperatures recorded by the road weather stations, it becomes evident that the C-SHRP model provides the closest results. Both the SHRP assumption and the LTPP model led to significantly lower pavement surface temperature estimates than the actual measured pavement surface temperatures. The absolute difference between the SHRP assumption and the measured lowest pavement surface temperatures is up to 8 ºC. The LTPP model results in a difference of 4–6 ºC lower compared to the measured lowest pavement surface temperature. In contrast, the C-SHRP model shows a difference of +1 to –2 ºC compared to the measured temperatures.

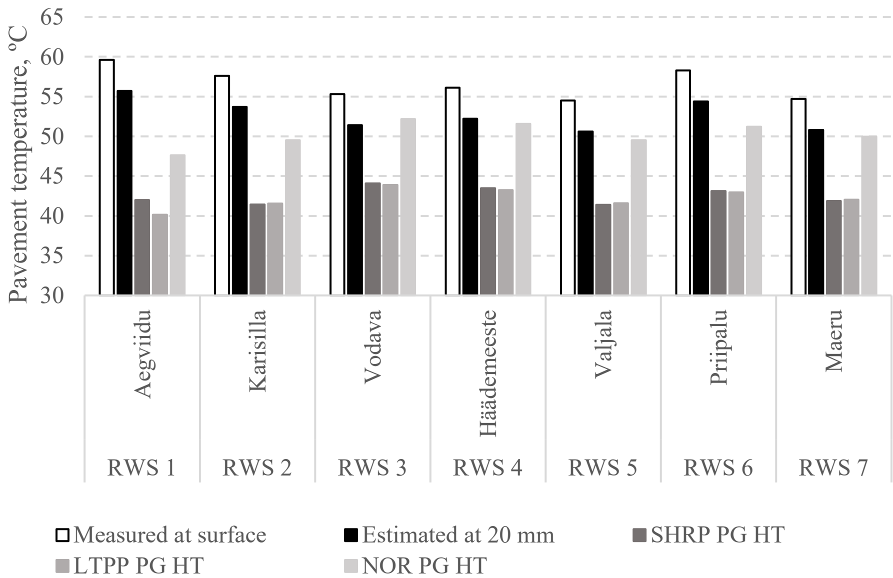

Figure 12. Pavement maximum temperatures recorded at road weather stations compared with calculated pavement maximum temperatures.

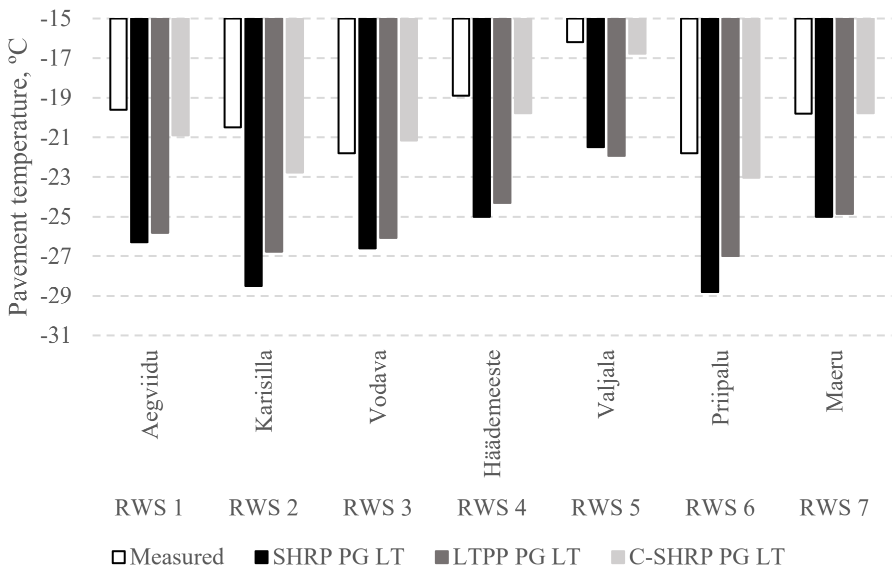

Figure 13. Pavement minimum temperatures from 2020 to 2022 recorded at road weather stations compared with calculated pavement minimum temperatures.

4.1.3 Superpave PG grading of Estonia

One of the aims of **Publication I** was to introduce PG grade regions (zones) of Estonian. The comparison between calculated and measured pavement temperatures indicated that Norwegian and C-SHRP models provided the closest results and therefore these models were used as the basis for determining the PG regions appliable for Estonia. The calculations were carried out based on the 7-day maximum and 1-day minimum air temperature data from weather stations indicated in Figure 7. Calculated pavement temperatures (PG regions) with 6 °C increments are described in Figure 14 and Figure 15.

The Norwegian model estimates pavement PG HT grades to range from 52 to 58 °C, while the C-SHRP model estimates pavement PG LT grades 22 °C. The results suggest that islands and coastal areas experience milder pavement temperatures, with colder temperatures predicted inland.

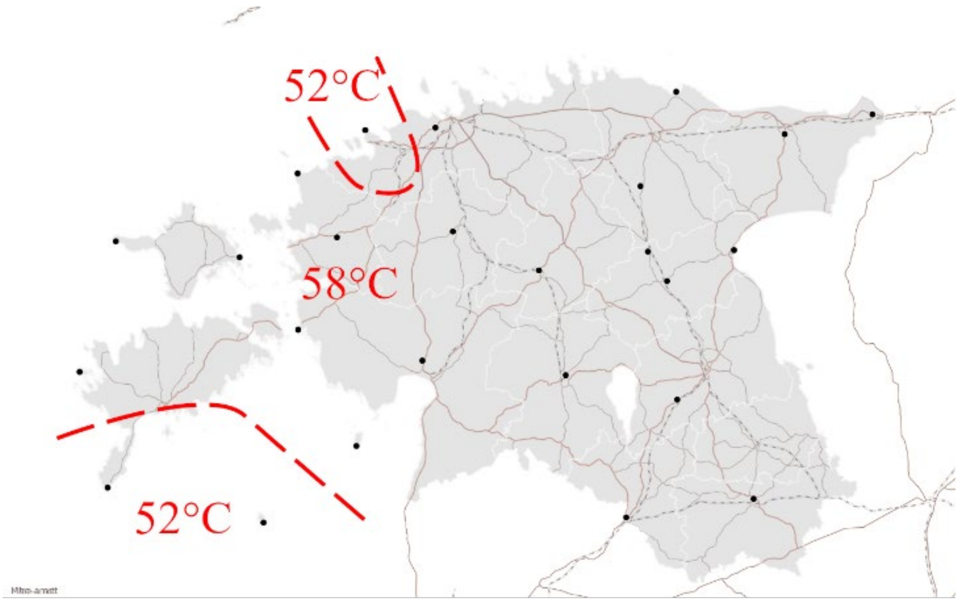

Figure 14. Pavement Superpave PG HT grades based on Norwegian model with 6 °C increments and ≥ 98% reliability.

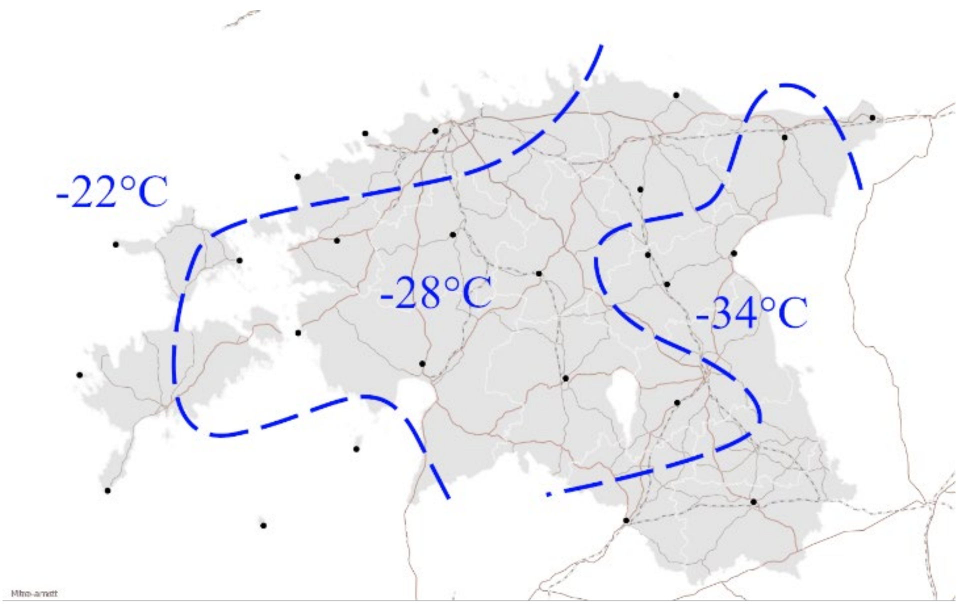

Figure 15. Superpave PG LT grades based on C-SHRP model with 6 °C increments and 98% reliability.

4.2 Pavement maximum and minimum temperature models

Comparison between SHRP, LTPP, Norwegian and C-SHRP pavement temperature prediction models in **Publication I** revealed variability in the calculated pavement temperatures. This could significantly influence the determination of Superpave PG HT and LT grades, not only for Estonia but for the entire Northern-Eastern European region. Data from road weather stations indicate that the commonly used LTPP pavement temperature prediction models are not suitable for Estonia, as they significantly underestimate maximum pavement temperatures and overestimate minimum pavement temperatures.

The Norwegian and C-SHRP models were in good agreements with the recorded maximum and minimum pavement temperatures in Estonia and were therefore recommended for use in pavement design temperature calculations. However, the Norwegian pavement temperature model lacks a depth variable, which could be used for calculating pavement temperatures at variable depths.

The aim of **Publication II** was to develop pavement temperature calculation models that would be the more accurate for Estonian conditions. To achieve this, air and pavement temperature data from the road weather stations indicated in Figure 9 were statistically analysed. The data for analysis ranged from December 2019 to February 2023.

4.2.1 Maximum pavement temperature regression model

In order to analyse the air and pavement temperature relationship in Estonia, the daily maximum air and pavement temperatures from June to September for the years 2020, 2021, and 2022 were extracted from the dataset for the Käru, Aegviidu, and Mõisaküla road weather stations. Daily maximum pavement temperatures, both at the surface and at 60 mm were correlated with daily maximum air temperatures, as shown in Figure 16 and Figure 17, respectively. The relationship between the maximum pavement temperature at 60 mm and the maximum pavement surface temperature is indicated in Figure 18.

A strong relationship was found between daily maximum air temperature and measured maximum pavement temperatures at surface and at 60 mm, with R^2 values of 0.79 and 0.85, respectively. Additionally, a very high correlation was observed between the maximum pavement temperatures at 60 mm and the maximum pavement surface temperatures, with an R^2 value of 0.96.

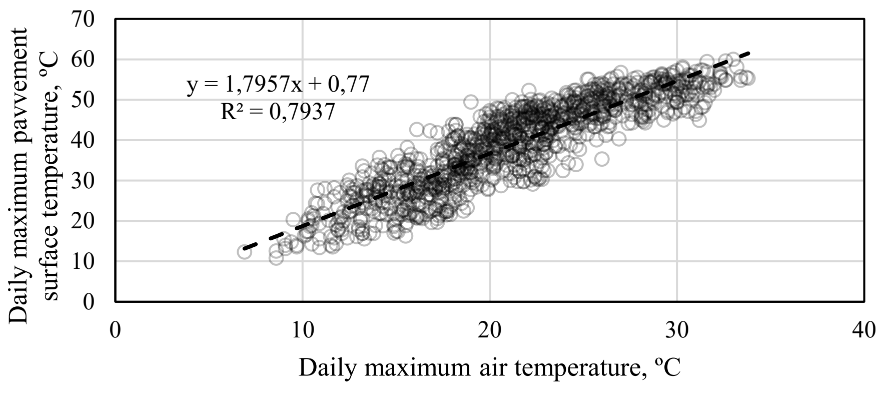

Figure 16. Daily maximum pavement surface temperature relative to daily maximum air temperature.

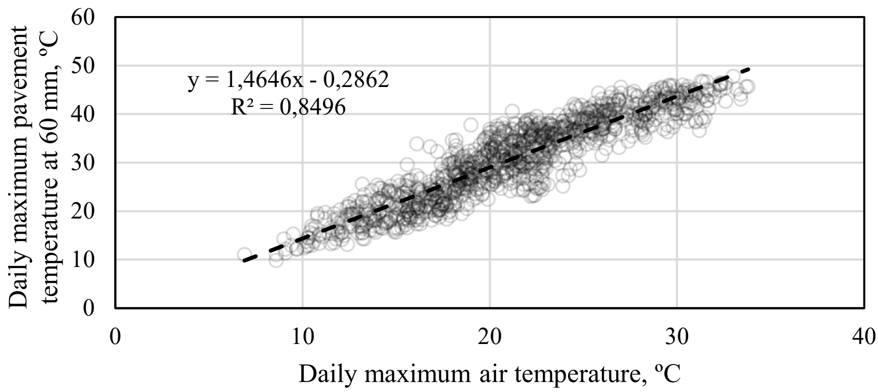

Figure 17. Daily maximum pavement temperature at 60 mm depth relative to daily maximum air temperature.

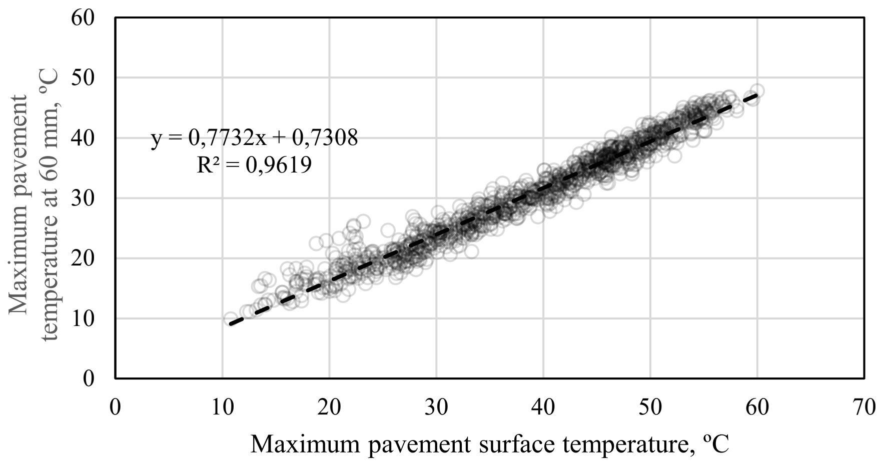

Figure 18. Daily maximum pavement surface temperature relative to daily maximum pavement temperature at 60 mm depth.

A multivariate regression model was developed (Equation 9) using daily maximum air temperatures, along with pavement temperatures at surface and at 60 mm depth, as input variables. The $Log_{10}(H+25)$ relationship was used as the depth variable, as it has shown to be the most effective predictor in previous air-pavement temperature correlation studies (Mohseni, 1998; Swarna et al., 2023a). The regression model, as presented in Equation 9, could be used for predicting maximum pavement temperatures at selected depths using daily maximum air temperature as an input. Based on N = 2188 observations, the model has an adjusted coefficient of determination (R^2) of 0.84 and a Standard Error (SE) of ±4.3 ºC.

$$T_{Pav,max} = 1.6302 \times T_{Air,max} - 16.8975 \times Log_{10}(h + 25) + 27.8947 \qquad \textit{Equation 9}$$

where:

$T_{Pav,max}$ – maximum pavement temperature at depth h, ºC;

$T_{Air,max}$ – daily or 7-day maximum air temperature, ºC;

h – depth from pavement surface, mm.

4.2.2 Minimum pavement temperature regression model

Daily minimum air and pavement temperatures from December to February in 2019 to 2023 for the Kauksi, Vodava and Sõmeru RWS were extracted from the dataset to examine air-pavement temperature relationship. The daily pavement temperatures at the surface and at 60 mm were correlated with the daily minimum air temperatures, as shown in Figure 19 and Figure 20, respectively. The correlation between the minimum pavement temperature at 60 mm depth and the minimum surface pavement temperature is also presented in Figure 21.

Similarly to maximum pavement temperatures, a strong relationship was found between the minimum air temperature and minimum pavement surface temperature at surface and at 60 mm, with R^2 values of 0.95 and 0.92, respectively. Similarly to maximum temperatures, a very high correlation was observed between the minimum pavement temperatures at 60 mm depth and the minimum pavement surface temperatures, with R^2 value of 0.97.

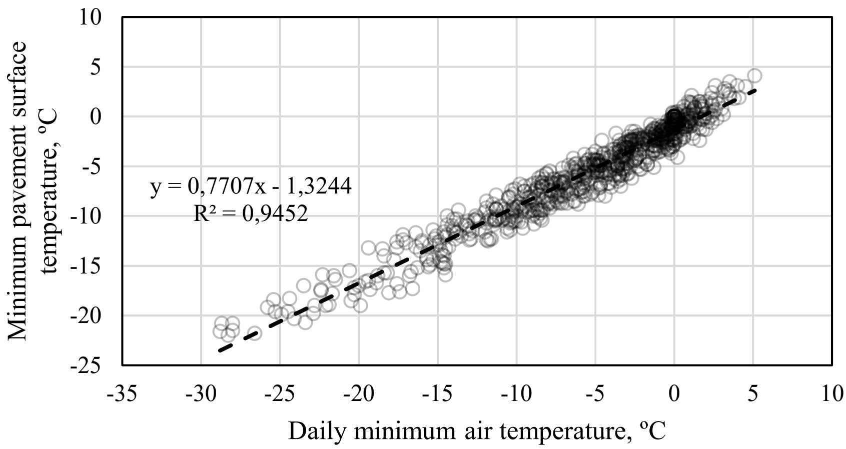

Figure 19. Daily minimum pavement surface temperature relative to daily minimum air temperature.

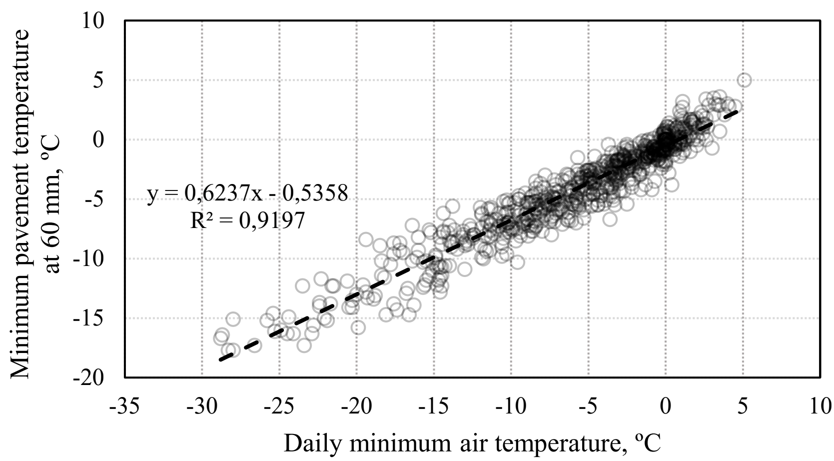

Figure 20. Daily minimum pavement temperature at 60 mm depth relative to daily minimum air temperature.

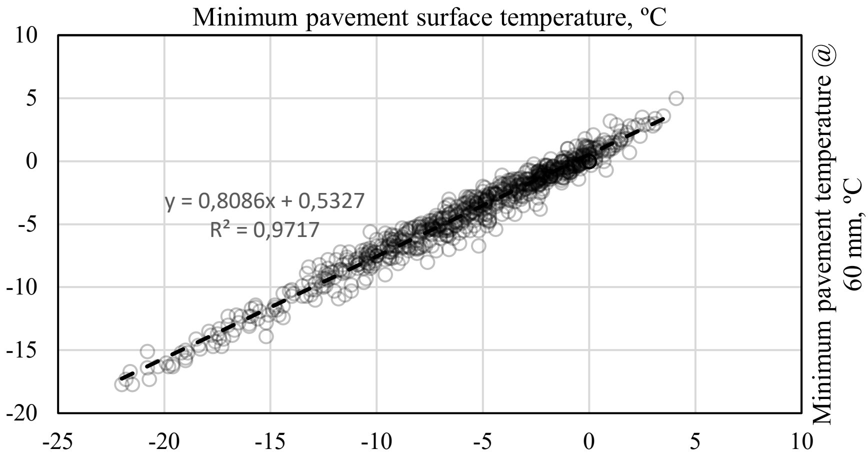

Figure 21. Daily minimum pavement surface temperature relative to daily minimum pavement temperature at 60 mm depth.

A multivariate regression model was developed (Equation 10) using the daily minimum air temperatures, along with pavement temperatures at surface and at 60 mm depth, as input variables. $Log_{10}(H+25)$ was used as the depth variable, similar to the model for predicting maximum pavement temperatures. The regression model can be used to predict minimum pavement temperatures at selected depths, using daily minimum air temperatures as the input. Based on N = 1474 observations, the model has an adjusted coefficient of determination (R^2) of 0.93 and a Standard Error (SE) of ±1.3 ºC.

$$T_{Pav,min} = 0.6944 \times T_{Air,min} + 3.4507 \times Log_{10}(h + 25) - 6.6132 \qquad \textit{Equation 10}$$

where:
$T_{Pav,min}$ – minimum pavement temperature at depth h, ºC;
$T_{Air,min}$ – daily minimum air temperature, ºC;
h – depth from pavement surface, mm.

4.2.3 Field validation of developed regression models

A separate analysis was carried out in **Publication II** to compare how well the established models estimate the pavement temperatures in Estonia. A comparison between calculated and measured pavement temperatures was conducted using temperature data from 12 randomly selected road weather stations throughout Estonia. Only road weather stations equipped with Vaisala pavement temperature sensors and at least 3 years of consistent pavement temperature data were included in the analysis. The locations of all the stations included for field validation are shown in Figure 9. The comparison was done at surface (h = 0 mm) and at 60 mm depth (h = 60 mm).

The differences between the measured and calculated maximum pavement temperatures for selected road weather stations are shown in Figure 22 and Figure 23. The absolute average discrepancies of the model for the surface and at a depth of 60 mm are 1,6 ºC and 2,7 ºC, respectively. The largest observed temperature discrepancies for the surface and at a depth of 60 mm are 3,1 ºC (Pärnamäe station) and 4,2 ºC (Priipalu station), respectively. This comparison showed that LTPP model would lead to lower pavement temperature estimates compared to registered temperatures both at the pavement surface and at the depth of 60 mm.

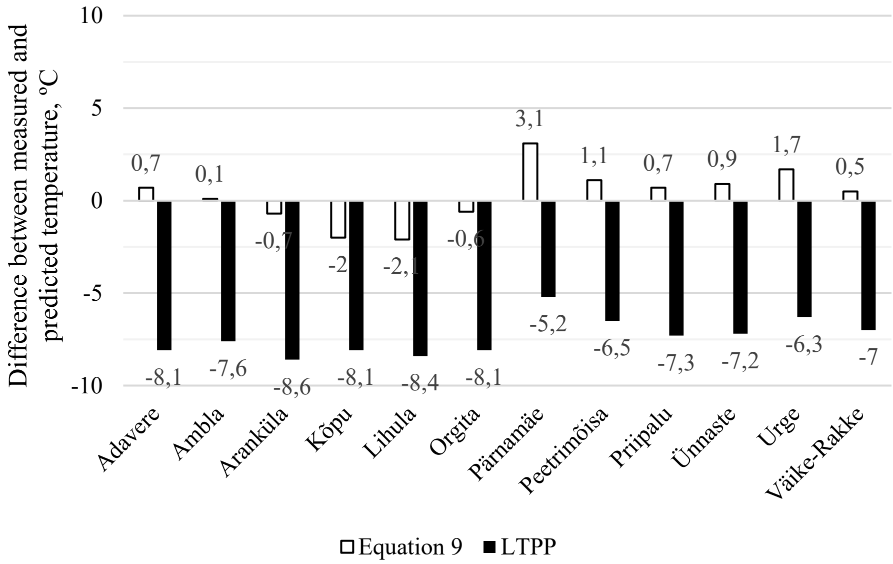

Figure 22. Difference between calculated and measured maximum pavement surface temperatures.

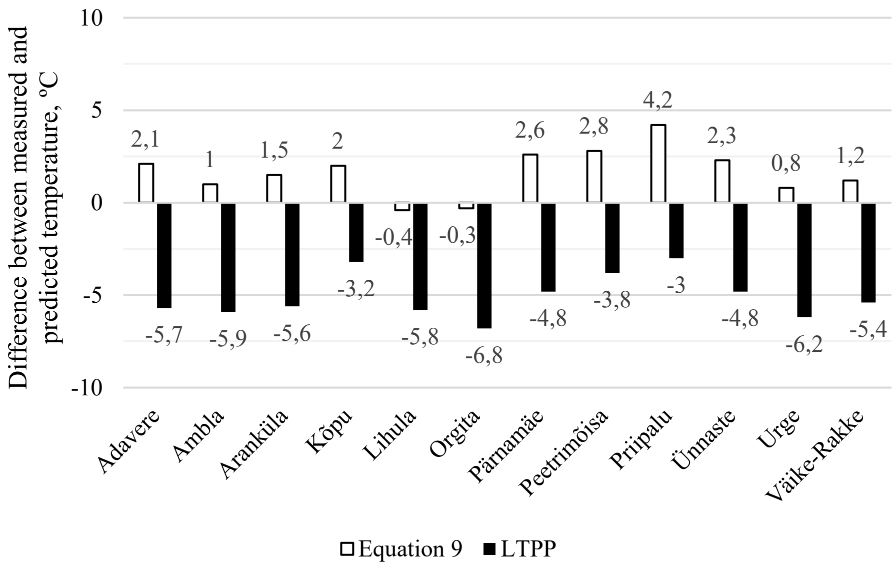

Figure 23. Difference between calculated and measured maximum pavement temperatures at 60 mm depth.

The comparative analysis for the minimum pavement surface temperatures is presented in Figure 24 and Figure 25. The absolute average discrepancies of the model for the surface and at 60 mm depth are 1,0 °C and 2,1 °C, respectively. The largest observed discrepancies for the surface and at a depth of 60 mm are 2,9 °C (Pärnamäe station) and

–2,6 ºC (Priipalu station), respectively. Like maximum pavement temperatures, the LTPP model would lead to lower pavement temperature estimates compared to measured temperatures. However, the C-SHRP minimum pavement temperature model is in good agreement with in-service temperatures. Nevertheless, based on road weather stations included in the validation analysis, the pavement temperature estimation model developed in **Publication II** provides slightly closer estimated temperatures compared to C-SHRP model.

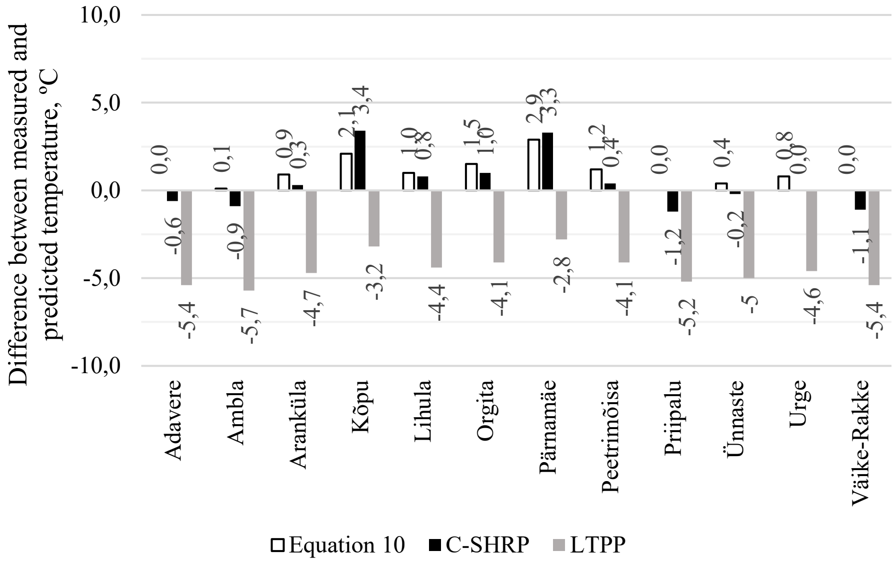

Figure 24. Difference between calculated and measured minimum pavement surface temperatures.

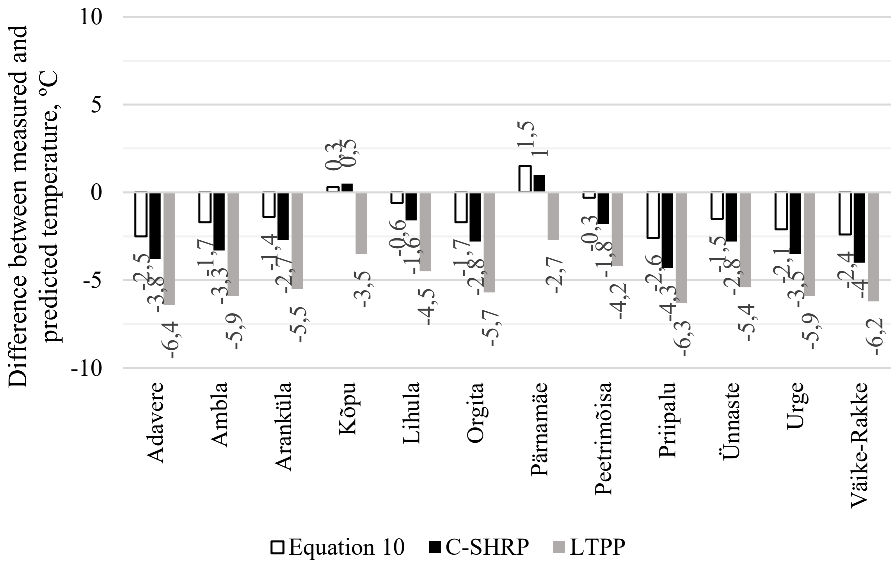

Figure 25. Difference between calculated and measured minimum pavement temperatures at 60 mm depth.

Developed statistical-empirical pavement temperature models closely match with the registered pavement temperatures in Estonia and were found to be more accurate than the LTPP and C-SHRP models in Estonian conditions. In conclusion, pavement temperature estimations can be made with reasonable accuracy across the latitudinal range of Estonia using the proposed correlation.

4.3 The impact of climate change to pavement temperatures in Estonia

Publication III analysed the potential impact of climate change on pavement temperatures and bitumen grade selection in Estonia. The impact was assessed using three commonly referred CMIP6 Shared Socioeconomic Pathway (SSP) projections – SSP1-2.6, SSP2-4.5, and SSP3-7.0. Pavement temperature analysis was carried out based on historical air temperature data of 25 meteorological stations indicated in Figure 7. The temperature data adopted in **Publication III** ranged from 2004 to 2024 (21 years). Historical air temperature data was converted to maximum and minimum pavement temperatures using statistical-empirical pavement temperature models developed in **Publication II** (Equation 9 and Equation 10). Average annual daily (1-day) maximum and minimum air temperatures and average annual 7-day average high air temperatures were considered in the calculations.

4.3.1 Calculated maximum and minimum pavement temperatures for current period

Based on temperature data from 2004 to 2024, the calculated maximum pavement surface temperatures are lowest in coastal areas and on the islands of Western Estonia. In these regions, the maximum summer pavement surface temperatures remain below 60 °C, whereas on the mainland, temperatures are close to or slightly exceed this threshold. Pavement maximum surface temperatures are described in Figure 28 as current temperatures.

The PG HT grade (h = 20 mm), determined based on the seven warmest consecutive days, is predominantly 52 °C for the coastal areas and islands of Western Estonia. According to data from the Sõrve station, the PG HT grade is 46 °C. On the mainland, the prevailing PG HT grade is 58 °C, while in Northeastern Estonia, it is generally 52 °C, except in Narva, where the PG HT is 58 °C. The PG HT regions of Estonia are indicated in Figure 26.

Coastal areas and the islands of Western Estonia experience warmer minimum pavement surface temperatures, with the PG LT grade (h = 0 mm) being mainly –22 °C. On the mainland, the predominant PG LT grade is –28 °C, with the lowest PG LT grade of –34 °C observed at two stations in Northeastern Estonia (Jõgeva and Jõhvi). The PG LT regions of Estonia are indicated in Figure 27.

PG HT and LT grades calculated with Equation 9 and Equation 10 for all included locations are indicated in Table 1.

Currently, penetration grade 70/100 is used throughout Estonia. Based on previous experience, PG LT grades –28 could only be met with high quality penetration grade 70/100 bitumen which show low tendency towards physical hardening (Lill et al., 2020a; Lill et al., 2020b). Regions with PG LT –28 and –34 grades could benefit from softer bitumen, e.g. penetration grade 100/150 or 160/220.

Table 1. Average daily maximum and minimum air temperatures and average 7-day maximum air temperatures with standard deviations (SD) over observed period of 2004 to 2024. Superpave PG HT and LT grades are calculated with 98% reliability based on 7-day maximum air temperature and 1-day minimum air temperature data, respectively

Station nr	Station name	1-Day T_{air} max		7-day T_{air} max		1-day T_{air} min		PG grade	
		T_{air} max °C	Std dev	T_{air} max °C	Std dev	T_{air} min °C	Std dev	HT	LT
1	Heltermaa	28,9	2,1	26	2,1	-19,2	5,2	52	-28
2	Väike-Maarja	30,3	1,9	27,3	2,2	-25,2	5,2	52	-28
3	Virtsu	29,5	2,2	26,5	1,9	-20,3	5,3	52	-28
4	Vilsandi	28,6	2,3	25,4	2,0	-14,5	5	52	-22
5	Viljandi	31,4	1,8	28,4	1,9	-24,1	5,6	58	-28
6	Türi	30,8	1,8	27,9	2,1	-23,8	4,9	58	-28
7	Tooma	30,8	1,8	27,7	2,2	-25,4	5,5	58	-28
8	Tiirikoja	29,6	1,7	26,7	1,6	-24,8	5,9	52	-28
9	Sõrve	26,9	2,0	24,3	1,7	-14,6	4,8	46	-22
10	Ruhnu	27,6	1,8	25,3	1,7	-13,3	4,3	52	-22
11	Ristna	28,5	2,3	25,5	2,2	-15,7	4,6	52	-22
12	Pakri	29,6	2,3	24,8	2,2	-17,5	5,2	52	-22
13	Lääne-Nigula	30,7	1,9	27,8	2,3	-22,7	5,0	58	-28
14	Kuusiku	30,5	1,7	27,8	2,1	-24,8	5,0	58	-28
15	Kihnu	29,2	1,7	26,3	1,9	-17,7	5,2	52	-22
16	Tallinn-Harku	30,4	2,1	27,0	2,5	-20,1	4,6	58	-28
17	Pärnu	30,6	1,5	27,6	1,9	-22,3	6,0	52	-28
18	Narva	31	2,2	27,6	2,3	-23,8	5,2	58	-28
19	Tartu	31,2	1,5	28,0	2,1	-24,6	5,8	58	-28
20	Dirhami	28,3	3,5	25,1	2,1	-18,0	5,1	52	-22
21	Jõgeva	31	1,8	27,9	2,1	-26,9	5,5	58	-34
22	Jõhvi	30,2	2,2	27,2	2,3	-25,9	6,0	52	-34
23	Kunda	30,7	2,6	26,6	2,6	-21,2	5,8	52	-28
24	Võru	31,7	1,6	28,6	1,9	-25,4	5,7	58	-28
25	Valga	31,4	1,6	28,4	1,9	-24,9	5,7	58	-28

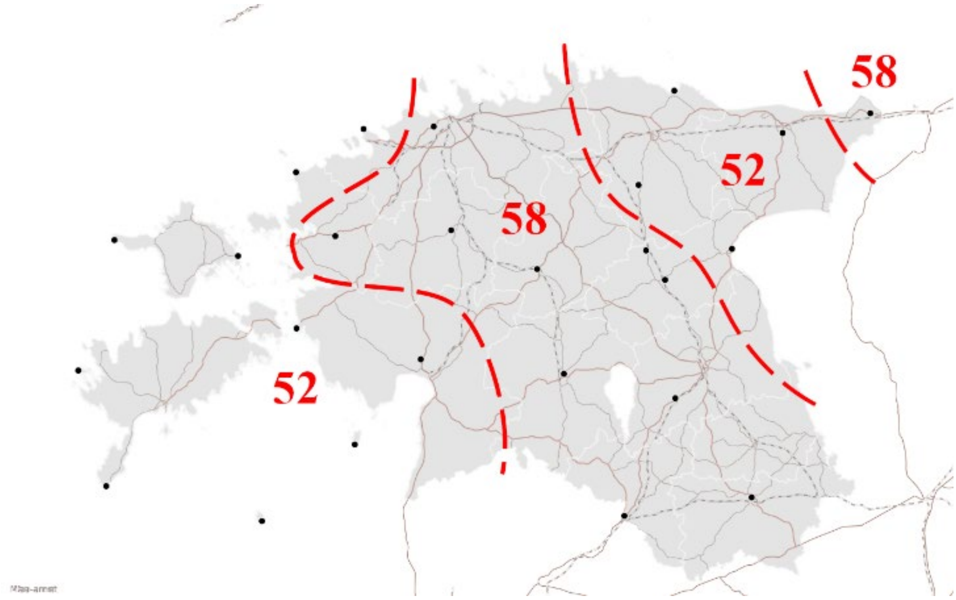

Figure 26. Superpave PG HT grades based on average 7-day maximum air temperatures from 2004 to 2024 (≥ 98% reliability).

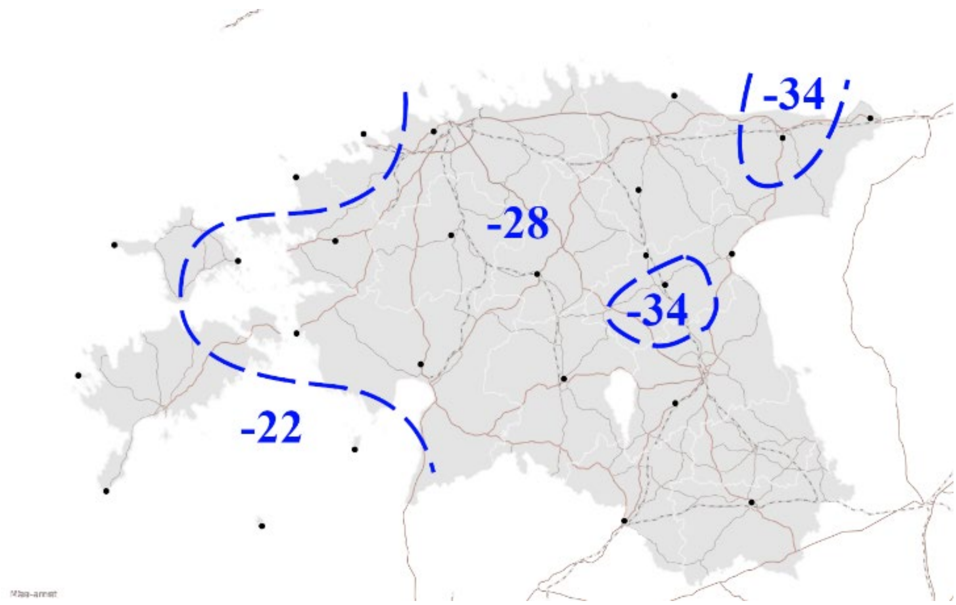

Figure 27. Superpave PG LT grades based on daily minimum air temperature data from 2004 to 2024 (≥ 98% reliability).

4.3.2 Calculated pavement temperatures based on different climate change scenarios

Publication III analysed the impact of climate change to calculated pavement maximum and minimum temperatures, using the pavement temperature estimation models introduced in **Publication II**. The analysis used the same weather stations as those in **Publication I** and indicated in Figure 7. The reference period was selected to be from 2004 to 2024. The analysis concentrated on three input variables: a) daily (1-day) maximum air temperatures; b) daily (1-day) minimum air temperatures and; c) 7-day maximum air temperatures. Table 1 indicates the average daily maximum and minimum air temperatures, the average 7-day maximum air temperatures, and the calculated PG HT and LT grades with at least 98% reliability for each included weather station in observation period 2004–2024.

Three periods were considered in the climate change analysis: near-, medium-, and long-term periods. The near-term period describes pavement temperatures from 2025 to 2044, the medium-term period covers 2045 to 2064, and the long-term period focuses on pavement temperatures from 2065 to 2084. The daily (1-day) maximum pavement surface temperatures during these periods under different SSP scenarios are described in Figure 28, Figure 29, and Figure 30.

In the near-term perspective (Figure 28), the pavement maximum surface temperatures are expected to increase by approximately 0–2 °C, depending on the selected SSP scenario. The SSP1-2.6 scenario results in a surface temperature increase of 0–1 °C, while the SSP2-4.5 and SSP3-7.0 scenarios lead to a 1–2 °C rise in maximum pavement surface temperatures. In the medium term (Figure 29), pavement maximum surface temperatures are projected to increase by 1–2 °C, 2–3 °C, and 3–4 °C under the SSP1-2.6, SSP2-4.5, and SSP3-7.0 scenarios, respectively. In the long term (Figure 30), the increases are projected to be 1–2 °C, 3–4 °C, and 4–5 °C under the SSP1-2.6, SSP2-4.5, and SSP3-7.0 scenarios, respectively.

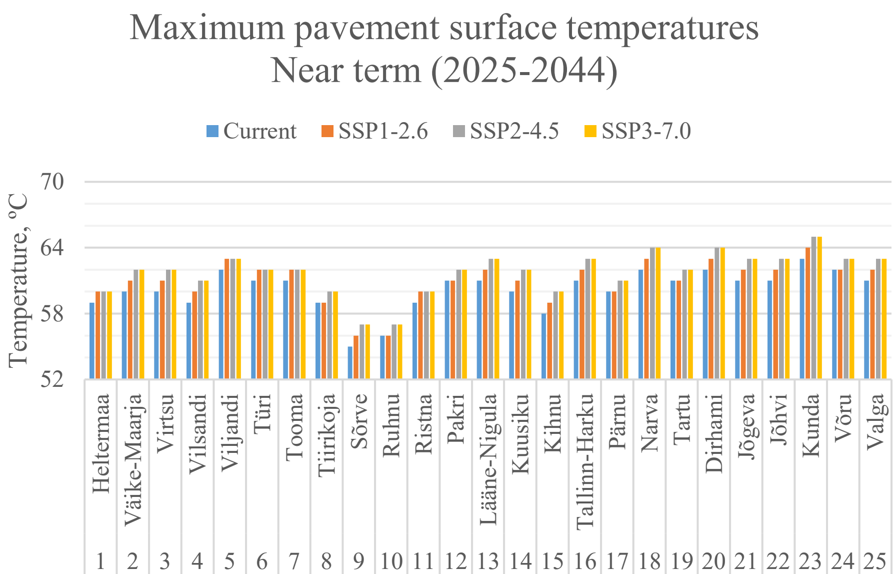

Figure 28. Estimated near term pavement maximum surface temperatures based on average 1-day maximum air temperatures from 2004 to 2024 (with ≥ 98% reliability).

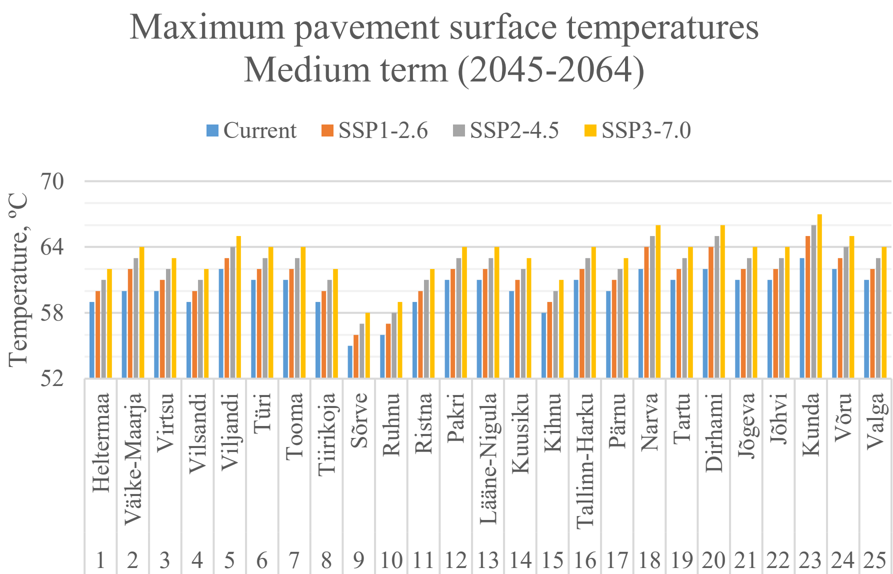

Figure 29. Estimated medium term pavement maximum surface temperatures based on average 1-day maximum air temperatures from 2004 to 2024 (with ≥ 98% reliability).

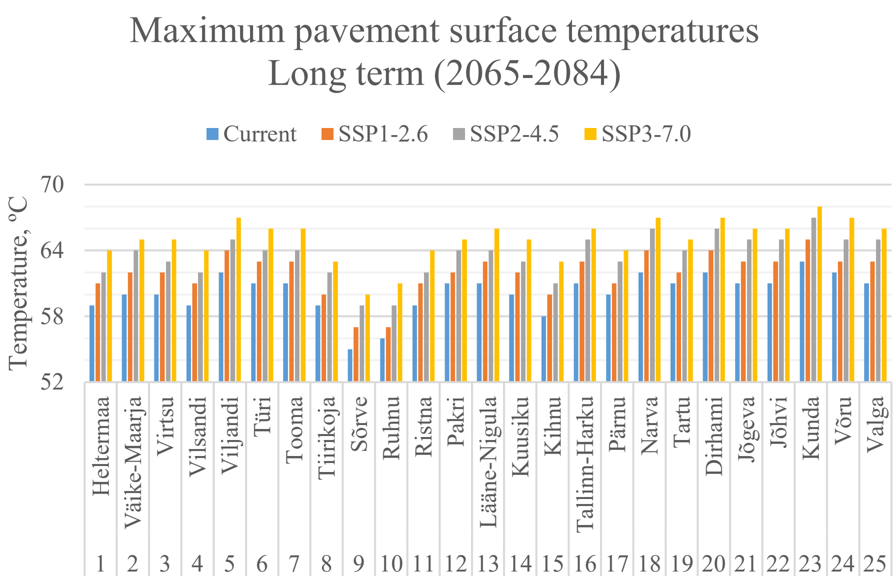

Figure 30. Estimated long term pavement maximum surface temperatures based on average 1-day maximum air temperatures from 2004 to 2024 (with ≥ 98% reliability).

The influence of climate change on pavement PG grades was evaluated by examining variations in pavement PG HT and LT temperatures. The calculations for PG HT and LT temperatures utilised the average 7-day maximum air temperature and the average 1-day minimum air temperatures and standard deviations listed in Table 1. The results for PG HT are illustrated in Figure 31, Figure 32 and Figure 33. The results for PG LT are shown in Figure 34, Figure 35 and Figure 36. All values shown in figures were rounded to the nearest whole number. Table 2 provides summarised overview of the changes in PG HT and PG LT grades for each station included in the analysis.

4.3.2.1 Near term scenarios (2025–2044)

Near term SSP1-2.6 scenario is predicted to increase the pavement PG HT grade from 52 to 58 ^{o}C in three locations (Väike-Maarja, Jõhvi and Kunda). Under the same SSP scenario, the PG LT grade is predicted to increases in four locations (Heltermaa, Tallinn-Harku PG LT temperature increases from –28 to –22 ^{o}C; Jõgeva, Jõhvi PG LT increases from –34 to –28 ^{o}C). In near term SSP2-4.5 and SSP3-7.0 scenarios, the PG HT grade is predicted to increases in five locations (Väike-Maarja, Pärnu, Jõhvi, Kunda from 52 to 58 ^{o}C; Sõrve from 46 to 52 ^{o}C). In the same SSP scenarios, the PG LT grade is predicted to increases in six locations (Heltermaa, Virtsu, Tallinn-Harku from–28 to –22 ^{o}C; Ruhnu from –22 to –16 ^{o}C; Jõgeva, Jõhvi from –34 to –28 ^{o}C).

4.3.2.2 Medium term scenarios (2045–2064)

Medium term SSP1-2.6 and SSP2-4.5 scenarios are expected to increase pavement PG HT grade in five locations: (Väike-Maarja, Jõhvi, Kunda, Pärnu from 52 to 58 ^{o}C; Sõrve from 46 to 52 ^{o}C). Under the SSP1-2.6 scenario, the pavement PG LT is predicted to increase in five locations (Heltermaa, Virtsu, Tallinn-Harku from –28 to –22 ^{o}C; Jõgeva, Jõhvi from -34 to –28 ^{o}C). In medium term SSP2-4.5 scenario, the number of stations where the pavement PG LT temperature is expected to increase is 12 (Heltermaa, Virtsu, Türi, Lääne-Nigula, Tallinn-Harku, Kunda from –28 to –22 ^{o}C; Jõgeva, Jõhvi from –34 to –28 ^{o}C; Vilsandi, Sõrve, Ruhnu, Ristna,from from –22 to –16 ^{o}C). In SSP3-7.0 scenario, the PG HT grade is predicted to increases in eight locations (Heltermaa, Väike-Maarja, Virtsu, Kihnu, Pärnu, Jõhvi, Kunda from 52 to 58 ^{o}C; Sõrve from 46 to 52 ^{o}C) and PG LT grade is predicted to increase in 17 locations (Heltermaa, Väike-Maarja, Virtsu, Viljandi, Türi, Lääne-Nigula, Kuusiku, Tallinn-Harku, Pärnu, Narva, Kunda from –28 to –22 ^{o}C; Vilsandi, Sõrve, Ruhnu, Ristna from –22 to –16 ^{o}C; Jõgeva, Jõhvi from –34 to –28 ^{o}C).

4.3.2.3 Long term scenarios (2065–2084)

In the long term SSP1-2.6 scenario, the pavement PG HT grade is predicted to increase in the same five locations as described for medium term SSP1-2.6 scenario. However, the PG LT grade is predicted to increase in 15 locations (Heltermaa, Virtsu, Türi, Lääne-Nigula, Kuusiku, Tallinn-Harku, Pärnu, Narva, Kunda from –28 to –22 ^{o}C; Vilsandi, Sõrve, Ruhnu, Ristna from –22 to –16 ^{o}C; Jõgeva, Jõhvi –34 to –28 ^{o}C). In SSP2-4.5 scenario, the pavement PG HT grade is expected to increase in 10 locations (Heltermaa, Väike-Maarja, Virtsu Tiirikoja, Ristna, Kihnu, Pärnu, Jõhvi, Kunda from 52 to 58^{o}C; Sõrve from 46 to 52 ^{o}C. PG LT grade is predicted to increase in 22 locations with only three locations remaining unchanged (Kihnu, Dirhami, Võru). In SSP3-7.0 scenario, the PG HT is predicted to increase in 13 locations (Heltermaa, Väike-Maarja, Virtsu, Vilsandi, Tiirikoja, Ristna, Pakri, Kihnu, Pärnu, Dirhami, Jõhvi, Kunda from 52 to 58 ^{o}C; Sõrve from 46 to 52 ^{o}C) and PG LT is predicted to increases in all locations. In four locations the PG LT increases by two grades (Heltermaa, Tallinn-Harku from –28 to –16 ^{o}C; Jõgeva, Jõhvi from –34 to –22 ^{o}C).

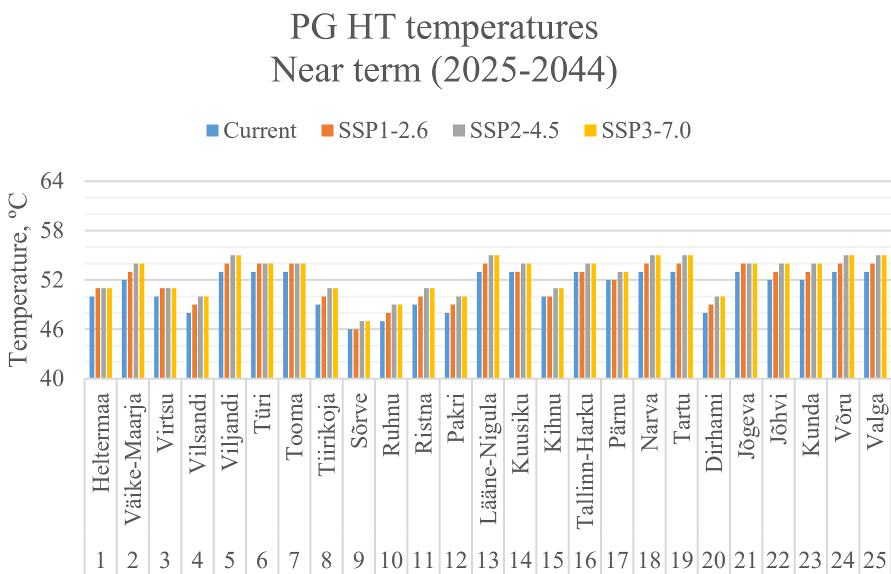

Figure 31. Predicted medium term pavement PG HT temperatures with ≥ 98% reliability.

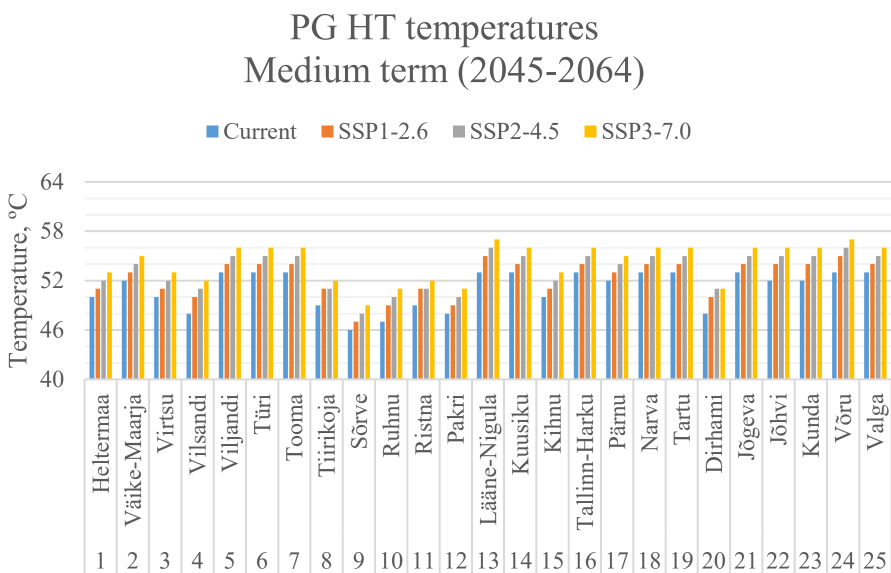

Figure 32. Predicted medium term pavement PG HT temperatures with ≥ 98% reliability.

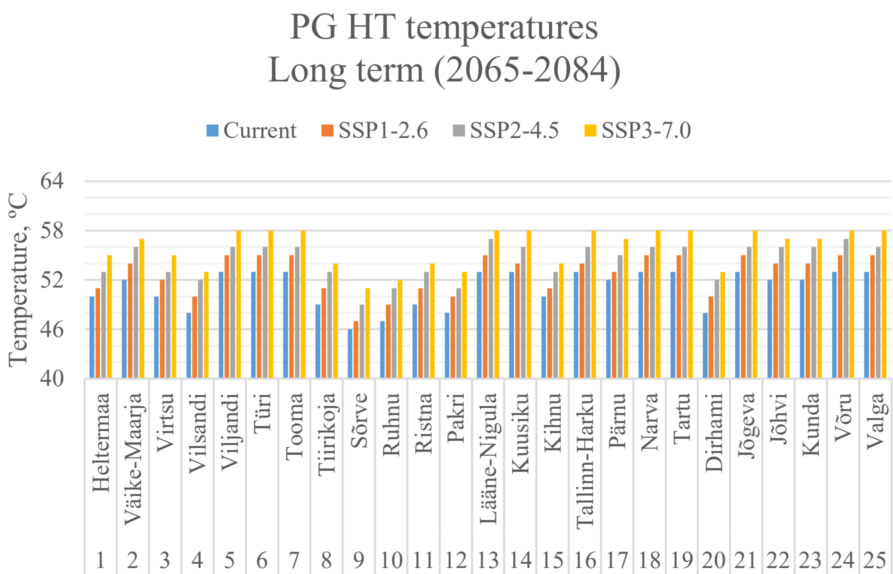

Figure 33. Predicted long term pavement PG HT temperatures with ≥ 98% reliability.

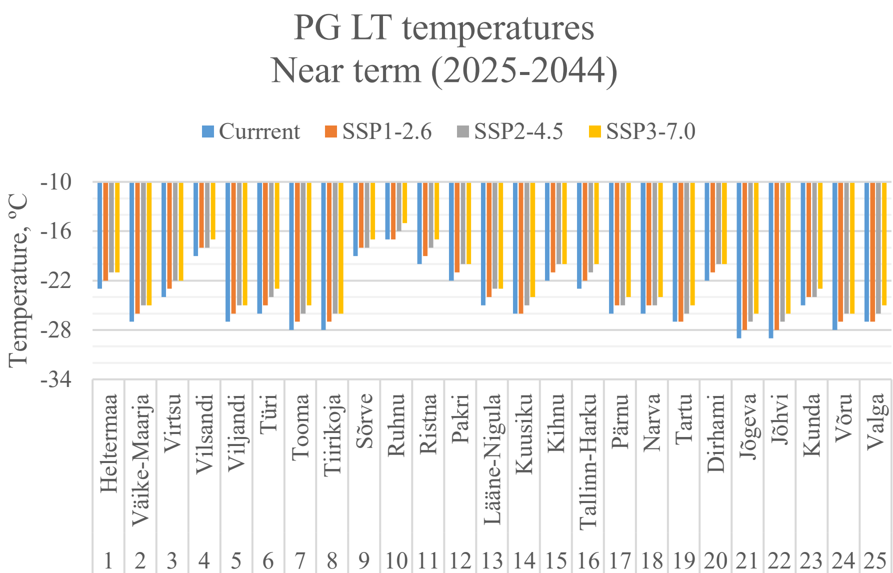

Figure 34. Predicted near term pavement PG LT temperatures with ≥ 98% reliability.

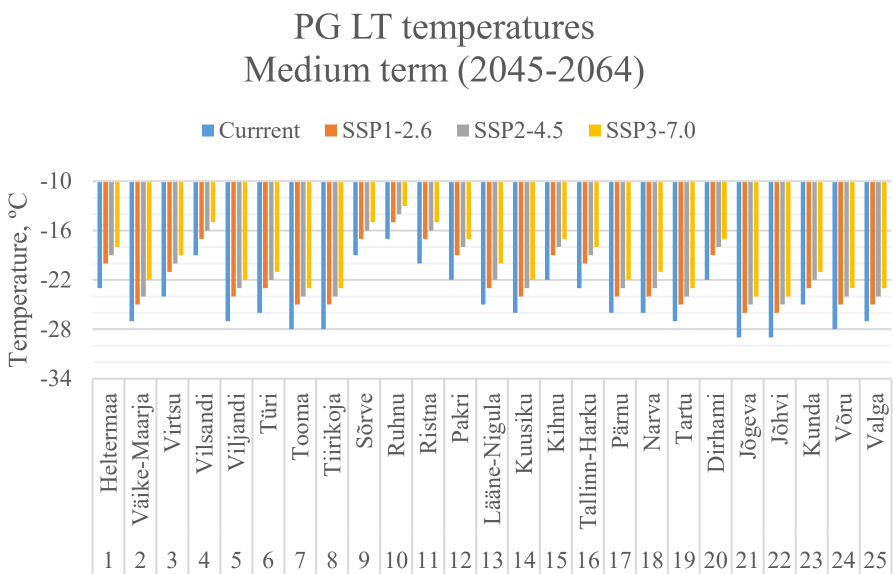

Figure 35. Predicted medium term pavement PG LT temperatures with ≥ 98% reliability.

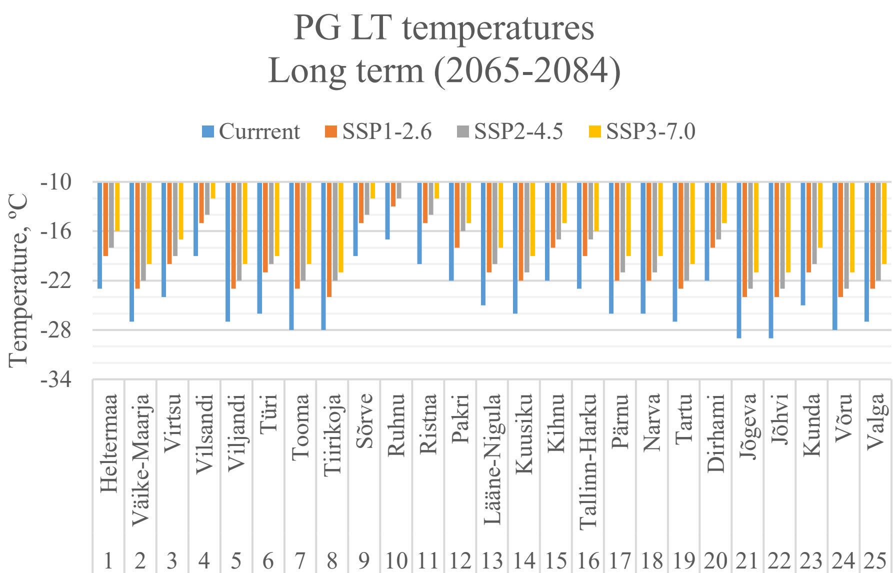

Figure 36. Predicted long term pavement PG LT temperatures with ≥ 98% reliability.

Table 2. Predicted shift in bitumen performance grades in near (N), medium (M) and long (L) term climate change scenarios. One and two plus signs refer to increase by 6 °C or 12 °C compared to current PG grades, respectively.

Station nr	Station	PG HT grade increase									PG LT grade increase								
		SSP1-2.6			SSP2-4.5			SSP3-7.0			SSP1-2.6			SSP2-4.5			SSP3-7.0		
		N	M	L	N	M	L	N	M	L	N	M	L	N	M	L	N	M	L
1	Heltermaa						+		+	+	+	+	+	+	+	+	+	+	++
2	Väike-Maarja	+	+	+	+	+	+	+	+	+						+		+	+
3	Virtsu						+		+	+		+	+	+	+	+	+	+	+
4	Vilsandi									+			+		+	+		+	+
5	Viljandi															+		+	+
6	Türi												+		+	+		+	+
7	Tooma															+			+
8	Tiirikoja						+			+						+			+
9	Sõrve		+	+	+	+	+	+	+	+			+		+	+		+	+
10	Ruhnu												+	+	+	+	+	+	+
11	Ristna						+			+			+		+	+		+	+
12	Pakri									+						+			+
13	Lääne-Nigula												+		+	+		+	+
14	Kuusiku												+			+		+	+
15	Kihnu						+		+	+									+
16	Tallinn-Harku										+	+	+	+	+	+	+	+	++
17	Pärnu		+	+	+	+	+	+	+	+			+			+		+	+
18	Narva												+			+		+	+
19	Tartu															+			+
20	Dirhami									+									+
21	Jõgeva										+	+	+	+	+	+	+	+	++
22	Jõhvi	+	+	+	+	+	+	+	+	+	+	+	+	+	+	+	+	+	++
23	Kunda	+	+	+	+	+	+	+	+	+			+		+	+		+	+
24	Võru																		+
25	Valga															+			+

Figure 37, Figure 38 and Figure 39 are illustrating Estonian PG regions based on SSP2-4.5 scenario for the near, medium and long term, respectively. Climate change scenario SSP2-4.5 was selected since this is often described as the most probable scenario. In near term the dominant PG HT-LT grade is expected to be PG 58-28. In near- and long-term climate change scenarios, the PG HT grade 58 remains to dominate. However, the dominant PG LT grade is expected to increase gradually from –28 to –22 °C. This would mean that requirements for bitumen low temperature properties could be relieved in the future. However, the area with PG HT 58 °C is expected to increase which requires more attention to be paid to bitumen high temperature properties.

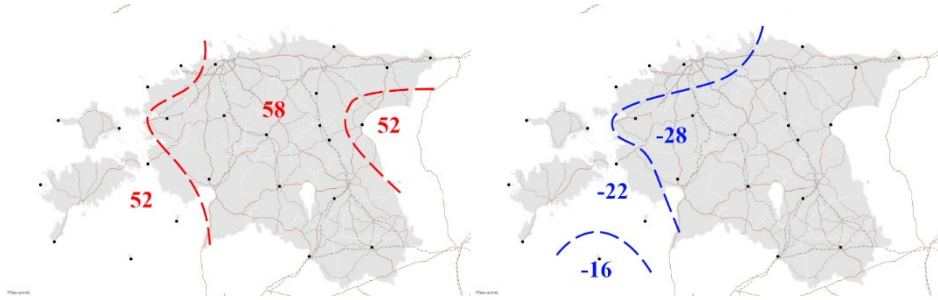

Figure 37. Superpave PG HT (left) and PG LT (right) pavement design temperatures in 2025–2044 based on SSP2-4.5 projection.

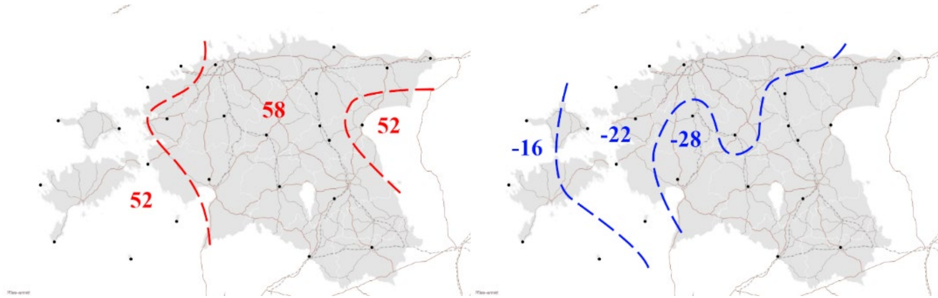

Figure 38. Superpave PG HT (left) and PG LT (right) pavement design temperatures in 2045–2064 based on SSP2-4.5 projection.

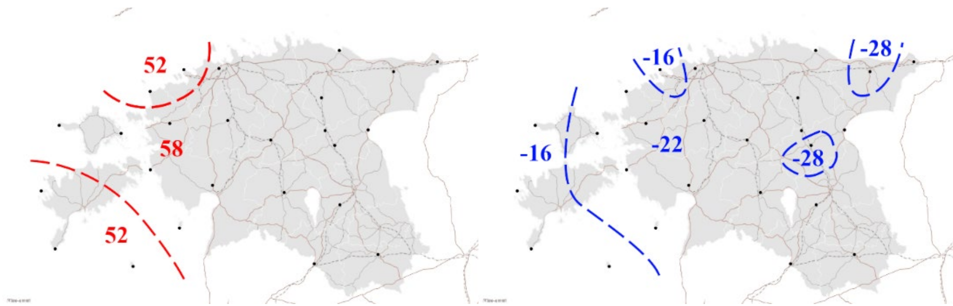

Figure 39. Superpave PG HT (left) and PG LT (right) pavement design temperatures in 2065–2084 based on SSP2-4.5 projection.

5 Conclusions

This thesis investigates which pavement temperatures would be applicable in Estonia according to Superpave Performance Grading principles. Over the course of the study, it was found that widely adopted pavement temperature calculation models usually adopted to determine pavement PG grades do not match with the actual pavement temperatures experienced in Estonia and would lead to erroneous pavement temperature estimations and bitumen grade selection. A separate pavement temperature estimation models were developed which were used to calculate the current and future pavement temperatures in Estonia.

The main conclusions from **Publication I** were following:

- Widely adopted LTPP pavement temperature models would under- and overestimate maximum and minimum pavement temperatures in Estonia, respectively.
- Pavement temperature models developed for Norwegian and Canadian climatic conditions show relatively good agreement with the actual registered pavement temperatures in Estonia.
- Based on Norwegian and Canadian pavement temperature models, the high temperature PG HT grades would be 52 and 58 °C. The low temperature PG LT grades would range from –22 °C near coastal areas in Northern and Western Estonia to –34 °C in Eastern Estonia.

Publication II developed two pavement temperature estimation models which can be used to determine pavement maximum and minimum temperatures for Estonian conditions. The conclusions of **Publication II** were:

- Pavement temperatures predicted with developed models are in good agreement with measured pavement temperatures in Estonia. The correlative models can be used with a fair degree of accuracy to predict pavement maximum and minimum pavement temperatures for Performance Grading in Estonia.
- Comparison between calculated and registered pavement temperatures showed that these models provide more accurate pavement temperature estimates compared to LTPP and Canadian models.

Publication III analysed the current and future pavement temperatures in Estonia based on pavement temperature estimation models introduced in **Publication II**. The impact of climate change to pavement temperatures was analysed based on three Shared Socioeconomical Pathways. The conclusions of **Publication III** are:

- Based on air temperature data from 2004–2024, the current pavement high temperature grades are 52 °C in coastal areas and islands, as well as in North-West Estonia, and 58 °C in mainland areas. The pavement low temperature grades are primarily –22 °C in coastal regions and islands, and –28 °C on the mainland. Only two small local regions would currently require bitumen with low temperature PG LT grade of –34 °C.
- Based on analysed climate change scenarios, the pavement high temperature PG HT grades remain to be 52 °C and 58 °C in all observed near-, medium- and long-term scenarios, but the area with PG HT 58 °C is expected to increase. In contrast, pavement low temperature grades are predicted to be more affected by climate change.

- When designing pavements for next 20 years, a PG LT grade of –34 °C would not be required. Considering the most plausible climate change scenario, the dominant PG LT grades would be –22 °C near coastal areas and islands, and –28 °C on the mainland.
- Analysed scenarios indicate warming trends, with both coastal and mainland areas undergoing notable shifts in PG grades. Climate change has a greater impact on low temperature PG LT grades, resulting in a narrower range of pavement temperatures across the region.
- Climate change leads to less strict requirements regarding bitumen low temperature properties but could increase the requirements regarding bitumen high temperature properties.

5.1 Recommendations for further studies

The completion of this study clarified several potential research directions that would require further investigation:

- **Improving the accuracy of the developed pavement temperature prediction models.** The pavement temperature model developed in this study is based on data from road weather stations that record temperatures on the pavement surface and at a depth of approximately 60 mm from the surface. To improve the accuracy and reliability of the developed models, additional research focusing on pavement temperature measurements at other depths is recommended. Also, since studded winter tires widely used by vehicles in Estonia causes pavement wear and rutting, potentially more accurate pavement temperature models can be developed if the actual distance between the surface and in-pavement sensors is known.
- **Improving the accuracy of the climate change predictions**. The current analysis used CMIP6 predicted increases in daily maximum and minimum air temperatures. The CMIP6 climate change model represents a combination of predictions from different climate models. It should be noted that not all models included in CMIP6 may provide reliable results for the Northern-Eastern Europe region. Future research should identify which models included in CMIP6 align better with the actual temperature changes in Northern-Eastern Europe. This could help to improve the reliability of predictions and enable more effective consideration of these forecasts.
- **Analysing the applicability of findings to Estonian asphalt pavements.** This study examines pavement temperatures derived from the principles of Performance Grading. It should be noted, however, that Performance Grading constitutes only one part of the fundamental principles of asphalt mixture design developed in North America. Further attention should be given to analyse, how Performance Grading could affect the properties of asphalt mixtures and therefore pavements in Estonia.

List of figures

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Abstract

Development of Pavement Temperature Estimation Models and the Impact of Climate Change to Pavement Temperatures in Estonia

Bitumen is a widely used thermoplastic and viscoelastic material in road construction and maintenance, with its properties being dependent on temperature and applied loads. Bitumen significantly influences the performance and durability of pavements. To ensure the long-term durability of pavements, it is essential to carefully select bitumen so that its properties align with the pavement's temperature and load conditions.

In Europe, harmonised standards are used for selecting bitumen, which are based on empirical penetration grades. The empirical tests referred to in these standards do not relate with the fundamental properties of bitumen or with its performance and durability. Additionally, the properties are determined for unaged or short-term aged bitumen. In the United States, a performance-based approach, known as Performance Grading (PG grading) was developed to ensure the suitability of bitumen properties with the pavement temperatures and loads experienced throughout its service life. Performance evaluation of bitumen involves test methods that assess the fundamental properties of bitumen at both high and low pavement temperatures. Particular emphasis is placed on evaluating the properties of bitumen after both short- and long-term ageing.

The aim of this research was to investigate which PG grades would be applicable in Estonia based on the principles of determining PG grades. As part of this research, different pavement temperature calculation models used for PG grade determination were compared. This comparison indicated that different calculation models may not provide reliable results for the Estonian region. To address this issue, two separate models for calculating pavement temperatures were developed during the study to estimate the maximum and minimum temperatures of Estonian road pavements. The models use daily maximum or minimum air temperatures as inputs. Additionally, the models can account for the calculation depth relative to the pavement surface. Comparisons with pavement temperatures recorded by road weather stations in Estonia showed that the models developed in this study provide more accurate results for Estonian conditions than other widely used models.

Using the models developed in this study and the principles of PG grade determination, the maximum pavement PG grade temperatures in Estonia are 52 °C and 58 °C depending on the region. The main minimum PG grade temperatures are –28 °C and –22 °C. There are also localised areas where minimum PG grade temperatures of –16 °C and –34 °C should be considered. Lower maximum and higher minimum pavement PG grade temperatures are predominantly found in the coastal areas and islands of Western Estonia.

Additionally, the study analysed the impact of climate change on the pavement temperatures in Estonia in the near, medium, and long term. The results indicate that, under all the examined climate change scenarios, the dominant maximum PG grade temperatures will remain to be 52 °C and 58 °C. However, the areas where a PG grade of 58 °C should be considered will increase in the future. In contrast, more significant changes will occur in the minimum pavement temperatures. For pavements designed for the next 20 years, the dominant PG low-temperature grades will be –28 °C and –22 °C, and there will be less need to account for the local areas requiring a PG grade of –34 °C.

In the medium and long term, climate change will reduce the area of regions requiring a PG grade of –28 °C, while the area requiring a PG grade of –22 °C will increase.

Currently, the bitumen used for asphalt mixtures in Estonia is predominantly penetration grade 70/100. Although this grade meets the requirements for maximum PG grades, issues with meeting the minimum PG grade –28 °C may arise, depending on the quality of the bitumen. In the near future, regions requiring the use of bitumen that meets the PG 52-28 or PG 58-28 grade requirements may benefit from using bitumen with penetration grades of 100/150 or 160/220 in the surface course layers of the pavement.

Lühikokkuvõte

Teekatte temperatuuride ennustamise mudeli väljatöötamine ning kliimamuutuste mõju teekatte temperatuuridele Eestis

Bituumen on teede ehituses ja korrashoius laialdaselt kasutatav termoplastne ning viskoelastne materjal, mille omadused sõltuvad temperatuurist ning rakendatud koormustest. Bituumenil on suur mõju teekatete toimivusele ja vastupidavusele. Et tagada teekatete pikaajaline vastupidavus, tuleb bituumeni valikult hoolikalt jälgida, et selle omadused sobituksid teekatte temperatuuride ja koormustingimustega.

Euroopas kasutatakse bituumeni valikul ühtlustatud tootestandardeid, mis põhinevad empiirilistel penetratsioonimarkidel. Tootestandardis viidatud empiirilistel katsetel puudub seos bituumeni fundamentaalsete omadustega ning selle toimivuse ja vastupidavusega. Lisaks määratakse omadusi vanandamata või ainult lühiajaliselt vanandatud proovidele. Ameerika Ühendriikides töötati välja toimivusmarkidel (PG margid) põhinev lähenemine, mille eesmärgiks on tagada bituumeni omaduste sobivus tee eluea jooksul kogetavate teekatte temperatuuride ja koormustega. Bituumeni toimivuse hindamisel kasutatakse katsemeetodeid, mis hindavad bituumeni fundamentaalseid omadusi teekatte kõrgetel ja madalatel temperatuuridel. Sealjuures pööratakse rõhku asjaolule, et bituumenite omadusi hinnatakse pärast lühi- ja pikaajalist vanandamist. Käesoleva töö eesmärgiks oli uurida, millised oleksid Eesti teekatte temperatuurid lähtudes PG markide määramise põhimõtetest.

Käesoleva uuringu raames võrreldi erinevaid mudeleid, millega teekatte PG markide määramiseks vajalikke temperatuure arvutatakse. Tulemusena leiti, et erinevad arvutamise mudelid ei pruugi Eesti regioonis anda usaldusväärseid tulemusi. Selle puuduse vältimiseks töötati uuringu käigus välja kaks eraldiseisvat tee temperatuuride arvutusmudelit, et arvutada Eesti teede maksimaalseid ja minimaalseid temperatuure. Mudeli sisendina kasutatakse päevaseid maksimaalseid või minimaalseid õhutemperatuure. Mudelites saab arvesse võtta ka arvutussügavust teekatte pinna suhtes. Võrdlused reaalsete teeilmajaamade poolt registreeritud teekatete temperatuuridega näitasid, et uuringus välja töötatud mudelid annavad Eesti tingimustes täpsemaid tulemusi võrreldes teiste laialt kasutatavate mudelitega.

Uuringuga välja töötatud mudelitega ning PG margi määramise põhimõtete alusel arvutatud Eesti teekatete maksimaalsed temperatuurid jäävad regiooniti 52 °C ja 58 °C vahele. Teekatete minimaalsed PG margi temperatuurid varieeruvad peamiselt vahemikus –28 °C kuni –22 °C. Lokaalselt esineb ka piirkondi, kus peaks arvestama minimaalse PG margi temperatuuridega –16 °C ja –34 °C. Madalamad maksimaalsed ning kõrgemad minimaalsed teekatte PG margi temperatuurid esinevad peamiselt Lääne-Eesti rannikualadel ning saartel.

Täiendavalt analüüsiti käesoleva uuringuga ka kliimamuutuste mõju Eesti teekatete temperatuuridele lähi- kesk ning pikas perspektiivis. Tulemustest järeldus, et domineerivateks maksimaalseteks PG margi temperatuurideks jäävad kõigi vaadeldud kliimamuutuste stsenaariumite korral 52 °C ning 58 °C. Tulevikus kasvavad nende piirkondade alad, kus tuleks arvestada PG margiga 58 °C. Seevastu toimub suurem muutus teekatete minimaalsete temperatuuridega. Järgmiseks 20 aastaks teekatteid projekteerides on domineerivateks PG LT markideks –28 °C ja –22 °C ning lokaalsete piirkondadega, kus peaks olema ettenähtud PG mark –34 °C enam arvestama ei pea.

Vaadeldud kesk- ja pikas perspektiivis väheneb kliimamuutuste tõttu PG –28 °C piirkondade pindala ning selle arvab kasvab PG –22 °C alade pindala.

Eestis kasutatakse täna asfaltsegude tootmiseks peamiselt bituumeneid, mille penetratsioonimark on 70/100. Kuigi see mark täidab ära PG high temperature grade nõuded, võib, sõltuvalt bituumeni kvaliteedist, tekkida probleeme PG low temperature grade –28 °C nõuete täitmisega. Lähitulevikus võib piirkondades, mis eeldaksid PG 52-28 või PG 58-28 margi nõuetele vastavate bituumenite kasutamist, olla kasu penetratsioonimargiga 100/150 või 160/220 bituumenite kasutamisest teekatete kulumiskihtides.

Appendix 1

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SUPERPAVE PAVEMENT DESIGN TEMPERATURES IN ESTONIA

KARLI KONTSON*, KRISTJAN LILL, ANDRUS AAVIK

Dept of Civil Engineering and Architecture, Tallinn University of Technology, Tallinn, Estonia

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Abstract. This paper introduces the maximum and minimum pavement temperatures of Estonian asphalt pavements in accordance with calculation models developed for North America and Norway. Historical meteorological data from 1992 to 2021 obtained from 25 different weather stations in Estonia were used as an input for the respective models. Comparison between the calculation models demonstrated high variability of the pavement design temperatures, thus significantly impacting the bitumen grade selection. Based on the road weather stations data, the Norwegian and Canadian models provide the most accurate pavement temperature estimations for Estonian conditions. Calculated upper and lower-bound pavement design temperatures varied between +52 °C to +58 °C and −22 °C to −34 °C, respectively. All models showed milder pavement temperatures and lower seasonal temperature amplitudes for coastal and offshore areas. The results also indicated the importance of validating model suitability as well as correlation with actual pavement temperatures in the Baltic region.

Keywords: bitumen, pavement design, pavement temperature, penetration grading, performance grading, Superpave.

* Corresponding author. E-mail: karli.kontson@taltech.ee

Karli KONTSON (ORCID ID 0000-0002-4029-3245)
Kristjan LILL (ORCID ID 0000-0002-2095-806X)
Andrus AAVIK (ORCID ID 0000-0002-8672-0345)

Introduction and background

On average, asphalt mixtures consist of 94–96 wt% (aggregate) and 4–6 wt% (bitumen). Although the content of the bitumen in asphalt mixtures is seemingly low, it has a vital role in ensuring performance and durability of asphalt pavements. Bitumen is a thermoplastic and viscoelastic construction material and its in-service behaviour is greatly influenced by environmental and loading conditions, i.e., temperature and forces induced by traffic.

Bitumen owes its unique properties to a complex chemical composition, consisting mainly of hydrocarbons with a variable structure, molecular weight and polarity, heteroatoms such as oxygen, sulphur, nitrogen and trace metals (Lesueur, 2009).

The rheological behaviour of bitumen varies with respect to temperature. As temperatures increase, the viscosity of bitumen decreases and asphalt pavements can become susceptible to plastic deformation and rutting caused by traffic induced loads (see Figure 1) (Delgadillo & Bahia, 2010). At low temperatures, the viscosity of bitumen increases, causing a transition to a solid-elastic state which leads to a reduced strain tolerance and more brittle behaviour. This, combined with temperature shrinking inherent to asphalt pavements, can lead

Figure 1. Extreme case of asphalt rutting on E67 Pärnu bypass. Rutting occurred on completely new asphalt pavement soon after paving in summer 2012

Figure 2. Typical pavement low temperature cracking with secondary fatigue cracking adjacent to transverse cracking

to premature pavement cracking, known as thermal cracking or low temperature cracking (see Figure 2) (Hesp et al., 2009).

Both rutting and low temperature cracking are among the most relevant types of pavement failure. Rutting results in a reduced service life of the pavement and can also negatively affect the safety of road users' due to the presence of pooled water increasing the risk of hydroplaning or ice formation (EC, 1999). Low temperature cracking is one of the main types of pavement failures in cold regions (Hesp, 2003). Cracks in the pavement allow water to penetrate into the pavement structure and when combined with seasonal freeze-thaw cycles, often leads to progressive pavement failure thereby reducing the service life of roads (Dawson, 2009).

It has been estimated that approximately 30–40% of the rutting and more than 90% of low temperature cracking can be attributed to the properties of bitumen (Dawley & Pulles, 1996; Hesp et al., 2009; Saarela, 1992). To ensure the durability of asphalt pavements, the bitumen and its properties must be carefully selected to meet the environmental conditions of the pavement.

In Europe, both unmodified paving grade bitumen and modified bitumen properties are selected based on widely employed empirical specifications known as penetration grading (CEN, 2009, 2010). The penetration grade for a given road or region is usually selected by the designer or user agency based on previous experience with given grade. Penetration grading relies mainly on the penetration test which measures bitumen consistency at 25 °C. The test is conducted by measuring the penetration depth of a specific 100 g needle after 5 s loading time in decimillimetres (CEN, 2015). Penetration grading is often supplemented with additional tests, such as the Ring and Ball (R&B) softening point and Fraass breaking point to measure and specify bitumen properties at high and low temperatures, respectively. The latter is the only option in European bitumen product specifications to specify the low temperature performance of the bitumen and is therefore often adopted in cold regions such as the Baltic and Nordic countries. However, penetration and Fraass breaking point tests are not related with fundamental physical properties of the bitumen and have shown to have somewhat inaccurate correlation with in-service pavement performance (Kennedy et al., 1994; Lill et al., 2020). Criticism towards penetration grading became more dominant after the crude oil crisis in 1970s in the USA, when crude sources used for fuel and bitumen production diversified. This was perceived by road industry as leading cause of variable in-service performance between bitumen with similar penetration grade (Adams & Holmgreen, 1986; Petersen et al., 1993).

1. Superpave Performance Grading and pavement temperature models

In the late 1980s, the government of the USA initiated the Strategic Highway Research Program (SHRP) to increase the durability and performance of asphalt pavements. The SHRP researchers concluded that bitumen properties should be generally selected based on two aspects: 1) *traffic load characteristics*, i.e., traffic speed and equivalent single axle load (ESALs) and 2) *environmental conditions* (temperatures) of the road to be constructed or remediated. One of the many SHRP outcomes was Superpave™ (**Su**perior **Per**forming Asphalt **Pave**ments) bitumen specification framework known as Superpave Performance Grading currently widely used in North America as the AASHTO M 320 specification. This adopted a completely new approach to specify bitumen. Penetration, softening point and Fraass breaking point tests were replaced with testing methods having a better correlation with pavement in-service performance (Kennedy et al., 1994).

Environmental conditions were divided into two criteria: 1) pavement high design temperature and 2) pavement low design temperature. Pavement high and low design temperatures refer to maximum and minimum pavement temperatures considered in selecting the appropriate bitumen grade. According to the Superpave Performance Grading, bitumen grades are denoted as "PG HT-LT", where "PG" stands for *Performance Grade* and "HT-LT" stands for *high temperature grade* and *low temperature grade,* respectively. PG grade indicates the range of temperatures, also known as Useful Temperature Interval (UTI), in which the bitumen is assumed to provide adequate resistance to both environmental and loading conditions. For example, bitumen with a grade PG 58-28, indicates that this bitumen is suitable to be used in asphalt pavements where the maximum and minimum pavement temperatures (pavement design temperatures) remain within the range: +58 °C to −28 °C. Both HT and LT grades are defined with 6 ºC increments. It is important to note that PG HT is determined by testing the short-term aged or unaged bitumen. Whereas PG LT is determined based on tests conducted on long-term aged bitumen (Asphalt Institute, 2011).

The lower-bound pavement design temperature, PG LT, was estimated to be equal to lowest 1-day minimum air temperature of the region (Kennedy et al., 1994). In this paper, this temperature boundary is denoted as SHRP PG LT.

$$\text{SHRP PG HT} = (T_{air} - 0.00618\varphi^2 + 0.2289\varphi + 42.2)\times 0.9545 - 17.78, \qquad (1)$$

where SHRP PG HT – pavement high design temperature at the depth of 20 mm, °C;
T_{air} – 7-day average high air temperature, °C;
φ – latitude of the specific road section under design or examination, °.

Both the maximum and minimum design temperature methodology was later reviewed based on factual data collected from 30 road test sections throughout North America, based on monitoring data as a part of the Seasonal Monitoring Program (LTPP-SMP) of the Long-Term Pavement Performance study. New pavement design temperature calculation models according to Equations (2) and (3) were proposed (Asphalt Institute, 2011; Mohseni, 1998). In this paper, these temperatures are denoted as LTPP PG HT and LTPP PG LT, respectively.

$$\text{LTPP PG HT} = \\ = 54.32 + 0.78T_{air} - 0.0025\varphi^2 - 15.14\log_{10}\left(H+25\right) + z\sqrt{9+0.61\sigma_{air}{}^2}, \qquad (2)$$

where LTPP PG HT – pavement high design temperature below surface, °C;
T_{air} – 7-day average high air temperature, °C;
φ – latitude of the specific road section under design or examination, °;
H – depth to surface, mm (for PG HT calculations, H = 20 mm is adopted);
σ_{air} – standard deviation of the 7-day average high air temperatures;
z – from the standard normal distribution table, z = 2.055 for 98% reliability.

$$\text{LTPP PG LT} = \\ = -1.56 + 0.72T_{air} - 0.004\varphi^2 + 6.26\log_{10}\left(H+25\right) - z\sqrt{4.4+0.52\sigma_{air}{}^2}, \qquad (3)$$

where LTPP PG LT – pavement low design temperature below surface, °C;
T_{air} – 1-day average low air temperature, °C;
φ – latitude of the specific road section under design or examination, °;
H – depth to surface (for PG LT calculations, H = 0 mm is adopted), mm;
σ_{air} – standard deviation of the 1-day low air temperatures;
z – from the standard normal distribution table, z = 2.055 for 98% reliability.

In Canada, a separate minimum pavement design temperature methodology was derived during the Canadian Strategic Highway

Research Program (C-SHRP). Pavement temperature measurements compared with air temperature indicated that SHRP PG LT assumption would lead to an overly conservative minimum pavement design temperature. Therefore, a separate Equation (4) was adopted, which gave reasonable correlation with real pavement low temperatures (Mohseni, 1998). In this paper, this temperature is denoted as C-SHRP PG LT.

$$\text{C-SHRP PG LT} = 0.859T_{air} + 1.7, \tag{4}$$

where C-SHRP PG LT – pavement low design temperature, °C;

T_{air} – 1-day minimum air temperature, °C.

From 1994 to 1998, the Norwegian road industry conducted a series of studies to adopt the Superpave principles in Norway. One of the products of these studies was the further development of the maximum pavement design temperature calculation methodology according to Equation (5), which is more compatible with environmental conditions for Norwegian pavements. In this paper, this temperature is denoted as NOR PG HT. The PG LT calculation method for Norway was kept the same as proposed by C-SHRP (Lerfald et al., 2004; Vegvesen, 2014).

$$\text{NOR PG HT} = T_{20\text{mm}} = \left(T_{\text{air}} - 0.0055\varphi^2 + 0.15\varphi + 36\right)0.9545 - 0.8, \tag{5}$$

where $T_{20\text{mm}}$ – pavement high design temperature (PG HT) at depth of 20 mm, °C;

T_{air} – 7-day average high air temperature, °C;

φ – latitude of the specific road section under design or examination, °.

The Superpave Performance Grading enables users to adopt different levels of confidence (reliability) by considering the standard deviations of the *7-day average high air temperatures* and *1-day minimum air temperatures* within the measured period. At least 20 years of air temperature data are needed to ensure reliable results (Asphalt Institute, 1996, 2011). According to the Asphalt Institute (1996), Superpave defines reliability as the "*percent probability in a single year that the actual temperature (7-day average high temperature or 1-day minimum air temperature) will not exceed the design temperatures*". For example, the mean 7-day average high air temperature in Tallinn, Estonia for a period between 1992 and 2021 was T_{air} = 26.8 °C with a standard deviation of ±2.5 °C. However, there is a 50% chance that mean 7-day average high air temperature will be exceeded in an average year. Assuming a normal distribution as well as the standard deviation of the mean 7-day average high air temperature, there is a 2% chance that 7-day average high temperature will exceed 26.8 + 2 × 2.5 = 31.8 °C.

Equations (2) and (3) of LTPP PG HT and PG LT design temperature have already included an option for reliability.

2. Objective

Currently, there is no information available in literature about bitumen Superpave Performance Grades suitable for Estonia and which calculation model provides the most accurate correlation with measured road surface temperatures. The objective of this paper is to calculate pavement design temperatures of Estonia according to models officially adopted in North America (LTPP and C-SHRP) and Norway, to compare the calculated temperatures with the highest and lowest pavement temperatures measured at randomly selected road weather stations and to select the most accurate calculation models for Estonia. The results are then used to develop pavement design temperature maps that can be used to select bitumen Superpave Performance Grades in different parts of Estonia.

3. Geographical location and climate

Estonia is located in Norther-Eastern Europe, bordering the Baltic Sea to the West and Gulf of Finland to the North. The total area of Estonia is 45 339 km^2 and the geographical coordinates range from 57°30’ N to 59°49’ N and from 28°13’ E to 21°46’ E. The country is situated in temperate climate zone and is influenced by both maritime and continental climate. The average annual air temperature (1991–2020) is 6.4 °C with the absolute maximum and minimum recorded air temperatures of 35.6 °C (11 August 1992) and −43.5 °C (17 January 1940), respectively. The warmest months are July–August and the coldest are January–February for all years (Estonian Environmental Agency, 2021).

4. Temperature data and analysis

4.1. Historical meteorological data

Recorded daily minimum and maximum air temperatures for 37 different weather stations were provided by the Estonian Environmental Agency. The period included in this study is from 1992 to 2021 (30 years). Therefore, weather stations with less than 30 years of daily temperature data were excluded from the analysis. This reduced the total number of suitable stations to 25. The global temperature data density resolution is one station per approximately 1800 km^2, the stations are fairly evenly distributed over the territory of Estonia (Figure 3).

For every weather station, the annual highest 7-day average air temperatures and lowest 1-day minimum air temperatures (incl. dates) were extracted for the PG HT and LT calculations, respectively. The daily minimum and maximum air temperature data from every station were subjected to verification to ensure that provided information was sufficient to be included in the PG HT and PG LT calculations. The most critical period for the pavement PG HT calculations was selected from June to September (incl.) and for the PG LT calculations from October to March (incl.). Additionally, the standard deviation of the 7-day average maximum air temperature and 1-day average low temperature was calculated.

In the rare cases where the daily minimum and maximum temperatures were missing for a limited period e.g., for a week or a month, the measurements from the nearby weather stations were analysed to assess the sensitivity of the PG HT and LT calculations due

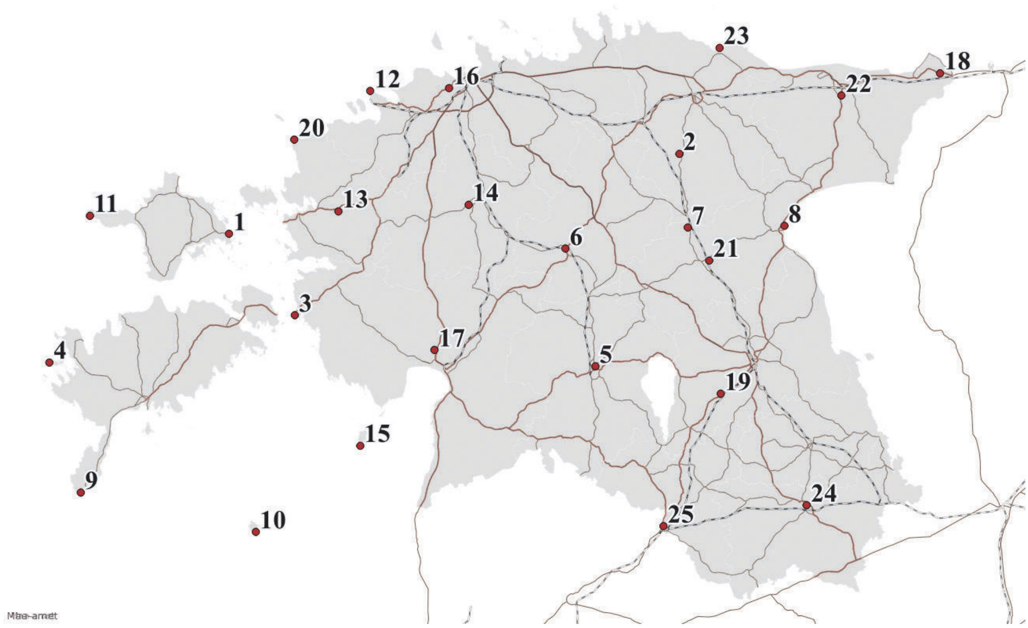

Figure 3. Locations and numbering of the weather stations included in the study

to the missing data. For example, if a weather station was missing the daily minimum air temperatures for a period between 1 December to 15 December and a nearby weather station indicated that the coldest temperatures for given year did not overlap with the period of missing data, then the data from the weather station were still included in the analysis.

4.2. Data from the road weather stations

Daily minimum and maximum recorded road surface and air temperatures from 2020 to 2022 from seven randomly selected road weather stations were collected to allow for comparison with the calculated pavement design temperatures. The highest 7-day average air temperatures and lowest 1-day minimum air temperatures leading to maximum and minimum recorded surface temperatures were extracted for the PG HT and LT calculations, respectively. Road weather stations included in this paper are described in Figure 4.

Figure 4. Locations and numbering of the road weather stations included in the study

5. Results and discussion

5.1. Calculated pavement design temperatures

Pavement high design temperatures (PG HT) were calculated based on LTPP (Equation (2)) and the Norwegian (Equation (5)) models. Pavement low design temperatures (PG LT) were calculated using LTPP (Equation (3)) and C-SHRP (Equation (4)) models. SHRP PG HT and LT results were included for comparison purposes only. All calculations accounted for reliability level of at least 98%. PG HT and LT calculation results have been rounded to the nearest whole number and are presented in Figures 5 and 6.

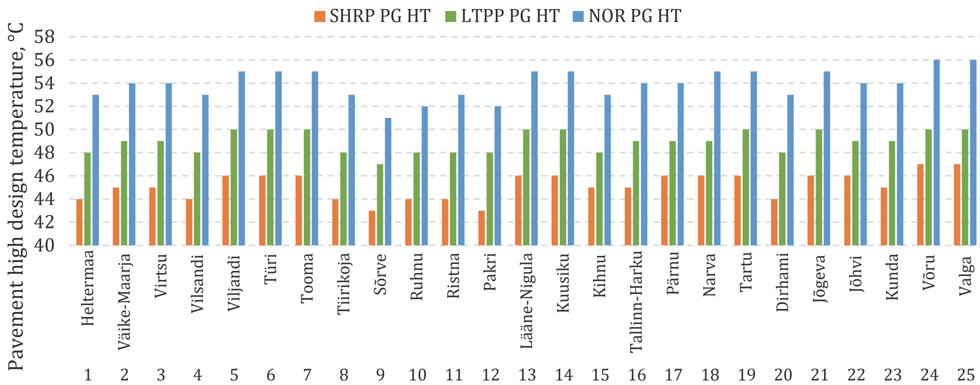

Figure 5. High pavement design temperatures according to SHRP, LTPP and Norwegian calculation models (98% reliability)

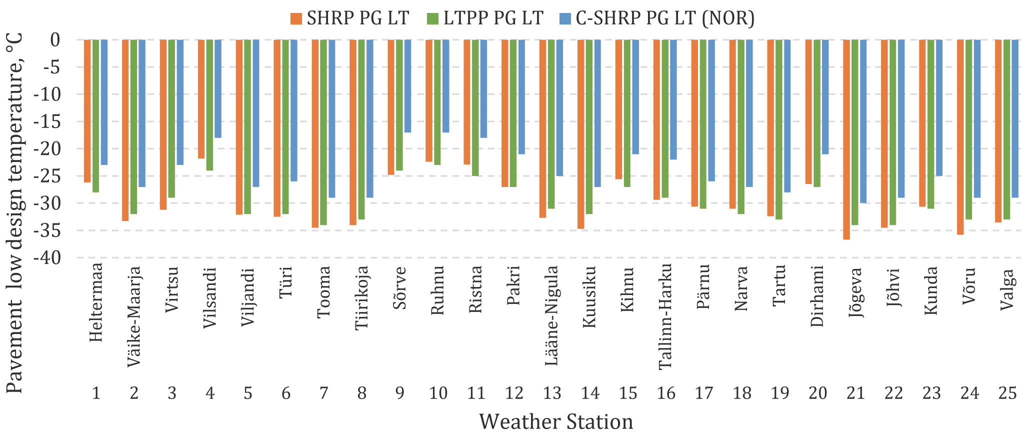

Figure 6. Low pavement low design temperatures according to LTPP and C-SHRP calculation models (98% reliability). SHRP temperature given based on lowest 1-day air temperature

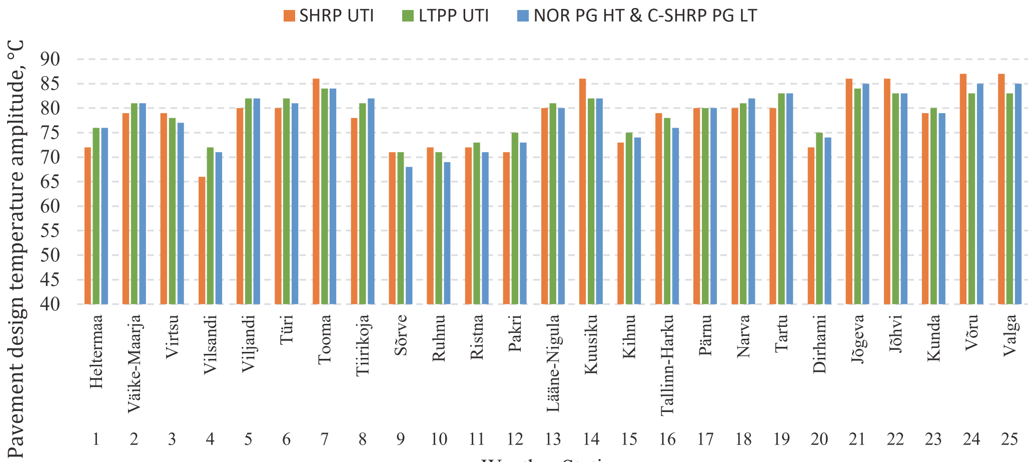

Figure 7. Absolute pavement design temperature differences based on respective calculation models

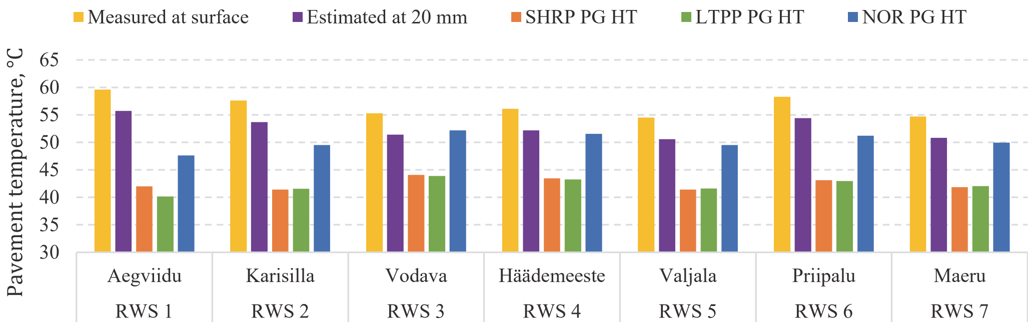

Figure 8. Highest pavement temperatures recorded at road weather stations compared with calculated temperatures

5.2. Calculated vs measured pavement temperatures

The highest recorded road surface temperatures and calculated pavement high design temperatures for every included road weather station with respective calculation models are presented in Figure 8. Pavement temperature at the depth of 20 mm was assumed to be 4 °C lower from surface temperature. This assumption is based on temperature-depth relationship adopted in Equation (2). Norwegian model shows the closest results with estimated temperatures at the depth of 20 mm. SHRP and LTPP models significantly underestimate pavement high temperatures and are not suitable for Estonian conditions.

The lowest recorded road surface temperatures and calculated pavement low design temperatures with respective models are presented in Figure 9. C-SHRP model provides reasonable pavement low temperature estimation compared with recorded temperatures. SHRP and LTPP models lead to significantly lower pavement temperature estimates compared with measured values, resulting in too conservative pavement low design temperatures and would significantly affect the Superpave bitumen low temperature grade selection.

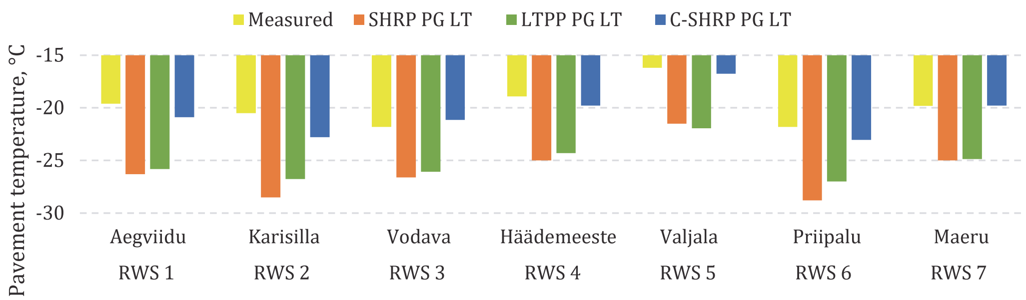

Figure 9. Lowest pavement temperatures from 2020 to 2022 recorded at road weather stations compared with calculated temperatures

5.3. Superpave pavement design temperature maps of Estonia

The pavement maximum and minimum temperature maps were developed based on Norwegian and C-SHRP models, respectively. According to the Superpave methodology, the bitumen grade is specified with 6 °C increments. Required bitumen PG HT and LT grades according to respective models with the aforementioned increments is displayed in Figure 10.

Pavement high design temperatures according to Norwegian model varies from 52 °C to 58 °C. Whereas pavement low design temperatures according to C-SHRP model varies from −22 °C to −34 °C. Results indicate milder pavement design temperatures for islands and coastal areas, i.e., colder pavement temperatures are expected inland, while milder temperatures are dominant near coastal areas and on islands.

Conclusions

This paper introduced maximum and minimum pavement temperatures for Estonian asphalt pavements in accordance with Superpave Performance Grading principles. Historical meteorological data from 1992 to 2021 obtained from 25 different weather stations in Estonia were used as an input for the respective pavement design temperature models. Different calculation models demonstrated variability of the pavement design temperatures, thus impacting the bitumen Superpave Performance Grade selection. Data from road weather stations indicate that LTPP models are unsuitable for Estonian conditions, leading to significantly underestimating maximum pavement

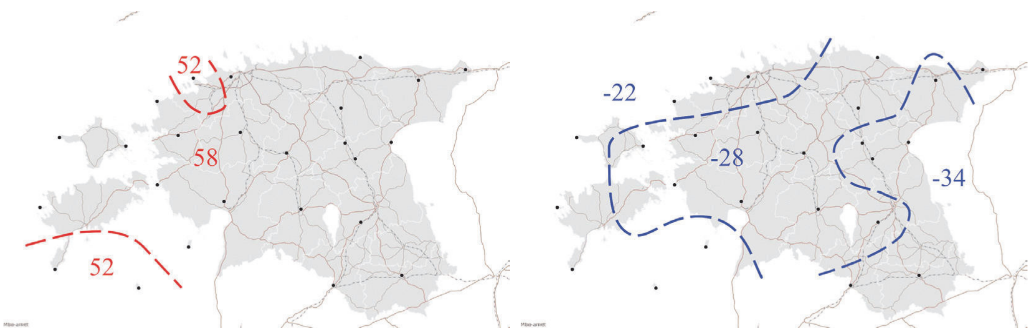

Figure 10. PG HT and LT grades according to the Norwegian PG HT and C-SHRP models with 6 °C increments (98% reliability)

temperatures and overestimating minimum pavement temperatures. Norwegian and C-SHRP models provide reasonable correlation with recorded pavement temperatures in Estonia and are therefore recommended to be used for pavement design temperature calculations. All included models indicate that milder pavement temperatures and smaller seasonal temperature amplitudes are expected for coastal areas and islands.

Estonian Superpave Performance Grades, according to Norwegian and C-SHRP models (≥98% reliability), are as follows:

- Pavement high design temperature (PG HT) varies from 52 °C to 58 °C.
- Pavement low design temperature (PG LT) varies from −22 °C for coastal areas and islands bordering the Baltic Sea and −28 °C to −34 °C for mainland area.

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Appendix 2

Publication II

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NOTE

Statistical-empirical pavement temperature prediction models based on data from road weather stations in Estonia

Karli Kontson, Kristjan Lill and Andrus Aavik

Tallinn University of Technology, Department of Civil Engineering and Architecture, Road Engineering and Geodesy Research Group, Tallinn, Estonia

ABSTRACT

Asphalt pavement properties are greatly affected by seasonal temperature variation. Binder grade selection needs to be aligned with forecasted operational pavement temperatures to maximise pavement longevity. Different widely adopted models for determining binder Superpave Performance Grading have been shown to provide inaccurate results for the Nordic-Baltic region, leading to under- or overestimation of operational pavement temperatures and erroneous selection of a suitable binder grade. This study proposes empirically formulated pavement temperature prediction models for Estonia based on temperature data collected from six different road weather stations in Estonia from 2019 to 2023. Models were derived using measurements from daily maximum and minimum air temperatures combined with pavement temperature profiles with depth.

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Asphalt binder; pavement design; pavement temperature; penetration grading; performance grading; Superpave

1. Introduction and background

Asphalt pavements consist mainly of mineral aggregates and binder. The latter component contributes to the bulk pavement behaviour and is highly temperature dependent. It has been estimated that up to 40% of the pavement rutting can be attributed to binder properties (Dawley & Pulles, 1996; Partl et al., 2013; Saarela, 1992) and up to 90% of the low-temperature cracking is caused by binder properties (Dawley & Pulles, 1996; Hesp, 2003). Incorrect binder selection reduces the expected working life and safety of the roads while increasing maintenance costs for the clients and taxpayers (EC, 1999; Hesp et al., 2009; Rys et al., 2017; Underwood et al., 2017; Yee et al., 2006).

In Europe, asphalt binder grades are determined based on empirical specifications known as penetration grading (CEN, 2009, 2010; Lesueur, 2009). This has been shown to have an inaccurate correlation with in-service pavement performance (Adams & Holmgreen, 1986; Kennedy et al., 1994; Lill et al., 2020; Petersen et al., 1993).

In the late 1980s, a dedicated research program known as the Strategic Highway Research Program (SHRP) was initiated in the USA to increase the durability and performance of asphalt pavements. The SHRP introduced Superpave™ (**Su**perior **Per**forming Asphalt **Pave**ments) binder specification framework known as Superpave Performance Grading currently widely used in North America as the AASHTO M 320 specification. According to the Superpave Performance Grading, binder grades are denoted as 'PG HT-LT', where 'PG' stands for *Performance Grade* and 'HT' stands for *high temperature grade* and 'LT' stands for *low temperature grade*. PG grade refers to the range of temperatures in which the binder is assumed to show adequate performance against both environmental and loading conditions. For example, 'PG 58-28', indicates that this binder is suitable to be used in asphalt pavements

CONTACT Karli Kontson karli.kontson@taltech.ee

where the maximum and minimum pavement temperatures remain within the range of +58°C to −28°C. PG grades are defined with 6 °C increments. PG HT is determined based on tests conducted on short-term aged or unaged binders. Whereas PG LT is determined by testing long-term aged binders (Asphalt Institute, 1996, 2011; Kennedy et al., 1994).

The required PG pavement temperatures for a specific road or section are calculated based on historical air temperature data. The initial SHRP introduced dedicated models to estimate maximum pavement temperature at a predetermined depth based on daily maximum air temperature. The minimum pavement temperature was assumed to be equal to the minimum air temperature (Kennedy et al., 1994). However, the SHRP models and assumptions led to under- and overestimating the in-service temperatures and were soon revised based on monitoring data from 30 road test sections throughout North America as a part of the Seasonal Monitoring Program of the Long-Term Pavement Performance study (LTPP-SMP). As a result, updated statistical-empirical pavement temperature prediction models (Equation 1 and Equation 2) were proposed by the researchers (Asphalt Institute, 2011; Mohseni, 1998).

$$T_{Pav,max} = 54,32 + 0,78 \times T_{Air,max} - 0,0025 \times Lat^2 - 15,14 \times LOG_{10}(H + 25) \tag{1}$$

Where:
$T_{Pav,max}$ – maximum pavement temperature at depth H, °C;
$T_{Air,max}$ – daily maximum air temperature, °C;
Lat – latitude of the specific road section, °;
H – depth from pavement surface, mm.

$$T_{Pav,min} = -1,56 + 0,72 \times T_{Air,min} - 0,004 \times Lat^2 + 6,26 \times LOG_{10}(H + 25) \tag{2}$$

Where:
$T_{Pav,min}$ – minimum pavement temperature at depth H, °C;
$T_{Air,min}$ – daily minimum air temperature, °C;
Lat – latitude of the specific road section, °;
H – depth from the pavement surface, mm.

LTPP-SMP models were developed based on data collected from various road sections ranging between latitudes 27° to 52°(Mohseni, 1998). It has been highlighted by different studies that these equations can be unsuitable for regions outside of these latitude boundaries, e.g. Canada, Baltic and Nordic countries (Kontson et al., 2023; Swarna et al., 2023). Considering the environmental conditions in Estonia, the LTPP-SMP model underestimates pavement maximum temperature by approximately 10°C and overestimates the pavement minimum temperature by more than 4°C (Kontson et al., 2023).

A separate minimum pavement temperature model (Equation 3) was proposed for Canadian conditions based on empirical data collected during the Canadian Strategic Highway Research Program (C-SHRP). This model displayed a good correlation to local minimum pavement temperature measurements (Mohseni, 1998).

$$T_{Pav,min} = 0,859 \times T_{Air,min} + (0,002 - 0,0007 \times T_{Air,min}) \times H + 1,7 \tag{3}$$

Where:
$T_{Pav,min}$ – minimum pavement temperature at depth H, °C;
$T_{Air,max}$ – daily minimum air temperature, °C;
H – depth from the pavement surface, mm.

When Norway investigated the adoption of the Superpave binder grading principles, they conducted a series of studies in the mid-1990s. This led to a further development of the maximum pavement temperature calculation methodology, more suited to Norwegian environmental conditions (Equation 4). In Norway, C-SHRP minimum pavement temperature prediction models are used (Lerfald et al., 2004; Vegvesen, 2014).

When applying this model to Estonian environmental conditions, a better correlation was obtained compared to the LTPP model (Kontson et al., 2023).

$$T_{Pav,max} = (T_{Air} - 0,0055 \times Lat^2 + 0,15 \times Lat + 36) \times 0,9545 - 0,8 \quad (4)$$

Where:
$T_{Pav,max}$ – maximum pavement temperature at 20 mm depth, °C;
$T_{Air,max}$ – 7-day average maximum air temperature, °C;
Lat – latitude of the specific road section, °.

Although the model is a better fit for Estonian environmental conditions, the Norwegian model lacks the means to account for varying depths from the pavement surface. The model is only suitable for calculating maximum pavement temperatures at 20 mm from the surface to determine pavement surface course binder grade as per Superpave criteria. The model does not include a depth variable to estimate pavement temperatures for lower asphalt layers, e.g. the binder and base course.

2. Objective

The main objective of this study is to develop statistical-empirical pavement temperature prediction models that could be used for estimating Superpave design temperatures in Estonia. Pavement temperature prediction models were developed based on air and pavement temperature data measured from six different Road Weather Stations (RWS) in Estonia. Three stations with the highest and three with the lowest pavement temperatures were selected for the empirical model derivation.

3. Geographical location and climate

Estonia is located in Northeastern Europe, bordering the Baltic Sea and Gulf of Finland. The geographical coordinates range from 57°30′ N to 59°49′ N and from 28°13′ E to 21°46′ E. The climate zone is temperate and is influenced by maritime and continental climate. The average annual air temperature is 6.4°C with the absolute maximum and minimum recorded air temperatures of 35.6°C (11 August 1992) and −43.5°C (17 January 1940), respectively. The warmest months are July–August and the coldest are January–February for all years (Estonian Environmental Agency, 2021).

4. Methodology

Different types of pavement temperature measurement sensors are used on Estonian roads, but many of these are aimed at winter road maintenance and therefore have limited measurement capacity in terms of the absolute maximum and minimum pavement temperatures. Recently, Vaisala DSR Road & Runway pavement temperature sensors with a wider temperature measurement range have been installed in Estonia which are capable of measuring temperatures reliably from −40°C to +80°C. This study focused on the RWS equipped with air temperature and the Vaisala pavement temperature sensors. The sensors are equipped with two PT-100 elements and measure pavement surface temperature and the temperature within the pavement at a depth of 60 mm from the surface, denoted as $H = 0\,mm$ and $H = 60\,mm$, respectively.

The adopted RWS for the maximum pavement temperature prediction models is located in Käru, Aegviidu, and Mõisaküla, and in Vodava, Kauksi, and Sõmeru for the minimum pavement temperature prediction models respectively. The data ranged between December 2019 to February 2023. The locations of the respective RWS are indicated in Figure 1.

The data are stored at a 10-minute interval to a Structured Query Language (SQL) database maintained by the Estonian Transport Administration. Air temperature, pavement surface temperature (H = 0 mm) and temperature 60 mm from surface (H = 60 mm) were converted to daily maximum and minimum temperatures for use in developing the air-pavement temperature regression

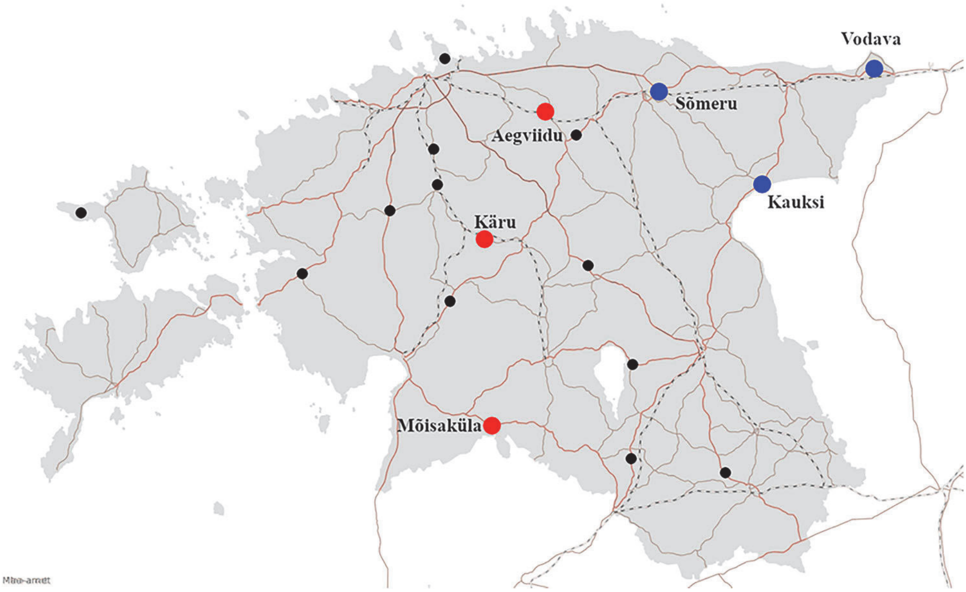

Figure 1. Locations of the six Road Weather Stations used for pavement and air temperature analysis; red – stations used for the maximum temperature model, blue – stations used for minimum temperature model, black – stations used for field validation.

models. A multivariate regression analysis was carried out in MS Excel using the Data Analysis feature allowing to take into account air temperature and pavement temperature at variable depths in the model.

5. Results

5.1. Pavement high-temperature regression model

The maximum daily air and pavement temperatures from June to September for 2020, 2021 and 2022 for the Käru, Aegviidu and Mõisaküla RWS were extracted from the dataset (N = 1094). Daily pavement temperatures for both the surface and at a depth of 60 mm are correlated against maximum daily air temperatures, presented in Figures 2 and 3 respectively. Maximum pavement temperature at 60 mm depth vs maximum pavement surface temperature is also presented in Figure 4.

There is a good relationship between the measured maximum pavement temperatures vs. maximum air temperatures, $R^2 = 0.79$ and 0.85 respectively. A very good correlation was observed for maximum pavement temperatures at 60 mm depth vs. maximum pavement surface temperatures ($R^2 = 0.96$).

A multivariate regression model was established (Equation 5) using the maximum daily air temperatures, as well as the pavement surface and depth temperature measurements (H = 0 and H = 60 mm) as input variables.

Log10(H + 25) was considered as the depth variable in this study since this has proven to be the most suitable predictor in previous pavement temperature correlation studies (Mohseni, 1998; Swarna et al., 2023). The regression model given in Equation 5 is proposed to act as a tool to predict the maximum pavement temperatures at predetermined depths using daily maximum air temperatures as the input. The regression model is based on N = 2188 observations and has an adjusted coefficient of determination $R^2 = 0.84$ and a Standard Error (SE) of ±4.3°C.

$$T_{Pav,max} = 1,6302 \times T_{Air,max} - 16,8975 \times LOG_{10}(H + 25) + 27,8947 \tag{5}$$

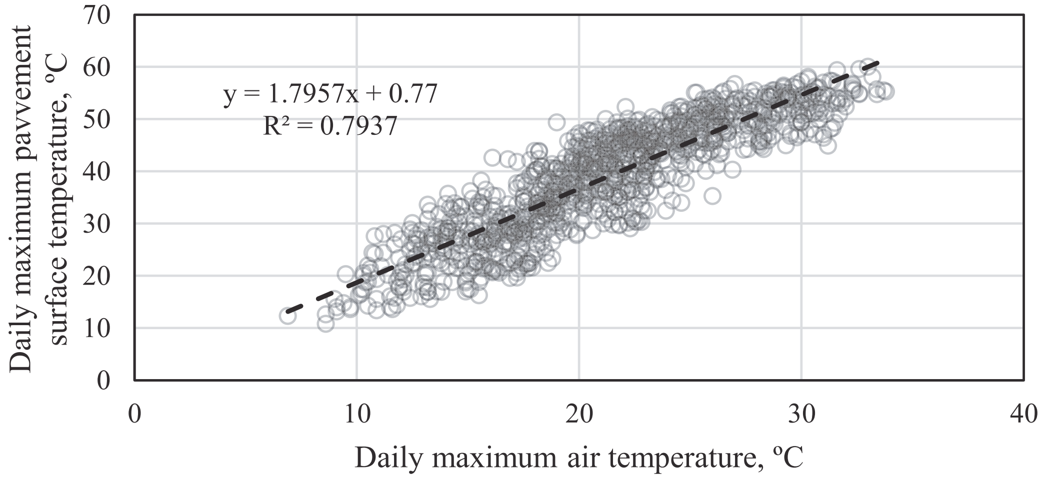

Figure 2. Relationship between daily maximum pavement surface temperatures and daily maximum air temperatures.

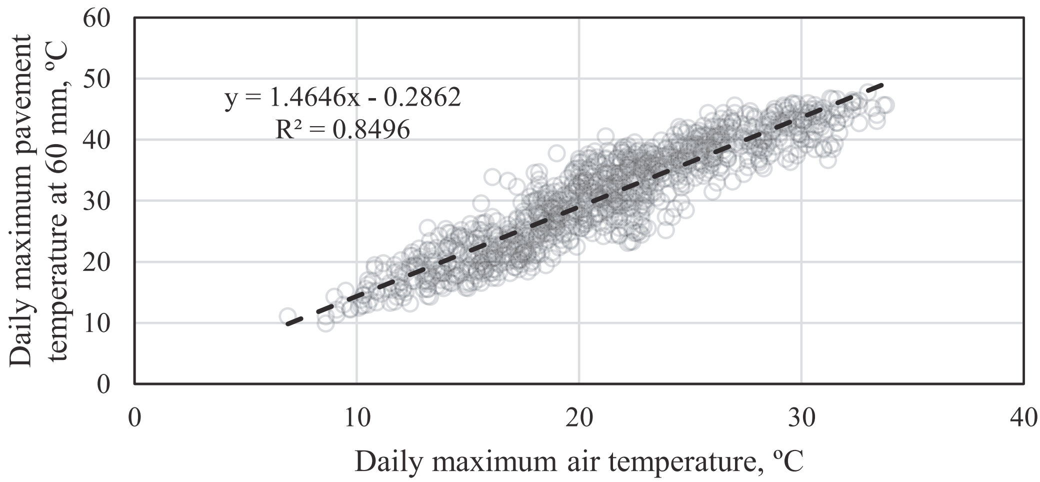

Figure 3. Relationship between daily maximum pavement temperatures at 60 mm depth and daily maximum air temperatures.

Where:
$T_{Pav,max}$ – maximum pavement temperature at depth H, °C;
$T_{Air,max}$ – daily maximum air temperature, °C;
H – depth from the pavement surface, mm.

5.2. Pavement low-temperature regression model

Minimum daily air and pavement temperatures from December to February 2019 to 2023 for the Kauksi, Vodava and Sõmeru RWS were extracted from the dataset (N = 753). Daily pavement temperatures for both the surface and at a depth of 60 mm are correlated against minimum daily air temperatures, presented in Figures 5 and 6 respectively. Minimum pavement temperature at 60 mm depth vs. minimum pavement surface temperature is also presented in Figure 7.

There is a good relationship between the measured minimum pavement temperatures vs. minimum air temperatures, $R^2 = 0.95$ and 0.92 respectively. A very good correlation was again observed

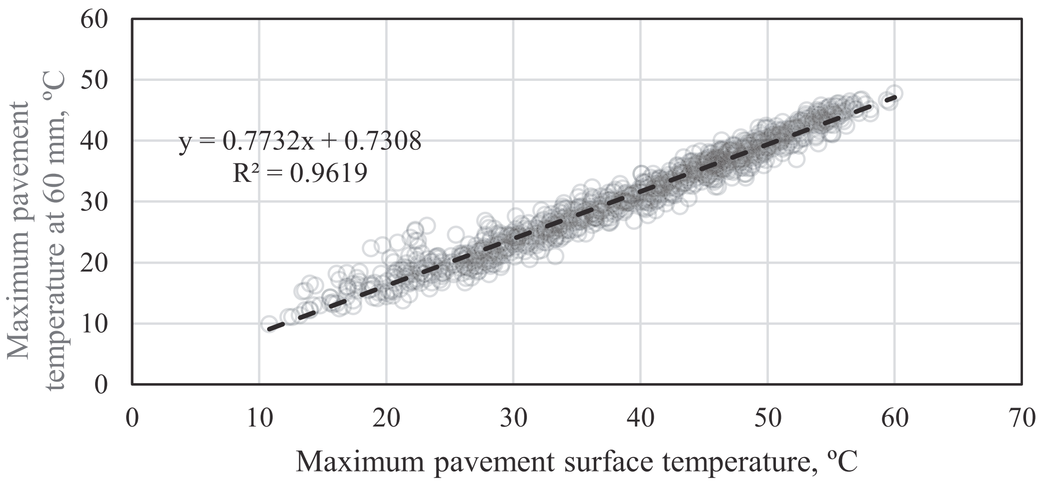

Figure 4. Relationship between daily maximum pavement temperatures at 60 mm depth and maximum daily pavement surface temperatures.

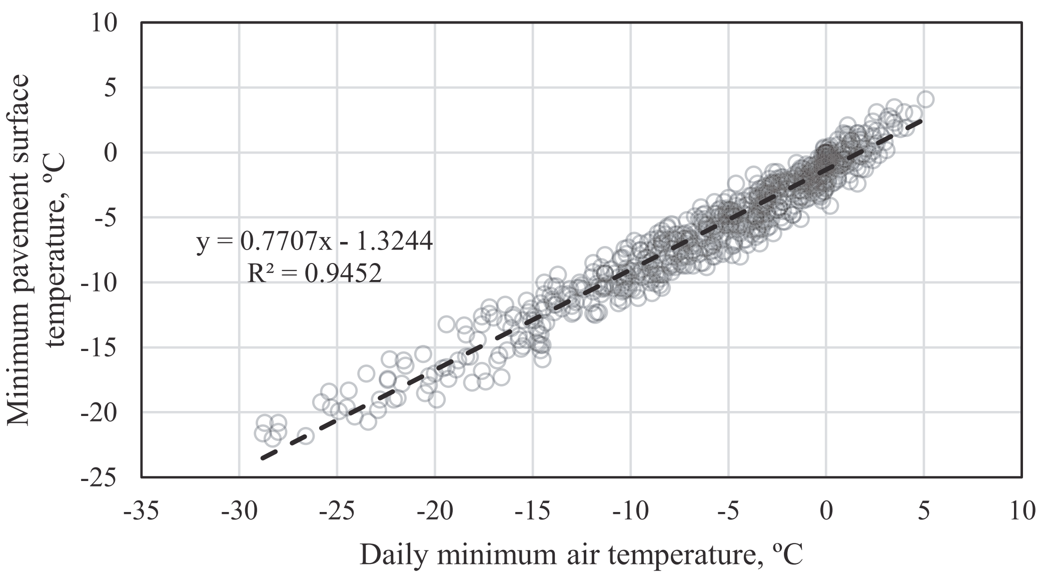

Figure 5. Relationship between daily minimum pavement surface temperatures and daily minimum air temperatures.

for the minimum pavement temperatures at 60 mm depth vs. minimum pavement surface temperatures ($R^2 = 0.97$).

A multivariate regression model (Equation 6) was established using the minimum daily air temperatures, as well as the pavement surface and depth temperature measurements ($H = 0$ and $H = 60$ mm) as input variables. A $\text{Log10}(H + 25)$ was used as a depth variable similar to the maximum pavement temperature prediction model.

The regression model presented in Equation 6 is proposed to act as a tool to predict the minimum pavement temperatures at predetermined depths using daily minimum air temperatures as the input. The regression model is based on $N = 1474$ observations and has an adjusted coefficient of

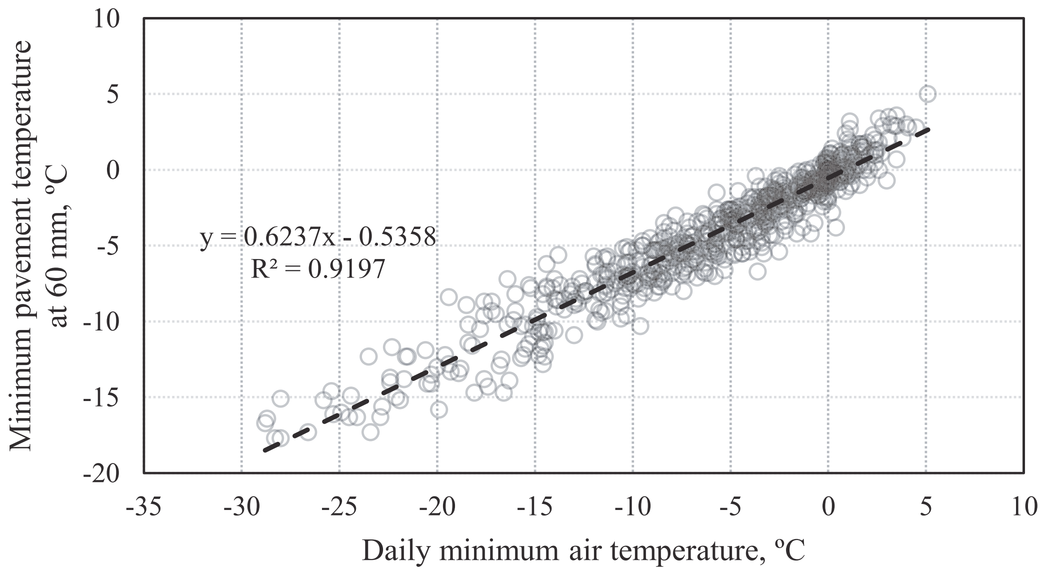

Figure 6. Relationship between daily minimum pavement temperatures at 60 mm depth and daily minimum air temperatures.

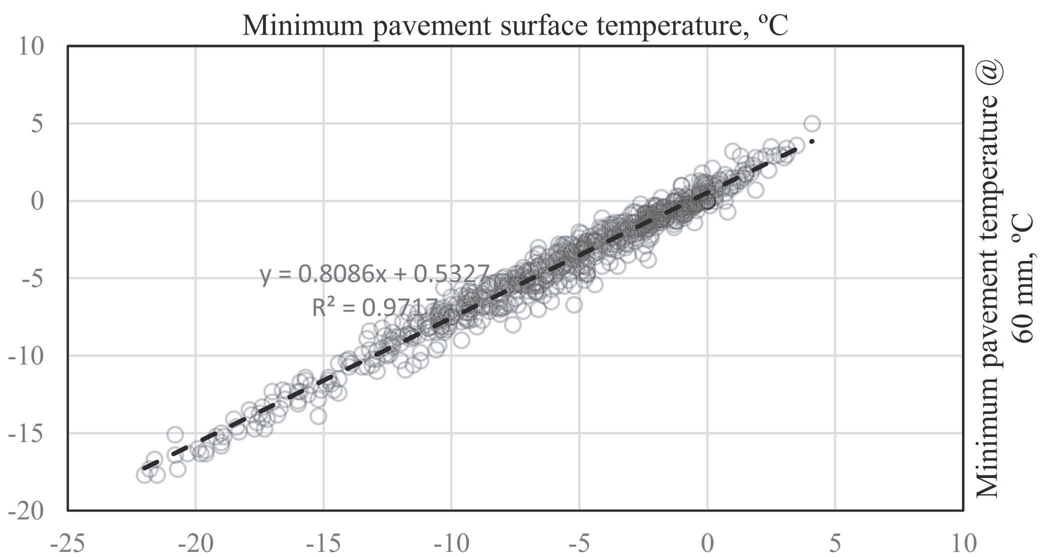

Figure 7. Relationship between daily minimum pavement temperatures at 60 mm depth and minimum daily pavement surface temperatures.

determination $R^2 = 0.93$ and a Standard Error (SE) of ± 1.3°C.

$$T_{Pav,min} = 0,6944 \times T_{Air,min} + 3,4507 \times LOG_{10}(H + 25) - 6,6132 \tag{6}$$

Where:
$T_{Pav,min}$ – minimum pavement temperature at depth H, °C;
$T_{Air,min}$ – daily minimum air temperature, °C;
H – depth from the pavement surface, mm.

6. Field validation

Calculated pavement temperatures at the surface and at the depth of 60 mm were compared with the maximum and minimum pavement surface temperatures from randomly selected 11 RWS throughout Estonia equipped with Vaisala pavement temperature sensors. RWS with at least 3 years of consistent pavement temperature data were included. The location of every included station for field validation is given in Figure 1.

The difference between the measured and the calculated maximum pavement temperatures are given in Figures 8 and 9. The largest observed discrepancy for surface and at the depth of 60 mm is 3.1°C (Pärnamäe RWS) and 4.2°C (Priipalu RWS), respectively. The absolute average discrepancy of the model for the surface and at the depth of 60 mm is 1.6 and 2.7°C, respectively.

Similarly, the comparative data for the minimum pavement surface temperatures is given in Figures 10 and 11, respectively. The largest observed discrepancy for the surface and at the depth of 60 mm is 2.9°C (Pärnamäe RWS) and −2.6°C (Priipalu RWS). The absolute average discrepancy of the model for the surface and 60 mm from the surface is 1.0°C and 2.1°C, respectively.

The empirical upper- and lower-bound road surface temperature models derived using Estonian-specific RWS data fit closely to the measured RWS values and are more accurate compared to LTPP and C-SHRP models. Therefore, road surface temperature predictions can be made with reasonable accuracy within the latitudinal extent of Estonia using the proposed correlation.

However, the model is developed by estimating that one of the temperatures is registered at 60 mm from the pavement surface. The true distance between the surface and the temperature sensor could

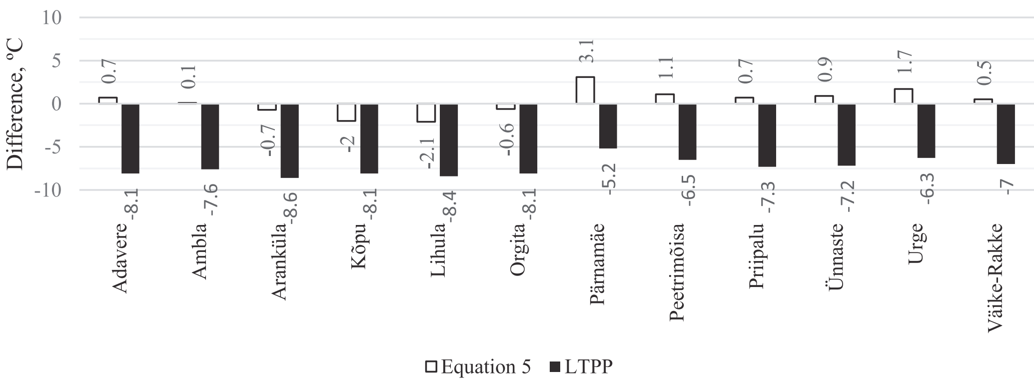

Figure 8. Maximum difference between measured and predicted maximum pavement temperatures (H = 0 mm).

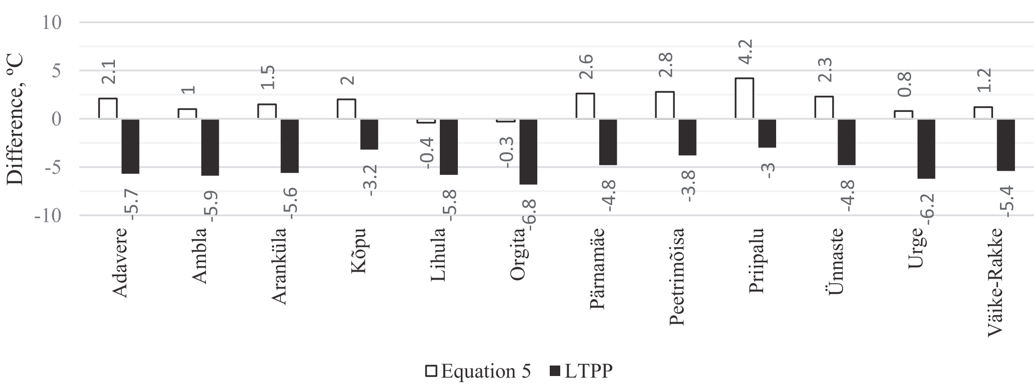

Figure 9. Maximum difference between measured and predicted maximum pavement temperatures (H = 60 mm).

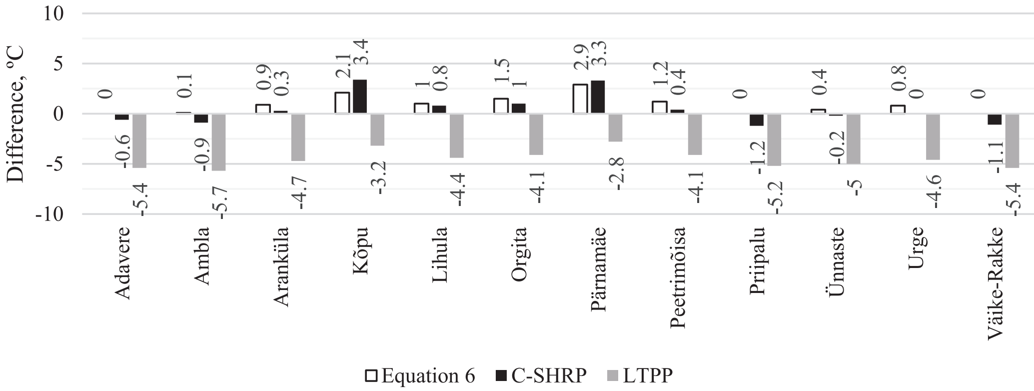

Figure 10. Maximum difference between measured and predicted minimum pavement temperatures (H = 0 mm).

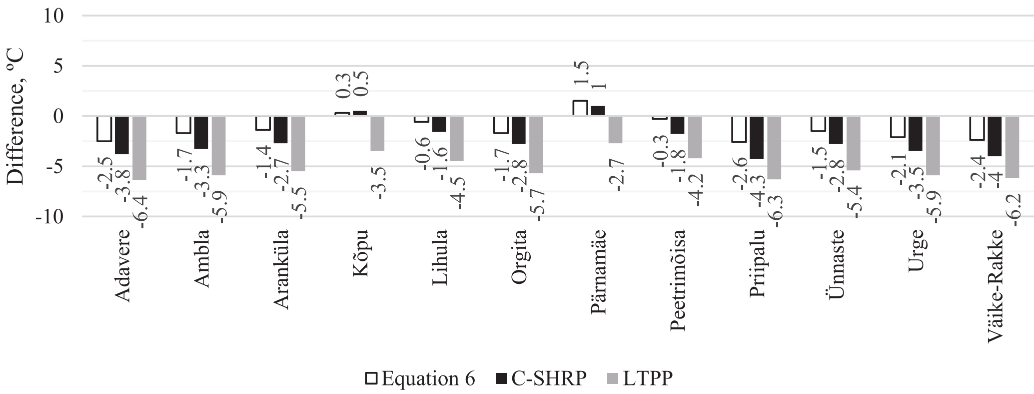

Figure 11. Maximum difference between measured and predicted minimum pavement temperatures (H = 60 mm).

be narrower due to rutting caused by extensive usage of studded tires in Estonia. Another limitation is that the model does not differentiate between pavement materials, which could affect the temperature dissipation in pavement with respect to depth. The models introduced in this paper are only applicable to asphalt pavements. It is suggested that these variables should be considered in the future if further refinement of the model is needed.

7. Conclusion

Different widely adopted pavement temperature prediction models can lead to erroneous results outside their validation region. The daily minimum and maximum air and pavement temperature data from six different Road Weather Stations in Estonia were analysed to establish two statistical-empirical regression models to predict the upper- and lower-bound asphalt pavement seasonal temperatures. Predicted surface temperatures are in good agreement with measured temperatures. The correlative models can be used with a fair degree of accuracy to predict pavement maximum and minimum pavement temperatures for Superpave Performance Grading in Estonia. The authors wish to point out that the introduced models are based on pavement temperature data from two reference depths (0 and 60 mm). Further analysis with data from a wider depth range would be beneficial to validate the models. The authors point out that the models may need to be reviewed in the future to account for the potential impacts of climate change on the accuracy of the models.

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Disclosure statement

No potential conflict of interest was reported by the authors.

Author contribution

Karli Kontson: Conceptualization, Data Analysis, Writing – Original Draft, Review and Editing of Final Manuscript. Reviewing and Editing of the Reviewed Manuscript.
Kristjan Lill: Data Collection, Analysis, Review of Final Manuscript.
Andrus Aavik: Conceptualization, Supervision, Review of Final Manuscript.

ORCID

Karli Kontson http://orcid.org/0000-0002-4029-3245
Kristjan Lill http://orcid.org/0000-0002-2095-806X
Andrus Aavik http://orcid.org/0000-0002-8672-0345

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Appendix 3

Publication III

Kontson, K., Lill, K., Ellmann, A., & Aavik, A. (2025). The Impact of Climate Change to Pavement Temperatures and Asphalt Binder Grade Selection in Estonia Based on Different Climate Change Scenarios. The Baltic Journal of Road and Bridge Engineering, 20 (1), 94–113. https://doi.org/10.7250/bjrbe.2025-20.655

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THE IMPACT OF CLIMATE CHANGE ON PAVEMENT TEMPERATURES AND ASPHALT BINDER GRADE SELECTION IN ESTONIA BASED ON DIFFERENT CLIMATE CHANGE SCENARIOS

KARLI KONTSON*, KRISTJAN LILL, ARTU ELLMANN, ANDRUS AAVIK

Department of Civil Engineering and Architecture, Tallinn University of Technology, Tallinn, Estonia

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Abstract. This paper examines the impact of climate change on asphalt pavement temperatures and Superpave asphalt binder Performance Grade (PG) selection in Estonia. Pavement temperatures were estimated using statistical-empirical pavement temperature prediction models tailored for Estonian conditions. The impact of climate change on pavement temperatures was assessed based on the latest climate change models, assuming three different climate change scenarios for the near, medium and long terms. Projected changes reflect warming trends, with both coastal and mainland areas experiencing substantial shifts in binder grades. Asphalt binder low temperature PG grades are more significantly influenced by climate change, leading to narrower pavement temperature ranges in the region.

Keywords: asphalt binder, climate change, pavement temperature, performance grading, Superpave.

* Corresponding author. E-mail: karli.kontson@taltech.ee

Karli KONTSON (ORCID ID 0000-0002-4029-3245)
Kristjan LILL (ORCID ID 0000-0002-2095-806X)
Artu ELLMANN (ORCID ID 0000-0002-5512-8461)
Andrus AAVIK (ORCID ID 0000-0002-8672-0345)

Introduction and background

Human activities, particularly the burning of fossil fuels (coal, oil, and natural gas), deforestation, and industrial processes have significantly increased the concentrations of greenhouse gases (such as carbon dioxide, methane, and nitrous oxide) in the atmosphere. These increased concentrations enhance the natural greenhouse effect, leading to global warming and climate change (Krinner et al., 2023; Myers et al., 2015; Ripple et al., 2024).

Rising global temperatures lead to extreme weather events, such as heat waves, which can damage roads, bridges, and buildings (FHWA, 2015; Pörtner et al., 2022). The performance of asphalt pavements is significantly influenced by temperature, with the asphalt binder playing a critical role in the pavement's overall behaviour. Higher air and pavement temperatures, particularly during warmer seasons, can lead to irreversible plastic deformations, such as longitudinal rutting and shoving, especially under heavy and slow-moving traffic (FHWA, 2015). In cold regions such as Northern Europe and North America, low air temperatures cause asphalt to become stiffer and less tolerant to strain, increasing the risk of brittle behaviour and premature pavement cracking. Furthermore, asphalt pavements are susceptible to temperature-induced shrinkage and low-temperature cracking, which further exacerbates the likelihood of premature pavement failure (Hesp et al., 2009; Yee et al., 2006). In the USA alone, the climate change is predicted to add from 13.6 to 35.8 billion US dollars to pavement costs based on different climate projections (Underwood et al., 2017).

1. Pavement temperatures and asphalt binder grade selection

Incorrect asphalt binder grade selection reduces the expected service life of roads, leading to higher maintenance costs for road users and taxpayers (FHWA, 2015; Underwood et al., 2017). Therefore, selecting asphalt binder properties based on local environmental conditions is crucial to ensure the durability and performance of asphalt pavements. In Europe, asphalt binder grade is still decided based on empirical specifications known as penetration grading (CEN, 2009, 2010). However, penetration grading has been criticised for its weak correlation with in-service pavement performance (Adams & Holmgreen, 1986; Kennedy et al., 1994; Petersen et al., 1993). To address these limitations, asphalt binder selection in North America is based on the Superpave (Superior Performing Asphalt Pavements) Performance Grading system, outlined in the AASHTO M 320 specification. According to Superpave principles, asphalt binder grades are denoted as PG HT-LT, where PG stands for Performance Grade, and HT-LT represents the high- and low-temperature grades, respectively. PG grades specify the temperature range, in which the binder is expected to perform adequately under environmental and loading conditions. For example, a binder grade of PG 58-22 indicates suitability for regions where pavement temperatures range from a maximum of +58 °C to a minimum of –22 °C. As per the Superpave specification, PG grades are typically defined in 6 °C increments. (Asphalt Institute, 2011).

The required asphalt binder PG grade for a specific road section is determined using historical air temperature data and predetermined air-pavement temperature relationship models. The pavement's maximum temperature is calculated using Equation (1), and the minimum temperature is calculated using Equation (2). These models were developed based on recorded temperature data from 30 road test sections across North America as part of the Seasonal Monitoring Program under the Long-Term Pavement Performance (LTPP-SMP) study (Asphalt Institute, 2011; Mohseni, 1998).

$$T_{\mathrm{pav,max}} = 54.32 + 0.78 \times T_{\mathrm{air,max}} - 0.0025 \times \mathrm{Lat}^2 - 15.14 \times \mathrm{Log}_{10}(H + 25), \quad (1)$$

where
$T_{\mathrm{pav,max}}$ – maximum pavement temperature at depth H, °C;
$T_{\mathrm{air,max}}$ – daily maximum air temperature or 7-day average high temperature for Performance Grading purposes, °C;
Lat – latitude of the specific road section, °;
H – depth from pavement surface, mm.

$$T_{\mathrm{pav,min}} = -1.56 + 0.72 \times T_{\mathrm{air,min}} - 0.004 \times \mathrm{Lat}^2 + 6.26 \times \mathrm{Log}_{10}(H + 25), \quad (2)$$

where
$T_{\mathrm{pav,min}}$ – minimum pavement temperature at depth H, °C;
$T_{\mathrm{air,min}}$ – daily minimum air temperature, °C;
Lat – latitude of the specific road section, °;
H – depth from pavement surface, mm.

Several studies have highlighted that these models may be unsuitable for regions outside the USA. Specifically, they tend to under- or overestimate pavement in-service temperatures at higher latitudes (Kontson et al., 2023, 2024; Swarna et al., 2023). To address this issue, the authors of this publication have developed separate pavement temperature prediction models using air-pavement temperature data collected in Estonia. The maximum pavement temperature for Estonian regions can be estimated using Equation (3), while the minimum pavement temperature can be estimated using Equation (4). Field validation demonstrated that these models reflect pavement in-service temperatures more accurately compared to the LTPP-SMP models (Kontson et al., 2024).

$$T_{\mathrm{pav,max}} = 1.6302 \times T_{\mathrm{air,max}} - 16.8975 \times \mathrm{Log}_{10}(H + 25) + 27.8947, \tag{3}$$

where
$T_{\mathrm{pav,max}}$ – maximum pavement temperature at depth H, °C;
$T_{\mathrm{air,max}}$ – daily maximum air temperature or 7-day average high temperature for Performance Grading purposes, °C;
H – depth from pavement surface, mm.

$$T_{\mathrm{Pav,min}} = 0.6944 \times T_{\mathrm{Air,min}} + 3.4507 \times \mathrm{Log}_{10}(H + 25) - 6.6132, \tag{4}$$

where

$T_{\mathrm{pav,min}}$ – minimum pavement temperature at depth H, °C;
$T_{\mathrm{air,min}}$ – daily minimum air temperature, °C;
H – depth from pavement surface, mm.

2. CMIP6 climate change model and projections

Climate change models are essential tools for scientists to analyse historical climate changes and predict future trends. These models simulate the physical, chemical, and biological processes of the atmosphere, land, and oceans. They are continually refined as research groups worldwide incorporate higher spatial resolutions, new physical processes, and biogeochemical cycles. These updates are typically aligned with the timeline of the Intergovernmental Panel on Climate Change (IPCC) assessment reports, with model outputs released in preparation for each report (Eyring et al., 2016; Krinner et al., 2023).

The latest IPCC Sixth Assessment Report (AR6) utilises data and projections from CMIP6 (Coupled Model Intercomparison Project Phase 6). CMIP6 is the most current and comprehensive set of climate simulations, supporting climate change assessments and projections. These projections are based on scenarios previously referred to as Representative Concentration Pathways (RCPs). However, IPCC AR6 now relies on Shared Socioeconomic Pathway (SSP) scenarios, which have replaced RCPs (Krinner et al., 2023). SSPs provide a more comprehensive framework by incorporating socioeconomic factors such as population growth, economic development, and technological advancements. This approach enhances the understanding of how these factors influence greenhouse gas emissions and climate change, offering a more holistic perspective for climate modelling and policymaking (van Vuuren & Carter, 2014).

There are mainly three SSP scenarios that are typically referred to (Krinner et al., 2023; van Vuuren et al., 2021):

- SSP1-2.6 represents an optimistic scenario where the world achieves rapid technological advancements, transitions to renewable energy, and addresses social and environmental issues, resulting in relatively low levels of climate change (with global warming likely staying below 2 °C by 2100). The "2.6" refers to the level of radiative forcing (measured in watts per square meter) by 2100.
- SSP2-4.5 is considered a "moderate" or "Middle-of-the-Road" scenario for both socioeconomic development and climate change, where emissions continue at a pace that leads to a temperature increase of approximately 2–3 °C by 2100.
- SSP3-7.0 is a high-emissions, low-development pathway characterised by slow economic growth, high inequality, and limited technological progress. It assumes a fragmented world with poor international cooperation, resulting in high emissions and significant climate change impacts. By 2100, this scenario could lead to a global temperature increase of 3.5–4 °C, which would have severe consequences for ecosystems, human health, and infrastructure.

While there is uncertainty about which SSP pathway will ultimately prevail, SSP2-4.5 is often considered the most likely because it reflects a continuation of current global trends: moderate socioeconomic development, gradual technological progress, and emissions reductions that are not aggressive enough to limit global warming to 2 °C or below (Scafetta, 2024).

3. Objectives

The main objective of this study is to assess the impact of different climate change scenarios on pavement temperatures and Superpave PG grading for Estonia. The assessment uses historical temperature data from 2004 to 2024 and evaluates the impact of climate change based on CMIP6 projections for SSP1-2.6, SSP2-4.5, and SSP3-7.0. The results of the analysis are then used to update the Superpave PG grading to better align with Estonian conditions.

4. Geographical location and climate

Estonia is located in Northern Europe, bordering the Baltic Sea and the Gulf of Finland. It shares land borders with Latvia and Russia and maritime borders with Finland and Sweden. The geographical coordinates of Estonia range from

57°30’ N to 59°49’ N and 21°46’ E to 28°13’ E. The climate is characterised by four distinct seasons, influenced by both maritime and continental climatic factors. The annual average air temperature is 6–7 °C. The highest recorded (11 August 1992), and lowest recorded air temperatures (17 January 1940) are 35.6 °C and −43.5 °C, respectively. The warmest months are June–August, and the coldest are December–February (Estonian Environmental Agency, 2021).

5. Methodology

Meteorological data analysis and projected climate change impact on pavement temperatures were divided into two stages:

Stage I – Calculation of pavement temperatures and Superpave PG HT and LT grades was based on historical temperature data from selected weather stations (Figure 1). Air temperature data from 2004 to 2024 was used for both PG HT and PG LT estimations, according to Equations (3) and (4), respectively.

Stage II – Assessment of the impacts of climate change on pavement temperatures was based on selected weather stations, for the near (2025–2044), medium (2045–2064), and long-term (2065–2084) periods. The calculations were based on the Shared Socioeconomic Pathways SSP1-2.6, SSP2-4.5, and SSP3-7.0 scenarios.

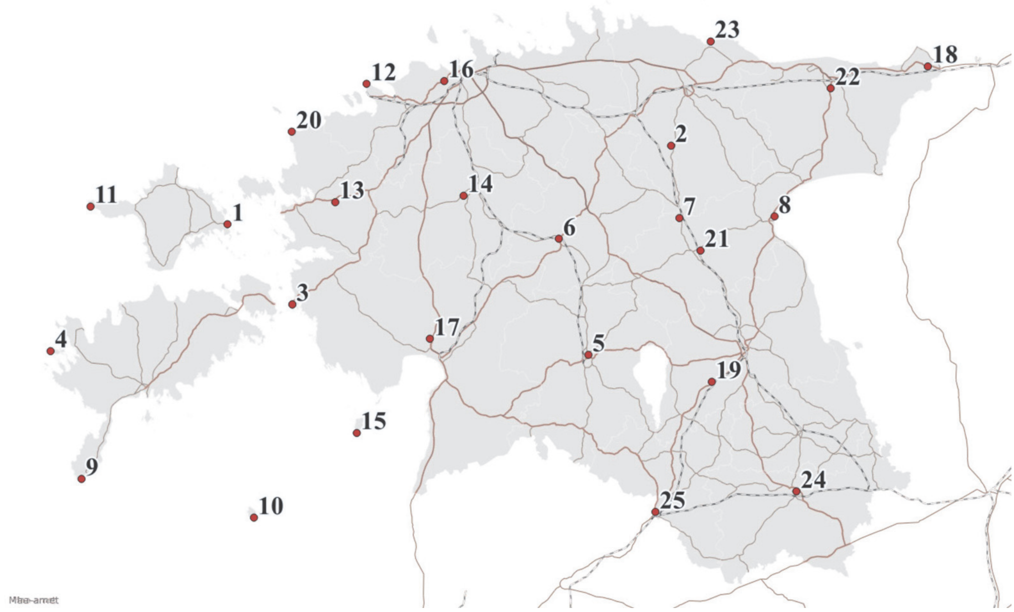

Figure 1. Numbers and locations of the meteorological stations used in the study

Pavement high temperatures are calculated based on historical 1-day average high and 7-day average high air temperatures using Equation (3). The 1-day average high air temperature is used for calculating the maximum pavement temperature. The 7-day average high air temperature is used as the basis for determining PG HT grades for a given station. PG HT grade determination depth is 20 mm (h = 20 mm) from pavement surface. Pavement PG LT grades are calculated based on 1-day average minimum air temperatures using Equation (4). PG LT grade is determined at the surface (h = 0 mm). To achieve at least 98% reliability, the standard deviations (2 × Std dev) of annual 1-day maximum, 7-day maximum, and 1-day minimum air temperatures were also considered in the calculations.

To account for the impacts of climate change in the pavement temperature calculations, changes in the daily maximum temperatures (TXx) for the June–August period and the daily minimum temperatures (TNn) for the December–February period (at 2 m above ground level) relative to the reference period 2004–2024 were extracted from the CMIP6 model. The CMIP6 model average TXx and TNn changes relevant to the Estonian region were derived from the CMIP6 model using the Copernicus Climate Change Service (C3S) online interface, specifically the Copernicus Interactive Climate Atlas. Three 20-year periods were considered for climate change projections: 2025–2044 (near term), 2045–2064 (medium term), and 2065–2084 (long term).

6. Results

The results of the meteorological data analysis for the selected meteorological stations are presented in Table 1. The table also indicates the current PG grades for the locations with reliability ≥98%, based on temperature data from 2004 to 2024. Figures 2 to 4 show the current and estimated changes in pavement maximum surface temperatures due to climate change over the near, medium, and long term. The current PG zone maps are presented in Figure 5.

The calculated pavement temperatures are lowest in coastal areas and on the islands of Western Estonia. In these regions, the highest summer pavement surface temperatures stay below 60 °C, while on the mainland, temperatures are near or slightly above this threshold. The PG HT grade (h = 20 mm), calculated based on the seven consecutive warmest days, is primarily 52 °C for the coastal areas and islands of Western Estonia. According to data from the Sõrve station, the PG HT grade would be 46 °C. On the mainland, the dominant PG HT grade is 58 °C, and in North-East Estonia, it is 52 °C (except for Narva, where the PG HT is 58 °C). Warmer minimum pavement surface temperatures prevail in coastal areas and on the islands of Western Estonia, with the PG LT grade (h = 0 mm) primarily at −22 °C. On the mainland, the dominant PG LT grade is −28 °C, and the lowest PG LT grade is −34 °C, which is observed at two stations located in North-East Estonia (Jõgeva and Jõhvi).

Based on temperature data from 2004 to 2024 and the currently implemented Superpave PG grading determination principles, the primary asphalt binder grades in Estonia would fall within the range of PG HT 52 to 58 °C and PG LT −22 to −34 °C. At present, penetration grade 70/100 is predominantly used across Estonia, which satisfies the required PG HT grades. However, PG LT grade −34 °C is not achievable with this penetration grade, and PG LT grade −28 °C is only achievable with high-quality penetration grade 70/100 binders having low tendency towards physical hardening (Lill et al., 2020a; Lill et al., 2020b). Regions with PG LT −28 and −34 could benefit from using softer binders, e.g., penetration grade 100/150 or 160/220.

Table 1. Average daily maximum and minimum air temperatures and average 7-Day maximum air temperatures with standard deviations (SD) over observed period of 2004 to 2024. PG HT and LT grades are calculated with 98% reliability based on 7-day maximum air temperature and 1-day minimum air temperature data, respectively

(table continues on the next page)

Station no.	Station name	1-Day T_{air} max		7-day T_{air} max		1-day T_{air} min		PG grade	
		T_{air} max °C	Std dev	T_{air} max °C	Std dev	T_{air} min °C	Std dev	HT	LT
1	Heltermaa	28.9	2.1	26.0	2.1	−19.2	5.2	52	−28
2	Väike-Maarja	30.3	1.9	27.3	2.2	−25.2	5.2	52	−28
3	Virtsu	29.5	2.2	26.5	1.9	−20.3	5.3	52	−28
4	Vilsandi	28.6	2.3	25.4	2	−14.5	5	52	−22
5	Viljandi	31.4	1.8	28.4	1.9	−24.1	5.6	58	−28
6	Türi	30.8	1.8	27.9	2.1	−23.8	4.9	58	−28
7	Tooma	30.8	1.8	27.7	2.2	−25.4	5.5	58	−28
8	Tiirikoja	29.6	1.7	26.7	1.6	−24.8	5.9	52	−28
9	Sõrve	26.9	2.0	24.3	1.7	−14.6	4.8	46	−22
10	Ruhnu	27.6	1.8	25.3	1.7	−13.3	4.3	52	−22
11	Ristna	28.5	2.3	25.5	2.2	−15.7	4.6	52	−22
12	Pakri	29.6	2.3	24.8	2.2	−17.5	5.2	52	−22
13	Lääne-Nigula	30.7	1.9	27.8	2.3	−22.7	5.0	58	−28
14	Kuusiku	30.5	1.7	27.8	2.1	−24.8	5.0	58	−28
15	Kihnu	29.2	1.7	26.3	1.9	−17.7	5.2	52	−22
16	Tallinn-Harku	30.4	2.1	27.0	2.5	−20.1	4.6	58	−28
17	Pärnu	30.6	1.5	27.6	1.9	−22.3	6.0	52	−28
18	Narva	31	2.2	27.6	2.3	−23.8	5.2	58	−28
19	Tartu	31.2	1.5	28.0	2.1	−24.6	5.8	58	−28

Table 1. (continue) **Average daily maximum and minimum air temperatures and average 7-Day maximum air temperatures with standard deviations (SD) over observed period of 2004 to 2024. PG HT and LT grades are calculated with 98% reliability based on 7-day maximum air temperature and 1-day minimum air temperature data, respectively**

Station no.	Station name	1-Day T_{air} max		7-day T_{air} max		1-day T_{air} min		PG grade	
		T_{air} max °C	Std dev	T_{air} max °C	Std dev	T_{air} min °C	Std dev	HT	LT
21	Jõgeva	31	1.8	27.9	2.1	–26.9	5.5	58	–34
22	Jõhvi	30.2	2.2	27.2	2.3	–25.9	6.0	52	–34
23	Kunda	30.7	2.6	26.6	2.6	–21.2	5.8	52	–28
24	Võru	31.7	1.6	28.6	1.9	–25.4	5.7	58	–28
25	Valga	31.4	1.6	28.4	1.9	–24.9	5.7	58	–28

Near-term climate change is expected to affect pavement maximum surface temperatures by approximately 1–2 °C, depending on the selected SSP scenario. The SSP1-2.6 scenario leads to an increase in surface temperatures by 0–1 °C, while SSP2-4.5 and SSP3-7.0 result in a 1–2 °C increase in pavement maximum surface temperatures. In the medium term, pavement maximum surface temperatures are projected to increase by 1–2 °C, 2–3 °C, and 3–4 °C under the SSP1-2.6, SSP2-4.5, and SSP3-7.0 scenarios, respectively. In the long term, pavement maximum surface temperatures are projected to increase by 1–2 °C, 3–4 °C, and 4–5 °C under the SSP1-2.6, SSP2-4.5, and SSP3-7.0 scenarios, respectively.

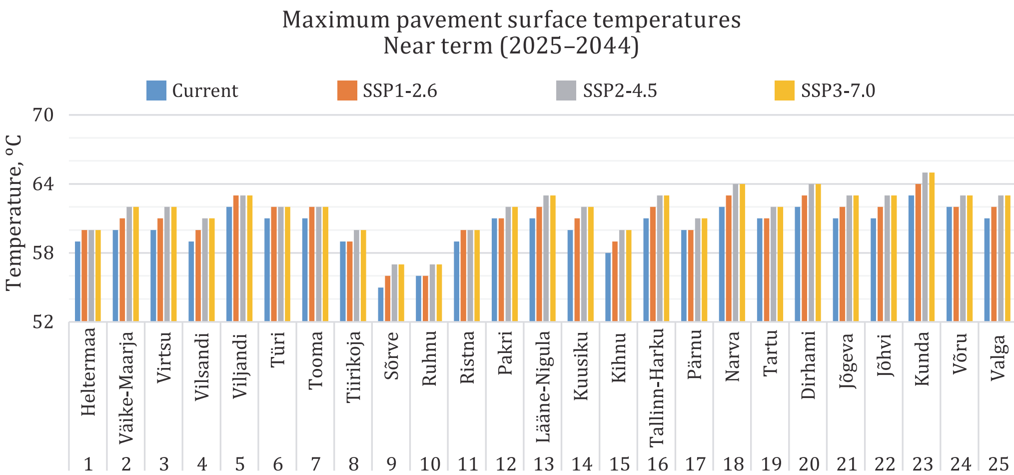

Figure 2. Estimated near-term pavement maximum surface temperatures based on average 1-day maximum air temperatures from 2004 to 2024 (with ≥ 98% reliability)

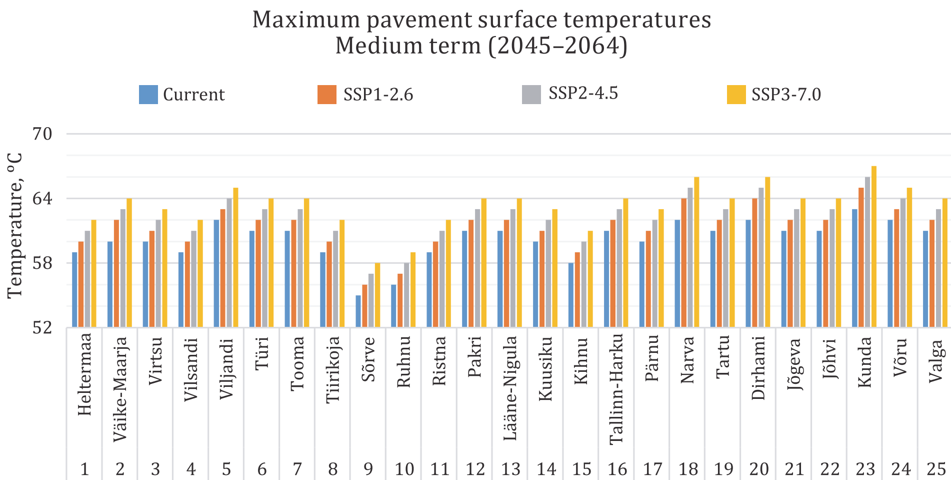

Figure 3. Estimated medium-term pavement maximum surface temperatures based on average 1-day maximum air temperatures from 2004 to 2024 (with ≥ 98% reliability)

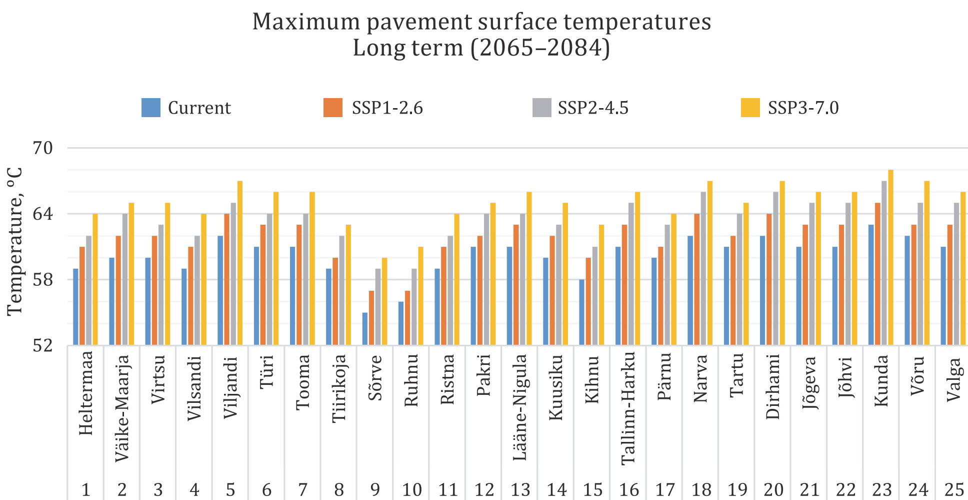

Figure 4. Estimated long-term pavement maximum surface temperatures based on average 1-day maximum air temperatures from 2004 to 2024 (with ≥ 98% reliability)

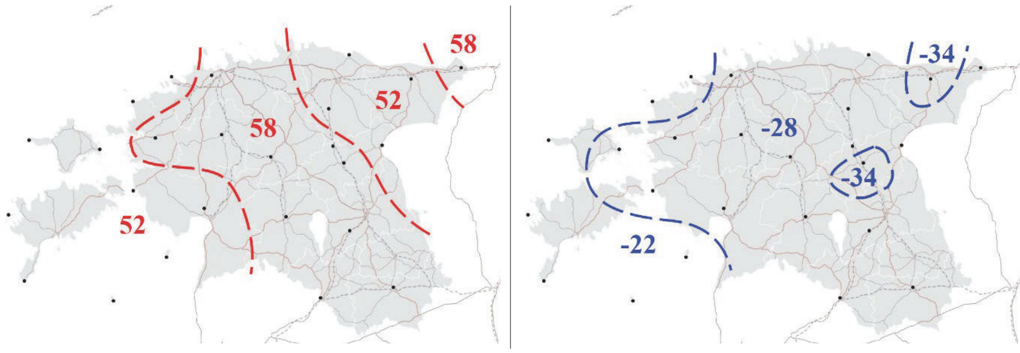

Figure 5. Calculated pavement PG HT and LT design temperatures based on air temperature data from 2004 to 2024

The impact of climate change on Superpave PG grades was assessed based on changes in pavement PG HT and LT temperatures. Calculations for PG HT and LT temperatures were made using 7-day maximum air temperature and 1-day minimum air temperature data, including the standard deviations described in Table 2. The calculated PG HT results are shown in Figures 6 to 8, while the PG LT results are presented in Figures 9 to 11. All results are rounded to the nearest whole number. Changes in PG HT and LT grades for each station are outlined in Table 2. Figures 12 to 14 depict the PG zones for the near to long term based on SSP2-4.5 scenarios.

6.1. Near-term scenarios (2025–2044)

In the near-term SSP1-2.6 scenario, the PG HT grade increases in three locations (Väike-Maarja, Jõhvi, Kunda from 52 to 58 °C). In the same scenario, the PG LT grade increases in four locations (Heltermaa, Tallinn-Harku from −28 to −22 °C; Jõgeva, Jõhvi from −34 to −28 °C). In near-term SSP2-4.5 and SSP3-7.0 scenarios, the PG HT grade increases in five locations (Väike-Maarja, Pärnu, Jõhvi, Kunda from 52 to 58 °C; Sõrve from 46 to 52 °C). In the same scenarios, the PG LT grade increases in six locations (Heltermaa, Virtsu, Tallinn-Harku from −28 to −22 °C; Ruhnu from −22 to −16 °C; Jõgeva, Jõhvi from −34 to −28 °C).

6.2. Medium-term scenarios (2045–2064)

In the medium-term SSP1-2.6 and SSP2-4.5 scenarios, the PG HT grade increases in the same five locations (Väike-Maarja, Jõhvi, Kunda, Pärnu from 52 to 58 °C; Sõrve from 46 to 52 °C). In SSP1-2.6 scenario, PG LT increases in five locations (Heltermaa, Virtsu, Tallinn-Harku from −28 to −22 °C; Jõgeva, Jõhvi from −34 to −28 °C). However, in SSP2-4.5 scenario, the number of locations where PG LT increases is 12 (Heltermaa, Virtsu, Türi, Lääne-Nigula, Tallinn-Harku, Kunda from −28 to −22 °C; Jõgeva, Jõhvi from −34 to −28 °C; Vilsandi, Sõrve, Ruhnu, Ristna,from from −22 to −16 °C). In SSP3-7.0 scenario, the PG HT grade increases in eight locations (Heltermaa, Väike-Maarja, Virtsu, Kihnu, Pärnu, Jõhvi, Kunda from 52 to 58 °C; Sõrve from 46 to 52 °C) and PG LT grade increases in 17 locations (Heltermaa, Väike-Maarja, Virtsu, Viljandi, Türi, Lääne-Nigula, Kuusiku, Tallinn-Harku, Pärnu, Narva, Kunda from −28 to −22 °C; Vilsandi, Sõrve, Ruhnu, Ristna from −22 to −16 °C; Jõgeva, Jõhvi from −34 to −28 °C).

6.3. Long-term scenarios (2065–2084)

In the long-term SSP1-2.6 scenario, the PG HT grade is increasing in the same five locations as described for medium term SSP1-2.6 scenario, but PG LT grade would increase 15 locations (Heltermaa, Virtsu, Türi, Lääne-Nigula, Kuusiku, Tallinn-Harku, Pärnu, Narva, Kunda from −28 to −22 °C; Vilsandi, Sõrve, Ruhnu, Ristna from −22 to −16 °C; Jõgeva, Jõhvi −34 to −28 °C). In SSP2-4.5 scenario, the PG HT grade increases in 10 locations (Heltermaa, Väike-Maarja, Virtsu Tiirikoja, Ristna, Kihnu, Pärnu, Jõhvi, Kunda from 52 to 58 °C; Sõrve from 46 to 52 °C. PG LT grade increases in 22 locations with only three locations remaining unchanged (Kihnu, Dirhami, Võru). In SSP3-7.0 scenario, the PG HT increases in 13 locations (Heltermaa, Väike-Maarja, Virtsu, Vilsandi, Tiirikoja, Ristna, Pakri, Kihnu, Pärnu, Dirhami, Jõhvi, Kunda from 52 to 58 °C; Sõrve from 46 to 52 °C). PG LT increases in all locations. In four locations, the PG LT increases by two grades (Heltermaa, Tallinn-Harku from −28 to −16 °C; Jõgeva, Jõhvi from −34 to −22 °C).

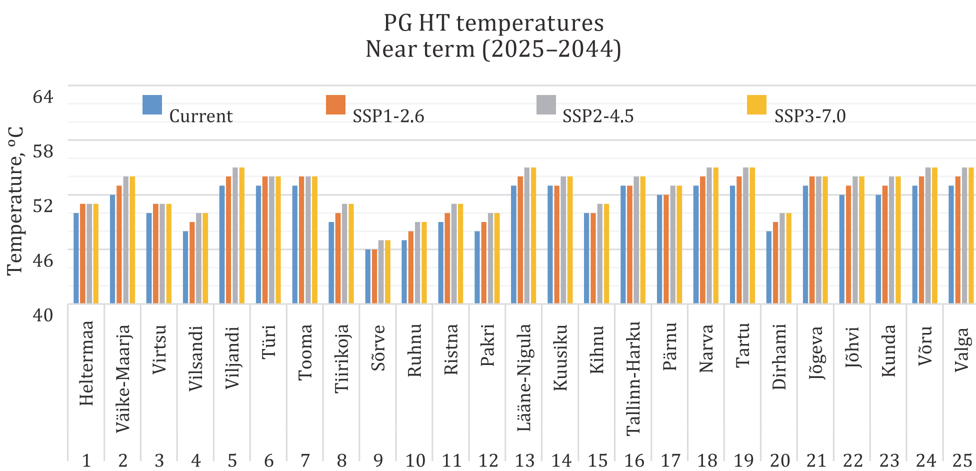

Figure 6. Pavement PG HT true grade temperatures at 20 mm depth under different near-term SSP scenarios (2025–2044)

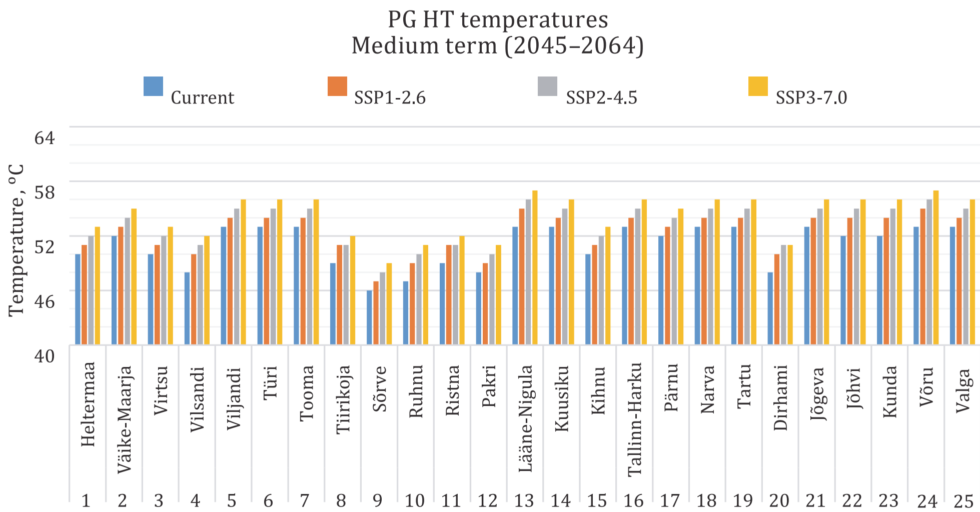

Figure 7. Pavement PG HT true grade at 20 mm depth under different medium-term SSP scenarios (2045–2064)

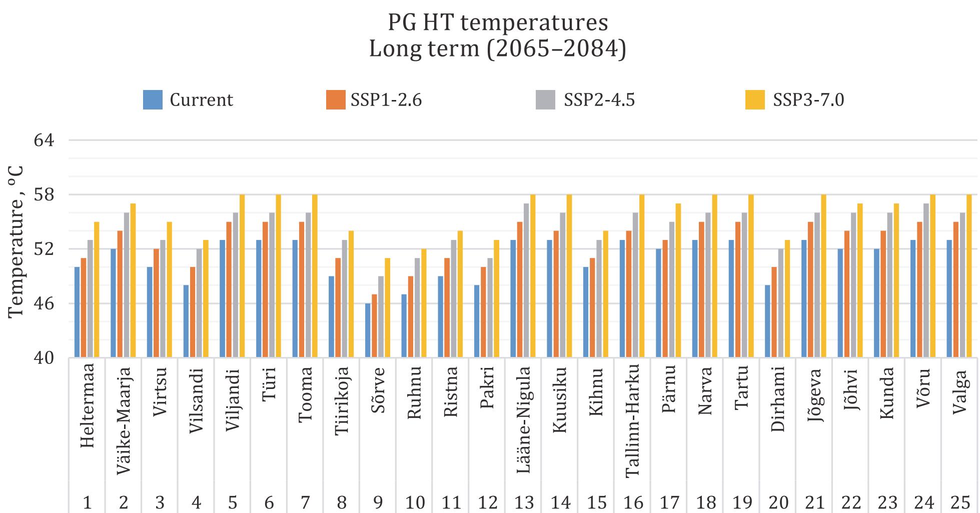

Figure 8. Pavement PG HT true grade at 20 mm depth under different long-term SSP scenarios (2065–2084)

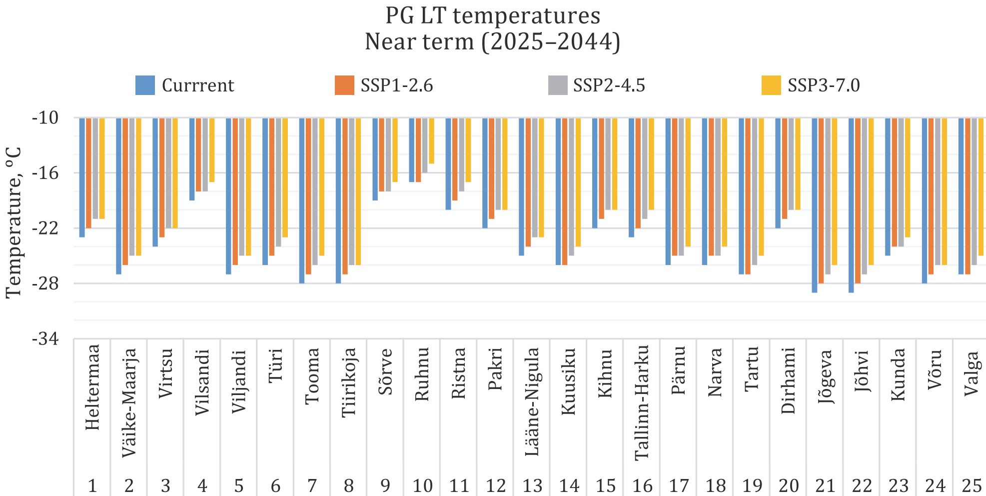

Figure 9. PG LT true grade under different near-term SSP scenarios (2025–2044)

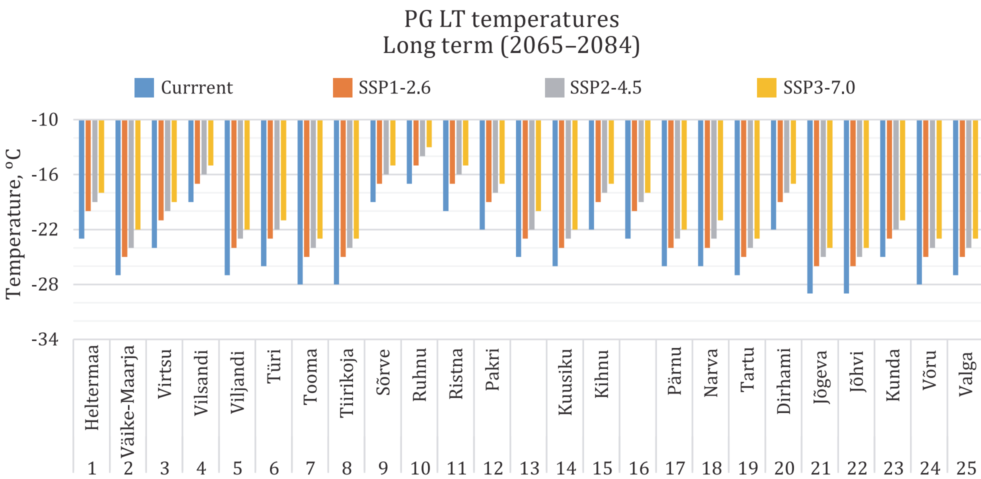

Figure 10. Pavement PG LT true grade under different medium-term SSP scenarios (2045–2064)

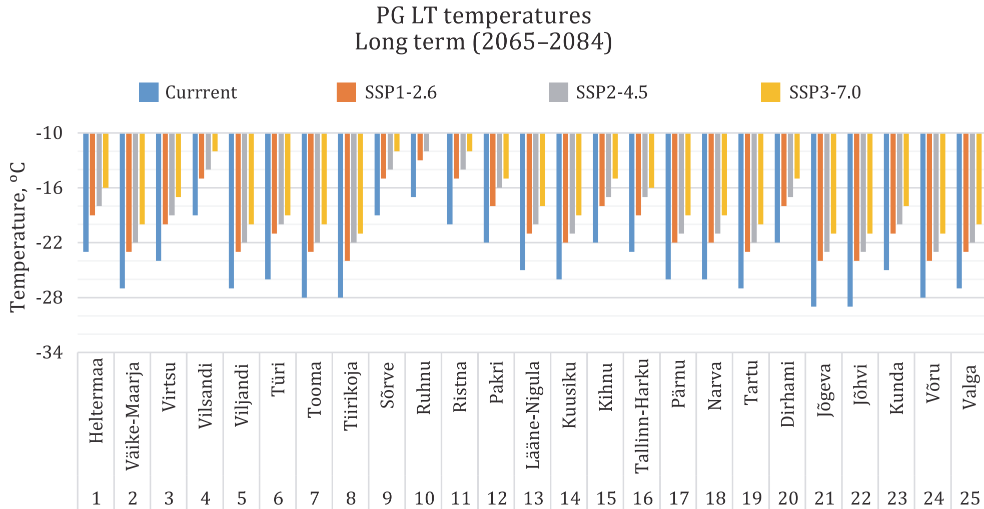

Figure 11. Pavement PG LT true grade under different long-term SSP scenarios (2065–2084)

Table 2. Shift in PG HT and LT Temperatures in different near- (N), Medium- (M) and Long- (L) Term SSP scenarios. One and two plus signs indicate an increase in PG HT and LT grade by one grade (6 °C) or two grades (12 °C), respectively (table continues on the next page)

Station nr	Station	PG HT grade increase									PG LT grade increase								
		SSP1-2.6			SSP2-4.5			SSP3-7.0			SSP1-2.6			SSP2-4.5			SSP3-7.0		
		N	M	L	N	M	L	N	M	L	N	M	L	N	M	L	N	M	L
1	Heltermaa						+		+	+	+	+	+	+	+	+	+	+	++
2	Väike-Maarja	+	+	+	+	+	+	+	+	+						+		+	+
3	Virtsu						+		+	+		+	+	+	+	+	+	+	+
4	Vilsandi									+			+		+	+		+	+
5	Viljandi															+		+	+
6	Türi												+		+	+		+	+
7	Tooma															+			+
8	Tiirikoja						+			+						+			+
9	Sõrve		+	+	+	+	+	+	+	+			+		+	+		+	+
10	Ruhnu												+	+	+	+	+	+	+
11	Ristna						+			+			+		+	+		+	+

Table 2. (continue) **Shift in PG HT and LT Temperatures in different near- (N), Medium- (M) and Long- (L) Term SSP scenarios. One and two plus signs indicate an increase in PG HT and LT grade by one grade (6 °C) or two grades (12 °C), respectively**

Station nr	Station	PG HT grade increase									PG LT grade increase								
		SSP1-2.6			SSP2-4.5			SSP3-7.0			SSP1-2.6			SSP2-4.5			SSP3-7.0		
		N	M	L	N	M	L	N	M	L	N	M	L	N	M	L	N	M	L
14	Kuusiku												+			+		+	+
15	Kihnu						+		+	+									+
16	Tallinn-Harku										+	+	+	+	+	+	+	+	++
17	Pärnu		+	+	+	+	+	+	+	+			+			+		+	+
18	Narva												+			+		+	+
19	Tartu															+			+
20	Dirhami									+									+
21	Jõgeva										+	+	+	+	+	+	+	+	++
22	Jõhvi	+	+	+	+	+	+	+	+	+	+	+	+	+	+	+	+	+	++
23	Kunda	+	+	+	+	+	+	+	+	+			+		+	+		+	+
24	Võru																		+
25	Valga															+			+

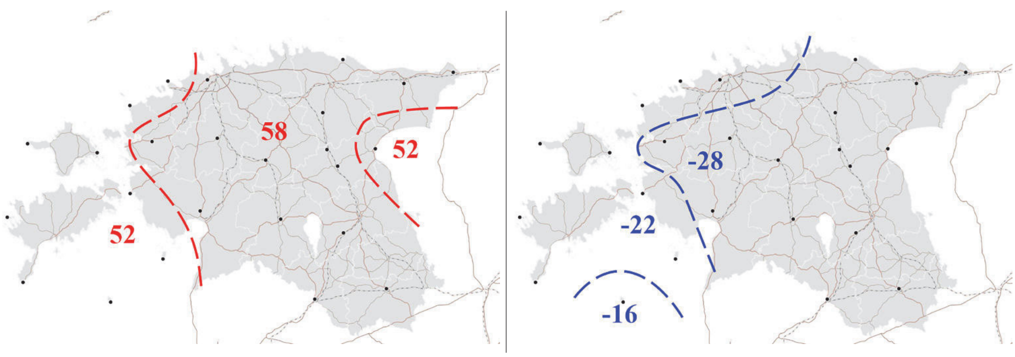

Figure 12. Predicted pavement PG HT and LT design temperatures in 2025–2044 (near term) based on SSP2-4.5

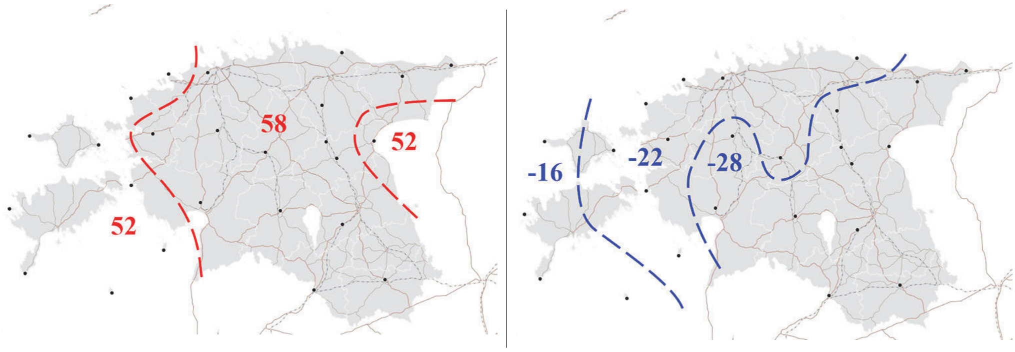

Figure 13. Predicted pavement PG HT and LT design temperatures in 2045–2064 (medium term) based on SSP2-4.5

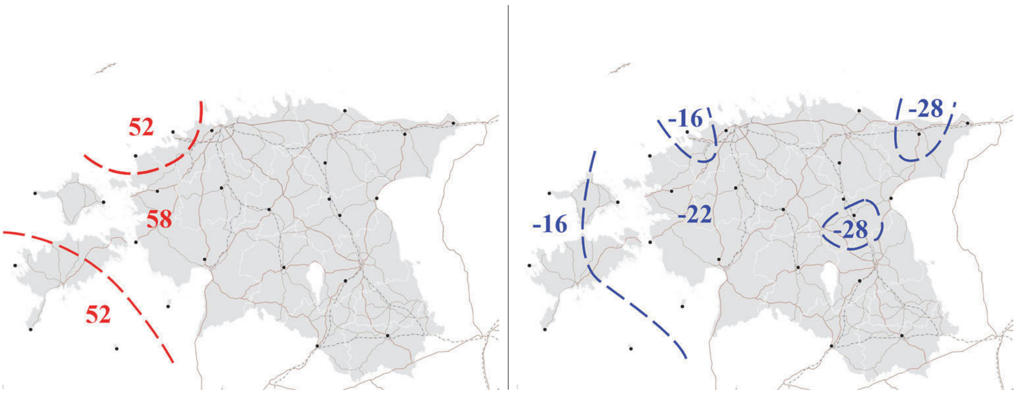

Figure 14. Predicted pavement PG HT and LT design temperatures in 2065–2084 (long term) based on SSP2-4.5

Conclusions

This paper examines the impact of climate change on asphalt pavement temperatures in Estonia. Pavement temperatures were estimated using statistical-empirical pavement temperature prediction models tailored for Estonian climatic conditions. The impact of climate change on pavement temperatures was assessed based on CMIP6 data for the Estonian region, assuming three Shared Socioeconomic Pathway (SSP) scenarios for the near-, medium- and long-terms. Based on the analysed data, the following conclusions can be drawn:

- Pavement temperature analysis reveals regional variations across Estonia, influenced by geographic location. Coastal areas and the islands exhibit the lowest maximum pavement temperatures, while mainland exhibiting the highest pavement maximum temperatures. Based on historical air temperatures from

2004 to 2024, which corresponds to current PG grade determination practice, the PG HT grades for coastal and island regions are 52 °C, dropping to 46 °C at Sõrve. On the mainland, the prevalent PG HT grade is 58° C, with North-East Estonia primarily at 52 °C. Warmer Superpave PG LT temperatures are noted in coastal and island regions, where the PG LT grade is primarily −22 °C. On the mainland, the dominant PG LT grade is −28 °C, with the lowest recorded grade of −34 °C near Jõgeva and Jõhvi in North-East Estonia. Currently, widely used penetration grade 70/100 binders in Estonia are suitable to meet required PG HT grades. However, only high quality 70/100 binders would cover PG LT −28 °C. Regions with PG LT −28 or −34 °C benefit from using softer binders to mitigate risks associated with low temperature cracking.
- All projected climate change scenarios indicate a general increase in both PG HT and LT grades across Estonia. These changes reflect warming trends, with both coastal and mainland areas experiencing substantial shifts in PG HT and LT grades. The LT grades are more significantly influenced by climate change, leading to narrower pavement temperature ranges in the region.
- Under the most likely climate change scenario SSP2-4.5, the PG HT grade is expected to be mostly 58 °C in the observed short-, medium- and long-term perspectives. For PG LT, the dominant grades in the short and medium-term will be −28 and −22 to −28 °C, respectively. In the long-term perspective, PG LT −22 °C will become the most dominant grade. The referenced changes illustrate that due to climate change, the requirements for asphalt binder properties and performance at low temperatures will become less stringent. Conversely, greater consideration must be given to the properties of the binder at high pavement temperatures.

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Disclosure Statement

The authors indicate that there is no conflict of interest.

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Curriculum vitae

Personal data

Name:	Karli Kontson
Date of birth:	01.12.1987
Place of birth:	Tallinn, Estonia
Citizenship:	Estonia

Contact data

E-mail:	karli.kontson@taltech.ee / karli.kontson@gmail.com

Education

2015–2025	Tallinn University of Technology, PhD
2012–2014	Tallinn University of Technology, MSc
2008–2012	Tallinn University of Applied Sciences, Dipl.Eng
1995–2007	Jüri Gymnasium

Language competence

Estonian	native
English	fluent
Finnish	average

Professional employment

2022–…	Rail Baltic Estonia OÜ, Senior materials engineer
2021–…	Tallinn University of Technology, Project manager
2018–2021	RB Rail AS, Team leader (construction materials and geotechnical engineering)
2018–2019	Tallinn University of Technology, Lecturer (road construction materials)
2013–2018	Estonian Road Administration, Project manager (Research and development)
2010–2013	Technical Centre of Estonian Roads, Laboratory technician/specialist

Supervised MSc theses

Kangur, M. (2025) Asfalteerimise, pindamise ja mössiga pindamise keskkonnajalajälje analüüs ja võrdlus [*Analysis and comparison of the environmental footprint of asphalt paving, surface dressing and slurry sealing*]. Supervised by **Kontson, K**. Master's thesis. Tallinn, Estonia: Tallinn University of Technology.

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Elulookirjeldus

Isikuandmed

Nimi:	Karli Kontson
Sünniaeg:	01.12.1987
Sünnikoht:	Tallinn, Eesti
Kodakondsus:	Eesti

Kontaktandmed

E-mail:	karli.kontson@taltech.ee / karli.kontson@gmail.com

Hariduskäik

2015–2025	Tallinna Tehnikaülikool, PhD
2012–2014	Tallinna Tehnikaülikool, MSc
2008–2012	Tallinna Tehnikakõrgkool, Dipl. insener
1995–2007	Jüri Gümnaasium

Keelteoskus

Eesti keel	emakeel
Inglise keel	kõrgtase
Soome keel	kesktase

Teenistuskäik

2022–…	Rail Baltic Estonia OÜ, ehitusmaterjalide vaneminsener
2021–…	Tallinna Tehnikaülikool, projektijuht
2018–2021	RB Rail AS, meeskonna juht (ehitusmaterjalid ja geotehnika)
2018–2019	Tallinna Tehnikaülikool, lektor (tee-ehitusmaterjalid)
2013–2018	Maanteeamet, projektijuht (uuringud ja arendus)
2010–2013	Teede Tehnokeskus AS, labori tehnik/spetsialist

Magistritööde juhendamine ja teaduspublikatsioonid

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